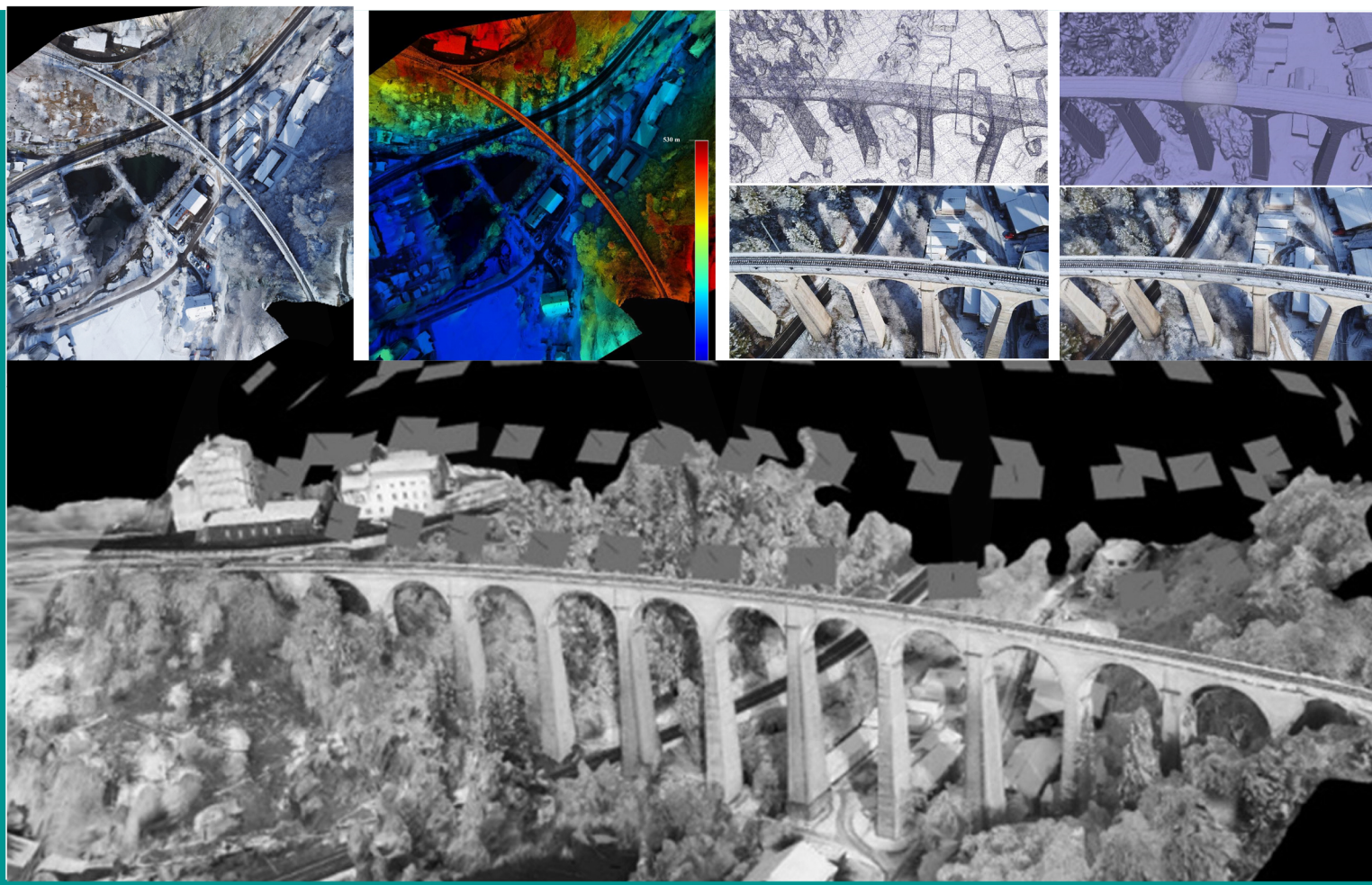
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Bulletin 109



Existing concrete structures life management, testing and structural health monitoring

State-of-the-art report

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fib CEB-FIP Bulletin 109

Existing concrete structures life management, testing and structural health monitoring

State-of-the-art report

Task Group 3.3

October 2023

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Approval for this bulletin

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Category:	**Approval by**:
Technical report	Task group and chairpersons of the commission
State-of-the-art report	Commission
Manual / Guide to good practice / Recommendation	Technical Council
Model code	General Assembly

Any publication not having met the above requirements will be clearly identified as a preliminary draft.

fib Bulletin 109 was approved as a state-of-the-art-report by the *fib* Technical Council in June 2023.

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Cover images: Combe Maran viaduct in St. Ursanne (CH), credit photo Hoffmann, M. (2021) created on the basis of publicly available UAV/UAS data of the Combe Maran viaduct in St. Ursanne (CH) created with Agisoft Metashape and 3DFlow Zephyr.

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ISSN 1562-3610
ISBN 978-2-88394-172-4
https://doi.org/10.35789/*fib*.BULL.0109
Layout by Corinne Bottollier (*fib*).
Printed by P+K Solutions GmbH & Co. KG

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Acknowledgements

This state-of-the-art-report was drafted by Task Group 3.3 - "Existing concrete structures: life management, testing and structural health monitoring" within the *fib* Commission 3 *Existing concrete structures*.

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DOI by chapter

Chapters	Authors	DOI
1	Andrade, Apostolidi, Chatzi, Duprat, Keßler, Lehky, Limongelli, Novák, Sousa*, Strauss.	https://doi.org/10.35789/fib.BULL.0109.Ch01
2	Bartolac, Bień, Górski, Keßler*, Küttenbaum, Kuzawa*, Ley, Maack, Mendler, Ryjáček, Santos, Verstrynge.	https://doi.org/10.35789/fib.BULL.0109.Ch02
3	Andrade, Chatzi*, Darò, Gentile, Górski, Limongelli*, Oliveira, Mancini, Shan, Sousa, Ueda, Verstrynge, Zabel.	https://doi.org/10.35789/fib.BULL.0109.Ch03
4	Andrade, Apostolidi, Duprat, Lehky, Matos, Novák, Sousa*, Sousa S, Strauss*.	https://doi.org/10.35789/fib.BULL.0109.Ch04
5	Braml*, Kainz, Keuser, Zimmert*.	https://doi.org/10.35789/fib.BULL.0109.Ch05
6	Ballio, Chatzi*, Cigada, Darò, Elfgren, Gentile, Górski, Küttenbaum, Limongelli*, Mancini, Oliveira, Rasol, Schmidt, Shan, Sousa, Torrent, Ueda, Verstrynge.	https://doi.org/10.35789/fib.BULL.0109.Ch06

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Foreword

The goal of *fib* Commission 3 *Existing Concrete Structures* is to define appropriate and reliable procedures to establish the safety of existing concrete structures and any associated requirements for interventions to extend the safe operation or working life of such structures. Furthermore, *fib* Commission 3 pursues the active dissemination of the outcomes of its work through the preparation of comprehensive guidelines for the assessment of existing concrete structures. These are also intended to support the development of a new *fib* Model Code which, for the first time, will also contain extensive technical information relating to existing concrete structures.

In view of the new challenges to be met with regard to the safety and life management of existing structures, the former *fib* Special Activity Group 7 Assessment and Interventions upon Existing Structures initiated a Working Group 3 entitled “A Through Life Management of Existing Structures” in 2013 with the aim of developing a technical recommendation on life measurement methods for the assessment of existing concrete structures. In 2015, *fib* Special Activity Group 7 was transformed into *fib* Commission 3 *Existing Concrete Structures*. The task of taking over the editorial work to finalise the technical recommendations as a *fib* Bulletin was taken over by Task Group 3.3 "Existing Concrete Structures: Life Management, Testing and Structural Health Monitoring" in 2019.

Four years of fruitful contributions and discussions within this working group (and the previous working group) have resulted in a technical recommendation that provides a basis for rational decision-making on issues related to life cycle management, inspection and monitoring of the structural condition of existing structures, as well as a comprehensive collection of background information and procedures for the assessment of existing structures.

We would like to thank the members of the task group for their work in developing this new *fib* State-of-the-Art report which is expected to serve as a valuable reference document, stimulate further research developments in the field of the assessment of existing structures and provide a practical basis for rational decision making for all those involved in this area of work including practitioners and code writers. Finally, we would like to express our appreciation and sincere thanks to the reviewers, Prof. Joan Casas and Prof. Emin Aktan, for their comprehensive and valuable comments and suggestions.

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Preface

The through-life management of our constantly ageing infrastructure is a basic requirement in order to ensure their structural safety and serviceability. Each structure experiences deterioration processes with time leading to a decrease of structural safety and serviceability. The design of new structures considers the expected deterioration for a defined period, the design service life. However, a frequent survey of structural safety controlling structural condition should be mandatory and a maintenance plan should be an integral part of the design. In addition, many structures have exceeded their design service life already or are very close to it leading to an increasing demand for condition assessment. On the one hand, assumptions made during design are not valid any more due to change of the loads, e.g., increasing traffic loads in terms of number and weights. On the other hand, design codes evolved over time in such a way that existing structures do not comply with today's standards. In all these cases, the through-life management is an important tool to maintain the accessibility of existing structures with known reliability.

In line with the new Model Code for Concrete Structures, which includes guidance for both – design of new structures and assessment of existing structures, the Task Group 3.3 focused on the compilation of a state-of-the-art guideline for the through-life management of existing concrete structures, including:

- Data acquisition by testing and monitoring techniques;
- Condition assessment for the evaluation of existing structures;
- Performance prediction using advanced methods;
- Decision-making procedures to perform a complete assessment of existing structure.

The overall objective of the through-life management is the assessment of the current condition and the estimation of the remaining service life under consideration of all boundary conditions.

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Nomenclature

List of symbols and abbreviations which are consistently used in this Bulletin. Symbols that are not listed here are defined at their location of first use. Some symbols are used in different sections of the document and therefore in different contexts of science with different meanings. For these, all meanings are introduced and separated by the slash symbol"/".

Roman and Greek symbols

a_{cc}	absorption of water by capillarity at time t_i
a_{ij}	transition probability from state i to state j
a	exposed area of the specimen / distance
A	cross-section area of the element
β	reliability index
$C(x,t)$	concentration function in point x at time t
C	ion concentration in the fluid
D	diffusion coefficient / accumulated fatigue damage
δ_w	density of water
E	electric potential across the element
$E[\ ]$	expected value
ε	difference between actual and predicted values
e^i	geometric element with dimension i
$f(\ )$	model function (meaning 1) / probability density function (meaning 2)
$f_{1\rightarrow i}(t)$	probability density function of the sum of the sojourn times from state 1 to i
F	Faraday constant (96,485 J. V^{-1}. mol^{-1})
I	absorption / current
J	flux of chloride ions
K	geometric factor
K_g	intrinsic coefficient of permeability to gases
K_w	Darcy's coefficient of water permeability
L	length of the element
m	mass
M_w	coefficient of water absorption or water absorption rate
μ	dynamic viscosity of the liquid / mean value
n_{Ei}	number of acting stress cycles associated with the stress range for steel and the stress level for concrete
N_{Ri}	number of resisting stress cycles at a given stress range for steel and stress level for concrete
$p(\ )$	probability
$p(\|)$	conditioned probability
p_f	failure probability related to a reference period (usually one year)
P	pressure
P_{ij}	probability of a transition between states i and j per unit of time
P_{corr}	penetration of corrosion attack
$\pi(\ )$	prior distribution representing the knowledge before collecting data
$\pi(\|)$	posterior distribution representing the updated knowledge considering collected data

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$Ø$	diameter
Q	flow rate of substance transported
R	universal gas constant (8.31 $J.mol^{-1}.K^{-1}$)
ρ	specific electrical resistance of the material
S_c	absorption coefficient or absorption rate
S_i	true state at time *i*
$S_{1\rightarrow i}(t)$	survival function of the sum of the sojourn times from state 1 to state *i*
s^i	model space with dimension i
σ	conductivity / standard deviation
t	time
t_{ini}	time of the initiation period
t_{prop}	propagation period
t_{SL}	design service life
T	temperature
$T_{i\rightarrow k}$	time the process will take to move from state *i* to *i*
θ	vector of parameters to be inferred
$u(\hat{y})$	standard measurement uncertainty
$U(\hat{y})$	expanded measurement uncertainty
V	electrical potential
V_{corr}	averaged value of the corrosion rate per year in the period considered
V_i	observed state at time *i*
w	water absorbed per unit area at time *t*
x	vector of observations / distance
$\hat{x}$	estimates of the input quantities
X_i	input parameters of the model
Y	measurand
$\hat{y}$	measured quantity value

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Acronyms and abbreviations

AMP	**A**sset **M**anagement **P**lan
ANN	**A**rtificial **N**eural **N**etworks
AR	**A**uto**R**egressive
ASR	**A**lkali **S**ilica **R**eaction
BIM	**B**uilding **I**nformation **M**odelling
BK	Bridge class (from German: **B**rücken**K**lasse)
BMS	**B**ridge **M**anagement **S**ystem
BWIM	**B**ridge **W**eigh-**I**n-**M**otion
C	**C**lass of the defect
CAN	**C**ontroller **A**rea **N**etwork (CAN bus)
CC	**C**onsequences **C**lasses
CCTV	**C**losed-**C**ircuit **T**ele**V**ision
CEAM	**C**onstrained **E**igenstructure **A**ssignment **M**ethod
CLS	**C**ondition **L**imit **S**tate
CNN	**C**onvolutional **N**eural **N**etwork
CNT	**C**arbon **N**ano**T**ubes
CoV	**C**oefficient **o**f **V**ariation
DEF	**D**elayed **E**ttringite **F**ormation
DF	**D**amage **F**eature
DFT	**D**amage **F**eature **T**hreshold
DI	**D**amage **I**ndicator
DL	**D**eep **L**earning
DT	**D**estructive **T**esting
E	Defect **E**xtent
EMA	**E**xperimental **M**odal **A**nalysis
EOCs	**E**nvironmental and **O**perational **C**onditions
FBA	**F**orce **B**alanced **A**ccelerometer
FDD	**F**requency **D**omain **D**ecomposition
FDTD	**F**inite-**D**ifference **T**ime-**D**omain
FE	**F**inite **E**lement
FEM	**F**inite **E**lement **M**odel
FNN	**F**uzzy **N**eural **N**etwork
FORM	**F**irst Order Reliability Method
FOS	**F**ibre **O**ptic **S**ensors
GPS	**G**lobal **P**ositioning **S**ystem
GPR	**G**round **P**enetrating **R**adar
GUM	**G**uide to the Expression of **U**ncertainty in **M**easurement
HCP	**H**alf-**C**ell **P**otential
HMMs	**H**idden **M**arkov **M**odels
I	Class of **I**ntensity
IE	**I**mpact **E**cho
ID	**ID**entification
IoT	**I**nternet **o**f **T**hings
IR	**I**nfra**R**ed
IRT	**I**nfrared **T**hermography
KL	**K**ullback–**L**eibler (divergence)
KPI	**K**ey **P**erformance **I**ndicator
L	**D**efect **L**ocation
LM	**L**oad **M**odel
LPV	**L**inear **P**arameter **V**arying
LS	**L**east **S**quares
LSF	**L**imit **S**tate **F**unction
LTI	**L**inear and **T**ime **I**nvariant

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LVDT	**L**inear **V**ariable **D**ifferential **T**ransformer
LZT	**L**ead **Z**irconate **T**itanate
ML	**M**achine **L**earning
MLR	**M**ultiple **L**inear **R**egression
MEMS	**M**icro-**E**lectro**M**echanical **S**ystems
MSC	**M**agnetic **S**liding **C**ollar
NDT	**N**on-**D**estructive **T**esting
NLFEA	**N**on-**L**inear **F**inite **E**lement **A**nalysis
OCC	**O**ne-**C**lass **C**lassification
ODS	**O**perational **D**eformed/**D**eflection **S**hapes
OMA	**O**perational **M**odal **A**nalysis
PC	**P**rincipal **C**omponent
PCA	**P**rincipal **C**omponent **A**nalysis
PCE	**P**olynomial **C**haos **E**xpansion
PDF	**P**robability **D**ensity **F**unction
PFA	**P**robability of **F**alse **A**larm (false indication of damage)
PGA	**P**eak **G**round **A**cceleration
pH	**P**otential of **H**ydrogen
PI	**P**erformance **I**ndicator
PMA	**P**robability of **M**issing **A**larm
PoD	**P**robability **o**f **D**etection
PSD	**P**ower **S**pectral **D**ensity
QC	**Q**uality **C**ontrol
QCP	**Q**uality **C**ontrol **P**lan
RC	**R**einforced **C**oncrete
RH	**R**elative **H**umidity
RMS	**R**oot **M**ean **S**quare
SAMP	**S**trategic **A**sset **M**anagement **P**lan
SDT	**S**emi-**D**estructive **T**esting
SF	**S**urvival **F**unction
SHM	**S**tructural **H**ealth **M**onitoring
SI	**I**nternational **S**ystem of Units
SLS	**S**erviceability **L**imit **S**tate
SORM	**S**econd **O**rder **R**eliability **M**ethod
St-ID	**St**ructural **ID**entification
SVD	**S**ingular **V**alue **D**ecomposition
SVM	**S**upport **V**ector **M**achines
TBM	**T**unnel **B**oring **M**achine
TCI	**T**echnical **C**ondition **I**ndex
TDR	**T**ime-**D**omain **R**eflectometry
TPM	**T**ransition **P**robability **M**atrix
UAS	**U**nmanned **A**erial **S**ystems
UAV	**U**nmanned **A**erial **V**ehicles
UHPFRC	**U**ltra-**H**igh-**P**erformance **F**ibre-**R**einforced **C**oncrete
ULS	**U**ltimate **L**imit **S**tate
UMTS	**U**niversal **M**obile **T**elecommunications **S**ystem
UPE	**U**ltrasonic **P**ulse **E**cho
VAR	**Var**iance
VBM	**V**ibration **B**ased **M**onitoring
VIM	**I**nternational **V**ocabulary of **M**etrology
WIM	**W**eigh-**I**n-**M**otion

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Glossary

Below, a list of technical terms, which are consistently used in this Bulletin, is given with brief explanations of their meanings to support the reader's understanding in the context presented.

A	Absorption	Process in which a substance permeates another, as in a liquid permeating, or absorbed by, a solid.
	Accuracy	Difference between the result of the measurement and the true value of the measurand (frequently provided by manufacturers).
	Acceptance criteria	A set of predefined requirements that must be satisfied to be considered accepted by a stakeholder.
	Acoustic emission method	Detects and monitors the release of ultrasonic stress waves from localised sources when a material deforms/fractures under stress.
	Acoustic method	Local impact and then measuring some properties of the specimen vibrations such as resonant frequency, decay time.
	Aleatoric uncertainty	Inherent uncertainty due to the randomness, which is irreducible and characterised by a probability distribution.
	Ambient vibrations	Vibration behaviour of a structure under ambient influences, i.e., natural forcing, e.g., wind or traffic, as opposed to artificial excitation.
	Analytical model	Mathematical relationship between the forces and imposed deformations exerted on a structure or a structural element and its response to those forces (e.g., deformations, displacements, or internal forces).
	Artificial Intelligence techniques	Algorithms that enable machines to perform tasks like speech recognition, problem-solving, and learning.
	Artificial Neural Networks	Artificial intelligence techniques, inspired by the biological neural networks that enable the behaviour of the brain.
	Asset	Infrastructure (resource) with economic value that an individual, corporation or country owns or operates.
	Assessment	See condition assessment.
	Asset management	Operating, maintaining, upgrading, and disposing of assets cost-effectively.
	Autoregressive model	Representation of a type of random process specifying the output variable as a linear function of its own previous values plus the influence of a stochastic term.
B	Bayesian statistics/ updating	Method of statistical inference in which Bayes' theorem is used to update the probability of a hypothesis as more evidence or information becomes available.
	Biological method	Process that is used to degrade and metabolise chemical substances and restore environmental quality.
C	Capacity (structural)	Ability of a member or a component, or a cross-section of a structure to provide load carrying functionality without mechanical failure e.g., bending resistance, buckling resistance, available ductility.
	Capacitive sensor	Sensor that detects nearby objects by their effect on the electrical field created by the sensor.
	Clustering technique	Machine Learning technique that involves the grouping of data points according to criteria (such as density or distance between members of a cluster).
	Condition	State of a system (structure and/or its components) considering deterioration phenomena (e.g., corrosion, carbonation).
	Condition assessment	A process of reviewing information gathered about the current condition of a structure or its components with respect to specific performance levels.
	Condition rating	Assignment of a rating to the performance or the condition of a structural system used to signify the presence and severity of faults/damage.
	Condition survey	The process of acquiring information relating to the current condition of the structure, recognizing defects, and deterioration that can affect its appearance, serviceability, and/or safety.
	Consequence	Outcome of an event including human, economic, and environmental contributions.

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	Consequences class	Classes corresponding to consequences of failure in terms of risk to life, injury, cultural damage, economical loss, and/or environmental damage.
	Corrosion	Natural oxidation of refined metal to a more stable form such as its oxide, hydroxide, or sulphide state, leading to deterioration.
	Cost function	Quantifies discrepancy between predicted and measured values and presents this in the form of a single real number.
D	Damage	Change in the condition of a structure or its components causing a reduction of the expected performance of the structure or its components
	Damage assessment	Process of ascertaining the severity of the damage to a structure.
	Damage detection	Process of ascertaining whether the damage to structure exists or not.
	Damage feature/ parameter	Quantifiable property or pattern sensitive to damage, which can be either directly monitored (e.g., strain) or extracted from monitoring data (e.g., modal characteristics from accelerometer measurements).
	Damage identification/ characterisation	Localisation and assessment, including ascertaining the cause of the damage and its consequences (see damage detection).
	Damage Indicator	A robust damage metric, computed based on damage parameters (or features).
	Damage localisation	Process of ascertaining where the damage to a structure is located.
	Damage mechanism	Refers to the root cause and evolution mechanism of damage processes, which includes corrosion, fracture, fatigue, heat damage, scour, and more.
	Damage prognosis	Prediction of remaining useful life of a damaged system.
	Darcy's law	Equations that describe the flow of a fluid through a porous medium formulated by Henry Darcy.
	Data-driven methods	Algorithms that extract information from measurements without the need for a physical model of the structure.
	Data acquisition	Sampling and processing of data collected by sensors.
	Decision maker	A person or entity, who is bestowed with the power to make strategic decisions, including, expansion, investment, and planning of maintenance and repair (in the infrastructure context).
	Decision-making process	Deciding process based on available or assumed information, resulting in the selection of a choice among alternative options, which may procure an action.
	Deep Learning	Type of machine learning based on artificial neural networks in which multiple layers of processing are used to extract progressively higher-level features from data.
	Defect	One of more flaws whose aggregate size, shape, orientation, location or properties do not meet specified acceptance criteria and are rejectable.
	Degradation process	Defined as degradation mechanisms, i.e., physical, chemical, and/or biological, or their composition that may reduce the performance of the asset over time.
	Degradation process	Process that reduces the performance of the asset over time.
	Dependent variable	Mathematical variable, whose value is determined by one or more other variables in a function.
	Design point	Closest point of the limit state surface to the origin in standard normal space.
	Destructive Testing	Experimental analysis carried out bringing the specimen to failure to determine its physical properties.
	Deterioration	Progressive or sudden reduction in the ability of a structure or its components to perform according to their intended functional specifications.
	Deterministic-based model	Model that does not consider randomness in the development of future states of the system.
	Diagnostic assessment/test	Tool or test for identification of the cause or explanation of the mechanism(s) by which a phenomenon affects the behaviour or the condition of a structure or its components, based on an investigation of signs and indications exhibited.
	Diffusion	Transport mechanism, driven by a concentration gradient, by which gases, liquids, or dissolved ions flow across a porous body.

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	Dynamic test	Test aimed to measure the dynamic response of the structure to extract information about its dynamic characteristics.
E	Eddy current sensor	Inductive sensors' technology, which uses non-contacting position sensors, for high resolution measuring the change of position of a specified target, by the change of current flow due to the variation of a magnetic field.
	Electrical resistance gauge	Element, which changes resistance when subject to strain.
	Electromagnetic method	Method that uses transmitters to create strong time-varying primary magnetic fields that induce electrical currents to flow in conductive materials.
	Electrochemical method	Technique that uses electrical stimulation to analyse the chemical reactivity of a sample surface or a solution.
	Empirical assessment	Dependent upon the training and experience of the expert.
	Environmental Condition	The state of the environment, soil, surface water, ground water, subsurface strata, or ambient air.
	Environmental exposure	Contact with chemical substances found in air, water, or soil as well as physical influences causing damage to construction materials.
	Epistemic uncertainty	Lack of knowledge in a phenomenon or process, as opposed to randomness, also known as systematic uncertainty.
F	Failure	State where the performance level of a structure or a structural element is inadequate, i.e., the achievement of a limit state.
	Failure probability	Probability of achieving a limit state within a defined reference period.
	Fick's laws	Describes the relationship between the rate of diffusion and the three factors that affect diffusion.
	Filtering	Data processing to remove frequency components of signals.
	Full-probabilistic model	Statistical technique used to take into account the impact of random events or actions in predicting the potential occurrence of future outcomes.
G	Gaussian space	A (complete) probability space together with a Hilbert space of centred real-valued Gaussian random variables defined on it.
	Gaussian variable	Random variable following a normal distribution.
	Geodetic technique	Technique for detecting and monitoring the position of points on the Earth's surface.
H	Hazard	An occurrence that has the potential to cause deterioration, damage, harm, or loss.
I	Ill-conditioned problem	A problem where a small change in the inputs variables leads to large variability in the output (dependent variable).
	Ill-posed problem	The solution of a problem does not exist for all data, or is not unique, or does not depend continuously on the data.
	Independent variable	A variable that stands by itself and its value is not affected by other variables involved in a problem.
	Indirect consequence	Consequences caused by the loss of serviceability of the structure or component
	Inductive sensor	Device that uses the principle of electromagnetic induction to detect or measure objects.
	Inertia sensor	Sensor that measures the acceleration and angular velocity of an object along three mutually perpendicular axes.
	Information	Processed, organised and structured data gained through measurement(s), analytical, numerical, or empirical methods.
	Inspection	An examination of a structure or its components to gather information about their structural and environmental conditions.
K	Key-Performance Indicator	A quantifiable measure of performance for a specific objective.
	Kullback–Leibler divergence	Measure of how one probability distribution is different from a second, reference probability distribution.
L	Law of total probability	Fundamental rule relating marginal probabilities to conditional probabilities.

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	Life-cycle (action)	Long-term action that extends over the entire lifespan of the structure, from the construction until the decommissioning or dismantling.
	Lifetime or life-cycle cost	Sum of all recurring and one-time (non-recurring) costs of the structure over the lifetime.
	Likelihood	Concept expressing qualitatively or quantitatively the chance that an event may occur in a specific period.
	Likelihood function	Joint probability of the observed data as a function of the parameters of the chosen statistical model, which assigns a probabilistic prediction to the observed data.
	Limit state	State beyond which the structure no longer satisfies the relevant performance criteria.
	Limit state function	Function for estimating the probability of achieving a limit state.
	Linear and time-invariant system	A system whose output for a linear combination of inputs forms a linear combination of the individual responses to those inputs and additionally the output is not a direct function of time, i.e., does not depend on the precise time of application of an input.
	Load carrying capacity	The ability of a structure or its component to withstand actions without mechanical failure.
	Load test	Programmed and controlled loading of the structure together with the measurement of the structure's reaction to the applied load.
	Long-term monitoring	Long-term observation of the response of a structure to loads and the environment by means of measuring equipment.
M	Machine learning	Computer algorithms that can improve automatically, based on experience and use of data (training) to predict (new) output values.
	Maintenance	A set of activities performed during the service life of the structure, intended to either prevent or correct the effects of minor deterioration, of the structure or its components to keep their future serviceability at the level anticipated by the designer.
	Mahalanobis distance	Measure of the distance between a point and a distribution, as a multi-dimensional generalisation of the idea of measuring how many standard deviations away the point is from the mean of the distribution.
	Maslow's hierarchy of needs	Theory of motivation which states that five categories of human needs dictate an individual's behaviour, mainly physiological needs, safety needs, love, and belonging needs, esteem needs, and self-actualisation needs.
	Markov model	Stochastic method to model randomly changing systems where it is assumed that future states do not depend on past states.
	Maximum Likelihood Method	Method of estimating the parameters of an assumed probability distribution, given some observed data, by maximizing a likelihood function.
	Measurement	Process to determine a value (if quantitative) or status (if qualitative) of a parameter.
	Measurand	Refers to the physical / chemical quantity that is subject to measurement (e.g., mass, power, or temperature).
	Mechanical action	Pressure, force, or imposed displacement, which can be either static or dynamic, leading to physical stress on a mechanical system or component.
	MEMS sensor	Chip-based sensors, known as a Micro Electro-Mechanical System, made up of a central unit and other components with a size in the order of micrometres.
	Metamodel	Model of a model, also knows as a surrogate model.
	Method of Moments	Method for the estimation of the parameters of a population.
	Migration	Microscopic motion of material caused by an external force.
	Modal damping	Ratio of the modal damping coefficient to the modal critical damping. The modal damping coefficient is a measure of how oscillations in a system decay after a disturbance. The critical damping is the value of the damping coefficient beyond which the system returns to its equilibrium position without oscillating after a disturbance
	Modal parameters	Natural frequency, damping ratio, and modal shape of each of the vibration modes of a system. They reflect the dynamic characteristics of vibrating systems.

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	Model-based method	Methods of addressing problems based on the use of a numerical or analytical model.
	Model calibration	Process of adjustment of the model parameters to obtain a model representation of the processes of interest that satisfies pre-agreed criteria or simulates the experimental evidence.
	Model-driven approach	Data interpretation approach performed comparing the responses of a structure with those predicted by a model of the structure.
	Model uncertainty	Basic variable related to the accuracy of a physical or statistical model.
	Model updating	Process of adjusting the parameters of a model so that its predictions are in agreement with measurements obtained by testing or monitoring.
	Monitoring	The process of managing (collect, process, transmit, and share) the information related to the structural performance, using technological devices.
	Monitoring method	Means by which the information about the condition/performance of an asset is acquired supported by a sensorial system.
	Monitoring plan/ policy	Instructions for systematic data acquisition that is specific to the structure being monitored. A procedure, or protocol for systematic data acquisition and specific to the structure being monitored.
	Multiscale modelling	Refers to a type of modelling in which multiple models at different scales are used simultaneously to describe a system.
	Multivariate statistics	Subdivision of statistics encompassing the simultaneous observation and analysis of more than one outcome variable, concerning the understanding of how they relate to each other.
N	Nernst-Planck law	Law that describes the conservation of mass equation used to describe the motion of a charged chemical species in a fluid medium.
	Non-destructive testing	Development and application of technical methods to examine materials or components in ways that do not impair their future usefulness and serviceability, in order to detect, locate, measure and evaluate flaws, to assess integrity, properties and composition, ant to measure geometrical characteristics.
	Nonlinear system	System in which the change of the output parameter(s) is not proportional to the change of the input parameter(s).
	Non-stationary system	System whose time-series response features a changing distribution and therefore statistical moments with time. It can include elements such as trends, cycles, and random walks.
	Non-parametric identification method	Identification algorithms that do not include the consideration of a parametrised system model.
O	Operational Conditions	Set of conditions for operating a particular system or process.
	Operational modal analysis	Identification algorithms for identifying the modal properties of a structure based on its response to ambient vibrations in operating conditions, i.e., no initial excitation or known artificial excitation is applied.
	Optical method	Physical-chemical methods of analysis based on the optical range of the scale of electromagnetic waves.
P	Palmgren-Miner rule	Allows assessing the structural fatigue damage in a component, commonly described by the S-N curve.
	Parametric identification method	Identification algorithms that include the consideration of a parametrised system model.
	Passive state/passivity	The state in which, by virtue of a protective oxide film, steel does not spontaneously corrode.
	Performance	The behaviour of a structure or a structural element as a consequence of actions to which it is subjected or which it generates, usually using a quantitative parameter (e.g., reliability index, ratio between resistance capacity and action effect).

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	Performance Goal/ Requirement	A condition for design, or an actual, potential, or intended option for intervention, aiming at meeting a specified performance criterion during the service life with appropriate reliability and in a sustainable way.
	Performance Indicator	A measurable/testable parameter (i.e., characteristic of materials and structures) that quantitatively describes a performance aspect.
	Periodic monitoring	Systematically performed short-term data acquisition, including the processing of information, about the structure, or its components.
	Periodic monitoring (triggered)	Collection of measurements employing a programmed criterium, usually a predefined threshold related to a parameter that is being measured which triggers the data recording.
	Permanent monitoring	Long-term data acquisition, including the processing of information, made continuously and autonomously, capable of providing real-time information about the structure, or its components, based on a permanently mounted system.
	Permeation	Penetration of a permeate (liquid, gas, or vapor) through a solid, which is directly related to the concentration gradient of the permeate, a material's intrinsic permeability, and the materials' mass diffusivity.
	Physics-based model	Model which uses physics-based equations to describe structural response or underlying processes (e.g., deterioration).
	Physical parameter	Measurable physical characteristic of a structure/system.
	Piezoelectric sensor	Device that uses the piezoelectric effect to measure changes in pressure, acceleration, temperature, strain, or force by converting them to an electrical charge.
	Posterior distribution function	Distribution that summarises the knowledge about uncertain quantities in Bayesian analysis, which is a combination of the prior distribution and the likelihood function.
	Pre-processing	Task relates to doing preliminary processing of something, such as data collected by some sensing system.
	Prior distribution function	Distribution of possible values for an unknown population characteristic that is formulated before one obtains any current data observations about the phenomenon of interest.
	Probabilistic model	Statistical technique that can consider the impact of random events or actions in predicting the potential occurrence of future outcomes.
	Probability density function	Function whose value at any given sample in the sample space can be interpreted as providing a relative likelihood that the value of the random variable would be close to that sample.
	Probability of Detection	Chance of detecting a defect and generally expressed by a curve that relates the likelihood of detection to the defect size.
	Probability of False indication	Chance of the model incorrectly predicting an event occurrence.
	Probability of Missing indication	Chance of the model incorrectly predicting no event occurrence.
	Prognostic assessment	Assessment or prediction of future performance (forecasting).
	Proof loading	Test to demonstrate the fitness of a load-bearing structure.
	Pyranometer	Type of actinometer used for measuring solar irradiance on a planar surface.
Q	Quasi-static load	External or internal loading, where inertial effects, i.e., acceleration of vibrations of the structure or its elements, are negligible or can be transformed to equivalent static counterparts.
	Quasi-static measurement	Measurements at very low, almost static rates, which do not record inertial effects, even if some quantities are allowed to vary slowly with time.
R	Radar technique	Detection system that uses radio waves to determine the distance (range), angle, or velocity of objects.
	Radiological method	Imaging technique using X-rays, gamma rays, or similar ionizing radiation.
	Random field	Random function, which takes a random value over an arbitrary domain, sometimes thought of as a synonym for a stochastic process.

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	Reference condition	Standard or benchmark against which current condition is compared.
	Reference period	Period used as a basis for assessing the statistical parameters of time-dependent variables and the target reliability.
	Redundancy	Presence of more than one load path that enables load redistribution within the structure (or assembly) in case of failure of one or more individual components.
	Regression methods	Mathematical models able to establish relationships between independent variable(s) and dependent variable(s).
	Reliability	Ability of a structure or a structural member to perform its intended function satisfactorily for its intended life under specified environmental and operating conditions. Reliability is usually expressed in terms of the probability of failure.
	Reliability index	Measure of the probability of failure.
	Remote sensing	Process of collecting data about the structural condition an asset at a distance (e.g., from satellite).
	Repair	Intervention taken to reinstate to an acceptable level the current and future performance of a structure or its components which are either defective or deteriorated to some extent so that their performance level is below that anticipated by the designer; generally, without restriction upon the materials or methods employed.
	Residual service life	Required or predicted remaining period in which the demanded structural performance is still satisfied; used in the redesign process for existing structures.
	Resilience	Ability of a structural system to resist the effects of extreme events, including the ability to recover original serviceability.
	Response function	Describes the cause (input) and effect (output) relationships for a system.
	Response surface method	Mathematical and statistical method for modelling and analysing a process in which the response of interest is affected by various variables.
	Rheological effect	Deformation exhibiting both viscous and elastic characteristics in response to applied forces or stresses.
	Risk	The expected adverse consequences associated with an event, an activity or a decision alternative.
	Robustness	The ability of a structure, subject to accidental or exceptional loading, to sustain local damage to some structural components without experiencing a disproportionate degree of overall distress or collapse.
S	Safety	Ability of a structure or structural element to ensure that no harm would come to the users and the people in the vicinity of the structure under any (combination of) expected actions.
	Sampling rate	The number of samples per second (or per another unit) taken from a continuous signal to make a discrete or digital signal.
	Sclerometric method	Non-destructive compression strength measurement of concrete.
	Semi-destructive testing	Under the prerequisite that the bearing capacity of the structure to be tested is subjected to no affection.
	Semi-supervised algorithm	A technique that combines a small amount of labelled data with a large amount of unlabelled data during the training of a Machine Learning algorithm.
	Sensor	Device that measures a physical quantity and converts it into a signal that can be read by an instrument (e.g., electric strain-gauge, fibre optic sensor).
	Serviceability	Ability of a structure or structural element to perform adequately for normal use under all (combinations of) actions expected during the service life.
	Serviceability limit	Conditions under which an asset is still considered fully operational.
	Short-term monitoring	Observation and data collection, by means of temporarily installed sensors, of relevant to a phenomenon an event-to-event change in asset behaviour that is unrelated to the long-term trend performed.
	Spatial variability	Variability caused by the construction process, exposure conditions, geometry of reinforcement, loads, among other spatial-related factors.

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	Spectral transformation	Transformations that relate to the spectrum (frequency domain representation) of a signal.
	Stabilisation diagram	A model order vs. a frequency plot, which aims to identify the stable parameters that represent the physical modes.
	Stakeholder	Person or organisation that has legitimate participation in a project, who is affected by an activity or that has any kind of reasonable interest in the structure, such as owners, users, local residents, contractors, general planers, and specialists.
	State	Condition of an asset at a specific point in time.
	Static test	Test performed through the application of controlled static loading on the structure, together with the measurement, of the structure's mechanical response for the identification of static characteristics with interest for assessment.
	Statistical-based model	Mathematical model that embodies a set of statistical assumptions concerning the generation of sample data, which represents, often in considerably idealised form, the data-generating process.
	Statistical moment	Parameter that summarises a distribution of data, e.g., mean for centre location, variance or standard deviation for dispersion, covariance, or correlation for dependence.
	Stochastic-based model	Tool for estimating probability distributions of potential outcomes by allowing for random variation in one or more inputs over time.
	Strain gauge	Device used to measure strains, usually a jargon synonym for “electrical resistance strain gauge”.
	Strengthening	Technical intervention on a structure aimed to improve its performance.
	Structural defect	Local changes to the material and/or geometric properties of a structural system, including changes to the boundary conditions and system connectivity, which adversely affect the system's performance.
	Structural Health Monitoring	Process of identifying the presence, the characteristics (location and type), and the severity of deterioration in a system based on information extracted from the monitoring system.
	Structural / system identification	Process of building mathematical models of mechanical/dynamic systems and of estimating physical parameters from observed data.
	Structural integrity	The ability of structural components to act together as a competent single entity.
	Supervised learning method	Artificial intelligence technique that attempts to discover the relationship between input attributes and a target attribute (output) using labelled datasets.
	System (model)	A representation (usually analytical or numerical), which can be used for understanding and simulating the behaviour of systems.
T	Thermal dilation coefficient	Material change, either in the form of shape, area, volume, or density, due to an increase in temperature.
	Thermographic method	Measurement technique to obtain an image of the heat distribution over the surface of an object.
	Threshold	Boundary defined to compare different states.
	Time-domain method	Technique for analysis of physical signals, mathematical functions, or time series of data, that operates in the time domain.
	Time-frequency method	Technique used for characterizing and manipulating signals whose statistics vary in time, in which a representation of the frequency domain reflects the behaviour of a temporally localised version of the signal.
	Time invariant method	Method that assumes that the system does not change in time.
	Transducer	Device, which converts energy from one form to another.
	Transition Probability Matrix	Matrix of a Markov chain that gives the probabilities of transitioning from one state to another in a single time unit.

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	Transfer function	Mathematical function that theoretically models the system's output for each possible input, related to a system, sub-system, or component.
U	Ultimate limit state	State associated with collapse or with other similar forms of structural failure.
	Ultrasonic technique	Non-destructive inspection test method that utilises sound waves that propagate along the surface of the structure's elements to detect cracks and defects.
	Unsupervised method	Method that does not require any labelled sensor data for automatic discovery of activity patterns.
V	Vibrating wire sensor	Sensor that operates on the principle that a tensioned wire, when plucked, will vibrate at its resonant frequency.
	Vibration-based monitoring	Monitoring of structural response, based on vibration information, for structure performance assessment, defects identification as well as remaining useful life prognosis.
X	X-ray technology	Non-destructive inspection method based on the use of X-rays to detect variations of density in the material, which is a function of the amount of radiation that passes through.

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Supportive normative documents

List of normative documents that are consistently used in this Bulletin along with a brief description of their scope and field of application.

Year	Title	Scope/Application Field
2020	AASHTO LRFD Bridge Design Specifications.	Design, evaluation, and rehabilitation of bridges. The specifications employ the Load and Resistance Factor Design (LRFD) methodology, using factors developed from current statistical knowledge of loads and structural performance. https://trid.trb.org/view/1704698
2020	RiZ-ING: Richtzeichnungen für den Ingenieurbau	German guideline using drawings to provoke a unified quality of details in the design process of road bridges. https://www.bast.de/BASt_2017/DE/Publikationen/Regelwerke/Ingenieurbau/Entwurf/RIZ-ING.html
2017	AASHTO 358: Standard Method of Test for Surface Resistivity Indication of Concrete's Ability to Resist Chloride Ion Penetration	Test method that covers the determination of the electrical resistivity of water-saturated concrete to provide a rapid indication of its resistance to the penetration of chloride ions. This test method applies to types of concrete where correlations have been established between this test procedure and long-term chloride diffusion procedures. Examples are discussed. https://standards.globalspec.com/std/10159996/AASHTO%20T%20358
2017	BEM-ING: Regelungen und Richtlinien für die Berechnung und Bemessung von Ingenieurbauten	Regulations and guidelines for the calculation and dimensioning of civil engineering structures. https://www.bast.de/BASt_2017/DE/Publikationen/Regelwerke/Ingenieurbau/Entwurf/BEM-ING_node.html
2017	RILEM TC PSC: Performance-Based Specifications and Control of Concrete Durability	Overview of significant research from recent years concerning performance-based design and quality control for concrete durability and its implementation. In engineering practice, performance approaches are often still used in combination with prescriptive requirements. https://www.rilem.net/global/gene/link.php?news_link=2015142902_0428-rilem-tc-psc-230-star-final-document-for-web-protege.pdf&fg=1
2016	DIN EN 15528: Railway applications	Line categories for managing the interface between load limits of vehicles and infrastructure. https://www.en-standard.eu/din-en-15528-railway-applications-line-categories-for-managing-the-interface-between-load-limits-of-vehicles-and-infrastructure/
2017	RI-EBW-PRÜF: Richtlinie zur einheitlichen Erfassung, Bewertung, Aufzeichnung und Auswertung von Ergebnissen der Bauwerksprüfung nach DIN 1076	Guideline for a unified survey, assessment, record, and evaluation of the results of bridge inspections according to DIN 1076. https://www.bast.de/BASt_2017/DE/Publikationen/Regelwerke/Ingenieurbau/Erhaltung/RI-EBW-PRUEF-Erhaltung.pdf?__blob=publicationFile&v=4
2016	RILEM TR-PCD: Performance-Based Specifications and Control of Concrete Durability	Overview of significant research from recent years concerning performance-based design and quality control for concrete durability and its implementation. https://link.springer.com/book/10.1007/978-94-017-7309-6

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Year	Title	Scope/Application Field
2014	ISO 55000:2014: Asset management — Overview, principles, and terminology	Overview of asset management, its principles and terminology, and the expected benefits from adopting asset management. It can be applied to all types of assets and by all types and sizes of organisations. https://www.iso.org/standard/55088.html
2013	MC2010: Model Code for Concrete Structures	Basis for future codes for concrete structures. It also considers new developments for concrete structures, the structural material concrete, and new ideas concerning requirements to be formulated, so that structures achieve optimum behaviour according to new insights and ideas. https://www.wiley.com/en-al/fib+Model+Code+for+Concrete+Structures+2010-p-9783433030615
2010	VDI 6200: Standsicherheit von Bauwerken – Regelmäßige Überprüfung	Assessment criteria and practical instructions for the regular verification of structural safety and recommendations for the maintenance of buildings. https://www.vdi.de/richtlinien/details/vdi-6200-standsicherheit-von-bauwerken-regelmaessige-ueberpruefung
2008	JCGM 100:2008: Evaluation of measurement data	Establishes general rules for evaluating and expressing uncertainty in measurement that can be followed at various levels of accuracy and in many fields. https://www.bipm.org/documents/20126/2071204/JCGM_100_2008_E.pdf
2008	JCGM 200:2012: International vocabulary of metrology	Set of definitions and associated terms are given for a system of basic and general concepts used in metrology, together with concept diagrams to demonstrate their relations. Additional information is given in the form of examples and notes under many definitions. https://www.bipm.org/documents/20126/2071204/JCGM_200_2012.pdf/f0e1ad45-d337-bbeb-53a6-15fe649d0ff1
2004	EN 1992-1-1: General rules and rules for buildings	Establishes general rules for the safety, serviceability, and durability of buildings https://eurocodes.jrc.ec.europa.eu/EN-Eurocodes/eurocode-2-design-concrete-structures
2002	EN 1990:2002 Eurocode - Basis of Structural Design	Establishes principles and requirements for the safety, serviceability, and durability of structures, by describing the basis for their design and verification, and gives guidelines for related aspects of structural reliability. https://eurocodes.jrc.ec.europa.eu/showpage.php?id=130
2002	EN1991-2: Actions on structures – Part 2: traffic loads on bridges	Defines imposed loads (models and representative values) associated with road traffic, pedestrian actions, and rail traffic which include, when relevant, dynamic effects and centrifugal, braking, and acceleration actions and actions for accidental design situations. https://eurocodes.jrc.ec.europa.eu/EN-Eurocodes/eurocode-1-actions-structures
2002	Eurocode 1 – Actions on structures	Gives design guidance and actions for the structural design of buildings and civil engineering works including geotechnical aspects. https://eurocodes.jrc.ec.europa.eu/EN-Eurocodes/eurocode-1-actions-structures
2002	ISO 15148: Hygrothermal performance of building materials and products	Determination of water absorption coefficient by partial immersion. https://www.en-standard.eu/iso-15148-hygrothermal-performance-of-building-materials-and-products-determination-of-water-absorption-coefficient-by-partial-immersion/

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Year	Title	Scope/Application Field
2001	JCSS: Probabilistic Model Code	Model code for full probabilistic design covering the basis of design, load models, resistance models, and examples. https://www.jcss-lc.org/jcss-probabilistic-model-code/
2021	TR DIBt: Technische Regel Instandhaltung von Betonbauwerken	Guideline for the protection and repair of structural concrete elements. https://www.dibt.de/de/aktuelles/meldungen/nachricht-detail/meldung/technische-regel-instandhaltung-von-betonbauwerken-neu-ab-januar-2021
1999	DIN 1076: Ingenieurbauwerke im Zuge von Straßen und Wegen – Überwachung und Prüfung	Engineering structures in connection with roads; inspection and test. https://webstore.ansi.org/Standards/DIN/DIN10761999
1985	DIN 1072:1985: Road and footbridges; design loads	Statements about load assumptions for traffic loads on bridge structures. https://webstore.ansi.org/standards/din/din10721985

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1. Introduction

1.1 Scope

This *fib* Bulletin, as part of the Task Group 3.3 work, focuses on the development of a set of practical state-of-the-art guidelines for the testing and Structural Health Monitoring (SHM), assessment, and decision-making procedures required for the acquisition of information for the through-life management of existing concrete structures, mainly focussing on:

- Testing and monitoring methods to acquire data able to inform about the condition of existing structures;
- Advanced condition and performance assessment for the evaluation of existing structures in their current condition;
- Modelling of performance prediction, by using advanced methods considering time-dependent changes, to evaluate future need scenarios;
- Practical decision-making procedures to perform a complete and coherent assessment of the existing structure;
- Portfolio of civil engineering as a demonstrator of the feasibility of the outlined tools and methods.

More specifically, the topics treated in this *fib* Bulletin include:

Chapter 2 presents an integrated classification and characterisation of modern testing and monitoring methods as a part of general strategy of condition survey of concrete structures. The scope of the chapter includes the description of the most common survey objectives and strategies, utilizing various as well as diagnostic testing and monitoring methods for a meaningful condition assessment of a structure or its components.

In Chapter 3, methods for damage identification for condition monitoring and assessment are introduced, mainly at the global level by using data collected during vibration-based tests and permanent monitoring systems. The extraction of condition parameters and indicators from measured data considers methods to account for the effect of environmental and operational actions.

In Chapter 4 degradation and prediction models are elaborated by detailing the deterioration mechanisms and levels to be predicted in the performance assessment. Models and methods for the prognosis of the behaviour of the structure, from deterministic-based to stochastic-based, are introduced. Furthermore, methods for continuously updating the accuracy of the characterisation of the future deterioration, based on the information from inspections/surveys and/or monitoring, are systematically introduced. Ultimately, Key Performance Indicators towards lifetime assessment of structures are outlined to support decision-making.

In Chapter 5, a survey and critical analysis of available evaluation/decision-making procedures for the assessment and through-life management of existing concrete structures is presented. Practical decision-making procedures to perform a complete and coherent assessment of existing structures are introduced. Considering the content of Chapter 2, 3, and 4, and supported by flowcharts, the decisions entailed by condition and performance assessment, that should be part of the assessment process, are addressed.

This chapter was written by Andrade, Apostolidi, Chatzi, Duprat, Keßler, Lehky, Limongelli, Novák, Sousa (main author), Strauss.

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Finally, Chapter 6 summarises a set of case studies to show how the tools and methods presented in Chapter 2, 3, 4, and 5 may be utilised in a holistic approach and from the practicing engineering point of view. More precisely, in each case study presented, a specific aspect is covered and related to the themes throughout covered in this *fib* Bulletin.

From a holistic perspective, Figure 1-1 offers how the different chapters have been framed within the scope of the *fib* Bulletin, by detailing their links and hierarchical relationships.

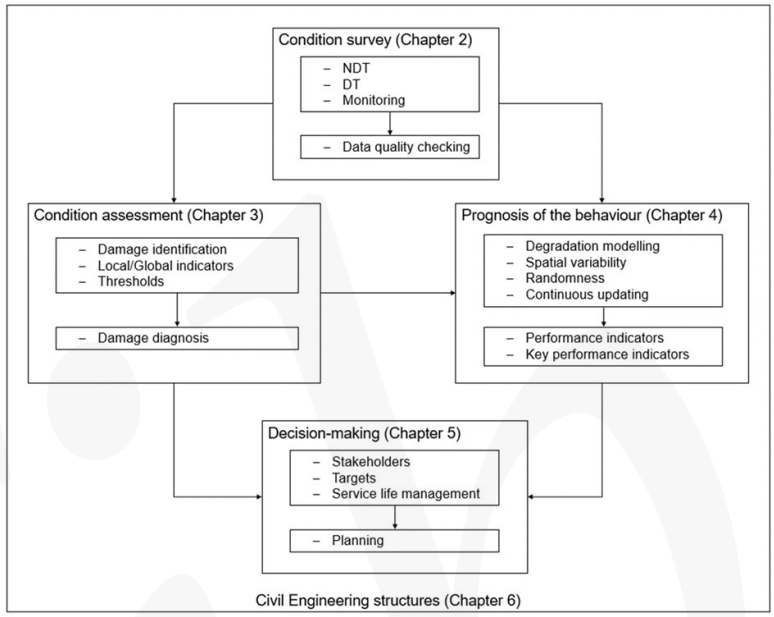


Fig. 1-1 Testing and Structural Health Monitoring (SHM) for life management of existing concrete structures: holistic perspective of the T.G. 3.3 *fib* Bulletin.

1.2 Detection of deterioration phenomena

In light of the above, Table 1-1 offers an overview of the deterioration processes that affect Reinforced Concrete Structures and from that appropriate inspection and monitoring techniques are described in this *fib* Bulletin.

More precisely, six main types of deterioration processes, both local: (i) corrosion, (ii) fatigue, (iii) freeze-thaw cycles, (iv) alkali-silica reactions, and global: (v) seismic damage, and (vi) scour global are covered. How these link to the previous version of Model Code 2010 and the future Model Code 2020 is also highlighted, in order to enhance the relevance of the presented *fib* Bulletin to the current state-of-the-art. Moreover, for each deterioration process a set of relevant case studies contained in Chapter 6 is also highlighted in Table 1-1. This aims to support the reader in better understanding how all chapters of the document are linked towards the ultimate goal – i.e., the acquisition of knowledge about the structural performance. It is worth mentioning that the list of case studies (presented in Table 1-1) is not exhaustive. Other case studies are included in Chapter 6 focussing on other aspects considered relevant for the ongoing investigation of structural performance.

Degradation, i.e., deterioration, of RC structures can be visually evaluated based on the crack patterns and/or appearance of its surface – external symptoms. Sometimes, similar patterns are observed for different types of attacks, which makes their identification a matter of specialists, requiring suitable analyses to address them and identify their origin.

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Table 1-1 Contextualisation of the covered deterioration processes.

Mechanism	**MC 2010**	**MC 2020**	***fib* Bulletin TG3.3 (section)**	**Case studies**
Carbonation induced corrosion of reinforcement	√	√	√(1.2.1)	6.2, 6.3, 6.4
Chloride induced corrosion of reinforcement	√	√	√(1.2.1)	6.2, 6.3, 6.4
Corrosion induced cracking and spalling of concrete	√	√	√(1.2.1)	6.2, 6.3, 6.4
Fatigue induced cracking or internal damage	√	√	√ (1.2.2)	6.1, 6.6
Scaling, surface spalling, or internal damage (disintegration) due to freezing-thawing with or without de-icing-agents	√	√	√ (1.2.3)	-
Alkali Silica Reaction (ASR) induced cracking and deformation (expansion)	√	√	√ (1.2.4)	6.14, 6.15
Seismic damage	√	√	√ (1.2.5)	6.12
Scour	√	√	√(1.2.6)	6.8

Hence, the first step for the assessment of degradation of concrete structures consists of the identification of the type of process(es) responsible for the attack, which can have a chemical, physical, biological, or mechanical origin or even a simultaneous occurrence of some of the previous. Table 1-2 summarises the main types of attack – i.e, degradation mechanisms – depicted by origin and symptoms, which in some cases may act together. It also gives the main specific types of attack in function of their origin and the main symptoms.

Beyond a certain level of development, the degradation mechanisms can be identified by qualified and experienced personnel during a visual/routine inspection, although its efficiency is limited, as once deterioration becomes visible, irreversible damage has taken place[1]. Conversely, early detection/identification approaches are much more appealing, although they bring some challenges to be carried out by visual inspection, as various degradation processes share similar symptoms/defects. In most cases, as mentioned, they act in a combined way. For constructions not visually affected yet, it is still possible to suspect potential future/ongoing distress concerning environmental conditions, employed concrete mixes and design, usage, and maintenance policy. Hence, attention must be paid to additional information that may confirm this suspicion to take early measures. Doing some surveys/tests when the structure is reaching its 10 years is seen as a suitable measure[2].

An accompanying effect, in the case of reinforcement corrosion, leaching, or biological attack, is the decrease of potential of Hydrogen (pH) of the concrete pore solution, which becomes even more pronounced when cracks exist since they facilitate the ingress of water. Therefore, corrosion is prone to occur after the onset of most of the chemical/biological degradation mechanisms and remains a major concern. Its detection and/or intensity level estimation are often necessary even if other primary degradation mechanisms seem prominent.

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The second step is to identify properly the possible degradation process – i.e., the model. For this, research has brought significant improvements in terms of knowledge, understanding, and modelling of degradation. These have been developed in the framework of recommendations, design codes, and/or standards to serve the maintenance strategies of existing concrete structures[1],[4], including a wide range of models for the same degradation process, affordable for the asset manager/engineer.

Table 1-2 Degradation mechanisms of concrete structures and associated classes of defects[3].

Degradation mechanisms		**Class of defects**					
		Deformation	**Destruction**	**Loss of material**	**Discontinuity**	**Contamination**	**Displacement**
Physical	Accumulation of inorganic dirtiness		□	□		■	■
	Cyclic freeze–thaw action		■	■	□	□	
	Erosion		□	■			
	Crystallisation		■	□	□		
	Extreme temperatures	□	□	□	□	□	□
	Creep	□					
	Relaxation	□			□		
	Shrinkage	□	□	□	■		
	Overloading	■	□	■	■		■
	Fatigue		■		□		
	Geotechnical condition changes	■		□	■		
Chemical	Carbonation		■				
	Corrosion	□	■	■	□	□	
	Aggressive compounds action		■	□		□	
	Chemical dissolving/ leaching		■	□		■	
	Reactions between material components	□	■	■	□		
Biological	Accumulation of organic dirtiness		□	□		■	□
	Activity of microbes		□	□		■	
	Activity of plants	□	□	□	□	■	□
	Activity of animals		□			□	

Note: ■-basic degradation mechanism, □-additional degradation mechanism.

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The third step is the collection of data[2] needed to calibrate the degradation model, which encompasses available data from the design phase, construction phase, and inspections/repair interventions. Ideally, this data serves to improve input/output model parameters. Depending on the model selected, (i) data usually available or; (ii) obtained with a limited investigation (e.g., concrete compressive strength, concrete porosity, atmospheric Relative Humidity (RH), concrete water content, reinforcing steel corrosion potential, carbonation depth), or; (iii) data from comprehensive investigations (e.g., chemical kinetics constants, number of crystals, interaction isotherms, corrosion parameters, creep coefficient) is appropriate. As these investigations are generally time-consuming and expensive, the asset manager/ engineer is encouraged, if knowledge is available, to capitalise data previously acquired for similar material, environmental and/or structural conditions. Nevertheless, this must not impair the quality of the model prediction, i.e., all data, expected to improve the knowledge of input/ output model parameters, must generally be used as guidance.

1.2.1 Corrosion

Corrosion is an electrochemical process inducing diffuse, or more local, dissolving metals. Under healthy conditions, i.e., in alkaline conditions (pH>10.5), the reinforcing steel embedded in concrete is known to develop a passivating film, composed of insoluble ferrous hydroxide formed by the corrosion process and forming a thin, tenacious, passive layer on the surface of the steel that protects the substrate until it is dissolved. Nevertheless, when the conditions become neutral or acid, the ferrous and ferric oxides that form remain soluble and the passivating film becomes unstable. The cause for this is the penetration of the so-called carbonation front (i.e., with a neutral pH - up to the steel bars). The other cause of the dissolution of the passivating film is the concentration of chlorides beyond a certain level (chloride threshold level) at the steel surface, or due to carbonation and/or chloride content, when the steel starts to corrode, oxides, with larger volume concerning the parent metal, develop. These oxides diffuse through the concrete pore network and may produce a radial pressure, which results in cracking of the concrete cover. This two-step process, comprising an initiation period and a propagation period, is depicted in Figure 1-2.

Reinforcement corrosion can be identified via direct (e.g., corrosion rate) or indirect indicators (e.g., moisture, electrical resistivity, or corrosion potential).

Direct indicators describe quantities that are straightforwardly linked to the evolution of corrosion, such as the corrosion rate, which is directly proportional to the degree of saturation and is further related to the resistivity and the porosity of the concrete. In this sense, the porosity and the degree of saturation are the main indicators of the progression of corrosion. These can be directly incorporated into degradation models, to determine - for example - the reduction in the diameter of the bars and, subsequently, the loss of concrete/steel bond. The latter is an important parameter for safety evaluation as it affects the cracking of the concrete cover and the bearing capacity of the concrete section.

Indirect indicators provide a qualitative assessment, typically delivered in terms of the probability of corrosion. Such indicators describe phenomena that affect the initiation, or development of corrosion, such as the increase of chloride concentration and the neutralisation of the alkalinity at the rebar surface (i.e., the pH value). The rate of ingress of chlorides or carbon dioxide increases with the concrete porosity and with the reduction of the binding ability towards the penetrating substance.

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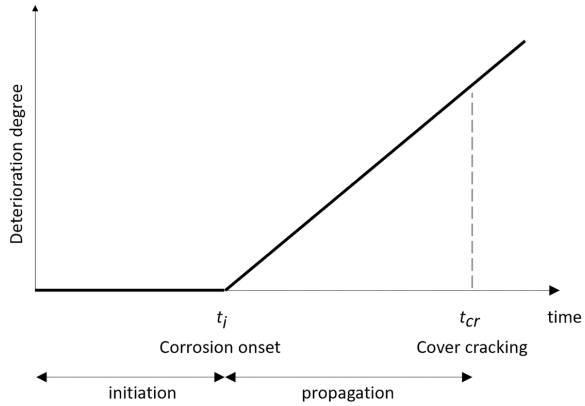


Fig. 1-2 Corrosion process composed of an initiation period and a propagation period.

It is worth noting that moisture is a necessary condition for corrosion to remain active, as in absence of water (i.e., below a certain level of humidity) corrosion does not progress. More precisely, in the latter, corrosion evolves so slowly that it can be assumed as not progressing in practical terms. The diffusion of chloride ions does, however, benefit from the presence of moisture. For carbonation to progress, the moisture level must lie below the threshold corresponding to active corrosion. In the state of carbonation, the saturation degree serves as a further indicator that is relevant for corrosion, which are summarised in Table 1-3. Examples of the effect of corrosion of both internal and external prestressing tendons in bridge decks are offered in Chapter 6[*,**].

Table 1-3 Corrosion-related indicators.

Process	**Indicator**
Corrosion initiation by carbonation	Porosity Proportion of hydrated phases Degree of water saturation pH value at rebar surface
Corrosion initiation by chlorides	Porosity Proportion of hydrated phases Degree of water saturation Chloride concentration
Corrosion propagation	Porosity Degree of water saturation (concrete resistivity) Corrosion rate Steel cross-section loss (local or homogeneous) Corrosion potential Polarisation resistance Concrete cover cracking level

* Case Studies 6.3 and 6.4. In the first case, where a failure of pre-stressing tendons was induced due to corrosion, a continuous SHM system allowed to both investigate the damage, as well as verify the strengthening works. In the second case, monitoring, via Micro-ElectroMechanical Systems (MEMS) accelerometers served for checking the condition of external tendons, in real-time, after the failure of an external prestressing tendon due to inappropriate grout composition.

** In Case study 6.2 an example of corrosion monitoring is reported.

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1.2.2 Fatigue

The phenomenon of fatigue consists of a decrease in the strength of materials at the increase of sustained load cycles. Structures subjected to fatigue need to be assessed in terms of their remaining fatigue life, to alleviate impending structural failure. Structural safety and serviceability under instantaneous actions are indirectly affected by fatigue damage, due to the induced stiffness reduction. Fatigue may lead to fracture for stress (or strain) levels lower than that those corresponding to the strength of the material under static loading. Structural materials subjected to fatigue loading are generally experimentally tested, to derive the so-called S-N diagram, which describes the relationship between the fatigue strength *S*, and the fatigue life, *N* (Figure 1-3). The fatigue life is measured in terms of the experimentally determined number of the loading cycles required for fatigue failure. The fatigue strength is evaluated in terms of the acting stress range either in compression or in tension. The acting stress range can be measured in terms of the monitored strain range, by attaching/embedding strain gauges to the material or components. The maximum stress level is computed as the sum of the stresses due to permanent and variable actions.

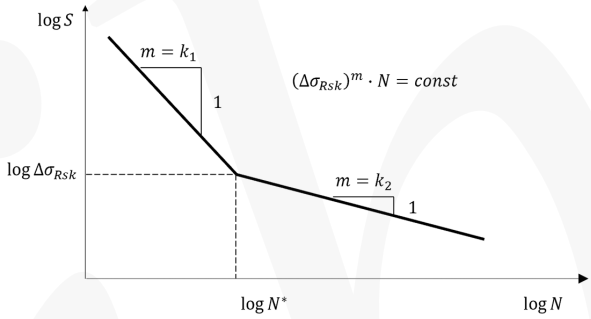


Fig. 1-3 Illustration of a typical S-N diagram (based on MC2010).

Concrete fatigue failure may occur under compression, tension, and shear. In the first two cases, the remaining fatigue life is assessed by estimating the stress acting in the concrete. This is not the case for shear failure under fatigue; the typical shear failure is punching shear failure of concrete slabs. As Figure 1-4 illustrates, the cracks propagate gradually at the tension face, ultimately leading to punching shear failure[5].



Fig. 1-4 Propagation of cracks in concrete slab due to fatigue[5].

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A commonly used indicator of fatigue damage, for an arbitrary magnitude of stress ranges n, is the accumulated fatigue damage, D (MC2010), which is computed under the assumption of the Palmgren-Miner summation rule, as follows:

$$D = \sum_{i=1}^{j} \frac{n_{Ei}}{N_{Ri}} \tag{1-1}$$

Where, n_{Ei} denotes the number of acting stress cycles associated with the stress range for steel and the stress level for concrete and, N_{Ri} denotes the number of resisting stress cycles at a given stress range for steel and stress level for concrete.

Depending on the reparability requirements (or damage limit state), Stage II or III in Figure 1-4 can be specified as a threshold condition. Finally, shear failure may also occur due to fatigue fracture of shear reinforcement. In this case, the remaining fatigue life can be estimated through the range of stress acting in the shear reinforcement.*

1.2.3 Freeze-thaw cycles

Freeze-thaw cycles cause frost damage – i.e, the combined effect of environmental actions and mechanical loading – in concrete that, due to cracking, scaling and pop-out, may result in loss of the concrete surface layer (Figure 1-5).

This is due to the fact that when concrete is subjected to alternate freezing and thawing, this results in stresses arising from the expansion of frozen water in a restrained space (i.e., a 9% increase in volume from liquid to solid phase). Hence, if these induced stresses become higher than the strength of either the cement paste or the aggregate particles, then cracking can occur and part of the volume increase will become permanent i.e., the damage will be accumulated, and it will increase with the number of freeze-thaw cycles.

Due to frost damage, both mechanical and durability-related properties of concrete deteriorate, a reduction is observed in both strength and stiffness in compression and tension. The reduction in tensile strength is higher than that in compression strength, affecting the relationship that binds the two, and which is usually evaluated for the case of intact concrete. In a recent study, a numerical model for the simulation of concrete with frost damage has been developed and is reported in Figure 1-6[6]. The model provides a relationship between stiffness and strength of concrete with frost damage based on measurements of the stiffness (i.e., the elastic modulus) of the surface layer of concrete. This is accomplished via ultrasonic wave velocity measurements, and coupled use of a numerical model, which enables the estimation of the degraded strength. Frost damage appears in the concrete surface as cracks, scaling, and pop-out, leading to loss of surface concrete area, which can also serve as an indicator of the extent of frost damage.

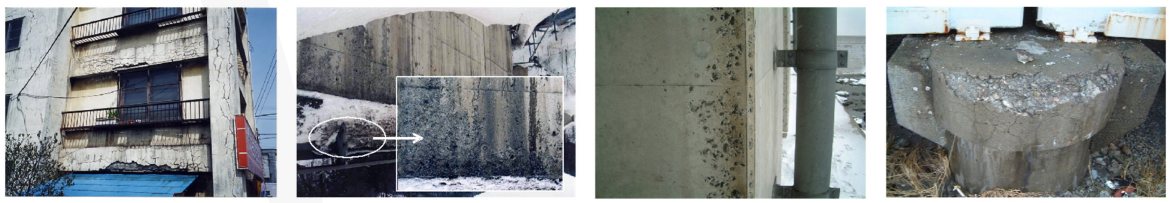


Fig. 1-5 Instances of damage relating to frost.

* Case studies 6.1 and 6.6 describe the monitoring system composed of strain gauges, installed on bridges, to monitor fatigue damage.

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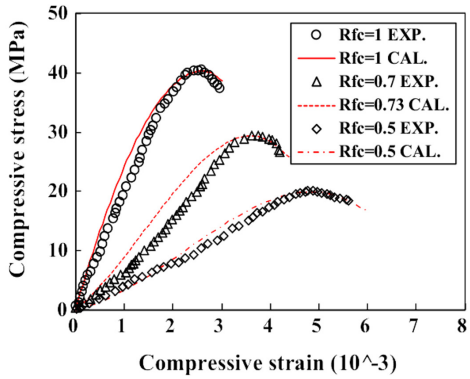


Fig. 1-6 Experimental (EXP) and calculated (CAL) stress-strain relationships of concrete with frost damage. Figure reused from[6].

1.2.4 Alkali-silica reactions

Concrete expansion may also occur as a result from the combination of two incompatible materials, mainly when the concrete has both a relatively high content of alkali in the cement and the aggregate is of a reactive type. Consequently, a reaction between alkalis released by the hydrated cement and aggregate containing a reactive form of silica occurs and is known as Alkali-Silica Reactions (ASR)[7]. Found in large bridges, viaducts, and dams, these can be subjected to swelling processes, due to internal chemical reactions, such as Alkali-Aggregate-Reactions (AAR). The development of those swelling reactions, which are determined by the concrete mixture (i.e., aggregates and cement type), moisture, and influenced by the thermal, hygrometric, and stress fields, can produce cracking and may contribute to the premature deterioration of concrete structures. The detection of swelling processes in concrete can be performed by identifying evidence of swelling reactions. This entails evidence that can be captured through visual inspections (i.e., through the microscope on concrete samples in the laboratory), for example, typical crack patterns such as surface cracks with hexagonal patterns ("craquelet") which are indicators of this type of damage.

Monitoring of the structural response in terms of stress field and/or displacements, coupled with the use of physics-based models, enables estimation of quantitative indicators of swelling. Physics-based models must simulate the temporal evolution of both the actions, due for example to seasonal and/or daily temperature variations or hydrostatic pressure variations in the case of dams, considering the delayed phenomena in concrete and the progressive cracking development.

To this aim, physics-based models must integrate chemical-mechanical interaction models to simulate factors that regulate the development of the swelling process over time such as concrete mixture, temperature and moisture fields, state of stress. Further to this, viscoelastic models accounting for maturation must be used to account for the concrete delayed behaviour and damage formulation to simulate the occurrence and propagation of cracking. Incremental techniques should be used for the time discretisation of actions due for example to seasonal and/or daily temperature variations and, in the case of dams , also hydrostatic pressure variations.

Using such models, indicators of swelling can be estimated in terms of:

- The evolution of the free strains measured in a free space (in a "cone") to enable the determination of the irreversible component.
- The evolution of displacements at some key locations. In this case, statistical models should be used to identify separately the effect of swelling from displacements due to other sources.
- The evolution with damage of the modal parameters extracted from the dynamic response of the structure.

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1.2.5 Seismic damage

Earthquake damage is linked to nonlinear behaviour, induced by yielding of the steel members, or reinforcement bars, and/or cracking of concrete caused by large deformations of structural elements, notably resulting in significant inter-storey drift for multi-storey buildings or large deck displacements for bridges. The demand in terms of ductility is thereby considered the main indicator of seismic damage. Depending on the data available for the estimation of damage, different indicators can be used mainly:

- Empirical Damage Indicators (DIs) are based on statistics of observed structural damage following earthquakes, which can provide valuable qualitative information on the overall seismic condition of the structural system. However, empirical evaluation cannot predict the strength reserve and the degree of structural damage[8].

- Strength-based DIs are indirect indicators that estimate the structural strength based on geometrical characteristics of the structure accounting for the possible deterioration. Useful properties for such calibration include the position of corroded areas; degraded concrete depth; cover dimension; mechanical properties of concrete. Visual inspection still plays an important role in the practice of structural defects detection and condition assessment using such indicators[9]. If field observations of damaged structures due to seismic loads are not available, the damage indicators need to be numerically estimated by nonlinear dynamic analysis using physics-based models.

- Response-based DIs can be computed when the structural response during the seismic event is measured by a monitoring system. In this case, the damage occurred during the event can be described in terms of indicators extracted from the measurements. Thus, the type of indicators is often tied to modifications of the vibrational characteristics of the structure, as well as to the wave propagation characteristics. Typical response-based seismic DIs can be classified in terms of the parameter used for their evaluation, more precisely:

  · DIs based on deformation: The ratio of the maximum deformation achieved during the earthquake and the yield deformation is defined as ductility ratio and is a main indicator of the seismic damage. Often the interstorey drift ratio is used for interstorey buildings as the deformation parameter. The maximum permanent drift, known as the residual drift, is closely related to the plastic deformation experienced by the structural system. However, the permanent drift might be difficult to identify from recorded responses due to biases introduced by signal processing[10]. Other indicators that can be retrieved from force-displacement diagrams recovered from recorded responses provide information on the damage related to stiffness degradation during the seismic loading. Some examples are the slope ratio, defined as the ratio of the slope of the loading to the slope of the unloading branches of the force-displacement ratio, or the modal effective stiffness defined as the slope of the modal force-displacement diagram[11].

  · DIs based on cumulative damage: The normalised cumulative rotation is defined as the ratio of the sum of the inelastic rotations during half cycles to the yield rotation.

  · Vibration-based DIs, whose treatment aligns with what is discussed in Chapter 3*.

1.2.6 Scour

As elaborated in a previous study[12] scour is defined as the excavation and removal of material from the bed and banks of streams as a result of the erosive action of flowing water[13]. Scour may occur in three forms: (i) general scour, (ii) contraction scour, and (iii) local scour. General scour is a natural process that is linked to changes in the morphology and parameters of the waterway (i.e., water flow and sediment quantity), leading to aggradation and degradation of the riverbed. Contraction scour

* Case study 6.12 describes the evaluation of seismic DIs for the case of a multi-storey building monitored with accelerometers deployed at selected storeys.

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occurs due to anthropogenic effects, e.g., bridge construction. Such processes reduce the channel cross-section, leading to accelerated flows and generated bed shear stresses, which can eventually mobilise sediments[14]. The third form - local scour - pertains to erosion effects that are localised around bridge piers and abutments and is attributed to flow separation effects resulting in horseshoe and wake vortices. Conditions that affect local scour, are mainly streambed erosion and sediment deposition. In absence of sediment delivery into the scour hole, the phenomenon is described as clear-water scour, while live-bed scour describes the case of sediment interaction, which is typically tied to lower ultimate scour depths than clear-water scour.

Scour evaluation was in the past primarily conducted by diving inspections, involving crude depth-measuring instrumentation, which naturally entails a dangerous practice, and also inaccurate as scour holes tend to refill upon the subsidence of floodwaters, thus affecting true depth assessment. In more recent years, remote monitoring solutions have been proposed for scour monitoring, primarily to what concerns the direct measurement of scour depth as the main indicator, including acoustic and electrical conductivity technologies, Magnetic Sliding Collar (MSC), Float-Out Devices, Time-Domain Reflectometry (TDR), Sonic Fathometer and Ground Penetrating Radar (GPR) systems*.

Very recently, monitoring of scour, which essentially results in loss of stiffness, via vibration-based methods[12], has been proposed as a low-maintenance scour monitoring alternative to more precise, but costly, NDT technologies.

1.3 Performance Indicators

When setting the goals for the asset management of infrastructure, one must recognise the multiple levels of objectives and multiple tiers posing requirements and creating constraints. The primary objectives of asset management are set at the highest strategic level by the Policy objectives, prevailing legislation, and administrative agreements (e.g., mobility, sustainability, resilience).

These strategic objectives are governing when the primary requirements are set for the function of the infrastructure during its full life cycle life and when the primary requirements are set for the properties that do not affect the basic functionality of the infrastructure but have an impact on the user expectations. These requirements are listed as the functional requirements and the non-functional requirements.

Both categories of requirements should be specified in terms of aspect requirements (Figure 1-7). Aspect requirements considered for infrastructure include reliability, availability, maintainability, and safety, RAMS, sometimes extended by including security, health, environment, economics, and politics, according to *RAMSSHE€P* (IM-SAFE D3.1, 2022).

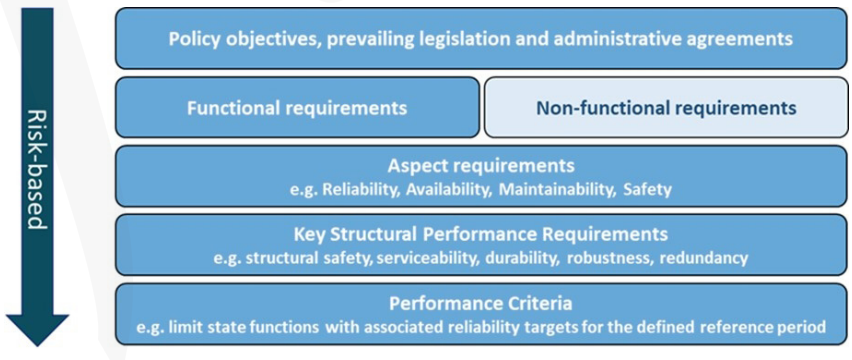


Fig. 1-7 Aspects and dimensions for identifying performance requirements, performance criteria and subsequently Performance Indicators (IM-SAFE, 2022).

* Case study 6.8 describes scour and the environmental actions on a scoured bridge.

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The presented concept is the basis for the performance and Key Performance Indicators. Moreover, this recent CSA project IMSAFE aims to harmonise the concepts and requirements from different committees and projects, such as TU COST 1406 and *fib* COM 8 to a CEN standardisation. From these activities, the aspects and characteristics defined in the following have been already derived.

Table 1-4 Concept of different requirements.

Requirement	**Concept**
Aspect requirements	Established using the (Key) Performance Requirements.
Key Performance Requirements (Key Performance Indicators, TU1406)	Main requirements set for the primary functions or properties for all aspects considered, specified in terms of performance. Established by means of the performance criteria, which are the quantitative limits, associated with a performance requirement, defining the border between desired and adverse behaviour.
Performance criteria	Threshold values about structural performances in the context of limit state design, that describe for each limit state the conditions to be fulfilled.

Examples of Key Performance Requirements considered for infrastructure include requirements concerning structural performance, which comprise, for instance, the requirements associated with structural safety, serviceability, durability, robustness or redundancy. These aspects and considerations elaborated in the IM-SAFE project for the CEN standardisation resulted in the following definitions of Performance Indicators and Key Performance Indicators.

- Performance Indicator (PI): A measurable and/or testable parameter (i.e., characteristic of materials and structures) that quantitatively describes a property of the structure and/or of the aspect of its performance and are used to qualify fitness of the structure for its purpose during service life.

- Key Performance Indicator (PI): Requirements set for the primary function(s) of an asset that further specifies the functional requirement(s), usually in terms of reliability, availability, maintainability, safety, security, health, environment, economics and politics, aiming at meeting specified functional requirements during the service life at appropriate service level.

It is important to note that the definitions of the indicators shown here do not only cover visual inspections but also test procedures, monitoring, SHM and modelling (Table 1-5). Further details on this can be found in Chapter 4 (section 4.4).

Accordingly, choosing suitable KPIs relies upon a good understanding of what is relevant for the organisation. Hence, various techniques to assess the present state of the structure, and its main tasks, are associated with the selection of PIs. These assessments often lead to the identification of potential improvements, so PIs are routinely associated with 'performance improvement' initiatives. A very common way to choose KPIs is to apply a management framework such as the balanced scorecard. The importance of such PIs is evident in the typical decision-making process (e.g., in the management of structures). When a decision-maker considers several options, they must be equipped to properly analyse the status quo to predict the consequences of future actions. Therefore, the proper usage of PIs is vital to avoid non-optimal decisions and minimise the risk[15], [16].

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Table 1-5 Performance Indicators

Cracks	Holes	Prestressing cable failure	Reinforcement bar failure / bending
Crushing	Debonding	Reinforcement bar failure/bending	Stirrup rupture
Rupture	Obstruction/impending	Stirrup rupture	Tensioning force deficiency
Delamination	Displacement	Tensioning force deficiency	Loss of section
Scaling	Deformation	Loss of section	
Spalling	Wire break	Prestressing cable failure	
Deteriorated mortar joints			
Frequency			
Vibrations/oscillations			

1.4 Requirements & extended Performance Indicators[1]

As stated above, PIs or KPIs should be causally related to the performance requirements – i.e., quantitative limits, associated with a performance requirement, defining the border between desired and adverse behaviour (see Table 1-4). These performance requirements can be formulated in the terms of deemed to satisfy, condition rating, or the statistically-based quantities shown below, and can thus also be directly related to visual inspections, tests, monitoring, or modelling. Apart from typical PIs listed in Table 1-5, in the following sections further extended PIs are described, serving as the basis for the performance assessment of structures.

1.4.1 Failure probability

The response of an engineering system is represented by a random variable Z called a safety margin:

$$Z = g\left(\boldsymbol{X}\right) \tag{1-2}$$

where $g(X)$, a computational model (e.g., degradation model), is a function of so-called basic random variables, $X = X_{1,}...,X_{Nvar}$, and also of other deterministic parameters. This random vector X follows a joint Probability Density Function (PDF), $f_X(x)$, where in general its marginal variables are statistically correlated. The failure probability, as a measure of unreliability, is defined by Eq. 1-3, where D_f represents the failure region where $g(X) \leq 0$.

$$p_f = P\left(Z \leq 0\right) = \int_{D_f} f_{\boldsymbol{X}}\left(X_1,\ldots,X_{N_v}\right) dX_1 \ldots dX_{N_v} \tag{1-3}$$

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1.4.2 Reliability index

Another relevant PI is the reliability index, β, defined in Eq. 1-4, where μ_z and σ_z are the mean value and the standard deviation of the safety margin β, respectively. In this case, β is the reciprocal value of the Coefficient of Variation (CoV) of Z, which can be interpreted geometrically as the minimum distance from the Limit State Function (LSF) $g(X)$ to the origin of the reference frame.

$$\beta = \frac{\mu_Z}{\sigma_Z} \tag{1-4}$$

Researchers[17] used this concept for generalised definition of β, by proposing to use the minimum distance from the LSF (usually nonlinear) to the origin in the uncorrelated normalised space U as a reliability measure, and given as follows:

$$\beta = \sqrt{u^T \cdot u} \text{ subject to } g(X)=0 \tag{1-5}$$

The relation between the failure probability, p_f, and the reliability index β, is approximately given by Eq. (1-6), where Φ denotes the standardised Gaussian distribution function.

$$p_f = 1 - \Phi(\beta) = \Phi(-\beta) \tag{1-6}$$

1.4.3 Risk

Another relevant PI is the risk, R, which holds the advantage to combine the effect of chances of some failure or disaster and the associated consequences, C, given as follows:

$$R = p_f \cdot C \tag{1-7}$$

where p_f is the probability of occurrence of failure and C, is the consequence associated with failure. Consequences consist of two parts: (a) direct consequences, C_{dir}, which are related to local components failure, and (b) indirect consequences, C_{ind}, which are associated with subsequent system failure. The former is considered proportional to the initial damage, since they include only the commercial cost loss (i.e., related to replace the damaged component/system), whereas the latter covers broader loss aspects, such as safety loss and environmental loss, in addition to commercial loss. Consequently, the risk caused by the failure of component i can be split into direct risk $R_{dir,i}$ and indirect risk $R_{ind,i}$:

$$\begin{aligned} R_{dir,i} &= p_{f,comp,i} \cdot C_{dir,i} \\ R_{ind,i} &= p_{f,comp,i} \cdot p_{f,subsys|comp,i} \cdot C_{ind,i} \end{aligned} \tag{1-8}$$

where $p_{f,comp,i}$ is the failure probability of component i, and $p_{f,subsys|comp,i}$ is the probability of subsequent system failure given the failure of component i. Hence, the total risk caused by the failure of component i can be quantified as:

$$R_{tot,i} = R_{dir,i} + R_{ind,i} \tag{1-9}$$

1.4.4 Redundancy

System redundancy is a PI that implies the absence of critical components whose failure would cause the collapse of the structure, and it is defined as the availability of system warning before the occurrence of structural collapse. In this context, redundancy index is defined as follows:

$$I_{red} = \beta_{sys} - \beta_{comp,1} \tag{1-10}$$

where β_{sys} is the system reliability index and $\beta_{comp,1}$ is the reliability index associated with the probability of failure of the first component. The larger the difference between these, the higher the system redundancy. It is worth noting that this PI should be combined with the information on other PIs, such as system reliability and risk.

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1.4.5 Risk-based robustness index

The ratio of direct risk to the total risk is defined as the risk-based robustness index, as shown in Eq. (1-11), where R_{dir} is the direct risk and R_{ind}, (Eq. 1-9) is the indirect risk.

$$I_{rob} = \frac{R_{dir}}{R_{dir} + R_{ind}} \tag{1-11}$$

1.4.6 Reliability-based robustness index

Structural robustness implies the insensitivity of the structure to local failures. Hence, another relevant PI is the reliability-based robustness index, I_{rob}, which is based on the concept of structural system redundancy, and expressed by Eq. 1-12, where β_{intact} is the reliability index of the intact system and $\beta_{damaged}$ is the reliability index of the damaged system.

$$I_{rob} = \frac{\beta_{intact}}{\beta_{intact} - \beta_{damaged}} \tag{1-12}$$

1.4.7 Resilience

The ability of a structural system to resist the effects of extreme events, including the ability to recover original functionality after such events, is known as resilience and it is expressed by Eq. 1-13, where t_o is the time occurring of extreme events, t_c stands for control time, and $Q_{(t)}$ is the percentage functionality of the system over recovery.

$$I_{res} = \int_{t_0}^{t_0+t_c} \frac{Q(t)}{t_c} dt \tag{1-13}$$

1.4.8 Hazard rate function

The instantaneous failure rate of a structural component, i.e., the conditional probability that the component will fail in a future time interval, given the fact that it has survived until the present time instant - is another relevant PI. For this, the survivor function, *S(t)*, representing the probability that a component/system is still surviving (i.e., not failed) at a given time, is defined by Eq. 1-14, where t_f is time to failure of the component or system.

$$S(t) = P(t_f \geq t) \tag{1-14}$$

Further to this, the hazard function, *h(t)*, which gives an indication of the rate of system failure, is defined as the limit for $\Delta t \rightarrow 0$ of the failure probability in the time interval $[t, t + \Delta t]$, given the system still surviving at time *t*, and averaged over the same time interval Δt. Eq. 1-15 allows its quantification, where *S'(t)* is the derivative of the survivor function, *S(t)*.

$$h(t) = \lim_{\Delta t \rightarrow 0} \frac{P\left[t \leq t_f \leq t + \Delta t \,|\, t_f \geq t\right]}{\Delta t} = -\frac{S'(t)}{S(t)} \tag{1-15}$$

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2. Condition survey: testing & monitoring methods

2.1 Introduction

Results of the structure condition survey form the basis of any management decision process. In civil engineering, each single structure is a prototype with variation of static system, material, foundation, load and environmental exposures. Every structure can experience specific degradation processes, determined by the combination of active degradation mechanisms and damage states (see Chapter 1) thus, it needs an individual condition survey concept, involving application of various testing and monitoring methods considering the objectives of the survey.

The diagnostic process starts long before the acquisition of the first data about the condition of the object under consideration. The basis of a successful structure diagnosis requires understanding of the capabilities and limitations of the available testing and monitoring techniques and procedures. The choice of diagnostic methods of concrete structures is closely related to the survey goals, which concern most often detection of defects and deterioration, characterisation of materials properties, and assessment of structural performance in normal or specific operational conditions.

The general structure of Chapter 2 is shown in Figure 2-1. The objectives of this chapter is to present an integrated classification and characterisation of modern testing and monitoring methods used in condition survey of concrete structures. The scope of the chapter includes the description of the most common survey objectives and strategies (see section 2.2), utilizing various diagnostic testing and monitoring methods (see section 2.3) for a meaningful condition assessment of a structure or its components. The proposed framework is independent of the age of the structure and can be applied to new structures and existing ones. The testing and monitoring methods presented (see 2.3) provide data collected through load independent tests, which usually provide local information (see section 2.4) and load dependent tests that provide information about the global structural behaviour (see section 2.5), as well as cover identification of environmental and operational actions on concrete structures (see section 2.6).

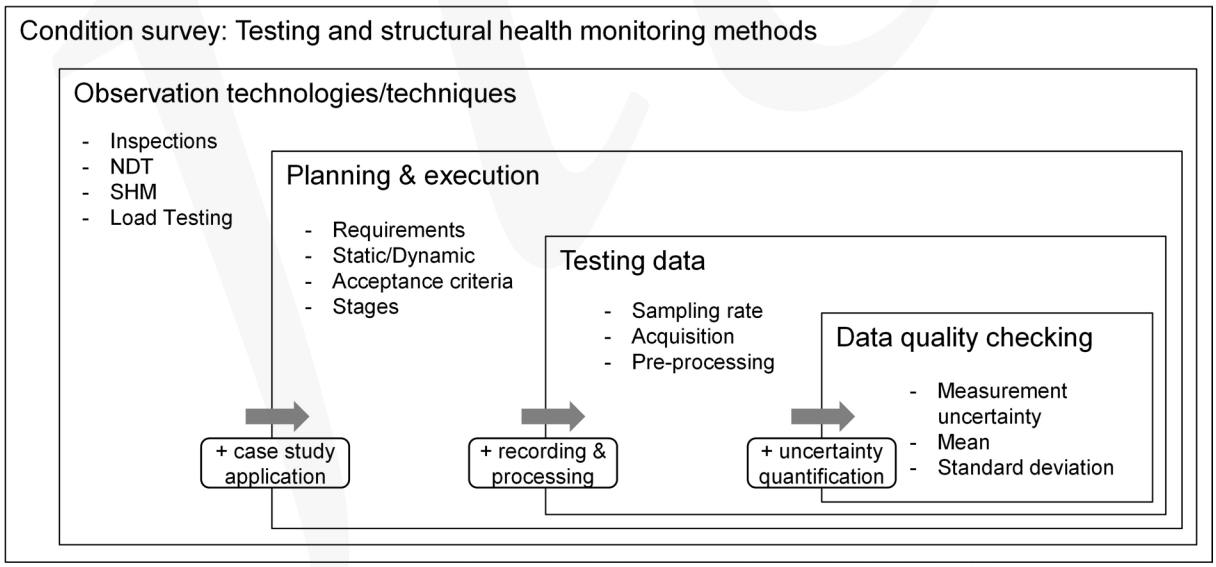


Fig. 2-1 Graphical representation of the structure.

This chapter was written by Bartolac, Bień, Górski, Keßler (main author), Küttenbaum, Kuzawa (main author), Ley, Maack, Mendler, Ryjáček, Santos, Verstrynge.

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Many testing and monitoring techniques are at various stages of development, ranging from universally acknowledged to developing field tests. Thus, this chapter addresses also the general characterisation and the evaluation of the usefulness of available testing and monitoring techniques in the context of assessment of concrete structures (see 2.3.2). This is essential in order to optimise the survey and monitoring process, since each technique has a specific range of applications. In general, various testing and monitoring methods are not alternative options and very often, various methods are applied together for effective diagnostics.

2.2 Strategies of testing and monitoring methods

Testing of concrete structures can be defined as a diagnostic procedure aimed at obtaining information about the current condition or performance of a structure or its components. Ad hoc tests can be performed in every moment of the structure's life without long-term planning. Expected results of each test is a value that characterises the behaviour or the condition of a structure or its components. The ad hoc tests are usually executed in the case of:

- doubts according to the structure condition or serviceability because of defects, e.g., after collision, fire, earthquake, etc.;
- re-opening the structure for operation after rehabilitation, strengthening, etc.

Monitoring of concrete structures can be defined as a procedure of systematic data acquisition based on a monitoring plan, which aim to detect changes in materials and/or structural properties. In a general sense, monitoring can be regarded as systematically performed testing[3], [18], [19], [20]. The monitoring systems are generally focused on detecting deterioration having as objective the evaluation of the structural performance in terms of structure safety (Ultimate Limit State, ULS), serviceability (Serviceability Limit State, SLS) and condition (Condition Limit State, CLS)[3], [21]. Results of the monitoring procedures create also bases for durability prognosis of structure, including evaluation of degradation processes, prognosis of condition changes in the future and finally lifetime forecast.

The possible strategies of both testing and monitoring methods applied during condition survey of concrete structures are presented in Figure 2-2. The relationships between various testing and monitoring policies in the context of using analogous diagnostic methods and techniques are presented in Table 2-1.

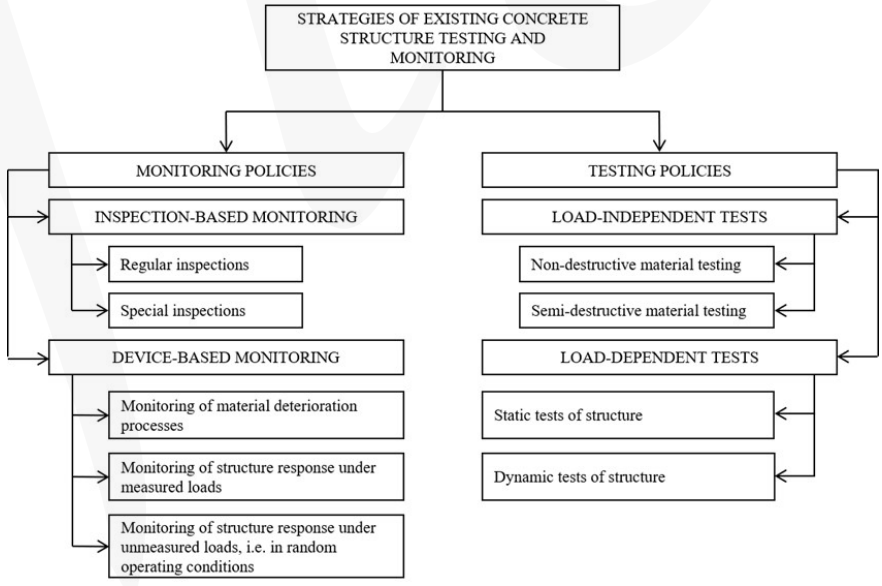


Fig. 2-2 Strategies of testing and monitoring of concrete structures[3], [21].

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Table 2-1 Strategies of concrete structures monitoring vs. testing policies[3], [21].

Strategies of monitoring of concrete structures			Testing policies			
			Load-independent tests		Load-dependent tests	
			Non-destructive material testing methods	Semi-destructive material testing methods	Static tests of structure	Dynamic tests of structure
Monitoring policies	Inspection-based monitoring	Regular inspections	■	■	□	□
		Special inspections	■	■	■	■
		Monitoring of material deterioration processes	■	■	□	□
	Device-based monitoring	Monitoring of structure performance under identified loads	□	□	■	■
		Monitoring of structure performance in random operating conditions	□	□	□	■
Legend: ■ – basic diagnostic methods, □ – additional diagnostic methods						

In the strategies of diagnostic tests, two basic types can be distinguished (Figure 2-2)[21]:

- load-independent tests, which provide results regardless of loads acting on the tested structure;
- load-dependent tests, which are based on effects of interaction between the tested structure and loads (static or dynamic) acting on the structure.

In general, load-independent tests are suitable to assess local condition, whereas load dependent tests can assess the global structural performance in terms of serviceability or safety.

Load-independent tests involve various Non-Destructive (NDT) and/or Semi-Destructive Testing (SDT) methods, as presented in handbooks and state-of-the-art analyses, e.g[21], [22], [23], [24], [25], [26], [27].

During load-dependent tests, response of the structure to the various loads (including traffic loads, environmental influences, etc.) is measured, recorded and analysed. Considering the general nature (static or dynamic) of the actions on structures, two fundamental load-dependent testing techniques can be specified: static tests and dynamic tests.

Strategies of structure monitoring can be divided into two main categories of policies: inspection-based monitoring and device-based monitoring (Figure 2-2)[3]. The inspection-based monitoring policy includes regular, as well as special (irregular), inspections. The basic functional components of a monitoring system that follows this policy are presented in Figure 2-3. Damage detection is founded mostly on visual field examinations and on the results of NDT and/or SDT tests using various types of techniques[3], [21]. The applied tests are load-independent. The inspection-based monitoring systems supporting the assessment of structural performance are usually based on the evaluation of a structure's technical condition, i.e., Condition Limit State, (CLS) and functionality[3]. Components of a load-independent inspection-based monitoring system are presented in Figure 2-3.

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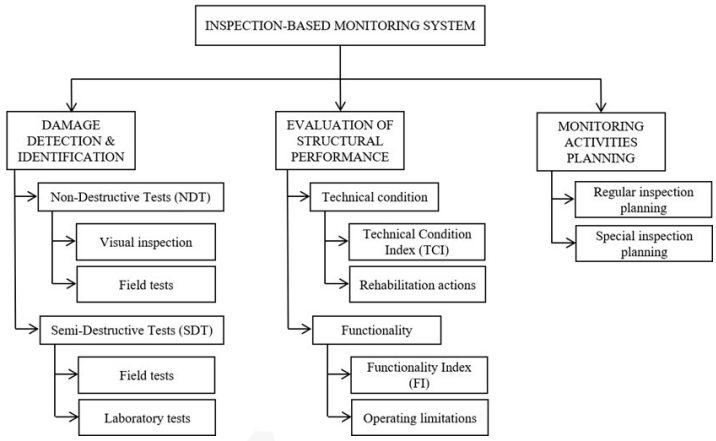


Fig. 2-3 Components of a load-independent inspection-based monitoring system[3].

The second category of monitoring policy, the so-called device-based monitoring (Figure 2-2, also called technical monitoring), is carried out through the observation of the response of a structure to loads and to the environment (including deterioration processes) by means of installed technical measuring equipment [3], [19], [20], [21], [22], [28]. The technical monitoring system is an installation for data acquisition and processing which continuously and autonomously provides real-time information about the structure or its component. The results of this type of monitoring, should contribute to the estimation of the structure's performance (safety, serviceability and condition associated with durability).

In this category, one can distinguish: monitoring of material degradation processes (carbonation, corrosion, chloride penetration, and rheological effects), monitoring of the structural response under measured loads (static, dynamic, and environmental actions) as well as monitoring of the structural response under unmeasured actions (Figure 2-2)[3]. The device-based monitoring is based on the deployment of transducers for sensing physical or chemical quantities, the application of programmable electronic equipment for acquiring, processing and communicating data, as well as the utilisation of respective algorithms. The short-term monitoring (usually lasting from 24 hours to a few days) is executed by means of temporarily installed sensors, while the long-term monitoring requires a permanently mounted system.

The overall objective of device-based monitoring is to support inspections by improving knowledge on a structure and its behaviour, including the recognition and evaluation of the condition of inaccessible areas. Basic functional components of a device-based monitoring system are presented in Figure 2-4[3].

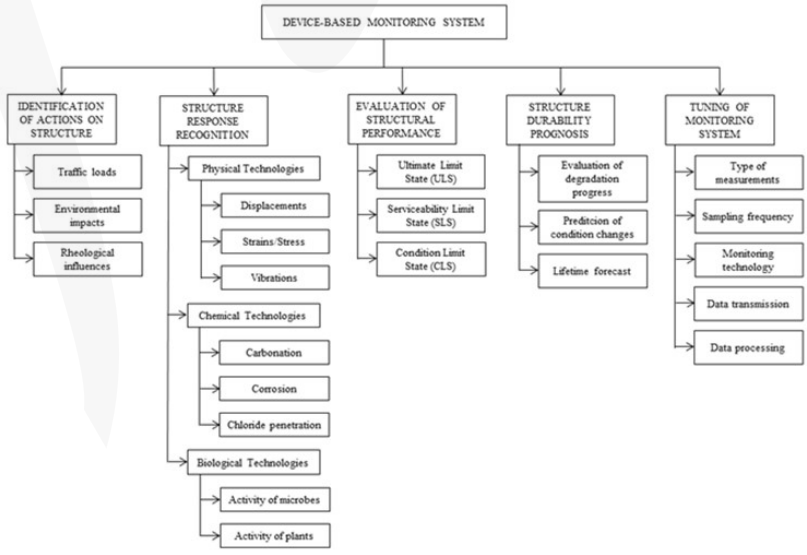


Fig. 2-4 Functional components of a load-dependent device-based monitoring system[3].

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Device-based monitoring is more commonly adopted for the following situations[3]:

- record-breaking long-span structures and/or structures with innovative structural/material solutions;
- very important structures of strategic meaning;
- damaged structures awaiting rehabilitation or replacement;
- selected reference structures, that are monitored for the improvement of design codes as well as degradation models and rehabilitation technologies.

2.3 Classification of testing and monitoring methods and techniques

2.3.1 General classification

Procedures of testing and monitoring of concrete structures require the application of problem specific measuring methods and techniques, which substantially differ from each other. In the case of ad hoc testing or inspection-based monitoring, a wide range of NDT methods and SDTmethods can be used*[21], [24], [25]. Classifications of the most popular NDT methods and techniques, which do not breach the integrity of the tested structures in any way, are presented in Table 2-2[21]. Additional information on optical, vision-based and wave-based monitoring systems of RC bridges can be found in the Annex.

A similar classification of the semi-destructive testing (SDT) methods that require material samples to be taken or any other minor breach of structural integrity is shown in Table 2-3. In this classification, the physical, chemical and biological methods applied during field and laboratory tests are distinguished.

Table 2-2 Classification of NDT methods and techniques[21].

Testing methods	**Measuring techniques**	**References**
Optical	Visual inspection Endoscopy Microscopy Geodesy / Global Positioning System (GPS) Closed-Circuit TeleVision (CCTV) Thermography	[23], [24], [25], [26], [27], [29], [30], [31]
Acoustic	Acoustic emission Impulse response Impact Echo (IE) Parallel seismic Ultrasonic techniques Ultrasonic tomography	[23], [24], [25], [26], [27], [32], [33], [34], [35], [36], [37]
Radiological	Radiographic techniques Computer tomography	[23], [24], [25], [26], [27]
Electromagnetic	Electromagnetic induction Radar techniques Magnetic techniques Electric potential / resistance techniques	[23], [24], [25], [27], [38], [39], [40], [41], [42], [43], [44], [45], [46], [47], [48], [49], [50], [51], [52], [53], [54], [55]
Laser	Laser static techniques Laser dynamic techniques	[23], [24], [25], [26], [27]
Sclerometric	Static techniques Dynamic techniques	[23], [24], [25], [26], [27]
Thermographic	Passive techniques Active techniques	[23], [24], [25], [26], [27]

* Case study 6.10.

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Table 2-3 Classification of SDT methods and techniques[21].

	Testing methods	**Measuring techniques**	**References**
Field Tests	Physical methods	Pull-out/Pull-off tests Penetration tests Water resistance test Pressure techniques (flat-jack test, water penetration test, etc.)	[23], [25], [27], [56]
	Chemical methods	Rainbow/pH tests Chloride test Alkali test Electroanalytical tests	[38], [55], [56], [58], [59], [60], [61], [62], [63]
	Electrochemical methods	Potential Concrete resistivity Corrosion rate	[50], [51], [58], [59], [60], [61], [62], [64], [65], [66], [67], [68], [69], [70], [71]
Laboratory Tests	Physical methods	Physical parameters of materials based on the testing of specimens Computer tomography techniques	[23], [24], [25], [26], [27], [36], [56], [72]
	Chemical methods	Quantitative analytical techniques Electroanalytical techniques Spectral analyses Chromatography analyses	[41], [51], [56], [67], [69], [73], [74], [75], [76], [77], [78]
	Biological methods	Advanced microscopic analyses "In vitro" culture techniques	[21], [22], [79], [80], [81], [82]

In device-based monitoring of concrete structures by means of installed technical measuring equipment, a wide range of measuring techniques and sensors can be engaged[3], [18], [63], [83], [84], [85]. The most popular measuring techniques and types of sensors used in technical systems for concrete structures monitoring are presented in Table 2-4. Many various techniques and measuring sensors are used in monitoring. In most cases, the sensors are attached directly to the structure, but vision-based systems and remote satellite radar systems are being developed, which will enable remote non-contact observation of the structure's behaviour. The so-called new generation systems are currently in the pilot study phase.

Table 2-4 Measuring techniques and types of sensors applied in monitoring of concrete structures[18].

Physical or chemical quantity	**Measuring techniques and sensors**	**References**
Linear displacement	Geodetic techniques Mechanical sensors Inductive sensors Vibrating wire sensors Capacitive sensors Eddy current sensors Fibre optic sensors Laser techniques Radar techniques Hydraulic sensors	[3], [63], [83], [84], [85], [86], [87], [88]
Angular displacement	Inclinometers Fibre optic sensors Micro-ElectroMechanical Systems (MEMS) sensors	[3], [63], [83], [84], [85], [86], [87], [88]
Strain / stress	Electrical resistance gauges Fibre optic sensors Vibrating wire sensors MEMS sensors	[3], [63], [83], [84], [85], [86], [87], [88], [89]

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Vibration velocity and acceleration	Piezoelectric sensors Capacitive sensors Inertia sensors Fibre optic sensors Inductive sensors Radar techniques Laser techniques MEMS sensors	[3], [63], [83], [84], [85], [87], [88]
Crack opening	Mechanical sensors Inductive sensors Fibre optic sensors Vibrating wire sensors	[3], [63], [83], [84], [85], [86], [87], [88], [89]
Bearing/cable reactions	Electrical resistance gauges Piezoelectric sensors Fibre optic sensors Vibrating wire sensors	[3], [63], [83], [84], [85], [87], [88]
Degradation processes	Acoustic emission sensors Fibre-optic chemical sensors Chloride level sensors Sensors of pH level Electrical potential Corrosion sensors	[3], [34], [41], [58], [60], [61], [63], [65], [66], [68], [75], [78], [84], [87], [88], [89], [90], [91], [92], [93]

The typical tools applied for the monitoring of loads and environmental influences acting on a structure are presented in Table 2-5[18].

Table 2-5 Measuring techniques and sensors in monitoring of loads and environmental impacts[18].

Physical quantity	**Measuring techniques and sensors**	**References**
Live loads	TV/video cameras Weight-in-motion systems	[83], [85], [94], [95], [96], [97], [98]
Temperature	Thermocouples Thermistors Electrical resistance thermometers MEMS sensors Fibre-optic chemical sensors	[3], [63], [83], [84], [85], [87], [88]
Humidity	Hygrometers MEMS sensors	[3], [63], [83], [84], [85], [88]
Wind	Anemometers	
Water flow	Water level sensors Water velocity sensors	

2.3.2 Classification according to survey goals

Goals of the load-independent tests and inspection-based monitoring can be categorised in three basic groups[3],[21]:

- identification and observation of changes of structure geometry, including geometry of the whole structure as well as all its elements (thickness, location of reinforcement and prestressing wires, etc.);
- determination and observation of changes of material characteristics and quality, such as: strength, modulus of elasticity, homogeneity, permeability, humidity, temperature or chemical composition;
- detection and characterisation of structure defects.

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For selected NDT and SDT methods and techniques, a comparison of their applicability in diagnostics of structure geometry, material characteristics and defects are presented in Table 2-6[21]. Because of continuous development of non-destructive load-independent testing techniques, areas of their use and application methods are expected to be extended and improved. It is also expected that new diagnostic methods and techniques will be developed.

Table 2-6 Geometry, material characteristics and defects of concrete structure detectable by non-destructive and semi-destructive load-independent tests[21].

Methods and testing techniques		**Geometry**			**Material characteristics**						**Class of defects**					
		structure geometry	element geometry	reinforcement/ wires identification	strength/modulus of elasticity	homogeneity	air/water permeability	humidity	temperature	chemical composition	deformation	destruction	loss of material	discontinuity	contamination	displacement
Optical (NDT)	Visual inspection	■	□						□		■	□	■	■	■	■
	Microscopy/endoscopy	■	□						□		■	□	■	■	■	■
	Geodesy/GPS surveying	■	■								■		□			■
	Laser techniques	■	■								■					■
	Infrared thermograph testing			□					□			□	□	□		
Radiological (NDT)	Radiological techniques			■								□	■	■		
	Radioscopic methods			□								□	■	■		
	Computer tomography			■								□	■	■		
Acoustic (NDT)	Impulse methods		■		■	■						□	■	■		
	Ultrasonic methods		■		■	■						□	■	■		
	Acoustic emission methods											□		■		
Electromagnetic (NDT)	Magnetic methods			■												
	Radar techniques	□	□	■								□	■	■		
	Electromagnetic induction methods			■												
	Electric potential/resistance techniques								□			■	■			
Physical (NDT & SDT)	Direct geometric measurements	■	■	□							■	□	■	■	■	■
	Penetration tests				■							□				
	Sclerometric methods				■	□						□				
	Pressure techniques						■					■				
	Field methods of testing physical properties				■	■						■				
	Laboratory methods of testing physical properties				■	■	■	■	■			■				

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Chemical (SDT)	Qualitative analytical techniques									□		□			□	
	Quantitative analytical techniques									■		■			■	
Biological (SDT)	Advanced microscopic analyses											■			■	
	"In vitro" culture techniques											□			□	
Legend: ■ – basic method/technique, □ – additional method/technique																

In load-dependent diagnostic tests, the response of the structure to known and measured or unmeasured loads as well as other environmental sources is measured, recorded and analysed. Static tests are usually performed for the identification of the mechanical characteristics of the structure in operational conditions as well as for the detection of some classes of defects (see Table 2-7)[21]. Dynamic tests are executed for recognition of dynamic characteristics, detection of defects and – finally – for determination of operational parameters of a structure (see Table 2-7 and section 2.5.4)[21]. Results of both static and dynamic tests are of the highest importance also for validation of theoretical models of the structures used in the diagnostic procedures[99].

Table 2-7 Typical results of load-dependent diagnostic tests and device-based monitoring[21].

Testing & monitoring techniques		**Typical effects**						
		structure stiffness	linear-elastic range	resonance hazard	dynamic parameters	material effort level	strain/stress distribution	damage presence
Static tests	Identified loads	■	■			□		□
	Random non-mechanical influences (temperature, etc.)	□	□			■	■	□
Dynamic tests	Identified loads	■	■	■	■		□	□
	Controllable mechanical exciters	□		■				■
	Random traffic loads (vehicles, pedestrians, etc.)	□	□	■	■	■	■	□
	Random non-mechanical influences (wind, etc.)	□	□	□		■	■	□
Legend: ■ – basic technique, □ – additional technique								

Results of load-independent and load-dependent diagnostic tests and monitoring form a basis for the evaluation of structure resistance and functional condition and – in the next step – for decisions in operation and maintenance of the structure.

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2.4 Load-independent tests: planning, execution and documentation

2.4.1 General requirements on testing

Whether a NDT method can be successfully used to obtain information about civil structures in general, and concrete structures in particular, depends on various prerequisites and boundary conditions. An essential prerequisite is the performance evaluation of the NDT inspection system (NDT-system) applied. The main components of a specific NDT-system are the equipment, the procedure, and the (trained/educated/qualified) personnel[100]. The (performance) evaluation of the test is part of quality assurance in inspecting a component with NDT methods. Such evaluations are of particular importance, if no standards or guidelines exist for the application of the testing methods, or if the inspection is highly safety-relevant, or when new techniques are applied or methods are set up in a different configuration[101].

In addition to the generally required (performance) evaluation, certain boundary conditions must be fulfilled for the successful conduction of an individual inspection, depending on the specific testing task. To assess the suitability of a testing method, it may be useful to compare such boundary conditions with the requirements based on three criteria: the physical, the technical and the economic feasibility. Physical constraints are referred to whether the selected inspection technique is basically capable to solve the inspection task. The radar method, for example, is unsuitable for locating tendons mounted at great depths and shadowed by close-meshed reinforcement near the surface. The technical requirements contain the testability of the specific component or structure and could include, for example, the accessibility of the component. The third requirement of the inspection is the economic efficiency, as testing should be both technically feasible and economically viable (Chapter 5).

2.4.2 Planning of NDT

As a starting point in planning NDT, the testing task and the quantity of interest (measurand) must be defined. The definition of a measurand, in turn, is directly linked to the inspection task. Aspects such as threshold values for the significance of an indication or expected measurement uncertainties should be considered in this early planning stage. The transformation of measurement results (see section 2.9.1) into structural parameters or basic variables, respectively, is described in section 2.8.3. Preceding sensitivity analyses can be used to determine the measurands, the locations (e.g., which cross-sections to assess). An example is the location of prestressing tendons in the area of maximum moments during the bending proof of a prestressed concrete bridge when as-built drawings are not available. When planning NDT, it is also necessary to clarify to what extent the test results should and can be destructively verified (e.g., by taking drill cores). The preparatory activities subsequently include preliminary on-site inspections, research using available technical drawings or test reports. During the pre-inspections on-site, practical boundary conditions such as the accessibility of the structure or the availability of supply media such as electricity, light, or water should be considered. Further, an on-site inspection provides the opportunity to identify boundary conditions influencing the measurement, such as coatings on the surface or power lines that might hinder the measurement. The study of technical drawings and existing test reports facilitate the decision on which test method can be used to solve the task optimally. It can be necessary to combine different test methods in order to increase the reliability of the results. Furthermore, it can be useful to gather additional information about the history of the environmental conditions such as temperature, humidity or freeze-thaw cycles at the test object since these can also have an influence on the results of the test. Thereby such knowledge is useful not only for planning, but also for subsequent interpretation of measured data.

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The detailed planning of the NDT follows the preparatory activities. Once the equipment has been determined, the measurement strategy is defined, considering the reliability of the expected results. The measurement strategy includes the test instruction or procedure, the definition of measuring areas (point, line, area), and criteria that can lead to aborting a measurement. This may be necessary if unknown aspects arise during the measurement that significantly influence the test result. Interruption of the test can also be considered if the test does not lead to the expected result. For the preparation of an NDT, initial thoughts should also be made about the type of documentation and support to adopt for the transfer and presentation of the results.

2.4.3 Reporting of NDT results

The description of the measurement conduction and the interpretation of the results of the NDTs are finally given in a test report. The test report has several important functions and is therefore an integral part of quality assurance. A comprehensively prepared test report creates transparency through documentation and thus enables traceability of the findings obtained. The test report is an archive of the test results with a clear and reproducible structure. It thus also represents a guideline for the personnel involved. Based on available test reports, existing procedures may be revised, or new ones can be drawn up. Furthermore, the information can be used for repeated measurements as part of regular (periodic) structural inspections. In addition, it is possible to transfer the knowledge gained to other structures.

The type and scope of testing reports can differ significantly depending on the requirements. A distinction can be made between external and internal test reports. The external test report is handed over to the client for invoicing the service. The client determines the type and scope of the report. Ideally, this is already done during the planning and preparation of the inspection. As a simple example, the markings applied directly to the component (e.g., markings for core drillings) can be regarded as a test result. Nevertheless, it is recommended that an internal test report is prepared in view of the above-mentioned aspects (e.g., quality assurance).

The test report must contain the required information about the inspection, expressed in a clear and comprehensible form. The report should provide information about the test object, the testing task, the personnel, the equipment used, the underlying test instructions or procedure and finally the test results. If deviations from the test instructions occur during the test or if precise instructions are missing, the relevant boundary conditions must be documented separately. An example of the minimum requirements can be given as follows:

- Test object: unambiguous designation, location of inspections, basic properties (e.g., age, general construction, material);
- Testing task: target, scope, underlying standards and guidelines, technical drawings, existing test reports, regions of interest;
- Test methods: technique(s), equipment (e.g., data acquisition, sensors, auxiliaries), settings (e.g., attenuation, filtering);
- Data acquisition / Performance of the measurement: general conditions (e.g., date, time, environmental influences, personnel ID, personal qualification, certificates), location of the measuring areas (e.g., measuring points, measuring grid), functional test/ calibration of the test equipment, data quality checks, deviations from the test instructions (e.g., coatings, impurities);
- Data analysis and evaluation:
    - Specification of software and parameters used (e.g., software name and version, filters, reconstruction algorithms);
    - Relevant information on signal processing (e.g., parameters of the filter used, truncated outliers/deletion of incorrect measured values);
    - Expression of measurement uncertainty (see section 2.9.1);

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· Preparation of the results in tables, plots, etc;

· Explanation of the result presentation (e.g., indication of the unit of the measurand (SI unit), colour scales);

- Summary including a statement regarding the predefined inspection task; deviations from the test planning, recommendations.

The state-of-the-art policy for the management of test results includes the digital recording and storage of the measurements. The data (raw data and processed data) can be handed over to the client in digital form, even in the case of large data volumes (e.g., monitoring). In this case, it is recommended to additionally hand over in digital form the metadata that contains the relevant information about the data. Section 2.9 provides further information about the Quality Control Process (QC).

2.5 Load tests: planning, execution and documentation

Due to the inevitable ageing of concrete structures over time, many of them are found, at some point in time, to have inadequate load-bearing capacity when assessed by standard calculation methods. On the other hand, others might reveal that their load-bearing capacity is still adequate even though for the in-situ conditions this was not expected. Both in the former or the latter scenario, uncertainties in the structure's performance exist and therefore, they need proper quantification. Load testing can provide information about the structure's condition (diagnostic load test) or about its load bearing capacity (proof loading tests). Such information enables a check whether the response of the bridge (structure) complies with the required performance.

2.5.1 Goals of load tests

In light of the above mentioned, the goals of a load test can be summarised in two levels[18], [102], [103]:

- Diagnostic load test focusing on confirming the response of the structure against the service loads. In other words, to verify the structure condition and the suitability of the design/analytical model by examining on a comparative basis, the results from the model and the observed behaviour of the structure under a certain percentage of the design live loads. This can be also seen as a validation of the design/analytical model. These are the most common type of load test since in many countries it is mandatory to perform a load test, which can be static and/or dynamic before the structure enters operation (including situation after strengthening).

- Proof load test focusing on improving the analytical assessment of an existing structure revealing the potential hidden safety reserve. This can be seen as direct proof that the structure can carry the total design live loads, which requires a careful execution of the test. Phenomena such as corrosion, delamination, alkali-aggregate reaction, etc. are possible motivations to conduct such a load test due to the difficulty in accurately evaluating these via models and therefore, giving important information about the effective structural performance and its actual level of safety.

It is worth noting that the comparison between results of two consecutive load tests (between either the diagnostic load test and a proof load test or two proof load tests) offers additional information about the structure deterioration rate.

2.5.2 Planning of load tests

To be successful a load test must consider a set of steps in order to obtain the most benefit from it, mainly:

- gather available information about the structure, i.e., the design, construction, and maintenance documentation such as design calculation reports, the design and as-built plans, QC records during

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construction etc. It is very important to acquire information about the age of the structure which gives indications of the materials and standards used at the time of its design.

- in-situ inspection to verify if the as-built information corresponds to the geometry and material parameters of the real in-situ structure. In addition, a careful inspection of the components of the structure is essential for detecting cracks or other relevant defects, which may play an important role in the structure performance.

- analytical or Finite Element Model (FEM) of the structure to simulate the test and define the suitable positions and magnitudes of the load to induce the expected range of deflections, strains, rotations, accelerations, etc. For new structures, the average material properties can be used as input in the model used for the design. For existing structures, it is important to properly consider eventual defects influencing structure parameters.

- instrumentation plan to collect data at the locations of the deflections, strains, rotations, accelerations, etc. estimated with the analytical FE model. In addition, temperature and RH must be measured to make allowance for their effect on the measurements. For this, prior calibration of the sensors, their position and installation setup, range of measurement, calibration factors, and sampling rate must be clearly defined. Some level of redundancy is recommended.

- loading setup and sequence that will be carried out and its objectives. In the case of a diagnostic load test, it is usual to increase the loads up to values corresponding to the characteristic combination level of the unfactored service load. On the other hand, in the case of a proof load test, the applied load will be representative of the factored load combinations. The loads may be applied using vehicles in the case of transportation infrastructures, whereas water tanks or dead weights are more usual for other types of structures (e.g., buildings). An assessment of the possible risks of the test and contingency actions must also be properly addressed.

- personnel requirements to conduct the load test. The definition of responsibilities, the required qualifications as well as experience and eventual authorisations from local authorities must be properly addressed.

- analysis of the results by comparing the collected measurements and the predicted ones with the analytical or FE model. This allows assessing the permanent deflections and the elastic or plastic behaviour showed during the load test against the design objectives and/or quantifying the hidden capacity.

2.5.3 Service loads, load arrangement, load test protocol

In the diagnostic load tests, the loads should be representative of service load levels, i.e., a certain percentage of the ULS load effect, dependent on the ratio between dead and live load, approximately, based on the load combination prescribed in the codes. In codes, live load models usually comprise distributed loads and concentrated loads. The total load should be applied to the structure incrementally in at least two steps and should remain at each level until the measured parameters become stable in time. For simply supported structures, the load should be applied span by span. For continuously structures, the loading arrangement should consider the overall structure behaviour. In this case, the loading will be carried out by placing loads in adjacent spans (aiming at maximum negative moments and maximum shear forces and pier reactions) and by loading alternate spans (aiming for maximum positive moments).

Loading cases to study effect of the spatial distribution of the applied load can also be included in the load test protocol.

The loading systems in diagnostic load tests need to fulfil the following requirements:

- Should be representative of the rating service loads;
- Loads should be manoeuvrable;
- Loads should allow repeatability to check the linear behaviour of the bridge.

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In a proof load testing, a load representative of the factored live load is applied to the structure. Since the load requirements are quite high, the desired effects are accomplished with special loading systems, such as heavy vehicles in the case of bridges. In any case, the test must be stopped if some abnormal event is observed and not identified in advance in the planning document. The two main loads to be defined and applied in a proof load test are the target value of the load and the maximum load attained in the test. The first one is defined based on the requirement of the allowable maximum load with a specified safety level and the second one must be defined in such a way that the test does not produce irreversible defects in the structure[99], [104].

2.5.4 Static test and dynamic load test

2.5.4.1 Static test

The static test is the most common load test carried out. In this kind of test, the load is positioned in each intermediate stage and the final stage long enough for the measurements to stabilise. The load is applied using particular systems (e.g., loading frames and hydraulic jacks). If the structure can carry this load without signs of distress, the test will be considered successful and the structure has been shown experimentally to fulfil the code requirements. To guarantee structural safety, the proof load test needs to be carefully studied and prepared. The test load must be applied progressively in a multiple loading and unloading procedure until the predetermined target proof load is reached or the end criteria (materials entering in post-elastic behaviour) is attained. Reliable sensors and monitoring techniques are necessary to guarantee the termination of the test without damaging the structure[99],[104].

The measurement parameters (e.g., deflections, strains etc.) that control the structural behaviour need to be constantly monitored and evaluated. In the event of nonlinear behaviour or signs of structural distress (e.g., cracking of the concrete, yielding, etc.), the load should be maintained for a certain period. The duration of the test and the accuracy of the results will depend on the time the load should be held until stabilisation of the outputs. It is important to have some criteria to decide when the stabilised value of the displacements is obtained.

The most usual load configuration for simply supported structures is the one that produces maximum static vertical deflection of the spans. Sometimes loads are increased in a stepwise pattern to obtain intermediate results. In the case of continuous structures, the most usual load configurations are those to obtain maximum static deflection (loading alternate spans) and those to obtain maximum negative moments and maximum shear forces in the supports line. In static load tests the basic measures most common are:

- Vertical deflections and longitudinal strains at midspans;
- Vertical deflections at supports (when the supports have displacements);
- Crack evolution (if they appear);
- Temperature and RH.

2.5.4.2 Dynamic Test.

Dynamic tests are performed for three main reasons:

- to assess the dynamic amplification factors; i.e., the coefficient amplifying the structural response when the load is applied in dynamic mode. This latter type of test is especially important in the case of bridges, which are carrying moving traffic loads. In this specific case, the test is carried out using the same type of loads used for the static test (heavy trucks or some type of railway vehicle, depending on the type of the bridge) that are driven along the bridge. The loading protocol comprises a set of vehicles equal to the number of lanes driving from one end of the bridge to the other side by side at a certain speed. Usually, the vehicles are driven at various speed levels which are determined according to the velocities of the vehicles that can appear in real traffic.

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Since it is quite challenging to drive multiple vehicles side by side at high speeds, this test can also be performed by using one vehicle, which can then safely drive at quite high speeds, even higher than those that can appear in real traffic. This situation is simpler in the case of railway bridges. Another subtype of the dynamic test is the braking test. In this type of test, a moving vehicle brakes on the bridge (at different speeds and with different braking power) to examine the horizontal movements of the structure.

- during the dynamic test, the speed of the moving load should be recorded and at least the vertical deflection in the midspans should be continuously measured. Additionally, strains, longitudinal (important for the braking test) and transversal displacements can be measured. Acceleration data are also usually recorded in the dynamic testing.

- to investigate the performance of structures that can be sensitive to dynamic actions (e.g., cable supported bridges that are sensitive to wind action and pedestrian bridges that are sensitive to pedestrians moving at marching and running speeds). The goal of this type of test is to verify if the structure is designed in such a manner that its behaviour under dynamic action does not put its load-bearing capacity in danger nor does it cause unpleasant feelings for people using it. Besides making the comparison of the obtained results with the design results, it is also preferred to compare the results with criteria indicated in the relevant codes.

- to estimate the dynamic characteristics of the structure, such as for instance modal parameters (natural frequencies, mode shapes and damping). The objectives and results that can be achieved through this type of test will be described in Chapter 3.

2.5.5 Acceptance criteria of load tests

Based on a static or dynamic load test, a structure can be positively evaluated if the following requirements are met:

- no signs of distress and reduction of the bearing capacity of the structure should be observed.

- the measured deformation parameters of the structure (e.g., deflections, strains, rotations) are less or equal to the predicted (calculated) ones.

- permanent deformation parameters (measured after unloading the structures) are within the limits prescribed by the relevant regulations (e.g., 20% of the maximal measured values during the test of concrete structures, whereas for prestressed concrete structures this value is limited to 15%).

- if cracks remain in the concrete structure after the test, these should be within the limits prescribed by the relevant regulations (e.g., max 0.1 mm).

- the measured dynamic parameters (i.e., natural frequencies, damping ratios and modal shapes) correspond to the calculated ones.

- the structural response to dynamic action is in accordance with the design considering primarily the dynamic amplification factor and the comfort of the bridge users.

2.5.6 Load test stages (pre, during, after)

On a chronological basis, and considering the time when the load test takes place, three main stages are identified with complementary objectives, mainly:

- Preparing the load test from the point of view of investigating the structure planned to be tested. The focus here is to define the objective of the test supported by available information, with eventual additional data collected in-situ, and simulations. By the end of this stage, a load test program, including a monitoring plan to observe the structure performance must be delivered.

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- Conducting the load test on the structure, as planned previously and guaranteeing that this is performed under suitable protocols of safety for both the structure and personnel involved. By the end of this stage, a database with the monitored data must be successfully collected. For this, it is important to double-check that no faults in the measurement procedure occurred.

- Analysing the monitored data gathered against the predictions made at the preparation phase. Based on a comprehensive analysis, this allows assessment of the elastic or plastic behaviour shown during the load test and/or quantifying the hidden capacity. By the end of this stage, a pragmatic conclusion should be achieved in terms of checking if the design objectives are being achieved or if there are hidden capacities and their quantification.

2.5.7 Quality control of load tests

During testing, it is critical to minimise errors or malfunctions during the collection of the monitored data. The required logistics associated with such tests justify even more such QC. The following aspects are outlined:

- Selection of the monitoring system type regarding its principle of functioning, i.e., an electric-based measuring system or an optic-based system. Aspects related to electromagnetic interference, level of RH and/or presence of water and desired number of measuring points should be criteria to decide in this aspect.

- Selection and calibration of sensors that will be used during the load test to observe the structure performance. Ideally, this should include a laboratory test to determine the calibration curve of each sensor, including the specific instrumentation chain to be used. Even though, usually, the sensors' manufacturer provides calibration sheets, this is recommended considering eventual hysteresis effects on the sensors from previous uses.

- Pseudo-real time visualisation of the monitored data, i.e., being able to quickly see the date in the field to double-check expected patterns and orders of magnitude. If the data exhibits abnormal or unexpected behaviours, this might help to identify potential problems in cable connections, data acquisition protocols faults and/or problems in power supply in some parts of the monitoring system.

2.6 Monitoring of environmental and operational loads

2.6.1 Environmental loads

Environmental loads caused by the local weather and climate condition can affect the performance of structures as well as their durability. The most important environmental parameters are temperature – air and structure -, humidity, wind, rainfall, chloride exposure etc. The temperature and especially temperature gradients among a single structure show high influence on the structural response. Consequently, each monitoring systems requires the consideration of temperature sensors*.

The measurement of humidity, in general, is related to some aspects of the behaviour of materials, such as creep and shrinkage of concrete or corrosion. Wind monitoring is based on the use of classical cup anemometers or sonic anemometers to measure the speed, direction and strength of gusts of wind[105], [106], [107]. There are many sensors available for the measurement of each of these parameters, or even others like rainfall, as well as several combinations for the measurement of two of them, and, finally, weather stations allowing the monitoring of all these parameters. Additionally, embedded sensors complement the weather station to gather information about the material parameter such as humidity, temperature, pH-value, chlorides, etc.

* Case studies 6.17 and 6.20

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Monitoring of seasonal loads, such as due to ice and snow in winter, is one of most crucial elements of to perform the safety assessment for large public buildings and infrastructure, industrial structures with long spans and large roofing areas not only in regions typical for exceeding snowfalls, icing and frosting but, due to climatic changes and anomalies, also in other regions of the globe formerly not affected by winter. Excessive snow, icing and frosting can impose new, unplanned, large loads on the structure, so monitoring systems allow undertaking proper actions by operators of buildings and structures. Monitoring systems for this type of load are based primarily on early warning systems referring to prediction and use of data from global systems, large areas Synthetic Aperture Radar and satellite mapping. During winter, direct monitoring is implemented with use of snow gauges, ultrasonic depth sensors, snow weight sensors and strain gauges. For ice and frosting loads on the structures, especially for cable-stayed ones, vibration-based methods are used which detect increase of mass. Recently winter loads are also being monitored with the support of Unmanned Aerial Vehicles (UAV) carrying vision-based monitoring systems.

2.6.2 Industrial load monitoring

This category of operational loads refers to industrial structures, which may be divided into the following general groups:

- Structures directly impacted by operational loads of the industry, as for example machine foundations subjected to dynamic loads of high magnitude and frequency;
- Structures indirectly subjected to industrial loads as for example structures protecting industrial installations affected by vibrations, blasts, acoustic waves and unfavourable environmental conditions.

Structures subjected to industrial loads should be supplied with monitoring system tailored individually considering the type of loads, their dynamic characteristic, technological requirements adjusted to demands of production process for its safe and effective operation. Due to the specificity of production, many industrial structures are affected by heavy loading and constant vibrations or stamping. In such cases dynamic behaviour monitoring and fatigue monitoring are recommended.

Additionally, industrial structures may be affected by elevated temperatures, sudden changes of temperature, high humidity and chemically aggressive environment. In such cases, monitoring of the structure's operational conditions should be applied.

It should be underlined, that due to elevated loads, and resulting vibrations transmitted by air and soil, monitoring may also be recommended for neighbouring structures not connected directly with the production process.

2.6.3 Traffic loads

2.6.3.1 Road bridges

Traffic loads on road bridges are specified in several codes, like EN1991-2 or AASHTO LRFD Bridge Design Specifications. However, the monitoring of those loads is relevant to such traffic models' calibration, as well as cases where a more refined analysis of traffic loads effects is required.

The technologies available for weighing traffic loads can be divided in the following groups:

- Weighing on static scales;
- Weigh-In-Motion (WIM);
- Bridge Weight-In-Motion (BWIM).

The use of classical static scales requires vehicles to come off the road onto a dedicated weighing site, where they must stop to perform the weighing, with obvious inconvenience due to the limited capacity and the impact on travel time.

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WIM and BWIM are alternatives to static weigh stations, which can measure a moving load without interrupting regular traffic. Both approaches provide the identification of the axle and gross vehicle weights of vehicles, as well as their speed.

WIM systems are placed in or under the roadway pavement and measure the weight of vehicles crossing that section. The common devices used for pavement-based WIM systems include bending plates, load cells, capacitance mats, and strip sensors. The only main inconvenience of these pavement-based WIM systems could be the traffic interruption during their installation and maintenance process.

BWIM techniques use an instrumented bridge as a weighing scale to estimate the vehicle weights. BWIM is a cost-effective technology with several advantages, since the installation of the devices is easy and safe, as it can be done without interrupting traffic. This technique is more accurate than pavement-based WIM systems and more durable, as the sensors are installed under the bridge, avoiding the direct exposure to the traffic. Due to these significant advantages, the BWIM technology has been playing an increasingly important role in bridge health monitoring and overweight truck enforcement.

2.6.3.2 Railway bridges

The traffic class, according to the EN 15528, specifies the load on the railway bridges. This code gives the maximum allowable unit and axle load on the railway track. The axle load, axle spacing and train speed can be detected by non-direct WIM methods (weight in motion). The results can be compared with the allowable load. These can be divided into the following groups:

- systems based on the deployment of a pressure sensor in a drilled hole in the rail or of other sensors on the web of the rail. The advantage is the relative simplicity of the installation, the sensor being close to the load acting on the rail surface. Variable numbers of sensors can be used to increase the precision of the weight detection and axle spacing.
- systems with pressure transducers deployed under the foot of the rail. An example is the commercial GOTCHA system, based on Fibre Optic Sensors (FOSs).
- systems with strain gauges glued to the rail, usually on the bottom surface of the rail. The advantage is the relatively low cost; the disadvantage is the complicated installation and the need to calibrate the sensors according to the known load and the actual stiffness of the rail support.
- systems based on the BWIM method, where the bridge itself with the strain gauges is used to determine the rail weight, axle spacing and axle load. The structural behaviour of the bridge should be clearly known.

The train braking or traction can be measured by the application of multiple strain gauges on the rail web, placed vertically. When the distance between sensors is known then acceleration can be derived from the time, needed to cross the sensors by the first axle of the train.

The traffic monitoring can be accompanied by the deployment of acceleration or velocity sensors, so that the bridge response can be compared with the limits, given in the Annex A2 of EN 1990.

It is also possible to add a digital camera, and based on the excitation of a first sensor, record a train passage. The images can be used to process and evaluate the type of the locomotives and carriages.

2.6.3.3 Pedestrian bridges

The traffic on the pedestrian bridges is monitored usually to check the dynamic behaviour of the bridge. That is why acceleration sensors are commonly used in the location, where the maximal acceleration can be expected, based on the modal analysis results.

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2.6.4 Other loads

Due to their influence on the safety and usability of the structure, special attention should be given to loads induced by construction works. Two categories of such operational loads may be distinguished:

- connected with the construction works directly affecting the existing structure;
- connected with construction works indirectly affecting the existing structure.

In the case of construction works of dynamic nature such as demolition works, tunnelling, pilling, heavy duty machines operating, etc., it is recommended to supply the structure with the dynamic monitoring for the time of the works. Monitoring may rely on existing monitoring system or can be based on temporary dedicated set-up.

The actual static scheme and the distribution of the masses should be taken into consideration while monitoring the dynamic behaviour of a structure directly subjected to construction and demolition works.

2.7 Number and location of monitoring sensors for local and global methods

Selecting an appropriate sensor arrangement is an application-specific task, where the procedure for sensor placement critically depends on whether local or global methods are pursued (see section 2.2). The sensor locations are generally limited due to operational conditions and interfering signals, limited accessibility to local components, and visualisation requirements. The number of sensors depends on the required damage sensitivity and localisation resolution, as well as on the acquisition and maintenance costs. If numerical models are available, the sensor placement can be optimised based on mathematical performance criteria.

2.7.1 Local Methods

Local methods assume that damage directly affects the measured response quantity, so the sensor's proximity to the anticipated damage locations is pivotal. The sensor placement derives from the analysis of anticipated damage mechanisms and the prediction of likely damage locations. The damage mechanisms can be predicted based on the experience with similar structural systems (archetypical damage reports) and, in case of structural damage, analysed in detail based on accumulating strains and stresses in FEMs. In combination with well-known loads, fatigue considerations can be made. The subsequent step is to analyse the structural response to the anticipated damage, and to select the most sensitive measurand and PI. Every component requires one sensor system at a location that is appropriate to measure the data necessary to derive the value of a selected PI. In many cases, not all components can be instrumented and a prioritisation is necessary, for example, based on the accessibility during site inspections and the consequences of damage on the safety and serviceability of the structure.

2.7.2 Global Methods

Global methods such as static or dynamic tests, measure parameters that are affected by the global state of the structure. For instance, dynamic tests are based on the premises that damage alters the mass, stiffness, or energy-dissipating properties of the structure, which in turn alter the measured dynamic response of the global system. To identify the existence of damage the sensitivity of modal parameters (e.g., modal frequencies) to changes of these structural properties is more relevant than the proximity to the examined damage locations thereby the damage location does not need to be known a priori and few sensors may suffice to monitor the entire structure to detect damage. More details about global dynamic methods will be provided in Chapter 3.

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2.7.3 Optimisation methods.

Due to the nature of the sensor placement problem, deterministic optimisation methods (based on Hessian matrices) are rarely used. Sequential sensor placements strategies, where subsets of sensors are systematically added or removed, cannot guarantee that the optimal sensor configurations are found, because the solution depends on the initial sensor choice and the number of sensors that are removed/ added in each iteration. Appropriate optimisation methods combine deterministic and stochastic search elements to overcome local minima and dependencies on the start condition. Such metaheuristic methods include genetic algorithms, simulated annealing, ant colony and bee swarm algorithms, mixed variable programming, and others. An important criterion when choosing an optimisation method is its capability to optimise multiple performance criteria simultaneously*.

Number of sensors: If the budget allows for a large number of sensors, it is sensible to choose a performance criterion that includes information on the optimal number of sensors. Alternatively, cost functions and approaches based on Pareto optimisation can help to compromise between performance and cost.

2.8 Testing data

2.8.1 Data recording frequency / acquisition

The number of tests and/or frequency of data acquisition or the number of monitoring spots/ sensors should be defined in accordance with the required level of confidence of the measurement. In addition, knowledge about mean values or standard deviations from pre-tests or pre-monitoring studies should be considered.

2.8.2 Statistical processing of testing data

The aim of statistical processing is to account for the uncertainty in the estimation of a parameter of interest, such as a material property, using a number of results to estimate parameters that can represent its distribution.

Usually, data acquisition is limited to a subset of the population, i.e., a sample. Quantitative conclusions are based on the sample, which is described by a random vector, assuming independent, identically distributed components. One result can be referred to as an observation and all the results as a measurement series. If an assumption has been made about the distribution of the observations, the parameters of the distribution of the population can be estimated using estimators. An estimator is considered as a function of a sample (the sample mean, for example, may be an appropriate estimator for the expected value of the population). The suitability of an estimator can be evaluated by requiring certain properties, such as unbiasedness, efficiency, or sufficiency. In metrology, either the value of the measurand is estimated (point estimator) or an interval containing the measured value with a degree of confidence is determined (interval estimator).

The concept of parameter estimation consists of the definition of a distribution type, the point estimation, and the verification of the results by comparison with competing distribution types. Interval estimators, in turn, specify in which interval the value of a parameter is realised referring to a previously defined certainty (in order to determine the precision of an estimate of a parameter). For this purpose, the confidence interval is often used in statistics, and the coverage interval in metrology.

* Case Study 6.11.

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The method of moments and the maximum likelihood method have been found to be particularly useful in engineering applications, provided that the distribution type is known, although the maximum likelihood method provides equivalent or better estimators than the method of moments. As an alternative, the Bayesian updating method can be applied to incorporate the results of inspection, testing and monitoring in "a priori" known distribution[108], [109], [110].

Physical reasoning, a review of histograms, or statistical hypothesis testing may be performed to evaluate the goodness of fit of an estimated model regarding a number of observations and to evaluate the suitability of the defined distribution type, respectively. A statistical test indicates how well a certain model performs, but not whether and which other models are better suited. It is recommended not to remove outliers from the sample without objective evidence that no information content about the population can be attributed to them.

The choice of a sample depends on the population of interest. Observations are to be selected randomly if inferences are intended to be drawn from the sample to the population. Systematic measurement errors must be identified and corrected. Statistical inferences must be based on independent, identically distributed (iid) observations. If additional measurements on objects other than the investigated structure (e.g., laboratory measurements) are required to achieve the test objective on-site, the permissibility of transferring the results to the structure shall be ensured and justified.

The statistical evaluation of a finite measurement series does not lead to the true value of a parameter of a random variable. Therefore, the statistical uncertainty should be estimated, and the number of observations increased if necessary. The number of observations necessary to obtain sound population parameters cannot be stated generally. In the case of a normal distribution, a rule of thumb of 30 observations can be often found in the literature.

2.8.3 Basic and nominal variables

A basic variable can be defined as an input variable in reliability assessment, which is modelled as a random variable representing a characteristic (e.g., a material property or a geometrical dimension), the effects of actions, or other quantities considered relevant about a system or structure.

The stochastic modelling of basic variables requires that relevant, reliable information on the quantity to be represented is available. Fundamental challenges in modelling include, but are not limited to correlation quantification, tail-sensitivity-problem, comparability of the models, and quantification of all relevant types of uncertainty[111], [112].

2.8.3.1 NDT-supported stochastic modelling of basic variables:

Both the values and the validity of calculation results, such as the probability of failure, depend essentially on the quality and quantity of the available knowledge about the considered system. Additional information can be acquired and processed at certain costs to increase the level of approximation of the computational models. The incorporation of observations made on site facilitates a more realistic assessment. Besides the comparability and verifiability of a stochastic model based on measured data, it is required to quantify and cover the uncertainty associated with the information acquisition and processing, i.e., measurement uncertainty.

Based on the definition of the limit state(s) and on the modelling of the initial basic variables, sensitivity analyses are performed to derive the relevance of the measurable quantities and the measurement specifications. This serves for the demand-oriented (efficient) definition of measurands and measurement procedures. The calculation of the measurement uncertainty is the main part in data analysis and serves to assess the quality of the measured information and to ensure its comparability. The measurement result, i.e., the value of the measured quantity and the attributed combined standard measurement uncertainty (see section 2.9.1), define the distribution of the quantity of interest. When transferring the measurement result into a basic variable, the statistical uncertainty (see section 2.8.2), the appreciation of prior knowledge, and the effects of competing models should be considered besides measurement uncertainty.

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2.9 Data quality

2.9.1 Measurement uncertainty

Measurements aim to determine the value of a measurand and thereby to generate knowledge. The (purely theoretical) true value of the characteristic of interest fundamentally remains unknown. A measured value approximates this value. Consequently, measurement uncertainties must be specified in order to evaluate the quality of the measured information. Measurement uncertainty cannot be eliminated even when measurements are conducted carefully and reflects the insufficient knowledge about the measured quantity value. A measured quantity value to which no measurement uncertainty has been attributed is useless.

Measurement uncertainty calculation serves to quantify the quality of a measurement result and to ensure its comparability. Thereby, confidence in measurements is established. Measurement uncertainty is a "non-negative parameter characterizing the dispersion of the quantity values being attributed to a measurand, based on the information used"[113]. Measurement uncertainty is composed of the random measurement errors, of the uncertainty in estimating identified systematic measurement errors, and of the unknown part of the systematic effects. The estimate of the systematic errors is referred to as bias and must be corrected.

The Guide to the Expression of Uncertainty in Measurement (GUM) framework consists of the main document: "JCGM 100:2008 Evaluation of measurement data — Guide to the expression of uncertainty in measurement"[114], its two supplements and further accompanying documents. The GUM framework is internationally harmonised, broadly applicable and provides the accepted rules for measurement uncertainty calculation. The definitions are given in the International Vocabulary of Metrology (VIM) [113].

In order to discuss how to quantify the uncertainty, let's indicate with Y a function of a number of input quantities X_i that are either necessary to compute Y and directly measurable or affect the outcome of the measurement. The relation can be described by an explicit model equation in the form $Y = f(X_i)$. In a second step, the identified and relevant input quantities must be modelled (stochastically). This quantification of input quantities can be done in two equivalent ways. Either measurement series are evaluated using statistical methods, provided that the n observations are independent (Type A evaluation). Often, in this case, the arithmetic mean corrected for the bias is the best estimate $\hat{x}$. The dispersion of $\hat{x}$ is described by the standard measurement uncertainty $u(\hat{x})$, i.e., the square root of the variance of the mean of the measured values. Note, that $u(\hat{x})$ corresponds to the standard deviation of the mean and not to the commonly used standard deviation (of the single values). If the observations are iid (cf. section 2.8.2), it is often assumed in Type A evaluation that an input variable is normally distributed. The arguments are based on the central limit theorem.

The second way to quantify an input quantity, the Type B evaluation, allows the determination of the standard uncertainty without using statistical methods. Instead, scientific evaluation or physical reasoning is permissible, which can be based on all available information, e.g., in form of manufacturer information or expert knowledge, about the variability of X_i. Thus, it is possible to incorporate (subjective) prior knowledge into measurement uncertainty calculation. Compared to Type B evaluation, the application of statistical methods (Type A) can lead to more unreliable results, if the number of observations is limited. Subsequent to the formulation of the measurement model consisting of the model equation and the stochastic models of the input quantities X_i, the measurement result representing the measurand can be calculated. For this purpose, the measured quantity value ($\hat{y}$) is determined by inserting the best estimates of the input quantities $\hat{x}$ into the model equation. The measurement uncertainty, in turn, expressed as standard deviation, is calculated applying the Gaussian error propagation law to the model equation and is called combined standard measurement uncertainty $u(\hat{y})$. An optional step is to convert $u(\hat{y})$ into the expanded measurement uncertainty $U(\hat{y})$ by multiplying $u(\hat{y})$ by a coverage factor k. The value of k depends on the intended level of confidence, i.e., the coverage probability p, and the distribution of Y. The values of k in case of a normal distribution are tabulated in common standard references depending on p.

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2.9.1.1 Expression of measurement results:

Two equivalent options are suggested. Either the measurement result is composed of the (estimated) measured quantity value $\hat{y}$, the combined standard measurement uncertainty $u(\hat{y})$, the expanded measurement uncertainty $U(\hat{y}) = k \cdot u(\hat{y})$, the coverage factor k, the coverage probability p, and the underlying distribution type of the measurand Y. Based on this information, the coverage interval $\hat{y} \pm U(\hat{y})$ can be described and interpreted unambiguously. Such an interval contains the true value(s) with a defined probability p. Alternatively, the measured value $\hat{y}$, the combined standard measurement uncertainty $u(\hat{y})$, and the distribution type of Y can be specified. Independent of the chosen expression of the measurement result, all information used in the calculation or related to the calculation must be documented comprehensively and transparently in order to maintain verifiability, deniability, and thus objectivity. Incorporated correlation should be indicated.

A useful part of the linearisation of the model equation for the calculation of $u(\hat{y})$ is the computation of the sensitivity factors performing sensitivity analysis. These are calculated from the partial derivation of the model equation to the individual input quantities X_i, from the uncertainty balance, and, analogous to the reliability analysis, indicate the relevance of the individual input quantities.

2.9.2 Reliability of test methods

In addition to the measurement uncertainty, the reliability of the test methods is essential to assess data quality especially for qualitative tests. The purpose of NDT is often to localise and to classify defects of components or an object. The questions are, if the defect of interest can be detected or not and if yes, how reliable is the defect detection. The generally accepted parameter to quantify the measurement reliability of qualitative tests is the Probability of Detection (PoD). Since the PoD is dependent on the defect size, the POD is often expressed in terms of a PoD curve with regard to the defect size.

Several approaches enable the evaluation of POD curves, the most common and simple ones are the so-called Hit/Miss model and the a vs. â model. The Hit/Miss model is suitable if the NDT provides only the results of the test, e.g., defect detected or not - which is typical for visual testing. If the testing system provides a signal, the signal-response POD is applicable, in which the evaluation relates the defect size (a) to the indication / signal-response ($\hat{a}$). Th evaluation of POD requires a certain number of data, which can be based on practical demonstrations on specimens enriched by numerical simulations[115], [116], [117].

In metrology, verification is considered as provision of objective evidence that a specific item, such as a measurement procedure, meets certain requirements (e.g., a maximum permissible measurement uncertainty). The validation additionally assumes that the specified requirements are appropriate in the considered case. Validation can be used, for example, to assess whether a specific measurement procedure is fit for the intended purpose. Either certain values of a measured characteristic (e.g., a minimum material strength) or permissible uncertainties could be demanded. These requirements are derived from principally flexible validation criteria such as a minimum level of approximation of an assessment model to be refined or a minimum robustness in the assessment of existing structures using the measured information. It should be noted that measurements are often conducted when knowledge of the characteristic to be represented by the measured data is insufficient for the structural assessment. The specification of required values of the measurand could thus not be suitable to evaluate the capability of a measurement procedure.

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3 Condition Assessment

This chapter describes the use of the information collected on the basis of the testing and monitoring techniques described in Chapter 2 for condition assessment of RC structures. The conceptual framework described in Chapter 3 is summarised in Figure 3-1 and relies on derivation of a set of indicators within the context of functionality (serviceability) and safety. This Chapter is limited to the methods and tools pertaining to the step of evaluating the current condition (diagnostic step). The further use of these indicators for modelling structural performance and for forecasting the remaining useful life and reliability, correspond to the prognostic step, which will be tackled in Chapter 4.

The chapter opens with an overview of the indicators used to describe the state of and the loading on the structure. Indicators are classified according to the method used to collect the information, namely visual inspections, NDT, and static and dynamic SHM – and the level of the assessment, which can comprise a local or global nature. A major part of this chapter is dedicated to methods and tools that are applied in the SHM context, which relies on continuous tracking of structural response, typically in the dynamic (vibration-based) sense. Different levels of damage detection are considered, from mere identification to localisation and quantification, as well to the updating of models from data. The influence of Environmental and Operational Conditions on DIs, which can often affect a robust assessment, is described at the end of the chapter, together with the issues involved in the definition of thresholds for reliable assessment of structural condition in terms of relevant limit states.

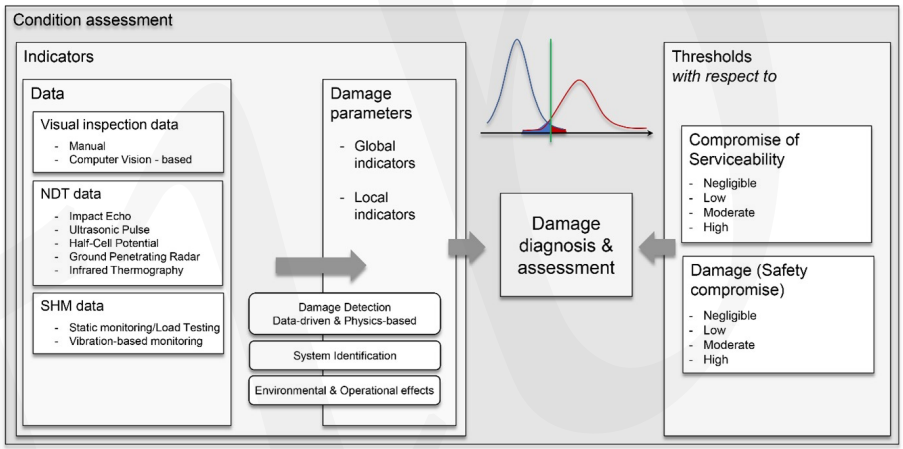


Fig. 3-1 Concept and framework of Chapter 3; Extraction of Damage Indicators (DIs) and definition of thresholds for condition assessment.

3.1 Introduction

Structures and their components deteriorate in time due to a diversity of factors, including ageing, repeated loadings, environmental actions, extreme loads, natural hazards and anthropogenic actions. As a result of these effects structural safety and/or serviceability are compromised, leading the structure to a condition limit state in which the design requirements are no longer fulfilled[118]. Condition assessment relies on interpretation of the information that is available regarding the state of the structure, or of its structural components, with respect to relevant condition limit states. ULSs are those concerning the safety of the structure and of its users, whereas serviceability limit states concern the functionality of the structure and its components under regular (operational) use.

This chapter was written by Andrade, Chatzi (main author), Darò, Gentile, Górski, Limongelli (main author), Oliveira, Mancini, Shan, Sousa, Ueda, Verstrynge, Zabel.

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Traditional methods for condition assessment are based on the acquisition of information through visual inspections and the use of condition scores based on the experience and background of the assessing inspector. Condition assessment practice differs quite significantly from country to country, however, condition scores are generally calculated based on the condition of structural elements and on the service provided by the structure, to account for the indirect consequences related to the loss of functionality[119]. Condition thresholds that define the condition rating (e.g., good, acceptable, deteriorated, bad and alarming) are also reliant on expert experience.

Such an approach, based on visual inspections and condition ratings, still forms the 'de-facto' tool for condition assessment all over the world. A survey of condition rating systems can be found in references[119], [120]. Inspectors infer the cause of possible degradation and damage from observations based on their judgment and knowledge. This approach has several advantages. It provides a general overview of structural condition at relatively low cost, uses simple tools, and does not always require previous knowledge of the potentially critical locations. However, it also has several drawbacks. It cannot acquire data in inaccessible areas, it lacks sensitivity to faults that cannot be detected by the human eye, and, not least, it may be affected by the subjectivity of the inspector. Furthermore, inspections may become costly with the increase of the level of detail and the frequency of inspections[121].

On the other hand, as described in Chapter 2, measurements provide an additional source of information in the objective form of measured data; a trait which can be exploited to increase the reliability of the assessment. The acquisition of measurements can be carried out via diverse complementary techniques, such as load testing (which primarily comprises a static approach), Destructive and Non-Destructive testing (DT and NDT), as well as short-term and long-term automated monitoring (SHM), which could refer to both dynamic (vibration-based) and static quantities (e.g., strains or deflections).

This chapter will focus on this latter suite of methods for condition assessment; these generally adhere to a stepwise process requiring:

1. Extraction of damage parameters, also referred to as damage sensitive features, from data gathered from NDT and SHM;
2. Computation of DIs from these Damage Features (DFs); these should additionally feature robustness against effects of operational and environmental variability;
3. Assessment of the structural condition through comparison of the derived DIs with thresholds, which define different condition limit states.

This chapter overviews the important aspects related to the analysis and interpretation of data that are gathered through surveying/inspection, NDT and continuous SHM for assessment of the current structural condition, or state. The aspect of prognosis of the evolution of the structural condition forms a focus of the subsequent Chapter 4.

As part of this chapter, the following distinction is made between the main classes of approaches for evaluating the condition and performance of RC structures[122]:

- Empirical schemes – these schemes do not include analysis procedures. They pertain to use of visual observations, information from measurements in their raw form (e.g., deflection/strain measurements), and analysis of the engineering drawing and specification documents.
- Scientific schemes – these schemes rely on use of more refined analysis procedures (e.g., Structural Identification (St-ID), diagnostic and prognostic modules relying on statistical processing, time series analysis, and/or machine learning, fusion of experimental data and models via the process of updating).

Critical to the distinction between these schemes is the type of technology used. Measurements that are indirectly linked to structural condition ("state"), such as ultrasonic or vibration-based measurements cannot be straightforwardly linked to empirical schemes and will thus require what is termed the "scientific" approach.

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Regarding available technologies, common examples of sensors used to gather continuous information on structural condition are sensors measuring vibrations (accelerometers, geophones, seismometers), conventional/vibrating-wire strain gauges, and FOS measuring strains, as well as piezoelectric sensors, which are increasingly used for guided-wave based assessment. Targeted pathologies as those pursued using NDT methods, as for instance the monitoring of moisture content or corrosion, are carried out with dedicated sensing solutions, measuring electrical resistivity, corrosion rate or potential[123]. Recent sensing technologies that are worth noting are further oriented toward use of smart conductive materials, such as cementitious carbon-based nanocomposites (graphene, CNT, etc.)[124], [125], self-sensing fibre-reinforced composites[126], [127], smart aggregates[128], [129], as well as 2D and 3D printed solutions[130], [131], [132]. Requirements for the data collection and transfer may differ based on the analysis method and on the parameters to be estimated. Different monitoring devices and data loggers come with different levels of sensitivity, resolution, precision, and signal to noise ratio, while it is further necessary to ensure weather and chemical protection. Other aspects to be considered are environmental influences and stability of the measurement quality, data storage issues and possible data transfer delays (e.g., for wireless sensor nodes), all of which form key points in the case of continuous SHM, but are less critical under a periodic measurement regime, as is the norm for NDT assessment.

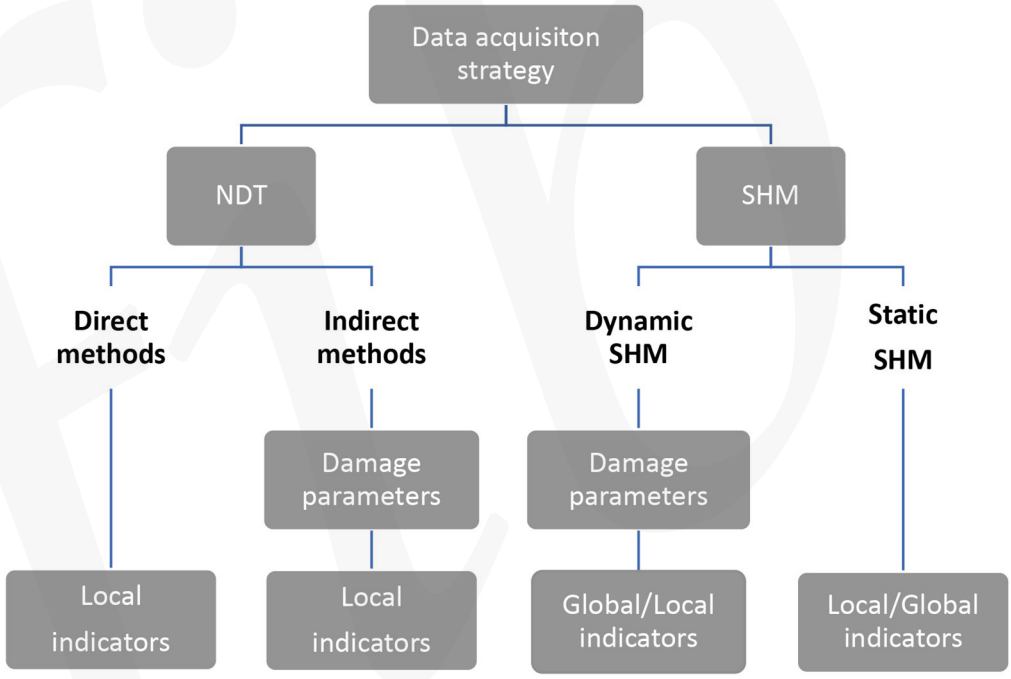


Fig. 3-2 Damage parameters and indicators extracted via NDT versus SHM.

Beyond the difference in enabling sensing technologies, a main distinction between NDT and SHM schemes, lies in the scale of the conveyed information with respect to structural condition. As presented in Figure 3-2, NDT can offer more targeted assessment in terms of localised damage phenomena, while SHM information is most typically referring to global quantities, especially when relating to vibration-based SHM. It should be noted, that monitoring of dynamic strain or acceleration has further proven capable of localised damage assessment[133]. A further defining separation of SHM versus NDT methods pertains to the frequency of data acquisition, which is carried out periodically - or on demand - for NDT schemes, versus a longer-term and continuous implementation for SHM.

Finally, the two schemes entail a different procedure in terms of data post-processing and analysis. NDT methods are typically classified as direct and indirect schemes. In the first case, damage parameters and indicators coincide with the measured quantity (measurand), as is the case for measurements of

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carbonation, chloride depth or corrosion rate, which serve as indicators of corrosion. Indirect methods additionally employ a usually physics-based model to relate measurements of, for example, temperature, moisture, porosity, or permeability to damage parameters. On the other hand, SHM methods are often indirect, as these typically require processing of acquired data for the extraction of the damage parameters. For SHM methods, the inclusion of a physics-based model in the loop results in the important separation between model-based and purely data-driven schemes, which will be discussed in this chapter. For a number of well-established SHM methods, such as vibration-based schemes, measurements normally relate to signatures of the global structural response (e.g., modal properties), which form the damage parameters. These are affected by environmental effects, requiring further processing for extraction of robust DIs.

SHM and NDT schemes thus, serve different assessment purposes and a proper combination of these approaches should be followed for ensuring the tracking of damage and degradation throughout the life cycle of a structure. Complexities, beyond the already mentioned challenge of varying EOCs, additionally stem from the possible coupling of degradation processes, which hinders a straightforward relation between DIs and degradation processes and necessitates fusion of diverse monitoring datasets.

In the next sections, after a short description of some of the damage and degradation phenomena that affect RC structures and the overview of relevant damage parameters, reported in section 3.2, the extraction of condition indicators from different assessment schemes, including inspection and vision-based schemes (section 3.3), NDT (section 3.4) and SHM (section 3.5) is described. Dependencies that may affect the assessment procedure are covered in section 3.6 and, while the procedures to define thresholds are illustrated in section 3.7.

3.2 The assessment chain from damage/loads to indicators

Structures and structural components may be affected by degradation and damage at the local or global scale. Local deterioration phenomena most often appear in the form of cracks or wear (e.g., bearings) due to corrosion, fatigue, freeze-thaw cycles, or alkali-silica reactions. Global phenomena, on the other hand, pertain to more extensive effects, possibly affecting entire structural components, as is the case of scour in bridge piers due to floods, or distributed damage induced by strong seismic shaking. In Chapter 1, a short overview of deterioration and damage mechanisms is reported.

3.2.1 Deterioration & damage indicators

Due to the different origins and acting timespan of diverse deterioration processes, a multiscale approach is typically necessary both in sensing as well as analysing extracted data for defining indicators that are capable of evaluating the effects of acting identified deterioration processes. In Chapter 1, the principal deterioration and damage mechanisms that affect concrete structures have been overviewed, along with relevant indicators and technologies that are suited for their assessment. Table 3-1 offers an example of the linkage between a particular damage mechanism to the measurand and appropriate technology, the associated damage parameter and suited DIs for the specific cases of corrosion and fatigue.

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Table 3-1 Example of the assessment chain: from damage mechanisms to Damage Indicators (DIs).

	Damage Mechanism	**Technology / Measurand**	**Damage Parameters**	**Damage Indicators**
Reinforcement Corrosion*	Corrosion initiation by carbonation	Permeability Chloride level sensors pH Sensors Electrical potential Polarisation resistance Corrosion rate sensors	Moisture Chloride presence Electrical resistivity	pH value at rebar surface
	Corrosion initiation by chlorides			Chloride concentration at rebar surface
	Corrosion propagation			Concrete resistivity Corrosion potential Polarisation resistance Corrosion rate Steel cross section loss (local or homogeneous) Concrete cover cracking level Concrete spalling level
	Fatigue	Strain gauges or indirect measurements of strain via acceleration / displacement measurements	n_{Ei}: denotes the number of acting stress cycles associated with the stress range for steel and the stress level for concrete. N_{Ri}: denotes the number of resisting stress cycles at a given stress range for steel and stress level for concrete	accumulated fatigue damage $D = \sum_{i=1}^{j} \frac{n_{Ei}}{N_{Ri}}$

3.2.2 Load(s) indicators

Both the sensors and the structural system itself are affected by environmental (e.g., temperature, humidity, wind) and operational (e.g., traffic) sources of variability of DFs and indicators (Table 3-2). This is particularly true for long-term SHM campaigns. It is thus beneficial to include additional sensors in a monitoring deployment, which can measure environmental effects, such as thermometers, anemometers and pyranometers. Complementary to the measurements targeting the recording of structural response are thus measurements which relate to the monitoring of loads and / or environmental influences. In certain cases, such measurements can also be used to derive exposure / hazards indicators to use in reliability or risk assessment procedures or to develop loading models to integrate in prognostic assessment. The measurement of these quantities is also relevant for the task of regularisation versus acting EOCs, which is addressed in the next Chapter 4.

Table 3-2 Loads Indicators (after Table 15 of the study[134]).

Class	**Measurand / Indicator**	**Technology**	**Measurement Purpose**
Environmental Conditions	Chloride Content	X-ray fluorescence Titration Photometry	Critical for assessment of corrosion and scour effects
	Temperature Humidity Moisture	Weather Station	Critical for condition normalisation, particularly in case of long-term monitoring

* Case Study 6.19 of Chapter 6 describes the use of NDT for prediction of carbonation-induced corrosion of Tokyo's National Museum of Western Art.

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Loads	Tensioning Force	Load Cells	Assessment of condition of tendons, cables and anchors
	Traffic	WIM System	Assessment of actual loads on the bridge and updating of the corresponding models and calculated PIs.
	Wind	Weather Station Anemometer Meteorological Mast	
	Earthquake	Seismometers, Episensors	Critical for seismic damage assessment

3.3 Visual inspection-based condition assessment

Visual inspection forms the default means of condition assessment for concrete structures and naturally lies in the class of empirical assessment schemes. This type of assessment is obviously dependent upon the training and experience of the expert, while a further ambiguity is due to the fact that no clear correlation exists between component level assessment and the rating attributed to the object (structure) as a whole, in terms of a condition score; these require expert judgement. Table 3-3[134] is overviews the rating systems adopted in different European countries for the case of bridge structures.

Table 3-3 Bridge condition rating in different countries - Table adapted[134].

Austria		**Croatia**		**Norway**		**France**	
Grade	Condition	Class	Condition	Class	Condition	Class	Condition
1	Very Good	0	No damage	1	Small damage	1	Good overall state
2	Good	I	Smaller Defects from Construction period.	2	Medium damage	2	Minor structural damage. Non urgent maintenance needed.
3	Satisfactory	II	Smaller Defects from Exploitation period.	3	Large damage	2E	Structure deterioration. Urgent maintenance needed.
4	Faulty	III	Defects that in long term decrease durability	4	Critical damage	3	Serious Structure deterioration. Urgent maintenance needed.
5	Bad	IV	Defects that in Foresseeable future decrease reliability.	Defects are categorised by using the following system of letters and numbers and combined with the above classes: M = Environment, B = Load capacity, C = Traffic safety, V = Maintenance cost.		3U	Structure deterioration. Urgent maintenance needed.
		V	Defects that present serious danger to safety of traffic.				
No clear correlation between component (element) condition rating and object (structure as aa whole) condition rating		Structure level condition is not determined from the above classes but from the influence of each element functionality on traffic safety, mechanical resistivity, stability, durability and general condition of element. Structural level is then determined by combining maximum elements level grades.		The element condition is related to the number of years before maintenance is needed and the condition rating is not levelled. The structure condition is quantified by calculating a character using condition from the element condition.		For the structural level condition rating number of levels and categories are the same as for element level: 5 levels, 1, 2, 2E, 3 and 3U, and the structure levels is the maximum of all element levels.	
Probabilistic (details given in Task 1 report)		None		None		Probabilistic in nature (Poisson's function is used)	

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3.4 NDT-based condition assessment

Another assessment method is delivered by non-contact, e.g., ultrasonic[135], Infrared Thermography (IRT)[136], or GPR*[137] technologies, which were already listed in the Annex (Table A-2). These can be used for drive-by, also referred to as on board, monitoring. In this case, assessment is delivered by means of properly equipped, or robotic vehicles. Certain NDT methods deliver direct indicators or parameters relating to damage, e.g., crack size or path detected by means of dye penetrant testing or Time of Flight Diffraction[138]. However, for a variety of NDT methods, including those related to wave-based assessment, indirect DIs can be obtained by means of appropriate post-processing. Typical processing schemes include 1D signal or image processing operations, such as wavelet-, filtering-, and cross-correlation based methods, or split-spectrum processing, as elaborated in a previous study[139]. As these approaches use similar processing to what is subsequently elaborated on SHM-based methods, we offer a relevant elaboration on processing is discussed, in the following Chapter 4.

3.5 SHM-based condition assessment

SHM describes a continuous process of measurements acquisition and automated processing in order to extract indicators of structural condition[140]. In this sense, it falls in the class of scientific, as opposed to empirical, means for condition assessment. Although monitoring may also be carried out over shorter time periods, the concept of SHM involves long-term sensor deployments on a structure. Based on the specific application and on the monitored phenomena, a multiscale and multi-parameter monitoring approach generally favours a more robust system identification and condition assessment.

Uncertainties relating to the sensing modules (signal-to-noise ratio, cabling noise, A/D conversion, outliers, environmental effects on sensors) and analysis methods (partial information, aleatoric and epistemic uncertainties) should be considered in the St-ID and condition assessment process. Particular attention is to be paid to the influence of the environmental conditions on the collected data. As will be elaborated upon in section 3.6, temperature and humidity often influence DIs and therefore procedures to account for these effects are crucial to appropriately characterise the structural behaviour and to avoid that such benign changes introduced into the system are interpreted as damage occurrence[141]. In summary, a complete condition assessment process relies on sound data treatment, reliable St-ID procedures, as well as coupling with expert engineering judgement, oftentimes supported by updated numerical models of the structure.

3.5.1 Static monitoring

Methods intended to provide information with respect to the static behaviour of a structure relate to both short- and long-term monitoring methods. For instance, the evaluation of relative or plastic deformations may be achieved both by means of a topographic network with measurements repeated in fixed time intervals (as elaborated in section 3.3), but also via a more refined continuous monitoring system comprising sensors able to monitor movement, displacement or rotations of the structure[142]. Similarly, for the task of crack detection, a defect can be quantified and evaluated via direct measurements, e.g., by means of periodic or one-off non-destructive tests (e.g., ultrasonic velocity tests, hammer sounding) or by means of a (continuous) SHM system comprising displacement transducers that monitor crack propagation over time and/or new cracks formation. The advantage of utilizing a continuous SHM system lies in the possibility to detect damage processes prior to their visual manifestation and to provide near-real time alerts when the deterioration mechanisms evolve. An example of long-term static monitoring for the case of tunnels is offered in Chapter 6**.

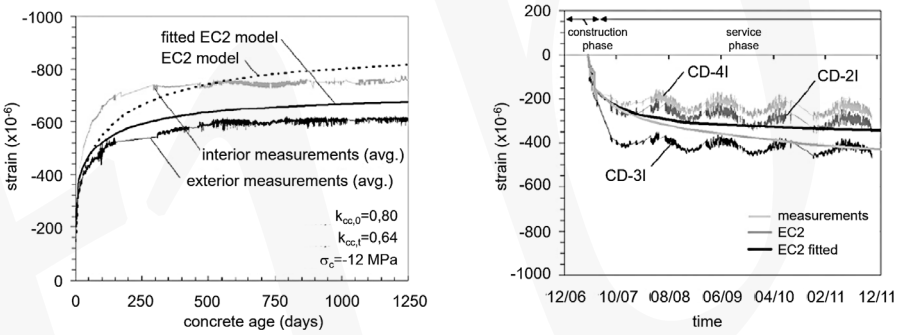
* Case study 6.10 and 6.21.

** Case Study 6.13, where global deformations and local rotations (using MEMS inclinometers) were tracked.

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As a further means to obtaining direct PI from static monitoring one could also consider the diagnostic and proof (static) load tests, typically conducted under application of increasing weight (truck load, concrete blocks, etc.), which were already overviewed in Chapter 2, as a typical class of testing methods. It should be noted that the combined use of proof load tests (static deformation shape) with continuous dynamic response monitoring is shown to significantly improve FEM updating procedures (as described in section 3.5.2.2.2).

Complementary to load tests (see Chapter 2), long-term static monitoring allows assessment of further parameters with important influence on the structural behaviour of concrete structures. Particularly concerning creep and shrinkage of concrete, it is worth noting that despite availability of relatively recent and comprehensive models, there is a lack of consensus on their utilisation due to substantial scatter in their predictions. Figure 3-3 illustrates an example of the importance of accurate modelling of creep deformations (plot in the left) in the structural behaviour over time (plot in the right). Beyond the accuracy in tracking the magnitude of deformations, it is the trend over time that can result in significant difference in the expected deformations/deflections for a service period (e.g., the impact on expansion joints and bearing devices). For illustration, in the results presented below, creep deformations are overestimated by 21% at the end of 1'250 days (i.e., ≅ 3.5 years) with a subsequent impact on assessment of structural deformation by around 11% for that same period. If creep is not correctly assessed, the increase of deformation with respect to what is expected can be falsely interpreted as damage or deterioration. Thus, similarly to the need for removal of environmental and operational effects, prior to application of damage-detection techniques, a corresponding procedure should be applied for creep effects.



(a) Creep deformations (b) Structural deformations

Fig. 3-3 Influence of accurate modelling of creep deformations[143].

Besides the issues mentioned about creep and shrinkage, laboratory tests revealed that the thermal dilation coefficient of concrete might differ significantly from the values stipulated in the codes. For instance, in a previous study[144], it was found that this coefficient was approximately 7.9×10^{-6} ºC^{-1}, which is 21% lower than the 10×10^{-6} ºC^{-1} specified in Eurocode 1 – EC1. In this context, long-term (static) monitoring can reveal unique information regarding:

- the creep and shrinkage patterns based on the analysis of collected measurements mainly from strain-gauges installed along the spans and supports, displacement transducers in bearing devices, and vertical displacements measured by suitable sensors' system.

- the thermal dilation coefficient based on the analysis of measurements from displacement transducers in bearing devices by analysing the correlation with temperature. For this, it is important to primarily use the uniform temperature component and to consider short periods (e.g., days/weeks) to avoid the influence of time-dependent effects related to creep and shrinkage of concrete, and steel relaxation.

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3.5.2 Vibration-based monitoring

A broad class of monitoring methods, the so-called Vibration Based Monitoring (VBM) methods, rely on dynamic monitoring data, such as acceleration or velocity records. VBM schemes operate on both forced, free and ambient vibration data, however - in the context of SHM - ambient vibrations are generally employed[145]. VBM methods are based on the premise that damage induces a shift in the stiffness, mass or energy dissipation capacity of the structure. The vibrational response is analysed by means of system identification methods to characterise the system or, in other words, to identify a mathematical model able to describe the structural behaviour. The parameters of this model can be physical properties, such as for example stiffness and mass properties that are directly linked to modal parameters, or statistical properties of the response to vibration, such as for example the coefficients of a regression function. Among these parameters, those sensitive to the variation of the dynamic characteristics of the structure, can be assumed as damage parameters or features and the indicators of damage can be defined in terms of a demonstrated shift from a reference condition. It must be noted that environmental changes, related for example to temperature, humidity, wind speed, etc. as well as operational changes due to the variability of the traffic loading on bridges or, more generally, the changes in applied loads and/or masses may affect these DFs. Their effect must thus be removed or accounted for in the damage detection procedure in order to avoid false or missing damage identifications. The treatment of environmental and operational effects can be carried out during the system identification procedure or can be included in the algorithms of damage identification. A schematic illustration of the process of VBM for condition assessment, from the stage of measurements collection to the stage of damage identification (diagnosis), is shown in Figure 3-4.

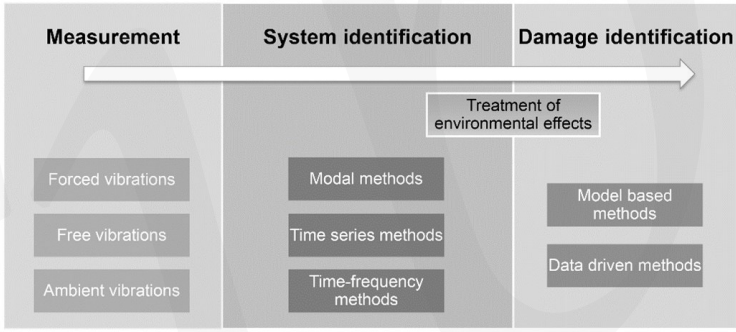


Fig. 3-4 Vibration-Based Monitoring (VBM) for condition assessment.

A more detailed classification of the methods of system identification to extract the DFs is given next. It is important to note that, due to their nature, vibration-based indicators typically provide information that is relevant to variations of stiffness, mass and energy dissipation capacity, but not direct information on the variation of structural strength. Section 3.5.2.1 offers a description of system identification methods (characterisation), while section 3.5.2.2 overviews damage identification methods (damage diagnosis).

3.5.2.1 Methods for structural identification

The term system identification, also known as St-ID, when used in the context of civil structures, forms the essence of what was in our introduction defined as the "scientific" approach to assessment. The term St-ID is used in different ways depending on the respective context. Engineering applications use numerical models such as, for example, FEMs that are directly related to physical parameters of a structural system. In these cases, the term system identification often refers to the estimation of physical model parameters, such that the numerical model best reflects the structural behaviour observed experimentally. As a result of this estimation of the actual physical parameters, a condition assessment is obtained. This procedure is more appropriately termed model updating or model calibration and is

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further elaborated in section 3.5.2.2.2. However, the term system identification more broadly pertains to the identification of a mathematical system model describing an input-output relation of a considered system without a direct relation to physical properties such as geometry or material parameters. This section discusses the use of the term in this (more formal) sense.

3.5.2.1.1 Parametric and non-parametric methods

Assessment of structural integrity, state of deterioration, or condition assessment can be performed in various ways using experimental data. In the simplest case, a measured quantity is directly compared to a respective threshold value that has been specifically defined. Another possibility is the computation of some metric that can be derived from the measured data by means of some mathematical transformations. One common example for such a metric are modal parameters identified via schemes that do not incorporate any further model. Both approaches, i.e., the comparison of measured data with some threshold and the direct identification of some metric, are referred to as non-parametric methods.

In several cases, the analysis of measured data incorporates the identification of parameters of a mathematical (and not necessarily physics-based) system model, which describes an input-output relation. From such model, further quantities, as for example modal parameters, can be derived in a second step of data analysis. Algorithms that include the consideration of a parametrised system model are termed parametric methods. In the context of parametric identification methods, the intermediate step of estimating the parameters of a system model is called system identification. The process chain from a physical test to the identification of parameters of different models is illustrated in Figure 3 5 for the example of the identification of modal parameters. In this diagram, the optional continuation of the analysis to the identification of model parameters of, for example, a FEM is also included.

The advantage of non-parametric identification methods can be seen in their lower degree of complexity, which is related to an easier numerical implementation and reduced computational costs. However, in many cases there are limitations with respect to their successful applicability, as, for example, in the identification of closely spaced modes (e.g., in structures with a rotationally symmetric cross-section). Parametric methods often allow for a more precise identification, for example of modal parameters, due to their higher degree of complexity, which causes on the other hand higher computational effort. However, the identification of several system models of different orders also offers the possibility for an assessment of the confidence or uncertainty of the identified quantities.

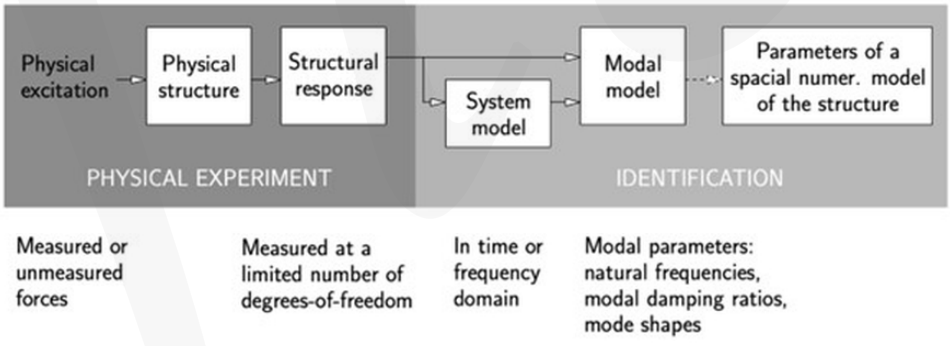


Fig. 3-5 Schematic illustration of the identification of modal parameters[146].

In relation to condition assessment of concrete structures, the most widely applied scheme in which both parametric and non-parametric methods are used is Operational Modal Analysis (OMA)[146], with some of the main OMA methods summarised in Table 3-4. While OMA is meant to be applied for the processing of output (response) signals, typically extracted from monitored structures during their ambient operation, it is closely linked to the procedure of Experimental Modal Analysis (EMA). EMA essentially comprises a testing protocol in which the structure is excited by one or several measured, i.e., known, dynamic excitation forces, often in the form of an impulse (e.g., hammer hits). The response of the structure to these forces is recorded, and the modal parameters in the frequency range of interest are inferred[147].

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Table 3-4 Examples for parametric and non-parametric methods for operational modal analysis.

Analysis domain	**Parametric methods**	**Non-parametric methods**
Time domain	Poly-reference least square exponential (p-LSCE)[148] Auto-regressive moving average[149] (ARMA) methods Covariance-driven stochastic subspace identification (SSI-cov)[150] Data-driven stochastic subspace identification (SSI-data)[151] Natural Excitation Technique (NExT)/Eigensystem Realisation Algorithm (ERA)	Ibrahim time-domain (ITD) method[152] Random decrement technique (RDT)[153] Principal Component Analysis (PCA) /Blind Source Separation Methods
Frequency domain	Poly-reference least square frequency domain (p-LSCF) method[154] Stochastic frequency-domain subspace method[155]	Frequency Response Function estimation Peak picking or Circle Fitting methods Least Squares Frequency Domain (LSFD) Frequency Domain Decomposition (FDD) method[156]

3.5.2.1.2 Non-linear and Non-Stationary Systems

While OMA requires the underlying assumption of Linear and Time Invariant (LTI) systems, in reality concrete structures can be nonlinear in nature (e.g., response to earthquake excitation shifting above yield/ultimate capacity, or nonlinear response of stays in cable-stayed bridges[157]). Moreover, their response is often non-stationary due to the nature of loads they are exposed to (e.g., heavy crossing vehicles for bridges[158]), or even to time-dependent effects on the mechanical properties of concrete (e.g., as creep, shrinkage and relaxation[159]). In this case, the system's response can be (excitation) amplitude-dependent, thus implying that the transfer function is no longer decoupled to the structural response, as is the case for linear systems.

Also here, a suite of methods is available for tackling such higher order complexities, which are once again separated into parametric and non-parametric methods[160]:

- Parametric methods assume a specific model structure and aim to identify the parameters included in the assumed model. An example of such a class of methods is found in nonlinear or time varying AutoRegressive models, which form an extension of the linear class listed in Table 3 5. An overview of Nonlinear AutoRegressive (AR) models with eXogenous input, as well as Linear Parameter Varying (LPV) models for structural response modelling[161], [162].

- As aforementioned, nonparametric methods are more general in that they do not impose a specific structure, with the system regarded as a black box. Several techniques have been proposed to this end, including wavelet-based approaches or methods relying on the Hilbert transform. Perhaps the simplest and more essential approach lies in utilisation of the spectrogram function (short-time Fourier Transform), which immediately delivers an overview of structural response in both the spectral and temporal domain and indicates potential nonlinearity and/or non-stationarity - as captured in the shift of modal frequencies within the duration of the analysis.

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Table 3-5 Examples for parametric and non-parametric methods for nonlinear and non-stationary systems.

Parametric methods (primarily in time domain)	**Non-parametric methods (primarily in a transformed domain)**
Linear parameter Varying Models[149] Nonlinear Auto-regressive models with eXogenous input (ARMA) methods[153] Smoothness Priors Time-Varying AR models (SP-TAR)[158]	Short-time Fourier Transform (Spectrogram) Wavelet based Analysis[163] Hilbert–Huang transform (HHT)/ Empirical Mode Decomposition (EMD)[164] Kernel PCA[165]

System identification is most often carried out with output only schemes since the input is seldom measured. However, when this is the case, for example in the case of seismic monitoring, input-output system identification approaches can be used. It should be noted that output only schemes are not only linked to vibration-based SHM. Guided wave-based methods, which can be applied within an SHM context, also admit output only treatment (with filtering and spectral transformations), while the same holds true for NDT data, provided that enough measurements are aggregated over the lifetime of a structure or structural component.

3.5.2.1.3 Practical Considerations for Structural Identification

Practical considerations on St-ID are mainly related to the noise contamination that is naturally present in the collected data, especially under ambient excitation, which often implies difficulties in distinguishing between noise (spurious) modes and physical modes. The selection of an optimal model order is here critical for robustly estimating modal parameters; a task which is achieved via stabilisation diagrams and appropriate clustering techniques[166].

3.5.2.2 Methods for damage identification

The purpose of St-ID is to infer the main system properties (e.g., modal parameters) or a model (parametric or non-parametric) that is able to reproduce the system response. Condition assessment relies on the potential to capture deviation from a reference, or "healthy" condition, which most often is the condition at the beginning of the monitoring period. DFs are characteristic properties, which may be used to capture deviations from a reference condition.

The system identification tools, overviewed in the previous section 3.5.2.1, for the case of linear and nonlinear/non-stationary systems, can now be re-examined in terms of their identified parameters, which serve as DFs, as elaborated in Figure 3 6. The Figure follows the classification used in the system identification section, namely in terms of frequency (modal), time, and time-frequency domain schemes.

DFs assume various forms, depending on the task at hand. These could relate directly to the monitored variables (e.g., strain) measured by means of dedicated sensors or tests, or to more indirect properties, such as continuous response functions or modal parameters. DIs, for example the change of the frequencies or mode shapes, are the metrics that quantitatively describe the presence of damage. Depending on selected DIs, more refined damage measurements may be required together with visual and/or in-depth inspections. The choice of the indicators should be such that their combined evaluation represents a univocal, exhaustive, and as much as possible unambiguous interpretation of the actual structural behaviour[167].

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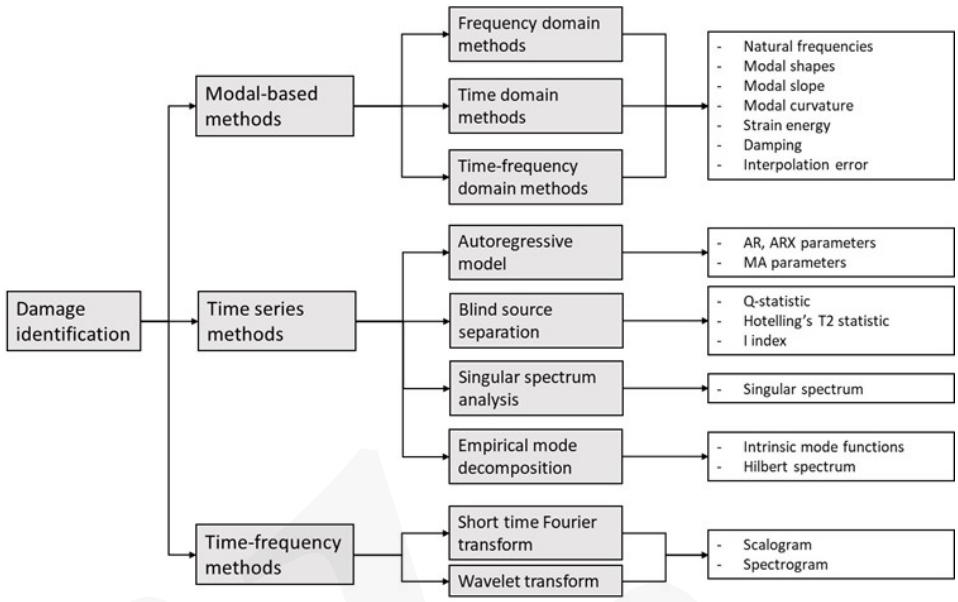


Fig. 3-6 Methods for Sructural Identification (St-ID), along with their corresponding Damage Features (DFs).

VBM methods for damage identification allows assessment of changes in the dynamic behaviour of the structure, e.g., linked to stiffness reduction or added mass to a system (e.g., due to icing) or change in energy dissipation capacity. Methods for vibration-based damage identification are classified in two main categories, namely data-driven and model-based approaches.

Data-driven methods extract DFs relying solely on the recorded response and on picking up shifts in damage sensitive features, with respect to those corresponding to regular operating conditions. These methods are hence attractive for adoption within real-time damage identification but the absence of a numerical model of the system often hampers the estimation of damage severity.

Model-based methods most typically rely on the use of the DFs to update significant parameters of a numerical, often FEM. They are usually less attractive for real-time damage identification, due to the computational cost of the updating process, but the availability of a FE model allows for higher-end damage identification and, if a degradation model is available or can be identified based on monitored data, they can further provide information regarding the remaining service life of the structure.

On the purely data-driven front, the advent of advanced and increasingly affordable monitoring technologies has allowed for continuous monitoring of structures based on use of heterogeneous sensors, which are able to deliver a "Big Data" stream of monitored information. Naturally, the treatment of this data requires suitable processing tools, which are typically tied to data-driven time-series analysis, statistical methods and Machine Learning (ML) schemes, e.g., Fuzzy Neural Networks (FNN) or Support Vector Machines (SVM).

According to Rytter's hierarchy different levels of refinement exist in the assessment of damage via VBM, namely i) detection, ii) localisation, iii) quantification and iv) remaining life prognosis[168]. This chapter refers to the first three levels, which correspond to diagnostic tasks, whereas the prognostic task will be covered in Chapter 4:

- Damage detection is the lowest level of damage identification and consists in the specification of the existence of damage[169]. Data driven methods have proven capable for damage detection, which may even be achieved on the basis of a single dynamic response measurement, provided that this captures significant features, e.g., natural frequencies, that are affected by damage. The most typical example of an indicator of the existence of damage is a shift in a modal property, i.e., in the natural frequencies and/or associated modal shape values, with respect to those estimated in reference (healthy) conditions.

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- Damage localisation usually requires the analysis of responses recorded at several locations, i.e., a higher spatial resolution in the description of the structural condition but to a certain extent can still be performed with a data driven approach provided spatially distributed DFs, for example modal shapes, are used.

- Damage characterisation, i.e., the evaluation of the type and severity of damage, typically requires a model-based approach to map the monitored response to different damage types and scenarios.

In the following sections, data-driven and model-based methods for damage identification, localisation and characterisation are briefly illustrated.

3.5.2.2.1 Data-driven methods

The term data-driven is used here to denote the absence of a physics-based model of the system in the assessment procedure.

Damage Detection

As already mentioned in the introductory sections, perhaps the most typical damage sensitive features, within the context of SHM, are the dynamic properties of a structure. Among these, modal frequencies, extracted from measurements using methods operating in the frequency domain, have been used in early studies to detect damage[170]. Modal frequencies are among the first DFs employed in SHM applications due to their ease of evaluation. However, in general, natural frequencies are not suitable for detecting small damage and for localizing damage. Modal damping has been proposed as a further Damage Feature (DF) since damage is often tied in increased energy dissipation and, thus, damping. This possibility was firstly highlighted by researchears[171], [172]. However, both research groups concluded that measuring modal damping from vibration data results in high uncertainty that compromises the reliability of damage detection.

Time domain methods have been overviewed in the corresponding system identification section. For the purpose of damage detection, the indicator is provided by the differentiation of the parameters of parametric schemes (e.g., autoregressive models), which here serve as DFs. In the case of non-parametric representations differentiation is picked up in terms of the time spectra and covariance properties of the collected time series, which are then employed as DFs. Time-frequency methods have also been employed for structural damage identification, relying on two-dimensional (i.e., time-frequency or time-scale) distributions. Chapter 6* presents an overview of the use of modal frequencies as defining features for detecting damage in concrete bridge structures.

Damage Localisation

Localisation of damage, intended often as the identification of stiffness loss in a region of the structure, forms the second stage of the Rytter hierarchy[168]. Most of the algorithms pursue damage localisation through the detection of irregularities in the deflected profile of the structure. Since the appearance of these irregularities is not evident and easily identified from the direct analysis of the deformed profile, most of these methods use features more sensitive to local irregularities, such as curvature[173], [174][175] or slope[176], which are related to respectively the first or the second derivative of the deformed profile[177], [178], [179], [180], [181], [182],[183]. Most methods analyse the deformed profile using modal shapes, while other methods make recourse to Operational Deformed/Deflection Shapes (ODS).

In the following Table 3-6, a non-exhaustive list of data-driven time invariant methods is described. Each method uses a different DF, but they all define the DI in terms of localised shifts of the DF. The features used by the different methods e.g., curvature, flexibility, strain energy, uniform load surface, interpolation error and flexibility are all affected by local changes of stiffness/curvature. All such features can be estimated directly from, or through differentiation of, the modal or operational shapes identified using the system identification methods previously described.

* Case studies 6.2 and 6.3

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The computation of curvature requires the use of estimators of the double derivative of the (modal or operational) deformed shapes that are very sensitive to the noise introduced during the acquisition and processing of measurements. For this reason, methods for the indirect detection of curvature changes using wavelet or spline functions have been developed. Characteristic examples are presented in Chapter 6*.

Table 3-6 Methods for damage localisation.

VBM methods for Localisation	**Reference**
Modal Curvature method	[173]
Strain energy method	[177]
Uniform load surface (ULS)	[178]
Flexibility method	[178]
Gapped smoothing method	[179]
Interpolation method	[180]
Wavelet transform applied on ULS	[181]
Transmissibility-based methods	[182]
Damage locating vector (DLV)	[183]

It needs to be reiterated, however, that information stemming from acceleration/vibration requires advanced damage levels, which affect the global properties of a system, for ensuring detection. What is more, localised damage might be more obvious in higher vibration modes that are not always easy to identify under ambient conditions. More recently, it has been found that strain-based modal shapes are more sensitive to local damage[133] **.

Due to the impact of uncertainties related for instance to noise contamination of acceleration sensors and to the further approximations and numerical errors introduced during their processing, the extraction of reliable DFs from vibration-based data is not always straightforward. Improvement in this field is supported by the increasing precision and decreasing costs of Micro-ElectroMechanical Systems (MEMs)-based sensing solutions and using large dataset processed using Artificial Intelligence techniques.

Damage Quantification

The quantification of damage severity forms an admittedly challenging task for data-driven methods, though it can be claimed that severity can relatively be deduced from the relative variation in the utilised DFs (e.g., modal frequencies, or curvatures of mode shapes)[184],[185] Even in cases where statistical methods are used, it is possible to recover some hint on damage severity by the probabilistic distances between the features corresponding to different damaged states using probabilistic distance metrics, such as Mahalanobis distance or Kullback -Leibler (KL) divergence[186]. However, this type of quantification is only relative and offers no precise estimation on of the quantity being modified due to damage (e.g., stiffness changes due to cracking).

The aspect of quantification may also be investigated from the point of view of quantifying accumulation of damage or deterioration, as is typically the case for fatigue assessment. In this case the severity of damage can be often straightforwardly quantified from strain data, when these are available, with use of a Palmgren-Miner hypothesis***.

* Case studies 6.5 and 6.15 exemplify the adoption of modal curvatures for condition assessment of a bridge and dam structure, respectively.

** An example of the application of acceleration-based data for damage localisation is offered in Case Study 6.6.

*** Case Study 6.8.

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Machine/Deep Learning based Schemes

In recent years, due to the increase of computational power, ML or DL schemes, which rely on use of Artificial Neural Networks (ANN) with blocks of layers forming the DL skeleton, have gained ground in data-driven SHM applications (Figure 3 7). DL schemes can fuse various types of monitoring information, analyse these and infer information that lies hidden in the data, such as damage patterns (for the case of SHM).

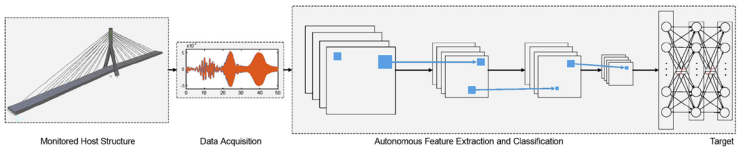


Fig. 3-7 Deep Learning (DL) based data-driven model in SHM[187].

DL/ML methods bear similarities, in their use within the SHM context, with statistical methods for SHM. In statistical methods for SHM, e.g., outlier detection, residual methods, hypothesis testing, the goal lies in defining probabilistic distance metrics (e.g., Mahalanobis distance or KL divergence) and thresholds on the basis of which damage can be detected or over classified in terms of severity. ML-based tools are similarly used in either a regression or classification mode. Damage detection is most often treated as a classification problem executed via a supervised, partially supervised, or unsupervised scheme. As elaborated in a previous study[188], supervised learning algorithms require a fully-labelled dataset for training, meaning that the training-set includes both observation data and descriptive labels (e.g., labels of faults, or types of damage) for the measured signals. The task of classification then inherently contains the notion of defining thresholds, since these are the defining boundaries of the clusters. Labelling is a crucial task for obtaining a performant ML algorithm and must be carried out by experts; thus, it still remains a time-consuming process, comprising both the strength and limitation of these techniques. Partially supervised algorithms are usually separated in semi-supervised and active learning schemes, which combine both labelled and unlabelled data. Active algorithms can interactively query a user (e.g., expert) to annotate unlabelled data for extending the original label-set. Semi supervised algorithms can exploit unlabelled data most notably via self-labelling, or use of a maximum-margin algorithm, such as the Support Vector Machine (SVM). For the first level of Rytter's hierarchy, i.e., damage detection, this can be simply achieved by a One-Class Classification (OCC) scheme, which simply looks for outliers/anomalies with respect to some baseline (healthy) data.

3.5.2.2.2 Model-based Methods

As aforementioned, in model-based methods, damage is detected, localised and quantified through the updating of the parameters of a numerical or probabilistic model[189]. Both stochastic, e.g., Bayesian[190], as well as deterministic approaches, e.g., Constrained Eigenstructure Assignment Method (CEAM)[191], can be used to this end.

When a model is inserted in the analysis loop, it is possible to more simply achieve higher-end SHM tasks, such as localisation and quantification, albeit at the cost of higher computational effort. Further to this, the availability of a model offers the potential to reach the highest level in Rytter's classification of damage identification that is damage prognosis. To this aim, a model of the future structural performance in terms of structural capacity and demand, and thereby of reliability, is needed to deliver a prognosis of the future service life. These aspects are treated in further detail in Chapter 4 of this Bulletin.

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Updating of a numerical model

The experimental data that are most typically used for the process of FE model updating are modal properties, which are typically extracted from measured response time histories by means of OMA techniques. Several types of modal data can be used; the simplest case pertains to exclusive use of natural frequencies or eigenvalues, which are known to be affected by changes in structural stiffness and can be measured accurately. Where a diagnostic load test (see Chapter 2) can be carried out, the calibration of the FE model should rely on both the static and dynamic response data acquired during the monitoring programme. Parameters extracted from measurements and computed using the FE model are confronted in a cost function, which is often expressed as weighted least squares fit between predictions - expressed in parametric form - and measurements. The optimal values of the model parameters are determined through minimisation of the cost function. Model updating is thus an inverse problem and is often prone to ill-posedness and ill-conditioning. Accounting for uncertainties due to measurement and modelling errors is therefore essential and may be accomplished within a Bayesian context[192]. Quantification is in this case achieved via the identified parameters, which typically relate to structural properties, e.g., stiffnesses. Their identification thus leads to a direct quantification of expected damage.

Updating of a probabilistic model

The most broadly applied approach for fusing models with monitoring data is Bayesian Model updating[193], which takes advantage of Bayesian statistics to treat parameters as random variables, thereby assigning probability distributions to these. Such a case is described in Chapter 6*. Bayesian statistics can handle complex problems by properly combining information even from different sources. The Bayes' theorem, the basis of Bayesian statistics, is able to incorporate the effect of information conveyed by new data (e.g., monitoring data collected by SHM systems) with prior knowledge (e.g., based on computation based on design reports) through the so-called likelihood function and the prior probabilities of a parameter of interest (e.g., mid-span span deflection, prestress loss, bearing displacement), respectively. The Bayesian updating leads to updated probability distributions of the parameters, according to Eq. 3-2. Further to this concept, in the cases of discrete events and continuous random variables, the Bayes' theorem presented can be rewritten as presented in Eq. 3-3 and 3-4, respectively. It is worth noting that the evidence is rarely available directly but can be calculated by using the law of total probability, according to Eq. 3-4.

$$posterior = \frac{likelihood \times prior}{evidence} \tag{3-1}$$

$$P(A \mid B) = \frac{P(B \mid A) \cdot P\left(A\right)}{P\left(B\right)} \tag{3-2}$$

$$p(\theta \mid x) = \frac{p(x \mid \theta) \cdot p\left(\theta\right)}{p\left(x\right)} = \frac{p(x \mid \theta) \cdot p\left(\theta\right)}{\int_{\Theta} p(x \mid \theta) \cdot p\left(\theta\right) \cdot d\theta} \tag{3-3}$$

$$P\left(B\right) = \sum_i P(B \mid C_i) \cdot P\left(C_i\right) \tag{3-4}$$

In the above formulations that are offered here as a summary for the purpose of completeness, according to the notation used in mathematical statistics[193], x is the vector of observations (e.g., strains, displacements, rotations), θ the vector of parameters to be inferred (e.g., bearing malfunctioning, prestress loss, pier settlement, loss of stiffness), $p(\theta|x)$ the posterior distribution of θ, $p(x|\theta)$ the likelihood function, $p(\theta)$ the prior distribution of θ, and $p(x)$ the evidence. In the case of absence of prior knowledge of the problem under analysis, a uniform distribution of the prior probabilities can be considered, since all possible states are equally likely, based on the available knowledge.

* Case Study 6.8.

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3.6 Treatment of environmental & operational effects

A critical issue relating to vibration-based SHM pertains to the influence of varying EOCs[194]. Environmental changes are usually related to temperature*, humidity, wind speed, etc. whereas the operational changes involve the traffic loading on bridges or, more generally, the changes in applied loads and/or masses. It is worth noting that the removal of EOC effects has been heavily addressed in the literature in the context of monitoring of natural frequencies, since these are prone to EOC susceptibility, with EOCs potentially masking changes due to damage. Methods for treatment of EOC variability are distinguished in two main categories: i) output-only methods (unsupervised) aiming to eliminate the EOC influence on the basis of output-only information[195],[196] and ii) input-output methods (supervised), which assume a measurement of the EOC inputs and try to derive a functional dependence between the measured vibration data and/or the extracted features with respect to measured operational conditions[197]. Output-only methods include the Principal Component Analysis (PCA)[198] or its nonlinear variants (kernel PCA, Factor Analysis). PCA is a multivariate statistical tool performing a linear transformation of data to a new coordinate system. PCA is typically used to reduce the dimensions of a data set by replacing a group of correlated variables with a new set of independent variables, designated as Principal Components (PC) that still account for those characteristics of the data set significantly contributing to its variance. The main idea is to identify the common trends in the measured/identified features and associate those common trends (producing the dominant contribution to variance) to the EOC effects.

The model-based (supervised) class is usually expressed in terms of a regression problem, where EOC information is obtained by appropriate environmental or operational (traffic in bridges) sensors. On the other hand, it should be noted that EOC factors also affect quasi-static measurements and the related DIs, with similar techniques suggested to remove the masking effects also from quasi-static measurements. Once the effect of all external factors on the measured/identified parameters has been successfully minimised or removed, any variation in the monitored features will be ascribed to structural changes.

A simple variant of the supervised class is based on the development of a regression relationship between the investigated features and the EOC factors. Such models can be divided in two categories, depending on whether the effects of input variables at previous time instants are accounted for or not. When the relationship is defined only between simultaneously measured data, the relationships are defined as static. However, this characteristic can turn into a limitation when trying to model dynamic processes, such as the heating up/cooling down cycles of structures. In this context, the dependence relationship can be enhanced by further considering the influence of EOC factors measured at previous time instants. Models of this class are referred to as dynamic.

When dealing with static regression models, the Multiple Linear Regression (MLR) is the statistical technique generally adopted to express the relationship between a single dependent variable (e.g., natural frequency, rotation, strain, deformation, etc.) and one or more independent EOC variables or predictors (such as temperature, humidity, etc.). The objective of MLR is to use the independent variables, whose values are directly measured, to predict the selected dependent parameter[199]. When a regression relationship is established between the dependent variable y (e.g., the modal frequency) and the predictors x (e.g., the temperature and the humidity), the value of the dependent variable is obtained through a linear or non-linear multiple regression equation. The latter is used for applications, where the need to model non-linear effects arises:

$$y_k = \theta_0 + \theta_1 x_k + \theta_2 x_k + \ \ldots \ + e_k \ (\text{linear}) \qquad (3\text{-}5)$$

$$y_k = \theta_0 + \theta_1 x_k + \theta_2 x_k^{\,2} + \ e_k \ (\text{nonlinear})$$

where θ_0 and θ_1 are the parameters of the regression relationship, and e_k is the prediction error, i.e., the difference between actual and predicted values of the dependent variable.

* Case study 6.17.

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In these equations a static regression form is considered: the dependent variable at time k depends only on the values of the predictors at this current time. A dynamic regression relationship can be alternatively established between the dependent variable at time k and the values of a predictor at time k as well as at (p-1) previous time instants:

$$y = \theta_0 + \theta_1 x_k + \theta_2 x_{k-1} + \ \ldots \ + \theta_p x_{k-(p-1)} + e_k \tag{3-6}$$

The parameters aggregated in the parameter vector θ, which characterises the model, are usually determined by means of Least Squares (LS) method during a reference (or "training") period, which covers a statistically representative sample of EOCs. It is worth mentioning that, among the supervised methods described in the system identification literature for removing the EOC effects, ARX models (Auto-Regressive models with eXogenous input[200]) are probably the most general methods, since they can address both static and dynamic regression, as special cases. The consideration of environmental effects into the monitoring and condition assessment of concrete structures, from the point of view of fatigue*, as well as for a wind turbine concrete support structure** is presented in Chapter 6. Additionally, Chapter 6 includes examples of scour in relation to environmental dependence with a link to decision support*** and the influence of the operational conditions, e.g., the water level, on the modal parameters of a dam****.

3.7 Thresholds for assessment

The assessment of the structural condition is ultimately aimed to support decisions for maintaining a prescribed performance level. Once the indicators of the structural performance have been identified from measurements and the impact of environmental variations has been accounted for or removed, the structural assessment is carried out by comparing such indicators with target values. These thresholds correspond to target performance levels of the asset that aim to guarantee both the serviceability and the safety of the monitored structures. The definition of the performance levels and relevant thresholds for the PIs[201], [202] in a risk-based management perspective can be carried out accounting for the direct costs related to the reliability of the structure on one side and for the economic and environmental costs related to its loss of functionality on the other. This is a key aspect of the decision-making process underlying asset management and maintenance scheduling.

Depending on the target performance levels the thresholds are meant to represent, their exceedance may trigger different corrective measures, ranging from low impact actions, such as an intensification of inspections and monitoring, or further structural assessment up to critical procedures, such as the immediate service interruption or repair/strengthening. The thresholds can be defined both for local and for global limit conditions. In the first case, threshold values are experimentally estimated. As an example, relevant to a corrosion indicator, a study[203] reports the corrosion rate as a function of time. Active corrosion occurs when the corrosion rate values lie above 0.1 μA/cm^2, whereas corrosion is arrested when corrosion rate is below this threshold, due to the reduction of water content caused by the external climate (lack of rain). The value 0.1 μA/cm^2 is, thus, assumed as a threshold for this indicator. Due to the high sensitivity of the corrosion rate with temperature and humidity, the indicator value should be averaged along a minimum period of 1 year.

* Case Studies 6.1 and 6.4.
** Case Study 6.20.
*** Case Study 6.9.
**** Case Studies 6.15 and 6.16.

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Table 3-7 and Table 3-8 below report the thresholds value adopted in the RILEM Recommendations[204], [205] for corrosion rate, corrosion potential and resistivity.

Table 3-7 RILEM Recommendation Thresholds for Corrosion Rate.

Corrosion Level	Corrosion Rate		Condition
	μA/cm²	μm/year	
Negligible	< 0.1	< 1	Passive state
Low	0.1 – 0.5	1 – 5	Depassivated in low humidity concrete
Moderate	0.5 – 1	5 – 10	Active corrosion in medium level humidity
High	> 1	> 10	Large areas corroding in wet concrete

The monitoring of local deterioration becomes relevant whenever it might affect the global behaviour. In these cases, a narrow and localised investigation must be performed in conjunction with the overall response monitoring. However, in general, the target performance of the structure is verified considering global indicators and relevant thresholds. Due to the uncertainties related to measurement accuracy, modelling and statistical scatter[167] that affect the values of the global DIs, the definition of their threshold value (DF_T in Figure 3-8) requires a trade-off between the probability of false and missing indications of damage (PFA and PMA in Figure 3-8). Most often the distribution of the DIs in the damage state is unknown, thereby the threshold values are determined based on the distribution in the reference structural state that is fixing a maximum acceptable value of the Probability of False Alarms (PFAs) in the reference state[203].

Table 3-8 RILEM Recommendation Thresholds for Corrosion Potential & Resistivity.

Corrosion potential (mV) $Cu/CuSO_4$ electrode	Resistivity (Ω.m)	Corrosion condition
More positive than -200	> 1'000 – 2'000	The corrosion rates will be low. The concrete is dry
Between -200 and -350	> 500 – 1'000	The concrete is moderately dry. Low corrosion rate
Between -350 and -500	> 100 – 500	Concrete is wet. Corrosion rate will be moderate
More negative than -500	< 100	Wet concrete of high porosity. High corrosion rate

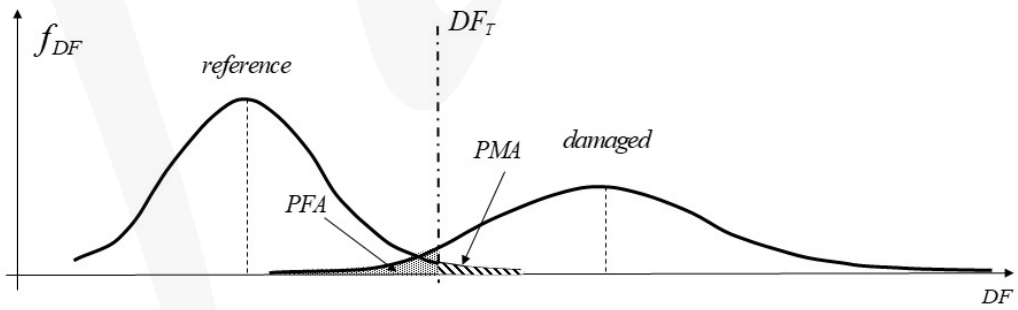


Fig. 3-8 Distributions of the damage feature DF, in the reference and damaged states. Probability of False Alarm (PFA); Probability of Missing Alarm (PMA); Damage Feature Threshold (DF_T)[203].

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As described in the previous sections, both data-driven and model-based approaches can be employed to identify the distributions of the DIs. In the second case, the model, calibrated and updated based on sensors measurement, can be used to explore several critical scenarios that are representative of the limit conditions for which the thresholds must be defined. In the practical applications, it is envisaged to define at least two categories of thresholds within the required safety levels. For new structures, the first category corresponding to a serviceability limit, and a second category corresponding to an unacceptable risk for human life. For existing structures, where the SLS limit is exceeded, the related serviceability verification can be disregarded and the first (lower bound) threshold may be referred to a lower corresponding level of reliability (Condition Limit States or Partial Damage Limit State), whilst the second (upper bound) limit can still correspond to an unacceptable risk for human life based on the target reliability for existing structures. Both thresholds can be referred to two different levels of risk for human life.

Protocols relevant to the actions to perform following the exceedance of a threshold (alert event) must be issued. These actions may include, for instance, the prompt communication of the alert to the decision maker for subsequent decision-making, based on the identified severity level. Moreover, protocols for data management must be established. For example, raw data relevant to the exceedance of a threshold and the issuing of an alert event should always be made available to the decision maker so that an in-depth analysis can be done, to confirm or neglect the urgency. Further to this, and to improve the alerting robustness, redundancy must be provided using measurements correlated in space and time, collected by groups of sensors located at optimised positions. This provides a redundancy of indicators, which in turn increases the reliability of the communication and can reduce the false alarms. Chapter 6* touches on the topic of optimal sensor positioning on a building for the purpose of seismic SHM.

As previously mentioned, when Artificial Intelligence and ML algorithms are employed, the task of classification inherently contains the notion of defining thresholds, since these are the defining boundaries of the clusters. This enables the automation of the issuing of alerts, however, retaining the decisions at the hands of specialists further allows engineering judgement to weigh in on the chosen policy. In this direction, experts in the loop system are currently under investigation for informing expert judgement and criteria into the alarms and data-driven decision support loop.

* Case Study 6.12.

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4. Performance prediction & modelling including advanced methods

Further to the condition assessment (Chapter 3), with the support of testing and monitoring techniques (Chapter 2), this Chapter 4 steps into the prediction of the expected performance of Civil Engineering structures along their lifetime. From a bottom-up approach, the chapter is organised as follows:

- Firstly, a review of degradation models towards performance prediction is introduced. More precisely, physics-based models, supported by a mathematical formulation (e.g., differential equations) that relate a specific parameter of interest with a set of input parameters. For a set of values in the input parameters, a single prediction is obtained (i.e., deterministic approach).

- Secondly, stochastic-based methods are then introduced to consider the fact that some of the mentioned input parameters may hold some variability associated with their quantification, including overtime. Hence, for a set of input parameters (i.e., including variability), a set of realisations are obtained for the prediction (i.e., a probabilistic approach). It is worth noting that the stochastic based methods are also frequently used to predict the damage and characteristics in their future development.

- Thirdly, updating techniques,.supported by Bayesian statistics, are introduced as a tool able to refresh the predictions given by the stochastic-based models. With this, the data that is continuously collected by one or more testing and/or monitoring techniques aforementioned (i.e., in Chapter 2) can be efficiently used to improve the accuracy (i.e., by reducing uncertainty) of the predictions.

- Finally, the outputs obtained from the modelling above assist the definition of specific PIs (e.g., crack's length, bearing displacement), which in turn allow the introduction of the concept of KPIs. The latter can incorporate other dimensions of the problem such as environment, politics, which are suitable for the lifetime assessment of existing structures.

Figure 4-1 offers a holistic perspective on the content of Chapter 4, by: (i) highlighting the progressive analysis approach adopted for the prediction of the performance of the structures through their lifetime (Figure 4-1(a)), and (ii) how the outputs obtained from these feeds into the quantification of the indicators (i.e., PIs and KPIs) towards lifetime assessment (Figure 4-1(b)).

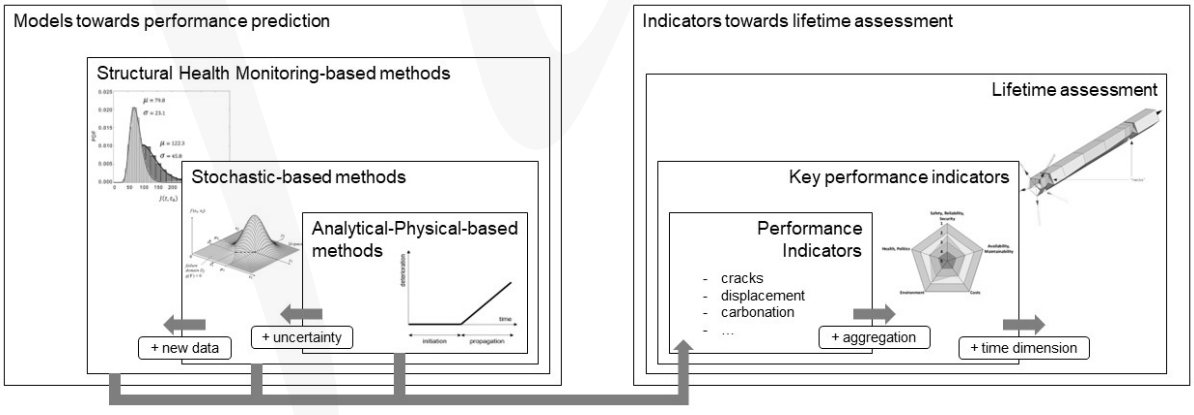


Fig. 4-1 Graphical representation of the content of Chapter 4 ; from prediction modelling to lifetime assessment.

This chapter was written by Andrade, Apostolidi, Duprat, Lehky, Matos, Novák, Sousa (main author), Sousa S, Strauss (main author).

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4.1 Models & parameters for degradation assessment

Degradation is a time-dependent process and therefore, the sporadic/periodic/permanent collection of data along the asset lifetime allows to better modelling of the expected deterioration rate. More precisely, quasi-continuous acquisition of data, e.g., by SHM and/or regular inspections supported by NDT and/or DT, allows to continuously updates (i.e., the accuracy by reduction of uncertainty) of the predictions given by models about the future behaviour of structures[206], [207].

Due to the random nature of most of the degradation processes, i.e., the uncertainty associated with the model parameters, predictions about the future performance of structures benefit from a global probabilistic approach, instead of solely relying on deterministic analyses[4], [208], [209]. For this, the statistical scatter of input parameters needs to be properly characterised and quantified, rather than the supply of a few deterministic values or a set of average values. With this, local and time-dependent scatter can be consistently incorporated in the models' prediction, regarding the expected behaviour of the asset. Additionally, spatial variability has to be considered for the sake of realism. This allows to properly modelling of the effects associated with the construction process, the exposure conditions, the geometry of reinforcement, the loads, among other spatial-related factors. It is worthy of note that by considering at least (some) input parameters as random, as described above, the question of whether the degradation model properly propagates the associated uncertainties becomes of paramount importance. For instance, most of the so-called engineer models describe degradation in one dimension (1D), although the phenomenon is usually three-dimensional (3D) and the random field of input parameters can be bidimensional (2D). This means that the computation strategy must be properly tailored to permit such an adaptation, in addition to the random and spatial variability issues mentioned.

Accounting for all these factors, comprehensively and consistently, provides the most robust approach to obtain reliable knowledge about the future performance of a given structure. Consequently, the residual service life of existing structures can be predicted accurately and the asset manager/engineer can plan more efficiently the maintenance actions that will be always needed along the structure lifetime (e.g., inspection and repair).

4.2 Performance models

As mentioned, further to the identification of the type of attack, the use of performance models is necessary for predicting evolution and possible deleterious consequences. For instance, in the case of corrosion, the rate of corrosion has to be measured, i.e. by collecting data[206], [207], [210], [211], [212], [213], [214]. These rates enable the prediction considering most of the decay curves can be assimilated to a two-step process (Figure 4-2(a)) with an initiation/incubation period followed by a degradation period. Chapter 6 presents a probabilistic prediction of the corrosion initiation time*.

The diagram in Figure 4-2 and the respective models can be in different forms, more precisely:

- Physics-based on a generic or universal law (i.e., differential equations solved either analytically or numerically);
- Stochastic-based on observations along time (i.e., empirical fitting-data models).

* Case Study 6.18.

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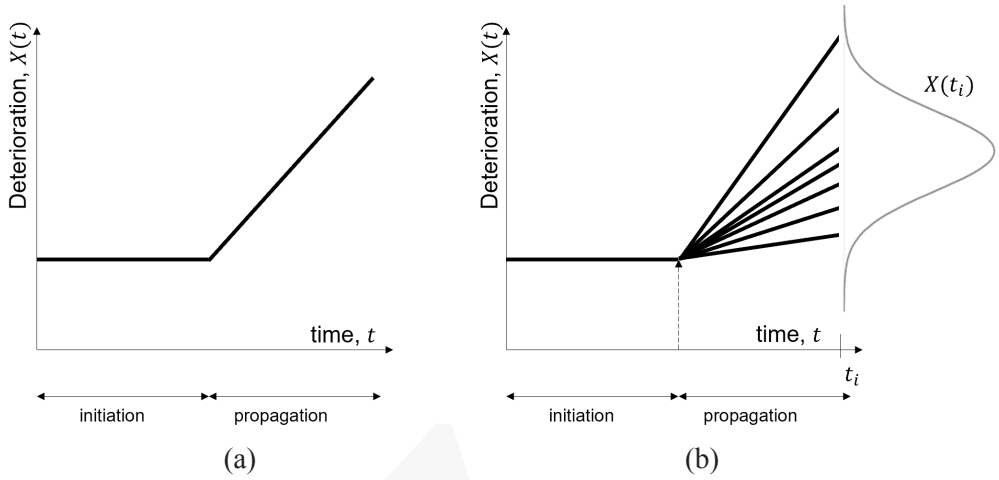


Fig. 4-2 Deterioration models: Two-step deterioration model with an incubation period followed by a propagation period: (a) Physical-based; (b) Stochastic-based [211].

4.2.1 Physics-based models

When equations are available to mathematically describe the physics of the degradation process, their resolution can be either analytical (i.e., if mathematical exact solutions exist) or by suitable FE/ numerical methods. The solution of the differential equations (either analytical or numerical) provides the evolution of the degradation process along time.

Physics-based models differ from each other concerning the sophistication or accuracy level. For instance, certain carbonation models derive from a diffusion physics with a stiff front of the portlandite consumption, without consideration of the modification of the microstructure, while others integrate refined kinetics of the consumption of all calcium hydrates, along with various diffusion physics, mass and moisture transfer, and reduction of the concrete porosity. The form of models is also linked to the sophistication level (i): so-called enhanced models are supported by implicit numerical solving approaches – FE method –, while (ii) so-called simplified or engineer models are explicit analytical expressions of the input parameters. In the same way, the model output at a given time may be the sole carbonation depth or a set of profiles as a function of the depth in concrete (e.g., carbon dioxide content, calcite content, porosity, pH, water content). Moreover, the combination of a set of models is suitable when each model satisfies a particular aspect of the degradation process, although the robustness of the modelling as a whole may become a challenging aspect.

When choosing a physics-based degradation model, the asset manager/engineer has to balance wisely the associated pros and cons. This balance depends on: (i) expertise regarding the type of model used (i.e., either FE /numerical or analytical models); (ii) the objective within the maintenance system; (iii) the expected accuracy of the prediction and; (iv) the capability to provide the model with its input parameters. Besides, the number of input parameters generally depends on the model sophistication, where the accuracy of the prediction is largely influenced by the quality of data used. The cost/benefit approach has recently proven to suit for helping the asset manager/engineer in this choice[215]. More precisely, the cost depends on the estimation method of the input parameters (either by expert advice or by in-site auscultation or with lab procedure both with the collection of data). This is rather an intellectual cost of operational investment (i.e., time spent) than an effective financial cost. Besides, the benefit depends on the accuracy of the predictions (assumed to be improved when observations hold high influence on the model output), which is weighted by the risk of lack of robustness, depending in turn on the complexity of estimating the input parameters.

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These models are based on well-known physical laws. For instance, they can be of transport of mass in porous media or of metallic corrosion, whereas those of diffusion differ in function of the driving force and both may or may not be influenced by the temperature. Table 4-1 shows some of the most common degradation processes that hold physics-based formulation.

Table 4-1 Degradation processes with physics-based formulation

Type of process	Driving force
Diffusion	Difference in concentration
Permeation	Difference in hydraulic pressure
Absorption	Capillary forces
Migration	Difference in electrical potential
Metal Corrosion	Difference in electrical potential

4.2.1.1 Diffusion

Diffusion is a transport mechanism, driven by a concentration gradient, by which gases, liquids, or dissolved ions flow across a porous body. The main material property governing the phenomenon is the diffusion coefficient, D, which depends on the pore structure of concrete and the nature of the transported substance, including its possible reactions with reactive phases in the cement paste. When gases, liquids, and dissolved substances are transported under a constant concentration gradient (steady-state) Fick's first law of diffusion applies, as given in Eq. 4-1, where the negative sign arises because diffusion occurs in the opposite direction of increasing concentration.

$$Q = - D \cdot A \frac{dC}{dx} \tag{4-1}$$

where Q is the flow rate of substance transported (mol.s^{-1}), dC/dx is the concentration gradient (mol.m^{-4}), A is the penetrated area (m^2) and D is the diffusion coefficient (m^2.s^{-1}).

In most practical cases, transient diffusion phenomena occur, by which the concentration C of the diffusing substance varies with location x and time t. In this case, the change in concentration $C(x,t)$ can be described by Fick's second law of diffusion. For one-dimensional flow assuming that there is no absorption process and gas solubility in the pore solution, Eq. 4-2 applies, which relates the unsteady diffusive flux to the concentration gradient. This also applies if enrichment or depletion of a diffusing substance is considered.

$$\frac{\partial C}{\partial t} = D\frac{\partial^2 C}{\partial x^2} \left(\pm \frac{\partial C}{\partial t} \right) \tag{4-2}$$

Assuming the diffusion coefficient and the surface concentration of the substance constant, Eq. 4-2 has an explicit solution (for a semi-infinite element and unidirectional diffusion) as follows:

$$C(x,t) = C_0 + (C_s - C_0) \left[1 - erf\left(\frac{x}{\sqrt{4.D.t}} \right) \right] \tag{4-3}$$

where, C_0 is the initial concentration of the substance (g.m^{-3}) and C_s is the surface concentration of the substance (g.m^{-3}).

Chapter 6* includes an example related to the failure of a significant number of pre-stressing tendons caused by a diffuse corrosive phenomenon in a prestressed concrete bridge.

* Case Study 6.2.

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4.2.1.2 Permeation

The most general permeation law, which is based on a generalisation of Darcy's law (strictly applicable to water flow) and extended to other liquids, assumes a laminar viscous flow and it reads as follows:

$$Q = K_l \frac{A}{\mu} \frac{dP}{dL} \tag{4-4}$$

where Q is the flow rate ($m^3.s^{-1}$), K_l is the intrinsic coefficient of permeability to liquids (m^2), A is the cross-section area of the body traversed by the liquid (m^2), μ is the dynamic viscosity of the liquid (Pa.s) and dP/dL is the pressure gradient across the body ($Pa.m^{-1}$).

The specific permeation of water is described by Darcy's law and expressed by Eq. 4-5:

$$Q = - K_w \cdot A \frac{\Delta h}{\Delta L} \tag{4-5}$$

where K_w is Darcy's coefficient of water permeability (m/s) and $\Delta h/\Delta L$ is the pressure gradient (m/m), with pressure expressed as water level.

The gas permeability law, assuming laminar viscous flow, is slightly different from Eq. 4 -5, to take gas compressibility into account and can be formulated as follows:

$$Q = K_g \frac{A}{\mu \cdot L} \frac{P_2^2 - P_1^2}{2\ P_1} \tag{4-6}$$

where K_g is the intrinsic coefficient of permeability to gases (m^2), A is the cross-section area of the body, traversed by the gas (m^2), P_2 is the gas inlet pressure (Pa), P_1 is the pressure at which Q is measured (Pa), and usually as the outlet pressure, L is the thickness of the body (m), and μ is the dynamic viscosity of the gas (Pa.s).

In Chapter 6*, a case study is presented, related to steel corrosion in some parts of exterior concrete beams. This phenomenon was attributed to carbonation and investigations on the coefficient of air-permeability has been conducted.

4.2.1.3 Absorption

Liquids, particularly water and solutions, may be transported into concrete by capillary suction or absorption. Water absorption may be expressed by Eq. 4-7:

$$w = M_w \cdot t^n \tag{4-7}$$

where w is the water absorbed per unit area at time t (m^3/m^2),t is the duration of water absorption (s), n a parameter lower than 0.5, and M_w is the water absorption rate ($m.s^{-0.5}$). Under testing conditions, M_w is quantified through the initial slope of mass or water uptake plotted over the square root of time. Besides, and according to RILEM TR-PCD, RILEM TC PSC, and EN ISO 15148, Eq. 4-8 and Eq. 4-9 show, respectively, the the parameter a_{cc}, which is an area-related increase in mass after time t ($kg.m^{-2}$) and the parameter $m(t)$ is the coefficient of water absorption

$$a_{cc} = \frac{m_i - m_0}{A} \tag{4-8}$$

$$m(t) = S_c \sqrt{t} \tag{4-9}$$

where A is the cross-section area of the specimen (m^2), m_i is the mass (kg) at time t_i, m_0 is the initial mass (kg) at the time t_0 and S_c is the designation of absorption coefficient or absorption rate ($kg.m^{-2}h^{-0.5}$ or $g.m^{-2}s^{-0.5}$).

* Case Study 6.18.

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The absorption parameter, calculated as in ASTM C1585, is defined as follows:

$$I = \frac{\Delta m}{a \cdot \delta_w} \tag{4-10}$$

where I is the absorption (mm), Δm is the mass gain (g) at time t, a is the exposed area of the specimen (mm²), δ_w is the density of water (g.mm-³). This equation is at the root of (i) test methods to measure chloride diffusion, and (ii) models to predict the penetration of chlorides into concrete with time.

4.2.1.4 Migration

Migration is a transport mechanism, driven by an electric field, by which ions move through the pore solution in concrete. The resulting flux of ions J is given by Nernst-Planck law of mass transport, in which convection can be neglected, leaving only the diffusion and the migration terms. For uniaxial flow, this can be expressed by Eq. 4-11.

$$-J = D\left(\frac{\partial C}{\partial x} + C\ \frac{z.F}{R.T}\ \frac{\partial E}{\partial x}\right) \tag{4-11}$$

where J is the flux of chloride ions (g.m-².s), D is the coefficient of diffusion (m².s^{-1}), $\partial C/\partial x$ is the ion concentration gradient (g.m^{-4}), C is the ion concentration in the fluid (g.m^{-3}), z is the electric charge of the ion, F is the Faraday constant (96,485 J. V^{-1}. mol^{-1}), R is the Universal gas constant (8.314J.mol^{-1}.K^{-1}), T is the absolute temperature (K), and $\partial E/\partial x$ is the gradient of electric potential across the body (V.m^{-1}). Inside the parentheses in Eq. 4-11, the first term represents Fick's First Law of Diffusion, whilst the second expresses the migration phenomenon. In theory, both are governed by the coefficient of diffusion.

As migration proceeds much faster than diffusion, in the few hours or days required by a migration test, the term of diffusion of Eq. (4-11) can be neglected and then, the diffusion coefficient can be calculated from the migration test data, provided the correct boundary conditions are fulfilled.

The transport of electrical charge by the ions presents in the pore solution, through the migration process, is responsible for the conduction of electricity through concrete. The electrical resistivity, ρ, is the specific electrical resistance of the material to the flow of electric current, which can be assessed by applying a current of intensity, I, and measuring the generated electrical potential V. For the bulk method, in which electrodes are coupled to opposite faces of a specimen of a cross-section A and length L, is expressed as follows:

$$\rho = \frac{V \cdot A}{I \cdot L} \tag{4-12}$$

For the surface, i.e., (Wenner) method (AASHTO 358), in which 4 aligned and equally spaced (by a distance a) electrodes are applied to the concrete surface, the electrical resistivity can be expressed as follows:

$$\rho = \frac{2\ \pi\ a}{K}\ \frac{V}{I} \tag{4-13}$$

$$\rho = \frac{1}{\sigma} \tag{4-14}$$

where K is a geometric factor (equal to 1 when the electrodes are applied on a semi-infinite flat surface, and different from 1 for other geometries). The conductivity is the reciprocal of resistivity and is given by Eq. 4-14, where σ is the conductivity (S.m^{-1}).

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4.2.1.5 Corrosion propagation

The model of the corrosion propagation can be expressed as follows[211], [212], [216]:

$$t_{SL} = t_{ini} + t_{prop} \tag{4-15}$$

where t_{SL} is the design service life, t_{ini} is the time of the initiation period, and t_{prop} is the propagation period. The initiation period is governed by diffusion processes of carbon dioxide and chlorides through the porous concrete.

The model of the propagation period, and according to MC2020 (*fib* Model Code 2020), is given by Eq. 4-16[217]:

$$t_{prop} = \frac{Ø_0 - Ø_t}{V_{corr}} = \frac{P_{corr}}{V_{corr}} \tag{4-16}$$

where V_{corr} is the averaged value of the corrosion rate per year (μm/year) in the period considered, P_{corr} is the penetration of corrosion attack (accumulated corrosion) (μm) during the period considered, $Ø_0$ is the initial diameter, and $Ø_t$ is the diameter at time t.

The change in the slope of P_{corr} overtime can be caused by cracking of the concrete cover (in general parallel to the bar direction) due to the expansive character of the iron oxides or to the formation of semi-protective rust. This cracking may either accelerate the corrosion process, if the structure is in continuous contact with water, or can slow down the corrosion because in the cracked area drying occurs faster. Hence, and further to the two-step deterioration processes introduced in Figure 4-2, a bi-linear or a non-linear evolution for P_{corr} with time can be modelled.

4.2.2 Stochastic-based models[ii]

In addition to the physics-based models, presented above, for the prediction of degradation processes, other methods are often used which allow a prediction of the damage or its specific indicators without a detailed description of the physical relationships. Among these methods are the stochastic methods presented in this section, which are also called prognosis models/methods with a temporal component. These forecasting methods also allow for the consideration of uncertainties in the model input variables as well as in the DIs under study characterised in terms of PIs.

Many of the common PIs simulate a high degree of precision and accuracy and usually only refer to the evaluation of the current condition or only allow for a preventive maintenance strategy. For a realistic approach to performance assessment, it is also necessary to be able to (i) represent the uncertainties in a model and also to (ii) include the uncertainties in the predictions (predictive maintenance strategies). The following models show some tools to capture these uncertainties in both the actual condition assessment and the predictive assessment, thus also allowing the uncertainties to be quantified in a statement. The models/tools presented here can be applied to information from visual inspections, testing, SHM and modelling.

4.2.2.1 Concept

A stochastic process $X=\{X(t), t \in T\}$ is a collection of random variables, i.e., for each instant in time, t, in the index set T (i.e., a subset of natural numbers or an interval, giving the set T the interpretation of time), $X(t)$ is a random variable – i.e., the state of the process at time t (e.g., Figure 4-2(b)). The set T can be countable, i.e., X as a discrete-time stochastic process, or T can be a continuum, which is called a continuous-time process. Any Realisation of X is called a sample path (e.g., for events occurring randomly in time, and $X(t)$ represents the number of events that occur in $[0,t_n]$, then a sample path of X corresponds to the initial event occurring at time 1, $X(t_1)$, the next event at time 2, $X(t_2)$, and so on until the event at the time t_n, $X(t_n)$.

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Moreover, a continuous-time stochastic process $\{X(t), t \in T\}$ is said to have[218]:

- Independent increments if for all $t_0 < t_1 < t_2 < \cdots < t_n$, the random variables; $X(t_1) - X(t_0)$, $X(t_2) - X(t_1), \ldots, X(t_n) - X(t_{n-1})$ are independent;
- Stationary increments: if $X(t+s) - X(t)$ has the same distribution for all t, meaning that it possesses independent increments if the changes in the processes' value over non-overlapping time intervals are independent;
- Stationary increments if the distribution of the change in value between any two points depends only on the distance between those same points.

4.2.2.2 Probabilistic approach

Failure probability, p_f, and reliability index, β, are reliability-based indicators, which allows characterizing efficiently the performance of structure in time – including degradation effect. To quantify them, one can use a fully probabilistic approach to consider the implicit uncertainties of the problem. Nowadays, attention is being given to the development of software tools devoted to the practical implementation of fully probabilistic approaches. Hence, important topics can thus be treated in a more proactive approach, such as the probabilistic vulnerability assessment of civil infrastructure systems followed by efficient decision-making processes.

Two main groups of reliability methods have been developed during last decades:

- Simulation methods:
  - · Crude Monte Carlo simulation;
  - · Stratified sampling techniques (e.g., Latin hypercube sampling);
  - · Advanced simulation methods (e.g., importance sampling, adaptive sampling, directional sampling, line sampling).
- Approximation methods:
  - · First/Second Order Reliability Method, FORM/SORM, or other approximation at the design point;
  - · Curve fitting techniques based on evaluating samples of safety margin;
  - · Perturbation techniques;
  - · Response surface methods.

From the many techniques to determine these PIs, two widely used and representative of both groups are described shortly.

4.2.2.3 Monte Carlo type simulation

Monte Carlo simulation technique is a well-known and widely used tool in structural reliability analysis, which consists of N simulations of response or LSF evaluations with different trials of the input vector X (Eq. 1-2) as follows:

- In the j^{th} simulation, a pseudo-random value $u_{i,j} \in \langle 0;1 \rangle$ is generated for the i^{th} variable;
- The realisation of variable X_i is then gained using the inverse transformation of Cumulative Distribution Function (CDF) according to Eq. 4-17, where $\Phi_{xi}(.)$ is the CDF of X_i, schematically represented in Figure 4-3 note that (different variables may have different CDFs).

$$x_{i,j} = \phi_{X_i}^{-1}\left(u_{i,j}\right) \tag{4-17}$$

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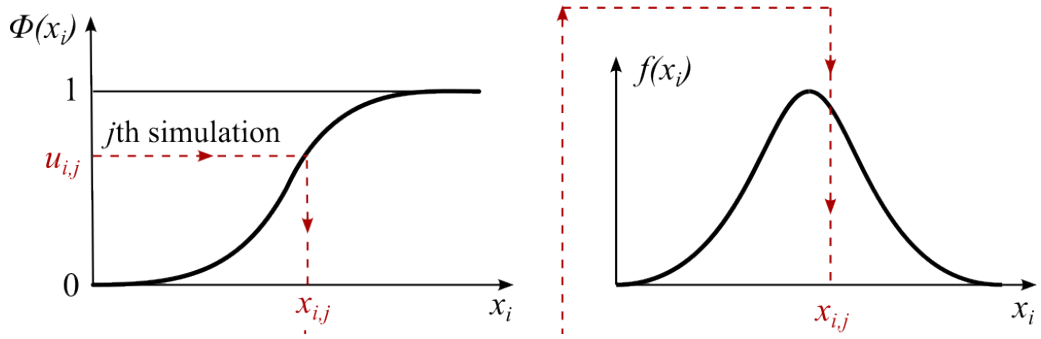


Fig. 4-3 Illustration of the inverse transformation for random variable X_i.

- After performing for all input random variables, i.e., $X = X_1, ..., X_{NV}$, the j^{th} simulation the function $z_j = g(X)$ is evaluated by using the related input variable representations.
- The final step relates to the statistical evaluation of set $Z = z_1, z_2, ..., z_j, ..., z_n$, rendering statistical moments of Z, and/or the probability p_f. The latter may be assessed as:

$$p_f \approx \frac{n(g \le 0)}{N} \tag{4-18}$$

Using mathematical formalism, and considering that the function $1[g(X)]$ has two values only, the probability of failure is obtained as follows:

$$p_f = \int_{\Omega_X} 1[g] f_X(X) dX, \begin{cases} 1[g] = 1 \dots g(X) \le 0 \\ 1[g] = 0 \dots g(X) > 0 \end{cases} \tag{4-19}$$

which can be solved numerically as follows:

$$E[p_f] = \frac{1}{N} \sum_{i=1}^{N} 1[g] \tag{4-21}$$

Finally, the CoV of failure probability approximately holds:

$$CoV_{p_f} = \frac{1}{\sqrt{N \cdot p_f}} \tag{4-22}$$

It is worth noting that the smaller the expected probability, the greater the number of simulations has to be performed. For example, when a $CoV_{pf} = 0.1$ is required and estimating p_f around 10^{-6}, a total of 10^8 simulations (N) is necessary to be performed.

In the context of time-intensive calculations, small-sample simulation techniques represent a rational compromise between feasibility and accuracy. Latin Hypercube Sampling (LHS) method, an advanced stratified sampling technique, gives a very good estimate of statistical moments of response using small-sample simulation[219].

The fundamental feature of LHS is that the range of univariate random variables is divided into N_{sim} intervals (i.e., the number of simulations). The values from these intervals are then used in the simulation process (i.e., by random selection, median, or the mean value). The selection of the intervals is optimised in such a way that the range of the PDF of each random variable is divided into intervals of equal probability, $1/N_{sim}$. The samples are chosen directly from the distribution function based on an inverse transformation of the univariate distribution function. The representative parameters of variables are selected randomly, and based on random permutations of integers, $k = 1,2,\dots,N_{sim}$, with every interval of each variable used only once. It has been shown that a preferable LHS strategy is the approach[220], where the representative value of each interval is the mean value (Figure 4-4) and quantified by Eq. 4-23, where f_i is the PDF of variable X_i, and the integration limits set according to Eq. 4-24. The sample averages equal exactly the mean values of variables and the variances of the sample sets are much closer to the target values compared to other selection schemes.

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$$x_{i,k} = \frac{\int_{y_{i,k-1}}^{y_{i,k}} x \cdot f_i(x)\,dx}{\int_{y_{i,k-1}}^{y_{i,k}} f_i(x)\,dx} = N_{sim}\int_{y_{i,k-1}}^{y_{i,k}} x \cdot f_i(x)\,dx \tag{4-23}$$

$$y_{i,k} = F_i^{-1}\left(\frac{k}{N_{sim}}\right),\ k = 1,\ldots,\ N_{sim} \tag{4-24}$$

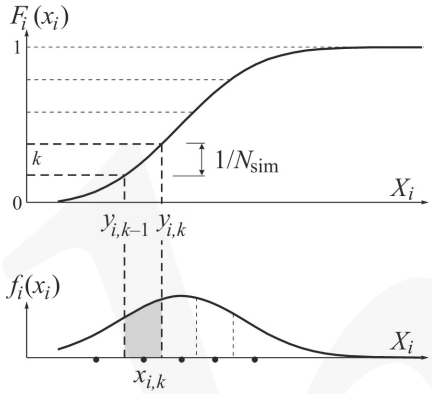


Fig. 4-4 Samples as the probabilistic means of intervals.

4.2.2.4 Approximation methods

The structural assessment of complex systems is usually time-consuming, leading to the utilisation of approximation methods to reduce the computational effort to a feasible level. The FORM has initially been proposed[17], where a linear approximation of the limit state surface in the uncorrelated standardised Gaussian space is used to estimate the probability of failure. For this purpose, it is necessary to transform the basic variables into uncorrelated standard Gaussian variables (i.e., in the U-space), which can be done according to Eq. 4-25.

$$Y_i = \frac{X_i - \mu_i}{\sigma_i},\quad i = 1,\ldots,N_v, \tag{4-25}$$

where μ_i and σ_i are the mean and standard deviation values of variable X_i, respectively. Thus, the probability of failure, p_f, in the transformed U-space (Figure 4-5) is expressed according to Eq. 4-26. As previously mentioned, the distance from the design point of the transformed LSF to the origin is the reliability index, β (dashed line in Figure 4-5).

$$p_f = \int_{g(\boldsymbol{Y})\le 0} f_{\boldsymbol{Y}}\left(Y_1,\ldots,Y_{N_v}\right)dY_1\ldots dY_{N_v}. \tag{4-26}$$

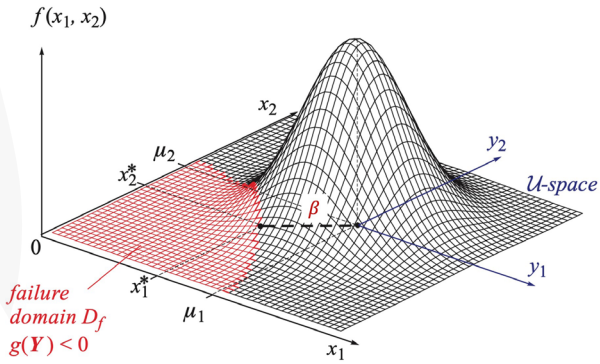


Fig. 4-5 Geometrical interpretation of failure probability (case of two-variate joint PDF $f_X(x_1,x_2)$).

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It is worth noting that the design point, which is the closest point of the limit state surface to the origin in standard normal space (dashed line in Figure 4-5). This is an important parameter since it corresponds to the point of maximum likelihood if the basic variables are normally distributed. The distance between design point and the origin is the reliability index, β, and according to Eq. 1-5. It can be shown that the probability of failure is approximately given by Eq. 1-6, which in the case of linear LSF and normally distributed basic variables yields to the exact failure probability. Although the calculation of failure probability using reliability index[17], [221] shows limited accuracy[222], it is a feasible alternative in many practical cases.

Response surface methods[222], [223], [224] belong to the category of methods, where the LSF is approximated by an advanced function/model. Originally, polynomial functions were used, although over the last decades enhanced metamodels, such as artificial neural network[225], polynomial chaos[226], support vector machine[227], and the Kriging metamodel[228], [229], have been used. After replacing the original LSF with one of these models, the reliability is calculated via classical simulation techniques (e.g., Monte Carlo method).

The regression coefficients of the used metamodel can be obtained by conducting a series of numerical "experiments". More precisely, a series of FE analyses with input variables selected according to the "experimental design", considering that the main area of interest lies in the region of maximum likelihood within the failure domain. Here, an iterative response surface approach is suggested. For polynomial response surface further information can be found in a previous study[222], whereas for Artificial Neural Network Response Surface Method (ANN-RSM) details can be found in another study[225]. Generally, the strategy is to position the new centre point reasonably close to the exact limit state $g(X) = 0$. In the latter method, the LHS method[219] is used to efficiently design experiments and stochastically prepare the training set for the artificial neural network. As illustrated in Figure 4-6, the application of ANN-RSM is divided into two main phases: (1) *the approximation phase*, where the original LSF is approximated by a suitable ANN using a series of numerical experiments where the input random variables are selected according to the LHS simulation scheme, and (2) *the reliability calculation phase*, where the approximated LSF is used instead in combination with classical simulation or approximation methods (e.g., Monte Carlo, FORM) for the calculation of reliability index, β, and failure probability, p_f.

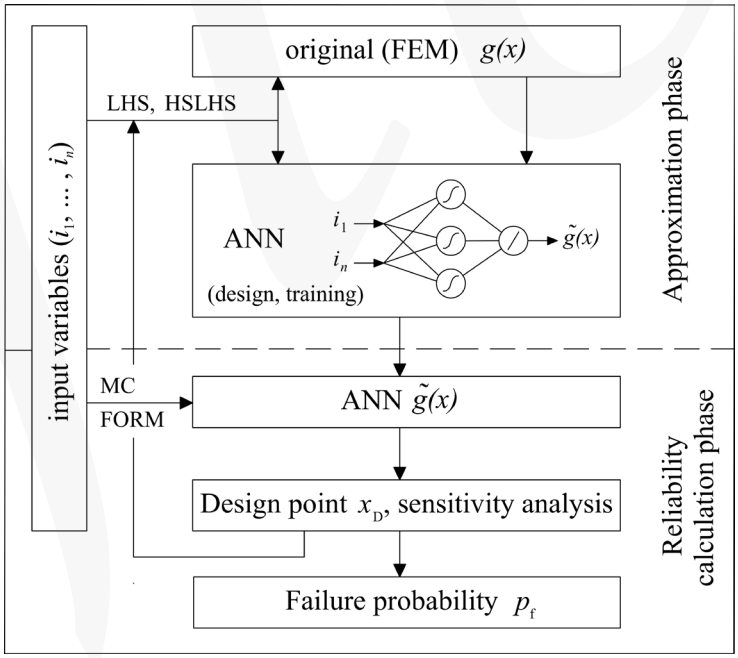


Fig. 4-6 Artificial neural network-based response surface method (ANN-RSM)[225].

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4.2.2.5 Discrete Markov models

Discrete Markov models are stochastic processes that describe physical systems, where the probability that a system will be in a given state j at time t_0, may be obtained from a known previous state i, at an earlier time t_1, but it is independent on its history, i.e., before time t_1 (Markov property)[230]. The probability of a transition between states i and j per unit of time is expressed as follows[231]:

$$P_{ij} = \Pr\{X_{t+1} = j \mid X_t = i_0\} = \Pr\{X_1 = j \mid X_0 = i\} \tag{4-27}$$

The probability of transitioning from all possible pairs (i,j) during a single period, may be assembled in the Transition Probability Matrix (TPM) of dimensions $n \cdot n$, where n is the total number of condition states and expressed by Eq. 4-28[231]. The condition of $P_{ij} = 0 \text{ ,if } i > j$ is assumed for modelling deterioration. Therefore, the system will remain in the same state during the discrete period or will move to a more deteriorated state.

$$TPM = \begin{bmatrix} P_{11} & P_{12} & \cdots & P_{1n} \\ P_{21} & P_{22} & \cdots & P_{2n} \\ \vdots & \vdots & \ddots & \vdots \\ P_{n1} & P_{n2} & \cdots & P_{nn} \end{bmatrix}, \; where \begin{cases} 0 \le P_{ij} \le 1, \; \forall \; i,j \\ \sum_{j=1}^{n} P_{ij} = 1, \forall i,j \\ P_{ij} = 0 \text{ , } if \;\; i > j \end{cases} \tag{4-28}$$

Utilizing a discrete Markov process, the state vector Q_t which corresponds to a vector containing the element rating for any time t, can be obtained as the initial condition state vector Q_0 multiplied by TPM to the power of t[232]:

$$Q_t = Q_0 \cdot P^t \tag{4-29}$$

For a newly constructed structure element at the time of the first inspection, the initial state vector will be equal to $Q_0 = [1\ 0\ 0\ 0\ 0\ 0]$[232]. Finally, the estimation of the condition rating as a function of time $R_{P,t}$ is obtained as indicated in Eq. 4-30[232], where R' is a vector of condition ratings. Consequently, the deterioration curve can be constructed by computing the expected value of the condition rating for each discrete time step over the lifetime.

$$R_{P,t} = Q_t \cdot R' \tag{4-30}$$

There are different ways to obtain the probability terms of the TPM. Probably the simplest is by estimating them based on purely the judgement of the engineer. Another way is by compiling the information of historical visual inspections into condition databases and calculate the probabilities from one state to another[233]. Several other methods to derive the TPM such as a duration model or Poisson regression model can be used[234], [235].

4.2.2.6 Semi-Markov models

Aging is mathematically defined as an increased probability of transition to a worse condition state as time progresses. Semi-Markov models are an extension of discrete Markov models where the aging effect can be captured through the random time that is inserted between state transitions[231]. This random time is referred to as sojourn (waiting) time and is denoted as T_{ij}, with the probability density function (PDF) designated by f_{ij}, and Survival Function (SF) designated by S_{ij}[236]. To estimate the transition probabilities in a semi-Markov process, it is necessary to calculate the sum of the sojourn times in the state, $T_{t,k}$, that is, the time the process will take to move from state i to k, which can be expressed as follows[236]:

$$T_{i \to k} = \sum_{j=1}^{k-1} T_{ij+1} \tag{4-31}$$

with $i = \{1,2,\ldots, n\text{-}1\}; k = \{2,3,\ldots,n\}$. Accordingly, the single-step transition probabilities can be determined[237]]:

$$P_{i,i+1}^{t,t+1} = \Pr\left[X(t+1) = i+1 | X(t) = i \right] = \frac{f_{1 \to i}(t)}{S_{1 \to i}(t) - S_{1 \to i-1}(t)} \tag{4-32}$$

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where $f_{1\to i}(t)$ and $S_{1\to i}(t)$ are the PDF and SF of the sum of the sojourn times from state 1 to state i, respectively. The TPM of the semi-Markov process is hence populated after generating all the transition probabilities using Eq. 4-32 , and given by:

$$TPM^{t,t+1} = \begin{bmatrix} P_{11}^{t,t+1} & P_{12}^{t,t+1} & 0 & \cdots & 0 \\ 0 & P_{22}^{t,t+1} & P_{23}^{t,t+1} & \cdots & 0 \\ \cdots & \cdots & \cdots & \cdots & \cdots \\ 0 & \cdots & \cdots & P_{n-1,n-1}^{t,t+1} & P_{n-1,n}^{t,t+1} \\ 0 & 0 & \cdots & 0 & 1 \end{bmatrix} \tag{4-33}$$

For this, the distribution of the sojourn times is assumed to follow a two-parameter Weibull distribution, and therefore, the SF and PDF of the sojourn times are respectively given, according to the study[236]:

$$S_i(t) = e^{-(\lambda_i t)^{\beta_i}} \tag{4-34}$$

$$f_i(t) = \lambda_i \cdot \beta_i (\lambda_i t)^{\beta_i - 1} e^{-(\lambda_i t)^{\beta_i}} \tag{4-35}$$

The parameters λ_i and β_i can be estimated from historical observations. To this end, two intervals of time can be defined, namely u and v *(*with $u \neq v$*)*, and the probabilities of a structural component remaining in a certain condition rating i for more than u and v years are assessed, that is $x_{i,u}$ and $x_{i,v}$ respectively [236]. These probabilities are computed from the relative frequency of events, similarly as for the Markov process[238]. Subsequently, the parameters λ_i and β_i are derived from the following expressions presented in Eq. 4-36 and Eq. 4-37[236] , respectively. Once both parameters are evaluated for every i, the TPM for the semi-Markov process can be computed.

$$\begin{cases} S_i(u) = e^{-(\lambda_i u)^{\beta_i}} \\ S_i(v) = e^{-(\lambda_i v)^{\beta_i}} \end{cases} \Rightarrow \begin{cases} \ln\left[S_i(u)\right] = -(\lambda_i u)^{\beta_i} \\ \ln\left[S_i(v)\right] = -(\lambda_i v)^{\beta_i} \end{cases} \Rightarrow ln\left(\frac{\ln\left[S_i(u)\right]}{\ln\left[S_i(v)\right]}\right) = \beta_i ln\left(\frac{u}{v}\right) \tag{4-36}$$

$$\beta_i = \frac{\ln\left(\ln\left[S_i(u)\right] - \ln\left[S_i(v)\right]\right)}{\ln(u) - \ln(v)};\ \lambda_i = \frac{1}{u}\left(-\ln\left[S_i(u)\right]\right)^{\frac{1}{\beta_i}} \tag{4-37}$$

4.2.2.7 Hidden Markov models

Monitoring data from historical inspections frequently contain measurement errors and selection biases[239], which affect the accuracy of the deterioration predictions. To address this issue, Hidden Markov Models (HMMs) have been used to incorporate the bias of the observations into the forecasting models[231], [239]. HMMs assume that there is some true condition state which is "hidden" to the observer [231]. In other words, the sequence of true states $S_1, S_2, ..., S_n$ at the inspection times $t_1, t_2, ..., t_n$ is hidden behind the sequence of the observed state $V_1, V_2, ..., V_n$ [231]. Therefore, considering the bias in the monitoring data allows the unobserved true condition states to be captured[239].

The sequence of the true states follows a simple Markov chain, so the probability of moving to state S_j, i.e., transition probability, a_{ij}, depends only on the state S_i, which can be expressed as $a_{ij} = P[q_{t+1} = S_j \mid q_t = S_i]$[240]. On the contrary, the observed sequence does not hold the Markov property[231].

The conditional probability of the observations given the true states corresponds to another study[231] and given as follows:

$$e_{ij} = \Pr\left[V_k = j \mid S_k = i\right] \tag{4-38}$$

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These probabilities are collected in an error or misclassification matrix E (n*n), where $0 \leq e_{ij} \leq 1$, and $\sum_{j=0}^{n} e_{ij} = 1$ [231]:

$$E = \begin{bmatrix} e_{11} & e_{12} & \cdots & e_{1n} \\ e_{21} & e_{22} & \cdots & e_{2n} \\ \vdots & \vdots & \ddots & \vdots \\ e_{n1} & e_{n2} & \cdots & e_{nn} \end{bmatrix} \tag{4-39}$$

The probability of correctly identifying the condition state corresponds to e_{ii}, while for all $i \neq j$, there exists a misclassification reflecting the variability in the inspections[231]. This variation is attributed to the fact that the condition rating is a qualitative measure affected by the subjectivity of the inspectors. This phenomenon was investigated by the FHWA[241]. The error probabilities referred to as emission probabilities in the formal notation for HMMs may be assessed by expert judgment or by maximum likelihood[231].

4.2.2.8 Gamma processes for deterioration modelling

Gamma function approaches applied to model deterioration are quite straightforward from the mathematical calculation point of view[242]. For the determination of the gamma process modelling, a random variable X needs to be considered, which is described by the gamma distribution with a shape parameter $\alpha > 0$, a scale parameter $\beta > 0$ and the following PDF in Eq. 4-40, where $\Gamma(\alpha)$ is the gamma function for $\alpha > 0$ as defined by Eq. 4-41.

$$Ga(x|\alpha,\beta) = \frac{\beta^{\alpha}}{\Gamma(\alpha)} x^{\alpha-1} \exp(-\beta\ x) \tag{4-40}$$

$$\Gamma(\alpha) = \int_{z-0}^{\infty} z^{\alpha-1} \cdot e^{z}\, dz \tag{4-41}$$

Graphical representationof a gamma process can be used to illustrate that successive gamma PDF or CDF functions are independent of each other. Consequently, the conditional distribution of variable X can only be formulated based on the current observation [243]. This property is very well suited for capturing typical deterioration processes in structural engineering. In practice, the projection of the deterioration into the future is best based on current observations of the condition and on those events that have led to the present condition. Historical deterioration profiles may be helpful, in addition to the projections, as a reference value for the comparison with the predictions. The inclusion of such profiles into the projections is recommended, but there is no need to record them for future use. It can be assumed that for deterioration processes, the shape parameter $\alpha\ (t)$ increases with time t, where $t > 0$ and $\alpha\ (0) = 0$. The gamma process with the shape parameter $\alpha(t) > 0$ and the scale parameter $\beta > 0$ is a continuous stochastic time process $\{X\ (t), t \geq 0\}$. The time dependent PDF of $X(t)$ is then given by:

$$f_{X(t)}(x) = Ga(x \mid \alpha(t), \beta) \tag{4-42}$$

With an expected value, variance, and CoV defined by, respectively:

$$E(X(t)) = \frac{\alpha(t)}{\beta} \tag{4-43}$$

$$Var(X(t)) = \frac{\alpha(t)}{\beta^2} \tag{4-44}$$

$$CoV(X(t)) = \frac{1}{\sqrt{\alpha(t)}} \tag{4-45}$$

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Both, expected value and CoV, are solely functions of the gamma process parameters and only indirectly dependent on time[243].

Historically, it was assumed that gamma processes can be used to model any form of deterioration with a constant scale parameter β and a time-dependent power law based on shape parameter $\alpha(t)$ of the form given by:

$$\alpha(t) = c \cdot t^b \tag{4-46}$$

It is generally assumed that sufficient engineering expertise on the expected form of the deterioration is available. Thus, a previous study[244] recommended using a constant exponent b, which can be adapted according to the considered deterioration process. For the corrosion of rebars, another study[245] recommends $b = 1$ while for sulphate attacks it is suggested $b = 2$, and for diffusion-controlled ageing $b = 0.5$. Bazant and his colleagues investigated the application of gamma processes to predict concrete creep, for which $b = 0.125$ is recommended[246]. The other parameters, however, are unknown quantities and need to be adapted by drawing on expert knowledge and/or statistical analyses such as the Maximum Likelihood Method, the Method of Moments or Bayesian Statistics. Another study[243] have used the Method of Moments for determining both values. Regarding special cases, the gamma process is called stationary if the deterioration is linear in time, i.e., when $b = 1$ in the power-law Eq. 4-46, and nonstationary when $b \neq 1$. In mathematical terms a process has stationary increments if the probability distribution of the increments $X(t+h) - X(t)$ depends only on h for all $t,h \geq 0$. Due to the stationarity, both the mean value and the variance of the deterioration are linear in time. An additional property of the gamma process with stationary increments is that the gamma density transforms into an exponential density if $t = c^{-1}$. When the unit-time length is chosen to be c^{-1}, the increments of deterioration are exponentially distributed with mean c^{-1} and the probability of failure in unit time i reduces to a shifted Poisson distribution[247].

4.3 Updating of performance models

Further to the stochastic-based models previously described, where some (or all) of the input parameters is considered as random variables (instead of deterministic values), these predictions can be enhanced by data, collected either from a monitoring system, NDT, visual inspection, or a combination of these. Moreover, once new data becomes available, the existing knowledge can be improved by properly combining both, i.e., the existent one (i.e., provided by the models) with the new knowledge (i.e., obtained from brought by the collected data). The method to combine these in a systematic and logical approach is the so-called Bayesian updating.

4.3.1 Single point updating

According to Bayes' theorem, updated information is achieved about a parameter θ (i.e., a parameter associated with the performance model) based on the initial state of knowledge (about that same parameter) and the knowledge gained from a set of observations y (i.e., data collected from a monitoring system, NDT, visual inspection, or a combination of these). For continuous random variables, Bayes' theorem is formulated in terms of PDF as follows:

$$\pi(\theta \mid x) = \frac{f(y \mid \theta) \cdot \pi(\theta)}{\int f(y \mid \theta) \cdot \pi(\theta) d\theta} \propto f(y \mid \theta) \cdot \pi(\theta) \tag{4-47}$$

where $f(y \mid \theta)$ is the conditional PDF of y given θ (i.e., sampling distribution or likelihood function that represents the knowledge obtained from the collected data), $\pi(\theta)$ is the PDF of θ (i.e., prior distribution that represents the current knowledge and before the collected data is used), $\pi(\theta \mid y)$ is the conditional PDF of θ given y (i.e., posterior distribution reflecting the updated knowledge after the knowledge obtained from the collected data has been considered in the analysis). The denominator of Eq. 4-47

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is a normalising constant, assuring that the area under the posterior distribution is equal to unity. It is important to realise that the prior must not be constructed with information derived from the observations, which implies that the prior should be independent of the likelihood, and therefore, the observations do not influence the choice of the prior. In other words, the prior represents the current level of knowledge about a parameter associated with the performance model as if the collected data did not exist.

4.3.2 Sequential updating

The described updating approach can expanded over the time dimension. More precisely, if the continuous random variable y is stochastic, i.e., it may be generated sequentially in time $y1,...,yi,...,$ and, in this context, it is intended to update inference on the parameter θ, this can be set by replacing the usual expression for a Bayesian updating scheme (Eq. 4-47) as follows:

$$\pi_{i+1}(\theta \mid x) \propto f(y_{i+1}|\theta) \cdot \pi_i(\theta),\ i = 1,2,\ldots,n \tag{4-48}$$

where it can be claimed that just before the next time step, i.e., $i + 1$, the knowledge of θ is summarised in the distribution $\pi_i(\theta)$ associated with the previous time step i. Consequently, the latter is used as a prior distribution for the next time step, $i + 1$. In such evolving context over time, it becomes evident that the characterisation of π_i is only required at time step i, and no earlier information needs to be retrieved. In other words, the latest π_i contains all knowledge relevant for updating when confronted with new information derived from the new collected data, $f(y_{i+1}|\theta)$.

4.3.3 Implementation

In the context of updating the prediction of a performance model up to a point-in-time in the future, i.e., t_f, when data collected from a monitoring system, NDT, visual inspection, or a combination of these becomes available at the present moment, i.e., t_i, a Bayesian approach supported by Eq. 4-47 and 4- 48 can be implemented. Sequentially, the methodology comprises the following five main steps (according to Figure 4-7):

- Step 1, Characterisation of the performance model prediction:

  Generation of a set of time-series for a selected model based on, for instance, a Monte Carlo simulation and the characterisation of the random variables. Quantification of the prior distribution $\pi\,[J(t,t_0)]$ and the mean, $\mu_J(t,t_0)$, and standard deviation, $\sigma_J(t,t_0)$, over time.

- Step 2, Characterisation of the model parameter from the data collected:

  Selection of a set of measurements over time. Quantification of the mean, $\mu_J(t,t_0)$, and standard deviation, $\sigma_J(t,t_0)$, over time.

- Step 3, Characterisation of the likelihood distribution of the model parameter:

  Generation of the likelihood distribution for $t = t_i$, $f\,[\tilde{J}(t_i,t_0)|J(t_i,t_0)]$, based on the mean and standard deviation calculated in Step 2.

- Step 4, Characterisation of the posterior distribution of the model parameter:

  Quantification of the posterior distribution for $t = t_i$, based either on the single point updating method, $\pi\,[J(t_i,t_0)|\tilde{J}(t_i,t_0)]$ (Eq. 4-47), or the sequential updating $\pi_{i+1}\,[J(t_i,t_0)|\tilde{J}(t_i,t_0)]$ (Eq. 4-48).

- Step 5, Characterisation of the posterior distribution of the model parameter at the targeted future date, t_f:

  Quantification of the posterior distribution for the target date (e.g. structure lifetime), t_f, $\pi\,[J(t_f,t_0)|\tilde{J}(t_i,t_0)]$, based on the extrapolation of the posterior distribution calculated in Step 4.

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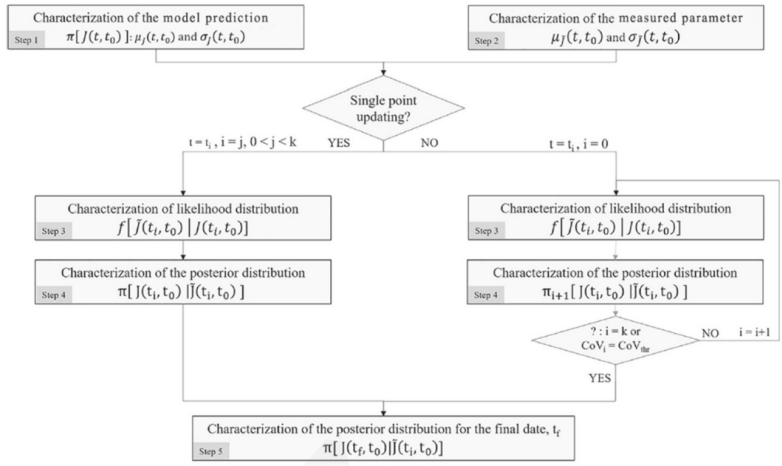


Fig. 4-7 Algorithm for the updating of performance prediction based on monitoring system, NDT, visual inspection, or a combination of these[248].

It is worth noting that when new information is continuously arriving, i.e., from a monitoring system, NDT, visual inspection, or a combination of these, the sequential updating algorithm presented in Figure 4-7 allows updating the uncertainty associated with a prediction from a stochastic analysis. More precisely, the value of CoV can be used to estimate the required observation period in order to achieve a posterior distribution with a targeted CoV, i.e., below a threshold value (see decision box below step 4 and on the right-hand side of Figure 4-7). This aspect is particularly useful when it is targeted to establish, in quantitative terms, the minimum requirements (i.e., optimal) (i.e., the period of time for the installation of a monitoring system, or the minimum number of NDT spread over time, or a combination of these) that is able to guarantee a specified level of uncertainty for the prediction supported by the performance model and collected data.

Five case studies presented in Chapter 6* introduce methodologies towards the enhancement of structural models supported by data collected by SHM systems installed on the structure.

4.4 Performance assessment towards lifetime assessment

4.4.1 Performance vs. Key-Performance Indicators

In section 1.3, the performance concepts have been presented, although these concepts might be interpreted somewhat differently at different levels and from different perspectives. It is, therefore, the goal of this section to also present these different approaches and perspectives of the PIs and KPIs in the following two sections, i.e., 4.4.1.1, 4.4.1.2, including extension of these concepts for monitoring and testing.

4.4.1.1 Performance Indicators according to the COST TU1406

Cost TU1406 objective was to develop a guideline for the establishment of Quality Control Plans (QCPs) in roadway bridges, by integrating the most recent knowledge on performance assessment procedures with the adoption of specific goals. Based on new approaches to quantify and assess the bridge performance, as well as quality specifications to assure an expected performance level, bridge management of ageing structures in Europe can be significantly improved. This focused on bridge maintenance and life-cycle performance at two levels: (i) Performance Indicators (PIs), and (ii) Performance Goals (PGs). Accordingly, the indicators shown in Table 4-2 were defined as PIs, depicted between PIs that can be collected through inspection, P_{Iinsp}, monitoring P_{Itest}, or tests P_{Itest}.

* Case studies 6.2, 6.4, 6.7, 6.8 and 6.22.

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Table 4-2 Performance Indicators (PIs) for concrete structures (COST TU1406).

Performance Indicator	**Inspection** P_{Iinsp}	**Monitoring** P_{Imoni}	**Testing** P_{Itest}
Cracks	✓	✓	
Crushing		✓	
Rupture		✓	
Delamination			✓
Scaling	✓		
Spalling	✓		
Holes	✓		✓
Debonding	✓		✓
Obstruction/impending		✓	
Displacement	✓	✓	✓
Deformation	✓	✓	✓
Wire break			
Prestressing cable failure		✓	✓
Reinforcement bar failure/bending		✓	✓
Stirrup rupture		✓	✓
Tensioning force deficiency		✓	✓
Loss of section	✓		
Deteriorated mortar joints	✓		
Frequency		✓	✓
Vibrations/oscillations		✓	✓

4.4.1.2 Performance Indicators according to *fib* Commission 8 (COM8)

fib Commission 8 (COM8) aims to identify concrete-related durability issues, considering current available information, by providing guidance on materials and methods to assist in optimal durability design of new and existing structures. Although an agreement for the definition of PI has not been achieved in COM 8 yet, the concept has been introduced in the drafts of MC2020. At present, PI has been understood as “a property or characteristic at the level of the material or of the structure that shows the degree of compliance with requirements specified in the design". For both new and existing structures, it has been identified four Levels of Approximation, LoA, (as defined in MC20210) have been identified, in which the PI are LoA 2, that is in between to design or assess following the prescriptive rules (LoA 1) of the existing codes and the use of “time explicit models” (LoA 3 and 4). The specification of PI, instead of the composition of the materials, is considered the most elemental strategy to design/assess performance. Moreover, threshold values for the PI have to be specified whose compliance is evaluated along the lifetime. Table 4-3 summarises the different levels of approximation for each deterioration mechanism.

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This aspect related to PIs, is addressed in Chapter 6, where several case studies are introduced including a brief description of PI related to each case study.

Table 4-3 Presently available approaches for SLD assessment.

Deterioration process	LoA 1	LoA 2	LoA 3	LoA 4	Avoidance
Carbonation induced corrosion	✓	✴	✴	✓	✓
Chloride-induced corrosion	✓	✴	✴	✓	✓
Freeze-Thaw	✓	✓	-	-	✓
Chemical attacks	✓	✴*	-	-	✓
Alkali Silica Reaction (ASR)	✓	✓	-	-	✓
Delayed Ettringite Formation (DEF)	✓	✓	-	-	✓
Abrasion	✴	✴	-	-	✓

✓ rules available in most of the codes and/or models assumed being mature enough for being implemented by skilled users;
✴ Limited rules: only available in some national codes, or still under validation and calibration;
- Unavailable: ongoing research;
* Depending on the nature of the chemical attack.

4.4.2 Key-Performance Indicators

Generally, the distinction between PIs and KPIs is not straightforward. For instance, KPIs relate to a whole structure or network and can be classified as follows:

- Reliability is the probability that a structure will fit its purpose during its service life. It is the complement to the probability of structural failure (safety), operational failure (serviceability), or any other failure mode.
- Availability is the proportion of time a structure is open for service. It does not include failure-related service outages but the ones due to planned maintenance interventions. For bridges, it can be measured as additional travel time due to a traffic regime.
- Safety is the situation of life and limb being protected from harm during the service life. Loss of life and limb due to structural failure is not included by this definition (reliability).
- Economy is related to minimizing the long-term cost of maintenance activities over the service life. Herein the user costs incurred due to detours and delays are not included when considering transportation networks.
- Environment is related to minimizing the harm to the environment during the service life.

The collection of PIs takes place at the inspection level. At the asset management level, KPIs are required, as shown in Table 4-4.

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Table 4-4 Relation between PIs and KPIs. Adapted[249].

KPI	PI	Definition (example provided for a bridge structure)
Structural integrity (S, R, S)	Safety (structural)	Related to minimizing the harm to people during the service life of a bridge. The loss of life and limb due to structural failure is not included.
	Reliability (structural)	The probability that a bridge will be fit for purpose during its service life.
	Security	Long-term functionality without much intervention.
(A, M)	Availability	Time proportion in which a system is in a functioning condition. Disruption originates from planned maintenance interventions.
	Maintainability	Facility with which a product can be maintained to repair the damage or its cause, repair or replace defective components without having to replace still-working parts, and avoid unforeseen maintenance measures.
Economy	Owner's costs	Adequate costs for construction and operation.
	Social costs	Acceptable and rare detours/accidents related to minimizing long-term costs and maintenance activities over the service life of a bridge. Herein the user costs incurred due to detours and delays are not included.
Environment	Greenhouse gas emissions	Associated with minimizing damage to the environment during the lifespan of a bridge and balancing emissions with the utility of the structure.
	Resource consumption	
	Waste generation	
(H, P)	Health	The absence of causes of diseases other than failure (for example, the use of asbestos), which in most cases is regulated.
	Politics	Includes the elimination of the causes of public protest, image protection, etc.

From a conceptual point of view, the mentioned KPIs may sometimes seem to overlap (e.g., Reliability and Availability) nevertheless, the combined performance of a structure given by different KPIs may be considered through a spider diagram as in Figure 4-8. In this case, each KPI is indicated on a qualitative scale, where the closer to the exterior of the diagram the better the asset is represented by that KPI. However, the overall performance is the combination of all considered KPIs by calculating the area enclosed by all of them, meaning that a larger area corresponds to a better performance.

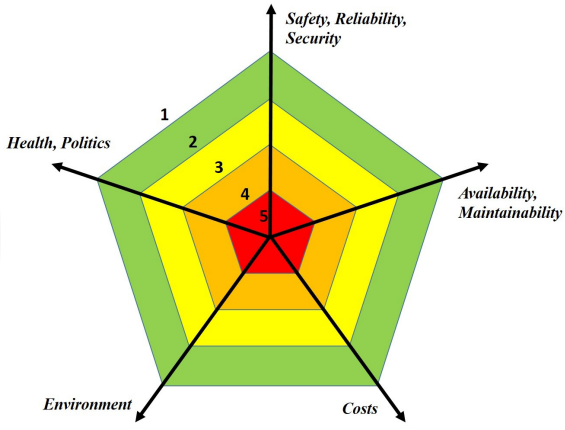


Fig. 4-8 Spider net diagram[119].

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The selected qualitative scale given to each KPI may be then correlated to a quantitative scale. As example, Table 4-5 indicates these correlations for reliability, safety and availability, as well as providing a scale for urgency of intervention. In the case of reliability, the mentioned scales are valid when the governing failure mode is considered. For that purpose, relevant failure modes can be defined based on design documentation and vulnerable/critical zones are to be considered for the failure modes. These critical zones are those segments and/or elements of a structure in which damage has the largest impact on the PIs.

Table 4-5 Scale for KPIs and the urgency of intervention[250].

Scale	Quantitative scale		Qualitative scale
	Reliability	**Serviceability**	**Urgency of Intervention**
1	> 4.00	> 2.50	Regular Inspection.
2	3.25 - 4.00	2.00 - 2.50	Reassessment should be performed to update the period between inspections.
3	2.50 - 3.25	1.50 - 2.00	Reassessment should be performed to plan an optimal time of intervention.
4	2.00 - 2.50	1.00 - 1.50	Reassessment and possible intervention shall be performed shortly after an inspection.
5	< 2.00	< 1.00	Immediate action/intervention is required.
Safety			
1	Injury return period > 100 years		No danger. It is very unlikely that a person could get injured because of the current bridge performance.
2	Injury return period - 75 years		It is very unlikely that a person could get injured because of the current bridge performance.
3	Injury return period - 50 years		It is unlikely that a person could get injured because of the current bridge performance. Intervention shall be performed before the next inspection.
4	Injury return period - 20 years		A person could likely get injured because of the current bridge performance. Intervention shall be performed shortly after the inspection.
5	Injury return period < 10 years		Immediate danger. A person could likely get injured because of the current bridge performance. Immediate action is required.
Availability			
1	1		No restrictions on traffic.
2	2		Weight, speed, and lane restrictions for heavy trucks.
3	3		Closure except for cars and regular lorries. Possible lane restrictions for regular lorries.
4	4		Closure except for cars. Possible lane restrictions for cars.
5	5		Complete closure.

The general framework ontology given in Figure 4-9 provides an entity relation where the “crow foot” symbolises the one-to-many relationship whereas the “crow foot with a circle” stands for one-to-zero or one-to-many relationships. In a broad sense, the diagram describes that a given observation of a certain property in a specific element and critical zone, which are related to a relevant failure mode, in combination with other data may have a relevant impact in a defined KPI expressed by a performance value.

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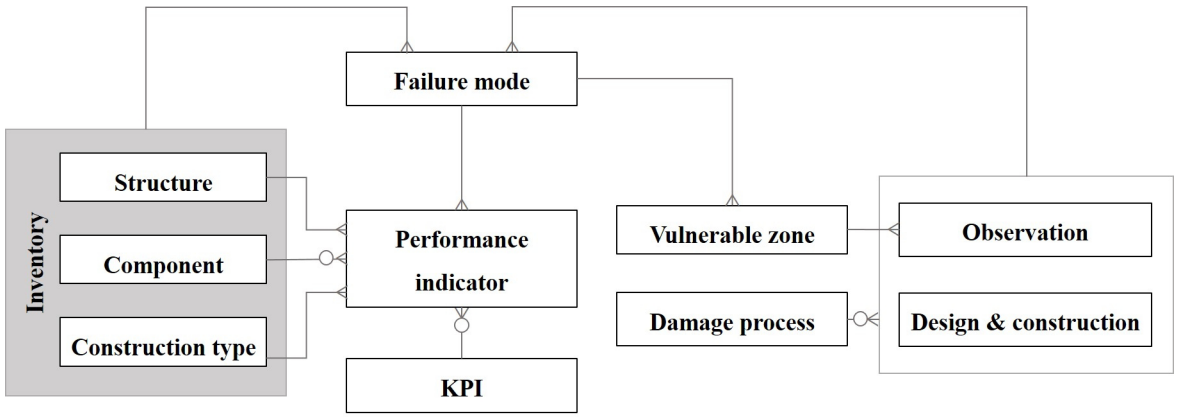


Fig. 4-9 The ontology of a General Assessment Quality Control Framework[250].

4.4.3 Key-Performance Indicators vs. lifetime assessment

A proper QCP specifies all activities and tools, needed to ensure quality throughout the lifetime of a structure/infrastructure. In the case of infrastructure, the QCP defines the extent and the interval of inspections or investigations and data necessary to estimate the KPIs and forecast their future development. QCP also includes a decision model that suggests maintenance actions based on the forecast of KPI. In this sense, the QCP overlaps with the Strategic Asset Management Plan (SAMP) and Asset Management Plan (AMP) as defined in ISO 55000.

The evaluation of KPIs based on PIs is crucial for an efficient QCP. For instance, and as exemplified in Figure 4-10 for reliability, the correlation may be considered based on a qualitative assumption related to conventional inspection framework leading to a condition rate. The critical/vulnerable zone/element is inspected along time and the detected change of condition rate may be related to a PI associated with resistance reduction. With that, reliability may be updated considering the resistance reduction associated to that PI (Figure 4-10).

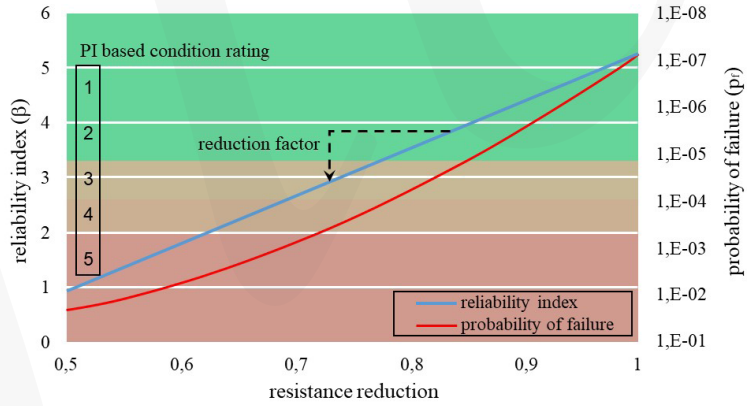


Fig. 4-10 The relation between KPI reliability and PI detection[249].

As mentioned, KPIs can be represented by a "spider diagram", which by extending it with an orthogonal axis allows to inclusion of the development of each KPI over time – a so-called "performance tube" as illustrated in Figure 4-11. Such representation is convenient to analyse and decide upon interventions and maintenance actions considering both failures due to a severe deterioration process or a specific hazard. In a broad sense, diagram necks correspond to time intervals of low performance, whereas segments of the tube with complete cross-section correspond to periods of normal performance. Moreover, the volume between the performance tube and the full pentagon solid may be considered as the performance deficit that is aimed to be minimised.

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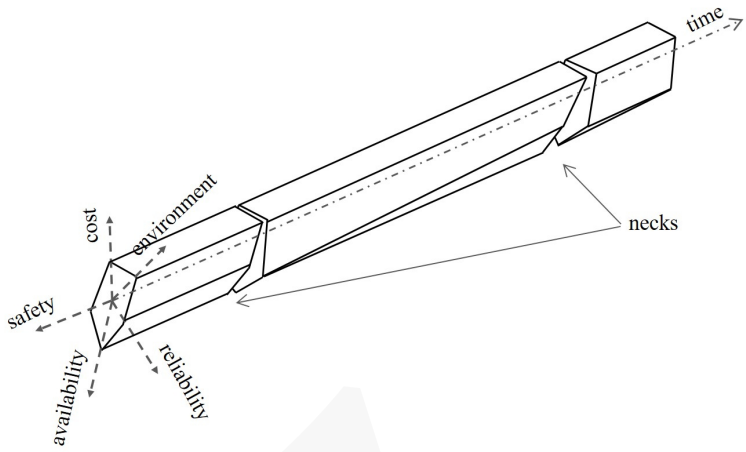


Fig. 4-11 Performance tube representing the evolution of KPIs over time[249].

Within a QCP, the KPIs will be evaluated for different maintenance scenarios, i.e., based on inspection/investigation or prediction, aiming at optimal and efficient use of resources. In that sense, KPIs must be time-dependent. Therefore, inspection, monitoring and testing are crucial to determine damage processes, which by having a detrimental effect on a structure are determinant on the performance prediction and consequent choice of preventative maintenance and eventual rehabilitation. In that sense, the QCP may be given in two stages: i) time static QC and; ii) time dynamic QC[249]. In the former case, thus for a specific period, quality control comprises: (a) preparatory work including inventory, conceptual weakness of design, material weakness, load changes, identification of vulnerable/critical zones, estimating a priori reliability; (b) onsite inspection including damage survey and detection, material property measurement, sample collection; (c) laboratory tests; (d) assessment of the reliability KPI based on resistance reduction estimates and reliability estimates; (e) assessment of the safety KPI. Whereas in the latter case, the QC comprises: (a) assessment of remaining service life by assessing the damage rate and forecast, time-dependent safety and reliability analysis; (b) maintenance scenario including reference scenario, - end of service life, preventative scenario, as well as long term cost, availability and reliability/safety estimates for scenarios; (c) decision-making supported by multi-objective/attribute optimisation, as well as monetise non-monetary KPIs and finding the optimal scenario.

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5. Decision making processes

5.1 Introduction

A decision-making process is defined as a process, where an individual, a group or an organisation reaches conclusions for its future actions[251]. This process is affected by several factors with different significance for those involved. The consideration of each factor depends on the interest of the respective group and can also be opposed to other aspects. Especially for existing concrete structures, there is a variety of current and future aspects to be considered. During the service lifetime of a structure, significant parameters of load carrying capacity or of durability can also change. In the context of planning, building, maintaining, monitoring or dismantling concrete structures, the responsible institution must consider all time-dependent interests of the various stakeholders.

During the process of condition assessment, the available information must be analysed, and the structural condition is rated using the available information. While engineering models can describe various attributes of a structure, there are also characteristics and views that will stay uncertain. Depending on the age of the structure, existing damage, load history and structural details, a decision for a further use during the remaining lifetime must be made.

In the remainder of this chapter the engineer in charge of an existing concrete structure is given a guideline on which decisions have to be made and general methods for making them. In the first part of this chapter the possible involved groups of stakeholders involved and their interests are introduced. The necessity and target of the decision-making process are pointed out. In the second part, specific decision-making channels are presented for selected topics. The third part gives some practical proposals for making decisions on performance modelling and shows typical decisions during a statical recalculation. Figure 5-1 gives an overview of the contents of this chapter.

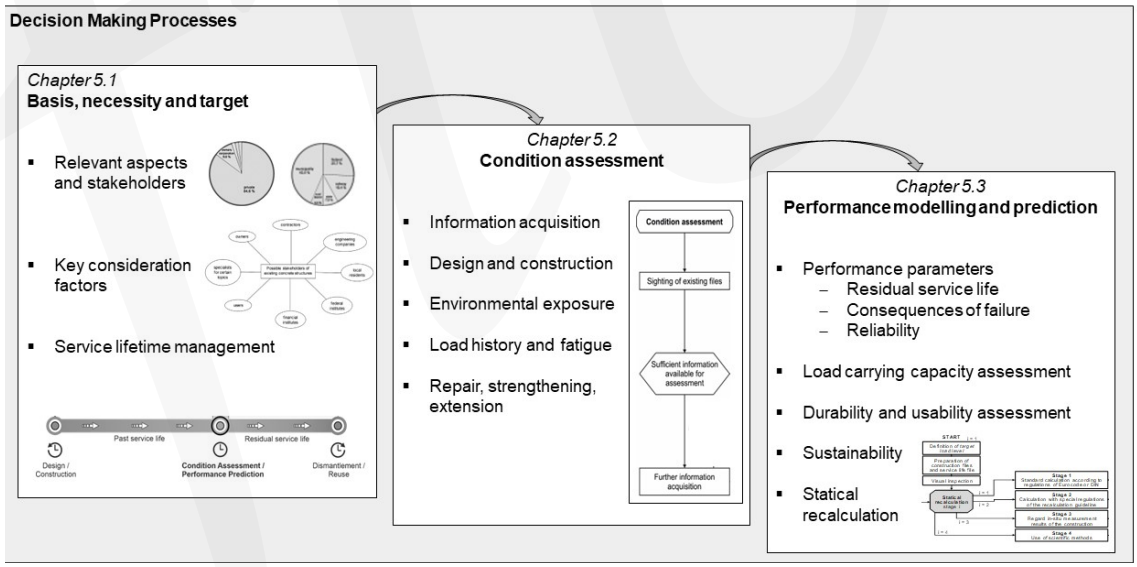


Fig. 5-1 Concept and framework of Chapter 5.

This chapter was written by Braml (main author), Kainz, Keuser, Zimmert (main author).

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5.1.1 Stakeholders of existing structures

A stakeholder is defined as a person or a group, which is affected by an activity or that has any kind of reasonable interest in the structure. The stakeholders of a concrete structure are its owners and users as well as local residents, contractors, general planners and specialists for certain topics, financial institutes and in the case of state-owned structures basically every citizen, see Figure 5-2.

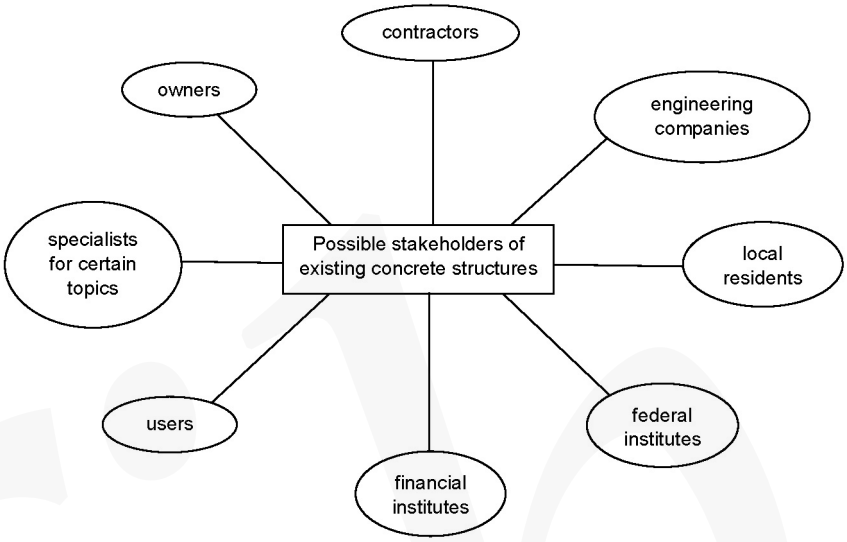


Fig. 5-2 Potential stakeholders of existing concrete structures.

These groups have different interests, personal qualification or technical knowledge and therefore, depending on the particular stakeholder, certain factors can be valued differently. The level of knowledge and specialisation is an important characteristic of a stakeholder, because with an increasing level of technical know-how, the type and number of possible decisions varies. Therefore, information about who owns or manages a structure is essential to know about for understanding certain decisions better. For example, there exist 39'500 bridges in the German federal highway network, which means only about every fourth bridge in Germany is directly under federal authority (see Figure 5-3). With a count of 67'000, most bridges in Germany are in the authority area of communal institutions such as municipalities and rural districts[254]. A specificity of bridges maintained by municipalities is that in many cases they are smaller than highway bridges and 70 percent of their total number is under the jurisdiction of municipalities with a population of less than 200'000 people[252].

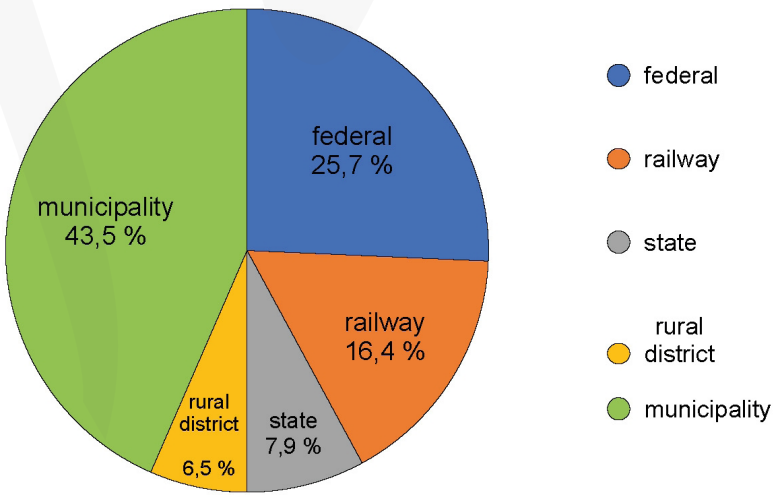


Fig. 5-3 Overview of the responsible authority for the maintenance of bridges in Germany related to the total number of bridges[252], [253], [254].

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With an increasing size of the responsible administration, there are more possibilities for specialised consultants and executive departments. Alongside the road bridges, about 25'700 railway bridges are managed by the Deutsche Bahn AG[253]. While almost every bridge is under administration of a government institution or the DB AG, the ownership structure of residential buildings is significantly different. The majority of residential buildings is in private ownership, see Figure 5-4.

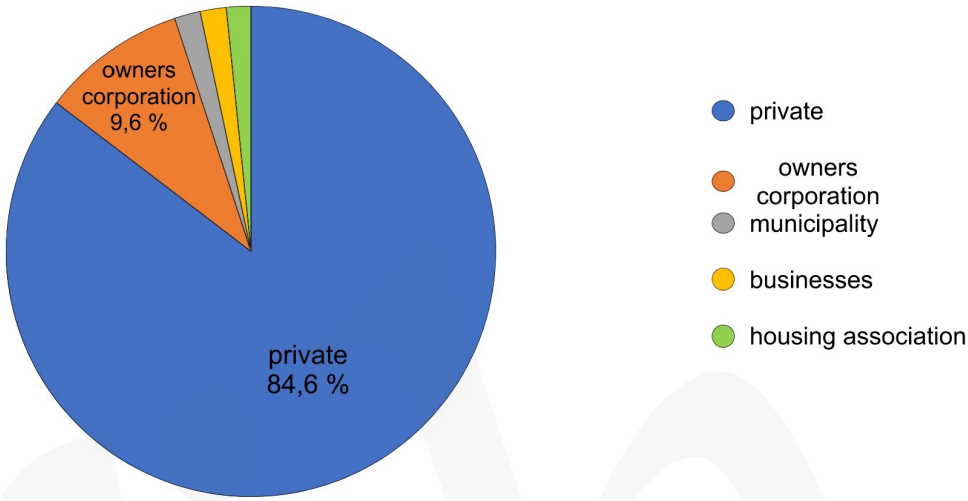


Fig. 5-4 Share of the owners of residential buildings in Germany[255].

5.1.2 Necessity & target of decision-making processes

The target of the decision-making process is to find reliable and reasonable choices for further actions according to the criteria defined as most important and by regarding the particular boundary conditions of individual structures. Making optimal decisions about existing concrete structures will be more and more significant in the future because of their number, current age, situation and condition. A standardised decision-making process is necessary to get comparable results for various structures, which is important for example for establishing a priority order for funding or maintenance[256], [257].

Several codes contain rating criteria for the standardised and consistent assessment of structures. The assessment process of an existing structure starts with the identification of the need for a decision. With monitoring a lot of data can be gained, from which the relevant information must be extracted and analysed. Depending on the existing knowledge of the engineer in charge, an understanding of the results is obtained and measures can be taken[258]. During this process, there are many decisions necessary at various points in time. But even with a standardised approach, e.g., the results of an in-situ inspection can depend considerably on the qualification, knowledge and experience of the individual engineer[259]. Commonly the short-term criteria are more relevant for the users of a structure whereas the owners are focused on long-term processes. For example most users are concerned about usability and functionality while costs and maintenance are the interest of the owners or administrators. Figure 5-5 and Figure 5-6 show a possible rating of factors in the case of bridges, and a possible ranking depending on the particular stakeholder.

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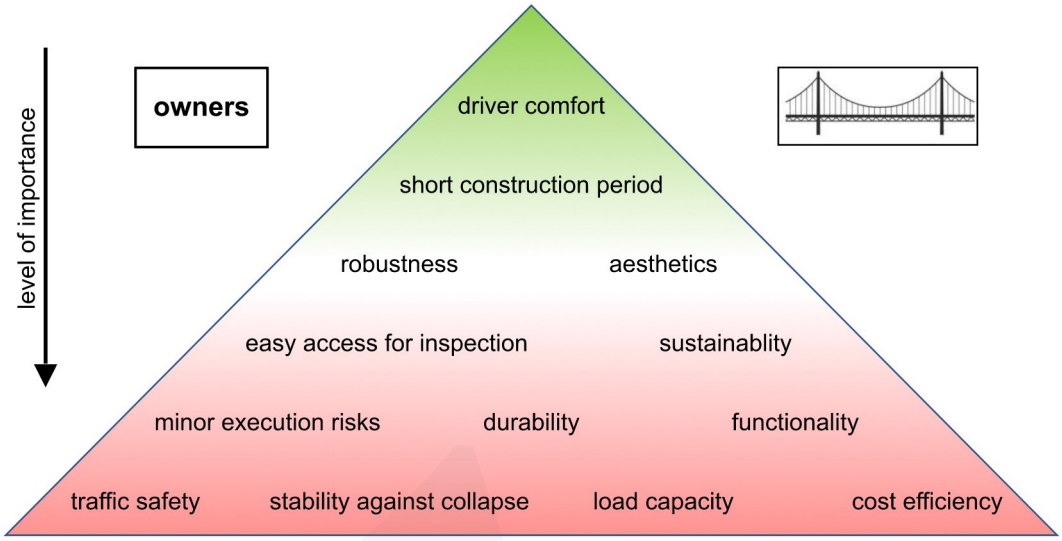


Fig. 5-5 Scheme of key consideration factors for the owners of bridge constructions in the design of Maslow's hierarchy of needs.

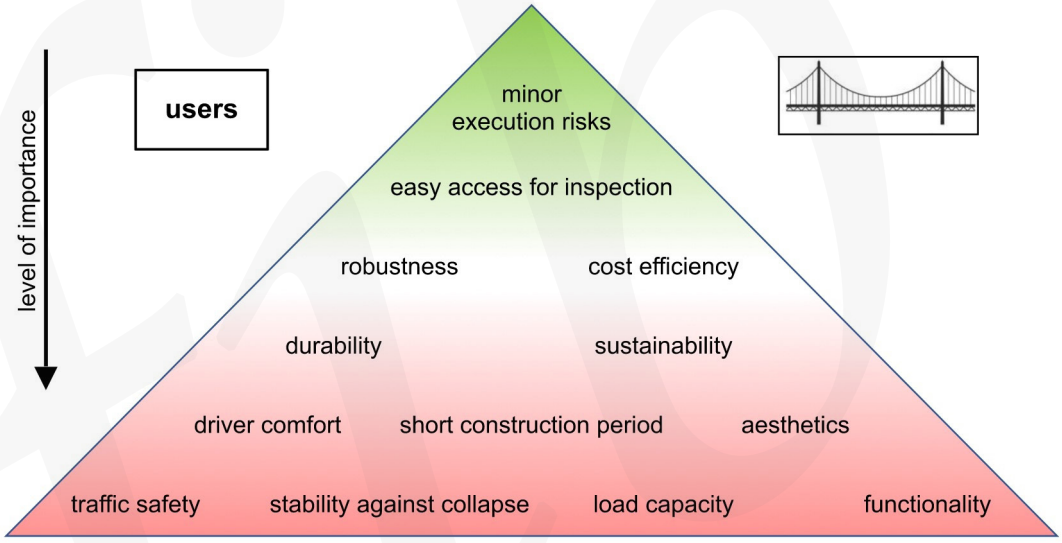


Fig. 5-6 Scheme of key consideration factors for the users of bridge constructions in the design of Maslow's hierarchy of needs.

In the following some aspects reported in the previous figures, and which might involve several decision alternatives, are briefly discussed.

The driver comfort involves vibration stresses for the passengers, free sight distance, road holding of the car or damaging effects on the cargo. It is mainly constrained by excessive sag or damage in the road pavement, which occur especially near the construction joints, due to heavy traffic.

Even though the valuation of the aesthetic of an engineering structure is an ambiguous and wide issue and the design depends on many individual circumstances, the engineer in charge has to identify the relevant boundary conditions and make a decision about their consideration. A collaboration between engineers and architects can lead to improved results in view of both construction and aesthetics. One has not only to consider the aesthetics of the bridge itself, but also its adaption into the landscape. An excellent decision support for the aesthetic and functional design of bridges can be found[260].

Robust concrete structures do not collapse suddenly and not without prior noticeable indication after an exceptional loading or after the failure of load bearing parts. Instead, the imminent collapse is noticeable with visible deformations or large cracks. The request for more robustness, especially for pre-stressed

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concrete structures, is a demonstrative example of the possibility of changing importance of aspects over time. At the time of the first application of pre-stressed concrete several decades ago, there was not much value attached to the robustness of a structure. But since then, several additional regulations were introduced into European codes, like the demand for additional reinforcement as well as a limitation to the applied forces on the tendons, and for modern structures robustness is highly relevant.

Short construction periods become more and more important, because of the heavily increased traffic volume in the last decades and especially for existing concrete structures, where construction work must be done while maintaining the use of the structure. Minimised traffic obstructions are desirable, not only in terms of reducing the higher accident frequency in construction zones but also to minimise economic impact due to extra time for redirections and traffic jams. Possible decisions to optimise the construction period can be relevant to the selection of construction methods, e.g., precast elements, or the construction sequences. On the other hand, these decisions also affect other aspects like costs or execution risks. For example, a shortened construction time can lead to higher construction costs, because of higher expenses for certain processes or prefabricated products. However, if one can optimise other areas with extended planning, like the construction phasing of the structure, both costs and construction time can be reduced. So, the different factors interlink with each other.

The importance of these subjective factors must be evaluated in an objective way considering the quality and uncertainty of data. Possible methods to do so will be presented in this chapter. The approach for regular inspections depends on the type of the structure. In this perspective three major categories of structures can be identified.

Temporary constructions have only a short service lifetime and are designed for fast changing demands, which is typical for industrial buildings. If there are no exceptional events, like a damage case or the wish for a change in their use, temporary structures usually require no decisions until the end of their planned operating lifetime. After the end of the service life, a decision on how to further proceed with the existing structure must be made.

For structures without regular inspections, e.g., residential buildings or office buildings, there are decisions to be made whenever exceptional events occur, or if the condition of the structure is not suitable for the current requirements. Possible reasons can be damage, that compromises the structural safety, or difficult marketability, which is important especially for residential and office buildings with outdated floor plans or equipment.

The third category are structures with regular inspections like bridges and halls. To enable inspection and to guarantee a safe working environment for the inspection staff, an easy access to the structure and its construction parts is required. Regular inspections of a structure generate a decision-making basis and based on this information, a decision for the further actions to implement must be made. For certain structures, there are standards for the execution of the checks. For example for bridges the DIN 1076 or for buildings the VDI 6200. In Germany, bridges and infrastructures are checked by default every six years in an intense and major checking procedure and, in between, after three years in a minor check. The assessment and evaluation of their results require decisions on the further steps. Possible strategies are a more detailed investigation or repair measurements. A continued usage of the structure without any repair can also be possible, but the circumstances may require usage restrictions. Between the recurring intervals of the inspections, decisions are still required in case of exceptional events like for the other types of structures mentioned before.

Demands on stability, usability and durability must be considered when making a decision for an existing structure considering the requirements related to the future usage of the structure. The most important aspect in this context is the structural stability of the building. The safety of life and limb is regulated by national laws, e.g., in Germany in the state building regulations. The owners of a structure are responsible for the stringent compliance of their property with the laws. The same goes for the environmental law, which is based on European regulations. In this respect, the economic interests of the owner are subordinate. Opposed to that, when it comes to decisions about usability and durability, there are still legal regulations to consider, but the economic interests of the owner have priority.

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5.1.3 Decision-making in service life management

The life cycle of a concrete structure can be divided into four main stages and graphically represented using a time beam, as shown in Figure 5-7 (according to MC2010). At the beginning, the whole structure is designed by owners, engineers and architects (stage 1). Later, the service life (stage 3) of a structure is framed by construction (stage 2) and dismantlement (stage 4).

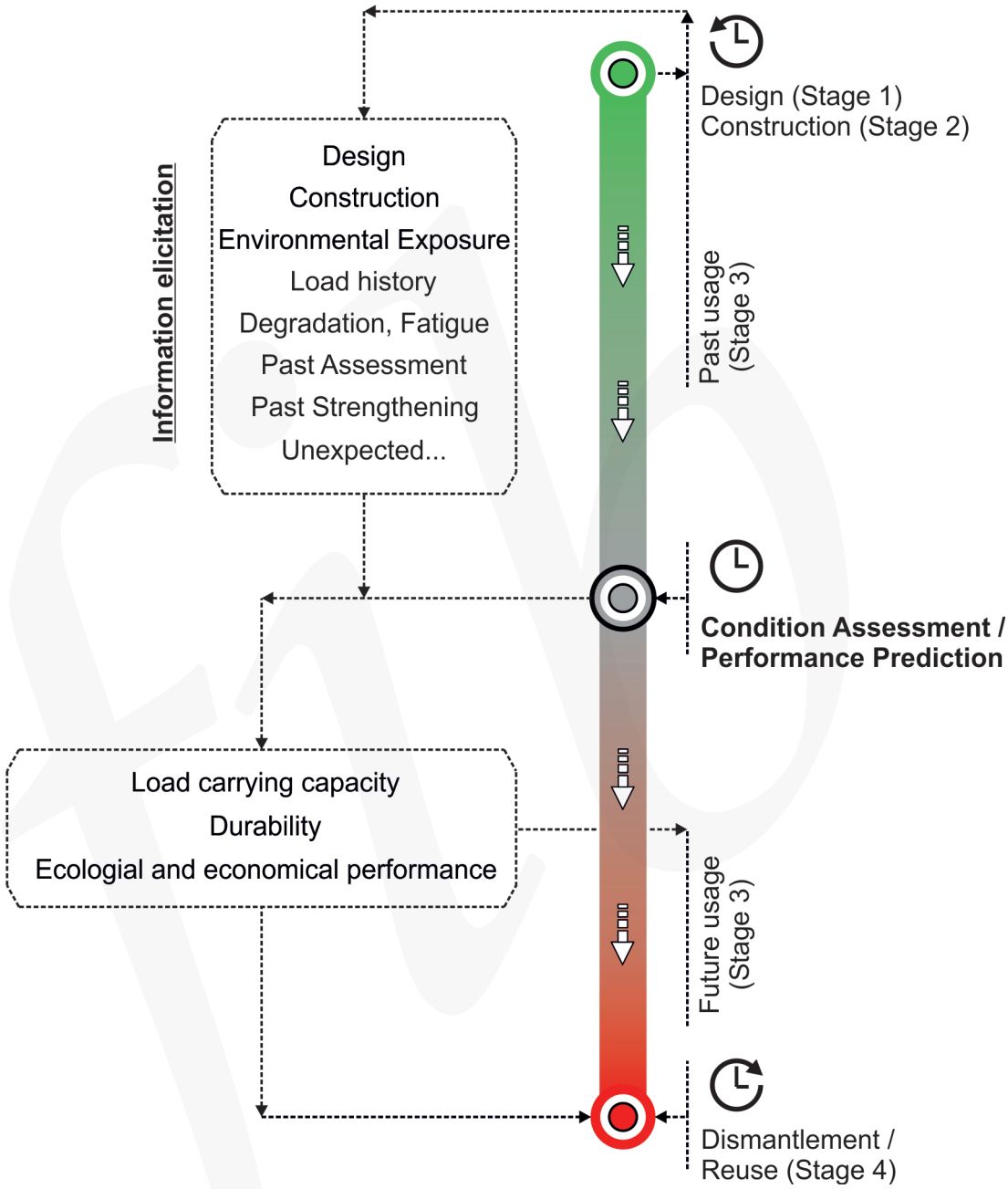


Fig. 5-7 Decision making for service life management.

During the design, a variety of decisions with influence on the whole service life of concrete structures have to be made. Excluding individual cases, these decisions are mostly made considering descriptive construction rules or assumptions based upon the current state of technical expertise at that point of time [261], [279].

Most common examples for these decisions are in general the selection of the material, the structural system or the cross section and, in detail, the choice of the concrete strength, the cement type or the thickness of concrete cover. Today's codes and guidelines, like the MC2010, provide necessary

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information for concrete structures to perform well with moderate costs for maintenance or repair over a period of 50 years, which is the standard design service life, and up to 80-100 years for bridges.

An example of the overall "as planned" performance of a structure, designed according to current guidelines and codes, is shown in Figure 5-8. While the durability of a structure gradually decreases, the load bearing capacity is expected to stay constant and additional costs for maintenance or repair to stay low. The time dependent development of a structure's environmental impact is not calculated at all in design codes.

However, at a later point in service life, the performance of a structure may turn out to have been overestimated. Especially in cases where structures are exposed to unexpected environmental / physical influences or simply exceed the planned service life, with relevant deviations from the "as planned" performance, for example as shown in Figure 5-8.

These deviations are the moving cause of condition assessment and performance prediction processes, methodologically described in the preceding chapters of this Bulletin. In the remainder of this chapter, the main decision making for condition assessment and performance prediction will be described in a practically oriented way.

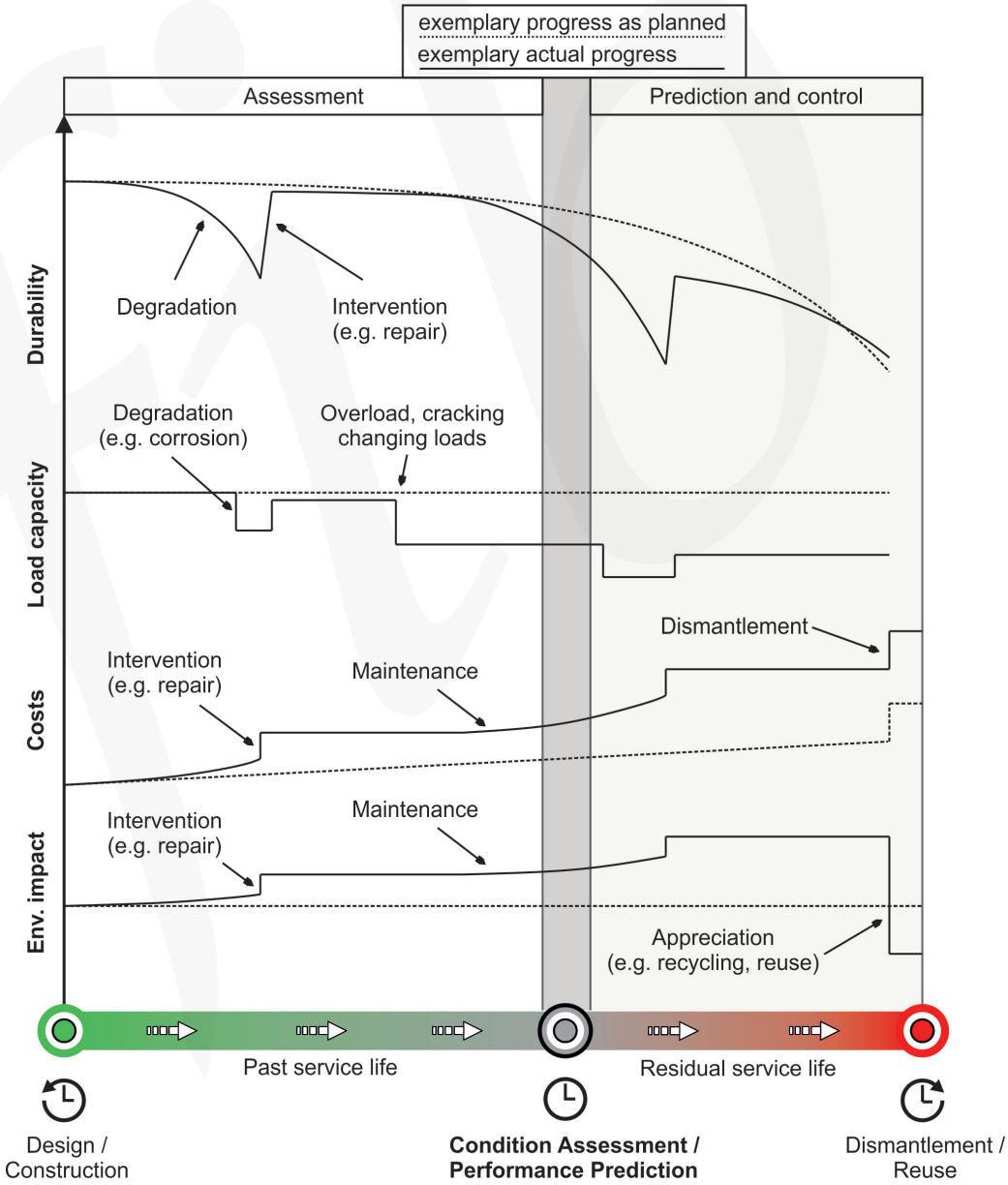


Fig. 5-8 Potential development of a structures performance during service life (adapted and extended[260]).

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5.2 Condition assessment

5.2.1 Overview

Throughout the stages of design, construction and use, several time dependent or time independent conditions occur, having a varying grade of influence on the future performance of a structure. Besides certain uncontrollable circumstances, these conditions are oftentimes the outcome of fundamental decisions already made during the life of a structure[262]. It is the intrinsic motivation of service life management to predict and evaluate the influence of such decisions on the performance of a structure. A fundamental approach thereby is the procedure of condition assessment, for which the main goal is the determination of a structure's current condition and present state in service life.

During the condition assessment process, the following questions arise:

- Which information is available and relevant?
- Which depth of information is required?
- How can the information be acquired?

Answers to these questions cannot be given in a general manner, since they are strongly correlated to individual assessment processes, as described in Chapter 2 and Chapter 3, or explicitly addressed by methods of performance prediction and modelling (see Chapter 4). Consequently, the following subchapters give the interested reader an overview of existing information with relevance to condition assessment procedures. The general procedure of information acquisition during condition assessment and for purpose of performance modelling is shown in Figure 5-9.

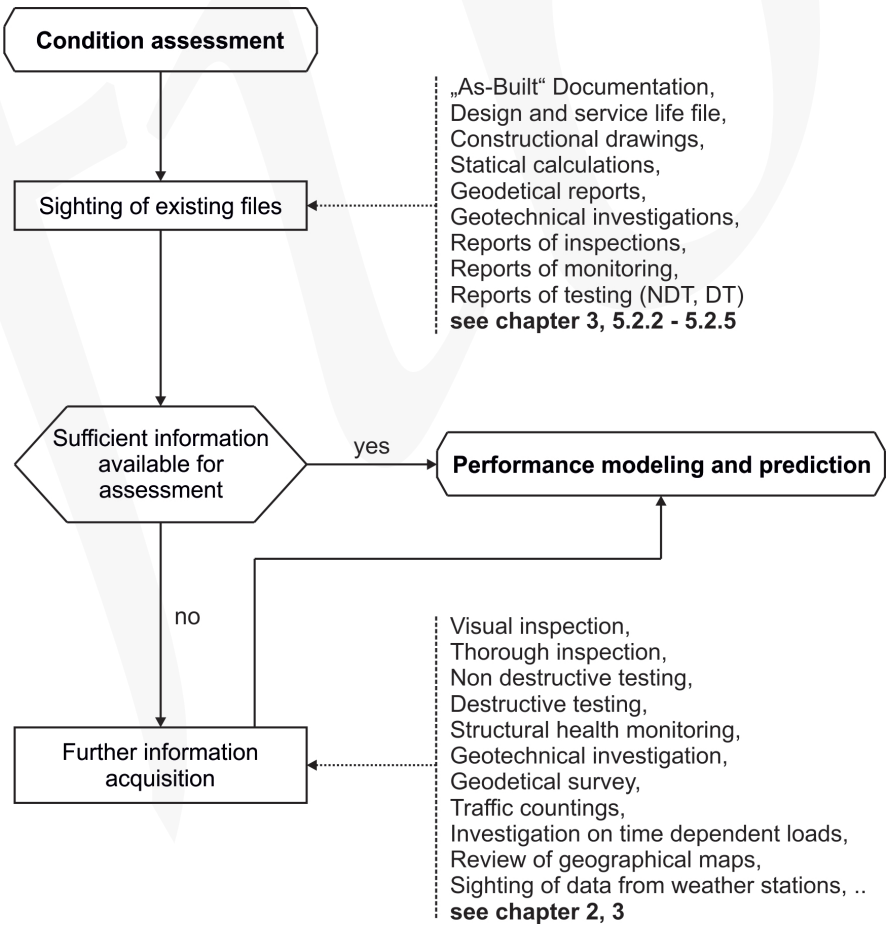


Fig. 5-9 Procedure of information acquisition in condition assessment processes.

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5.2.2 Design & construction

The design of RC elements and structures mainly implies calculation-based specifications of the expected load bearing behaviour, the provision of structural details and the selection of building materials. Nowadays the design is carried out adhering to international standards and guidelines, like for example EN 1990 and EN 1992. Existing structures nevertheless may have been designed adhering to older documents or special circumstances which makes it necessary to gather the information that is deviating from today's standard. Furthermore, during the construction stage, desired or accidental deviations from the design may have occurred.

During the process of condition assessment, decisions have to be made, on which information from the design and construction stage is available and how it is considered to be acquired. The different types of available information are listed in Table 5-1 and are briefly described below.

5.2.2.1 Structural Details

Structural details have a high influence on the load capacity, durability and consequently the performance prediction of RC structures[263]. Possible aspects are the concrete cover depth, the amount and position of reinforcement or tendons, technical equipment and drainages. Typical details for bridges in Germany are predefined in code drawings like the RiZ-ING, which were first introduced in 1972 and, since then, steadily updated to give approved construction details and support the decision-making process for typical details. In the first instance, structural details can be evaluated by examining of existing files and drawings, and partially by visual inspection. If such documents are not available, or if a higher reliability of the information is required, it is necessary to decide between different methods of DT and NDT, described in depth in Chapter 2, to acquire the required information.

5.2.2.2 Materials

Knowing about the main properties of construction materials used in an existing structure is essential for both the condition assessment and the performance prediction. The necessary depth and reliability of such information depends on the type of performance prediction method. For instance, for a detailed deformation analysis, it is necessary to know about the current modulus of elasticity of the concrete. This type of information can be acquired by existing documents relevant to the structure. If those are not available, or a higher precision of information is required, several DT and NDT methods can be executed (see Chapter 2). Based on the test results, the design values of the material properties can be estimated. A typical example of interest for the planning engineer is the characteristic concrete strength for the calculation of which e.g., EN 1990, for instance, provides a procedure based on in-situ information.

5.2.2.3 Load Bearing

The load bearing behaviour of an existing structure and its structural parts must be analysed thoroughly for the purpose of performance prediction and modelling. This evaluation is regularly performed through the analysis of existing static calculations and construction plans. It has to be decided if a static recalculation is necessary to deliver the relevant data. This might be necessary in cases where for example the original calculation was performed according to outdated standards, or simplifications have been made due to the unavailability of automatic computing resources.

5.2.2.4 Design deviations, incidents and damage

Deviations from design, which may have been introduced during construction, should be identified during the assessment process. In addition to that, design codes represent the generally accepted technical rules at the time of design, but not necessarily state-of-the art technology. Examples of the past for such deficits are the values provided for stress corrosion cracking, and aspects related to the impact of fatigue or temperature on the structural performance. In combination with information from potential incidents and damage during construction, this information is highly relevant for the purpose of assessment

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and performance prediction. Information about potential and unknown design deviations, incidents and damage can only be acquired by DT and NDT testing methods or e.g., geodetic survey (see Chapter 2, Table 2-2, Table 2-3). Therefore, decisions have to be made on the methods of information evaluation.

Table 5-1 Basic information evaluation for decision making from design and construction.

Design			**Construction**	
Structural Details	**Materials**	**Load Bearing**	**Design Deviations**	**Incidents/ Damage**
Thickness of concrete cover	Cement type and water-to-binder ratio	Simplifications	Geometry	Insufficient concrete cover
Anchorage of reinforcement bars	Concrete strength and modulus of elasticity	Utilisation	Materials	Improper deformation
Lapped joints of reinforcement bars	Reinforcement strength and modulus of elasticity	Redundancies	Construction sequence	Insufficient compacting
Minimal reinforcement	Weldability of reinforcement	Fatigue	Reinforcement	Insufficient grouting of tendons
Quality of strut-and-tie models	Bond characteristics of reinforcement	Settlements	Structural parts	Exceeding settlements
Bearings	Fatigue strength of reinforcement	-	Bearings	Cracking
Split tensile reinforcement	Fatigue strength of concrete	-	-	-
Coupling joints (tendons)	Sealing arrangements	-	-	-
Drainage	-	-	-	-

5.2.3 Environmental exposure & degradation

Environmental impacts are one of the most common damaging causes of damage for concrete structures. Any concrete structure is exposed to several environmental influences during its service lifetime and at various stages. The current exposure can be measured directly, whereas past events, which might have caused deterioration, must be recorded to know about them. In a first step, the status of the structure must be examined. Depending on several factors like severity, area of application, exposure time or damaging potential, various aspects must be considered during the decision. To assess the appearance and intensity of the potential damage the engineer in charge has several options. Based on the condition of the structure, an estimation for the future degradation should be made. The decision about the type of test to perform must be made before there is knowledge about the result. Since every investigation involves costs, this decision should consider the additional value the test will bring[196]. When deciding about the use of testing methods, the individual structure must be considered, e.g., access to potential attacked spots when covered under asphalt or in intense traffic. If inventory documents are missing, this rather reasonable approach is compromised. The basic information evaluation for decision-making in terms of environmental exposure and degradation is summarised in Table 5-2.

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5.2.3.1 Chloride

Possible contact with chlorides can have significant influence on the durability of a concrete structure and should therefore be examined during the assessment. Information about the nearby use of de-icing agents can be gained by inquiring of the owner, manager or responsible authority for the structure. In many cases, the decision about the use of de-icing agents is discussed during the design process and documented in the construction files. Further information about existing damage can be acquired by using the methods mentioned in Chapter 2, Table 2-2. The critical chloride content depends on various factors and therefore it has to be evaluated individually by an expert, if certain numbers are exceeded. As part of this examination, the areas where concrete should be removed have to be determined. A specialist should be consulted if the chloride content exceeds 0.6 wt.%. in RC components and 0.2 wt.% in case of pre-stressed concrete elements. In both cases, the amount is related to the cement mass. This consultation must take place, irrespective of whether there is evidence of corrosion on the concrete surface or not[264], [265].

5.2.3.2 Freeze-Thaw

Recurring freeze-thaw attack can damage a concrete structure. The intensity of freeze-thaw stress can be reliably identified based on data from weather stations. The deterioration mechanism and possible damage are explained in section 1.2.3.

5.2.3.3 Carbon dioxide

Carbonation starts from the concrete surface, continues over time initially and depassivates the reinforcing bars. Because the loss of the alkaline protection enables damage due to corrosion of the reinforcement, information about the intensity of the carbonation is very important to acquire. The depth of ingress is of special interest in any reinforced concrete structure, because in order to decide about the remaining service life the engineer in charge has to know about the condition of the passivation of the rebar steel. The deterioration mechanism is described in section 1.2. Possible NDT and DT methods are shown in Chapter 2, Table 2-2. Carbonation is slowed by low water-cement-ratios or intense post-treatment and depends on the environmental conditions. Its maximum speed occurs at a RH from 50 to 70% while dry or completely wet components barely carbonate.

5.2.3.4 Moisture

Permeating water can infiltrate concrete elements via cracks or due to damaged sealants. It is a common factor which has often consequences for usability or other non-concrete parts. The reinforcement is also more likely to corrode in a moist environment than it is in a dry place. Information about moisture can be found through on-site visual inspection of the affected element, ideally at a time or season where the impact is easier to detect visually, e.g., after long rain periods. Information about moisture can be accumulated for certain accessible spots with measuring methods as described in Chapter 2. A decision about possible methods for renovation, e.g., application of sealants, can be made with consideration of the cost-benefit ratio and accounting for the available knowledge about the ambient conditions.

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Table 5-2 Basic information evaluation for decision-making: environmental exposure & degradation.

Chloride	**Carbon dioxide**	**Moisture / Temperature**	**Mechanical**	**Chemicals**
Nearby usage of de-icing agents	Average carbon dioxide content in air	Surface moisture content	Abrasion due to vehicles	Nearby location of chemical emission
Chloride content	Nearby location of high carbon emission	Average air moisture content	-	Chemicals in ground water
Depth of ingress	Depth of ingress	Penetrating water	-	Chemicals in soil
	-	Direct exposure to rain and wind	-	-
		Ambient temperature range		
Physical and chemical attack to concrete				Chemical attack to concrete
Chemical and electrochemical attack to reinforcement				

5.2.3.4 Mechanical

Mechanical impacts can appear due to repeatedly passing vehicles or forklift traffic. Furthermore, a natural abrasion due to wind, rain and sand as well as ageing due to weathering and ultraviolet rays is usually present. To avoid damage due to these impacts, several codes provide a support for the decisions e.g., by limiting the water-cement ratios or by giving minimum values for the compression strength. The stress level can be identified based on the information about the acting forces, acquired from the documentation in the service-life file, from traffic counting or from information about the passing vehicles. If there is any damage due to abrasion present, decisions must be made about how long the use can be continued and what replacement options might be possible. One can also decide to perform DT or NDT to get more information about the material resistance.

5.2.3.5 Chemicals

Chemical attack to concrete, which can have various sources and consequences, should ideally have been addressed during the design process. Choosing concrete properties and structural arrangements according to the demands ensures a durable structure. During the assessment process, information about possible chemical impact can be acquired in a first step at nearby locations from data relevant to chemical emission or chemicals in the ground water. If there is suspicion of local chemical attack because of e.g., a traffic accident or typical damage symptoms, a decision for testing methods is suggested.

5.2.4 Load history & fatigue

5.2.4.1 Permanent or static loads

For many structures the loading history is important as well as the current loading condition. For concrete structures the permanent loads are of major interest. The construction weight from design can be reviewed based on information available in construction files and plans. Changes in static loads happen in nearly every structure during construction and the construction weight can change due to addition, renovation, modifications or strengthening measures afterwards. Ideally those changes are documented and available. The construction weight of the in-situ structure can be determined by visual inspection and taking measurements. The amount and strength of pre- or posttensioning systems can be determined with a review of static calculations or plans, while the exact state at a certain time is hard to predict regarding losses in force, shrinkage and creepage. The assumptions relevant to the effect of shrinkage and creepage can be retrieved from the static calculations. During the assessment process one has to decide if those assumptions are reasonable based on past and current knowledge about those factors. If necessary, geodetic surveying can provide information about soil movements which can generate internal stresses, especially in statically indeterminate structures.

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5.2.4.2 Quasi-static loads

The loads due to wind, snow, temperature or traffic can be measured over a certain time and by examining traffic counts or data from weather stations. The actual loads on the structure are not the same as the scheduled loads in the computational limit states. At present, object-specific load approaches are rarely used while in most cases approved and generalised loads produce sufficient results. But especially for existing concrete structures additional load bearing capacity can be gained by a realistic model of the traffic loads. The BEM-ING (in particular the guideline for static recalculation of road bridges) gives the option, dependent on the number of cars per day on the bridge, to use former traffic load models instead of LM1.

5.2.4.3 Fatigue loads

With an increasing age of concrete structures, more and more attention must be devoted to the effects of fatigue which are particularly important in case of heavy load traffic. Even if the intensity of the loads can be lower than the values corresponding to the ULS, repeating load changes can cause damage on concrete structures over time. In addition to the increased traffic loads in the last decades, typical construction flaws of the past can lead to significant fatigue damage due to high stress peaks. Close attention should be paid to notches, bending radius and welding spots at rebar elements. Continuous vibrations can also induce fatigue stresses. While information about temperature and normal traffic is available in many cases, the exact number of heavy load crossing is often uncertain and the decisions about the further use are often based on visible damage. In past years, special systems for measuring the actual traffic loads have been developed, so new knowledge can be gained in the years to come. Components with damage or known structural flaws should be monitored and further examined at regular intervals.

5.2.4.4 Exceptional loads

In emergency situations temporary high loads, e.g., due to impact, explosion, earthquake, flood or fire, may develop. In the aftermath of such exceptional loading situations, a rapid decision is necessary, especially if major damage is observed. Information about the intensity of the stress can be determined through knowledge about the impact, like explosion strength or earthquake intensity. Further damage can be found by visual inspection. Information about the external load, like fire load or impact intensity can be measured a lot easier than the actual effects on the concrete structure.

Table 5-3 summarises the main information under evaluation for decision-making in terms of loading history and fatigue.

Table 5-3 Basic information evaluation for decision making: load history and fatigue.

Permanent / static loads	**Quasi-static loads**	**Fatigue loads**	**Exceptional loads / Earth quake**
Construction weight from design	Temperature differences		Impact
Construction weight after strengthening	Wind		Explosion
Expansion load	Common traffic		Earth quake
Post- and pretension**	Snow	Continuous vibrations	Flood
Shrinkage*	Special traffic		Fire
Creepage**			Temporary overload
Settlements			

* time dependent.
** time and load dependent.

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5.2.5 Past assessment, repair, strengthening & extension

A whole existing structure or its components may have been assessed, repaired or even strengthened multiple times during its past service life. Examples for such events are tabulated in Table 5-4. Furthermore, the structure might have been modified in different ways, for example due to changing demands on the use. The tracking of such time-varying events can be expensive if only fragmentary documentations exists. During the process of condition assessment, decisions have to be made, on how the information about past actions on the structure can be acquired, and on the depth and reliability of such information. Available options for decision-making are introduced in Table 5-4.

Table 5-4 Basic information evaluation for decision making: Past assessment, repair, strengthening and extension.

Assessment	**Repair**	**Strengthening**	**Extension**
Inspections	Partial concrete renewal or repair	Exchange of bearings	Drainage
Destructive (DT) and non-destructive testing (NDT	Sealing arrangements	Change of support conditions	Extension of cross sections
Monitoring	Partial renewal of reinforcement bars	Additional reinforcement	Additional, non-load bearing structural parts
Technical evaluation	Belated grouting of tendons	Additional posttensioning	Additional bearings
Statical recalculation	Crack fillings	Extension of cross sections	
	Drainage		

5.3 Performance modelling & prediction

5.3.1 Overview

In Chapter 4 of this Bulletin, different methods for performance prediction and modelling for existing concrete structures have been introduced, together with relevant performance parameters and indicators. The following subchapters give a practical proposal for making decision on the following questions:

- When is an assessment necessary?
- Which performance parameters must be chosen?
- How is the outcome of an assessment process rated?

On the examples of durability and load carrying capacity assessment procedures, available decision-making options are depicted in flowcharts, see Figure 5-10 and Figure 5-11. Starting with the condition assessment of the relevant structural parts according to Chapter 2, Chapter 3 and section 5.2, and proceeding with the definition of performance criteria, the basis for further decision making is provided for the person in charge of the assessment procedure. If major degradation of load bearing structural parts already occurred or is expected to occur in the near future, it is necessary to start a performance prediction or modelling procedure.

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5.3.2 Performance parameters

5.3.2.1 Residual service life

The residual service life of an existing structure is a key performance criterion and an indispensable decision to be made at the very beginning of performance modelling and prediction. The following criteria may be considered when choosing:

- Use and or conversion plan;
- Cultural or economic importance of the structure;
- Cost-benefit analysis.

The current condition of the existing structure is an essential aspect for those considerations. If structural damage is present (e.g., due to corrosion of reinforcement bars), their future impact on the structure should be examined by an assessment process. If the expected, calculated residual service life is higher than the target service life, the assessment procedure can be concluded by specifying potentially necessary future actions, defined during the assessment process (e.g., future monitoring). In any other case, the assessment procedure should yield proposals for intervention options (e.g., repair of deteriorated structural parts).

5.3.2.2 Consequences class

The failure of a structure or structural part leads to different consequences in terms of risk to life, injury, cultural damage, economical loss and / or environmental damage. The knowledge of these possible consequences is of high interest for the calculation of a construction's reliability in its design or assessment process. So called Consequences Classes (CC) according to Table 5-5 were introduced in international standards for the classification of structures, in terms of the ratio ρ between the total costs in case of failure and the construction costs (see MC2010, JCSS, VDI 6200)[262]. Structures of the Consequences Class 3 are usually of minor public interest and the possible consequences of a failure are rather small, e.g., material damage and financial costs. Typical examples for this class are agricultural structures or objects without public access. In CC 2 there is danger to life and health in case of a failure, so the safety demands for e.g., office buildings or bridges have to be higher. If there is danger for life and health of many people or if the structures are very significant, like theatres, hospitals or stadiums, then the structure would be assigned to CC 1.

Table 5-5 Consequences Classes and examples for classification according to JCSS, VDI 6200.

CC	Description	Examples: buildings	Examples: infrastructure
1	Damage to life and health for a lot of persons, major environmental damage	Theatres, hospitals, high rise buildings, stadiums, congress halls, multipurpose arenas	Road and railway bridges, supporting walls, noise barriers and traffic sign bridges on main roads
2	Damage to life and health for many persons, serious environmental damage	Television towers, office buildings, industrial buildings, apartment buildings, power stations, production plants, train stations, airport buildings, shopping malls, museums, theatres, schools	Road and railway bridges, supporting walls, noise barriers and traffic sign bridges on minor roads
3	Material damage and financial loss, low environmental damage, risk to individual persons	Agricultural structures, silos, masts, detached residential houses, apartments	Objects of infrastructure without public access

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5.3.2.3 Reliability index

Both the chosen target residual service life and the consequences class have a direct correlation to the target reliability index of a structure, further described in section 4.3.1. The target reliability index β is a measurement tool for the safety of a structure, that is based on statistical and probability calculations, and corresponds directly with the failure probability P_f, where Φ is the standard normal CDF (see also Eq. (1-6):

$$P_f = \Phi(-\beta) \tag{5-1}$$

The failure probability must be differentiated from the actual rate of failure for structures. If the probability of failure exceeds certain values, the structure can be rated as unsafe, but this does not mean that there is an immediate risk of collapse.

Table 5-6 shows the different target reliability indices corresponding to the ultimate and serviceability limit states for existing structures with different spans of residual service life. Table 5-7 presents tentative values of the reliability indices considering the chosen consequences in case of failure.

Table 5-6 Target reliability indices for existing structures (MC2010, JCSS).

Limit states	**Target reliability index β**	**Reference period**
Serviceability (SLS)	1.5	Residual service life
Ultimate (ULS)	In the range of 3.1 – 3.8*	50 years
	In the range of 3.4 – 4.1*	15 years
	In the range of 4.1 – 4.7*	1 year

* Depending on the costs of safety measures for upgrading the existing structure.

Table 5-7 Tentative target reliability indices for a one-year reference period in ULS (JCSS).

Relative cost of safety measures	**Consequences of failure**		
	Minor	**Moderate**	**Large**
Large	3.1	3.3	3.7
Normal	3.7	4.2	4.4
Small	4.2	4.4	4.7

The assessment should yield the current reliability index of the whole structure or parts of it, which subsequently has to be compared against the target reliability index. If the calculated reliability index is higher than the chosen target reliability index, the assessment procedure can be concluded by specifying potentially necessary future actions, defined during the assessment process (e.g., future monitoring). In any other case, the assessment procedure should yield proposals for intervention (e.g., repair of damaged structural parts, strengthening). These proposals are the foundation for further decision-making processes about intervention or other consequences like dismantlement, as depicted in Figure 5-10.

5.3.2.4 Load carrying capacity

During condition assessment, it is especially relevant to investigate the current loading of the structure. As soon as the current load cases exceed the original assumptions from the design stage, a verification of the load carrying capacity is required.

5.3.2.5 Durability and usability

Decision making processes in the durability and usability assessment procedure can be depicted in a flowchart according to Figure 5-11.

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5.3.2.6 Sustainability

Worldwide there are numerous rating systems for sustainability of buildings like the US-American LEED (Leadership in Energy and Environmental Design) and the British BREEAM (BRE Environmental Assessment Method)[266]. A typical life cycle analysis contains the definition of decision alternatives, the determination of the analysis period, a cost estimation as well as a life-cycle cost computation and a comparison of decision alternatives[267]. In Germany there is also a guideline published by the BASt[266], which gives possible criteria to make a decision about the ecological and economic performance of a structure.

Possible criteria for a rating of sustainability can be[266]:

- Impact on the global environment (e.g., leakage, emissions);
- Occupation of resources (e.g., primary energy consumption);
- Life cycle costs;
- Progression (e.g., due to caused traffic obstructions);
- User satisfaction and functionality;
- Health and comfort aspects (noise protection, comfort, traffic safety);
- Engineering, execution and construction quality;
- Maintainability (e.g., accessibility, strengthening or extension possibilities).

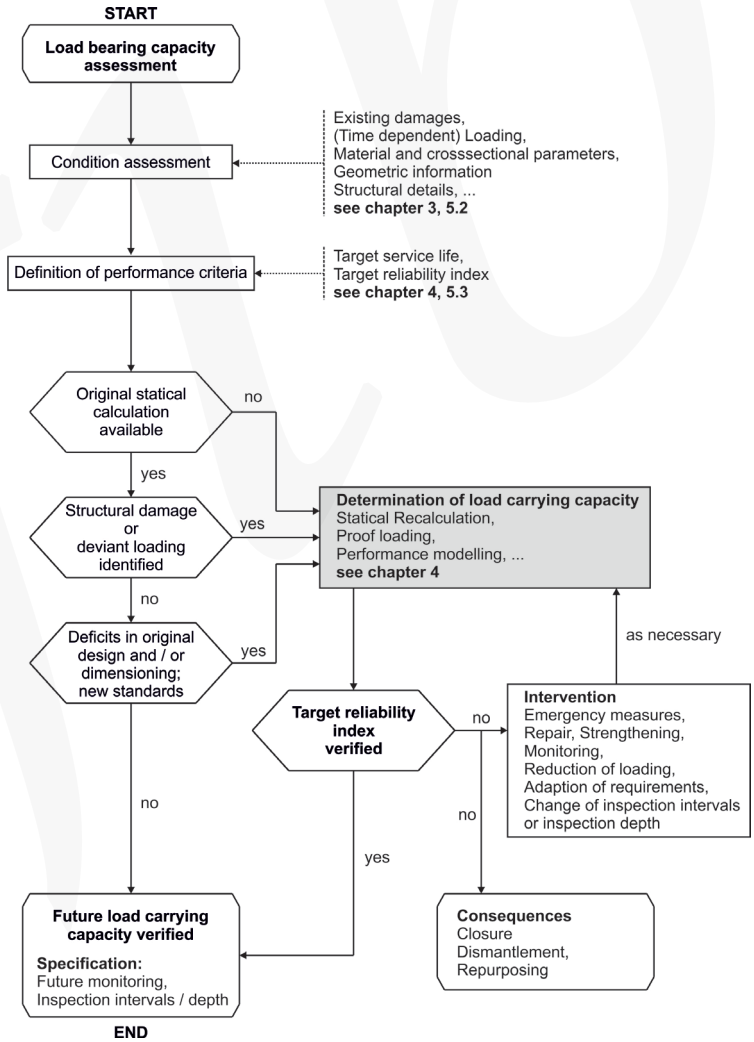


Fig. 5-10 Decision making processes for load carrying capacity assessment.

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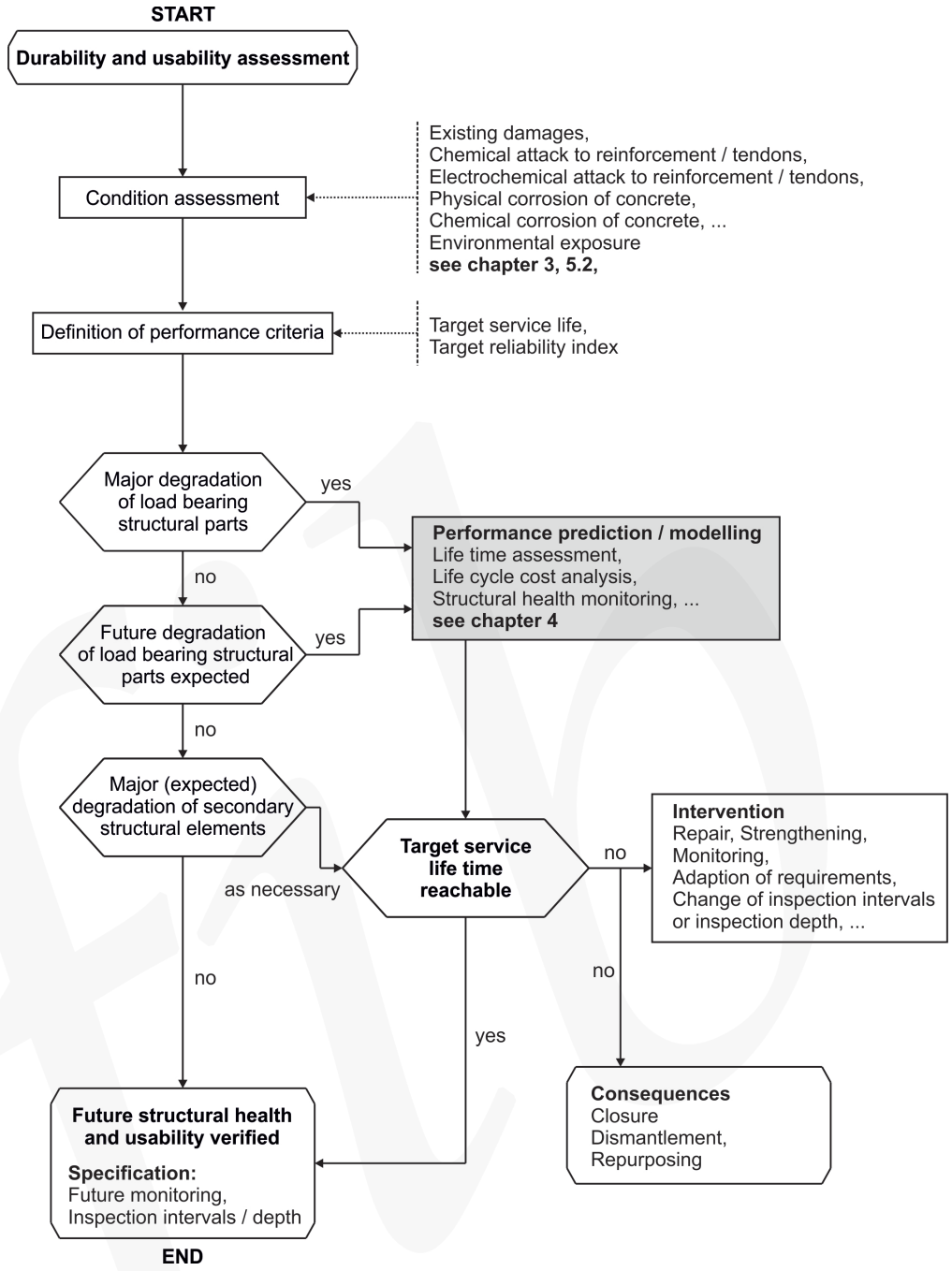


Fig. 5-11 Decision making processes for durability and usability assessment.

5.3.3 Typical decisions during the static recalculation of existing structures

A possible procedure of a load carrying capacity assessment is given in the following on the example of a static recalculation of existing bridges in Germany. In 2011 a guideline for the static recalculation of road bridges in Germany (see BEM-ING) was introduced. The goal of this guideline is a more realistic assessment of the load carrying capacity and usability of existing bridges, with special regards to both the increasing traffic on the one hand and to the advanced construction technology on the other hand. Since its introduction in May 2011, another amendment has been released in April 2015. By using the standardised approaches, a lot of good experience for the assessment of existing bridges was seen since its introduction[268]. The new BEM-ING will integrate those documents to one superordinate regulation.

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The static recalculation is performed according to Figure 5-12. In a first step, one must decide which impacts the structure should be able to withstand. Then the actual recalculation is hierarchically structured into four stages with different possibilities, characteristics and complexity.

In stage 1, the load model and the validation of the load bearing capacity are exclusively performed and proofed according to the rules of the current Eurocodes or former DIN-Technical Report. In the original static calculations of many existing bridges the exact geometry, e.g., oblique-angled or cross-sectional changes or the exact position of tendons, were not considered accurately, due to the computational limits of that time. By using modern calculation methods like FEM for plates, load bearing reserves can be mobilised.

If the target load level cannot be reached by using the general regulations of current codes, one can decide to perform a recalculation of the next stage 2 and use some specific, supplemental methods, which are given in the recalculation guideline of BEM-ING:

- Calculate of the shear force resistance by varying the shear compression angle;
- Verify the shear force resistance by using the principal stress criteria;
- Allow local deficits in shear reinforcement (cutting of the shear force coverage);
- Consider certain stirrup forms for the shear reinforcement;
- Verify the compression chord connection with additional regulations;
- Regard the tendons as longitudinal torsional reinforcement;
- Determinate material parameters by sampling;
- Modify partial safety factors for certain aspects;
- Reduce the factors for spreading of post tensioning forces;
- Modify the material factor for long term impact;
- Reduce of the target load level.

For most bridges a recalculation up to stage 2 can be performed by appropriately specialised engineering companies, while the regulations of stage 3 or higher are only used in some special cases, because of the complexity and effort, even if they have very large potential[270]. In stage 3, one can decide to include results of in-situ measurement in the calculation of the specific bridge like:

- Monitoring;
- Implement measured properties of the construction from DT or NDT;
- Perform a proof loading under in-use loads;
- Measure deformations on critical spots and perform strain measurements.

Stage 4 allows explicitly the decision for the use of scientific methods like not yet in regulations for analytical models, physical non-linear calculations, non-linear FEM and non-linear crack simulations as well as probabilistic methods, pressure arch models for shear resistance or a modified compression field theory.

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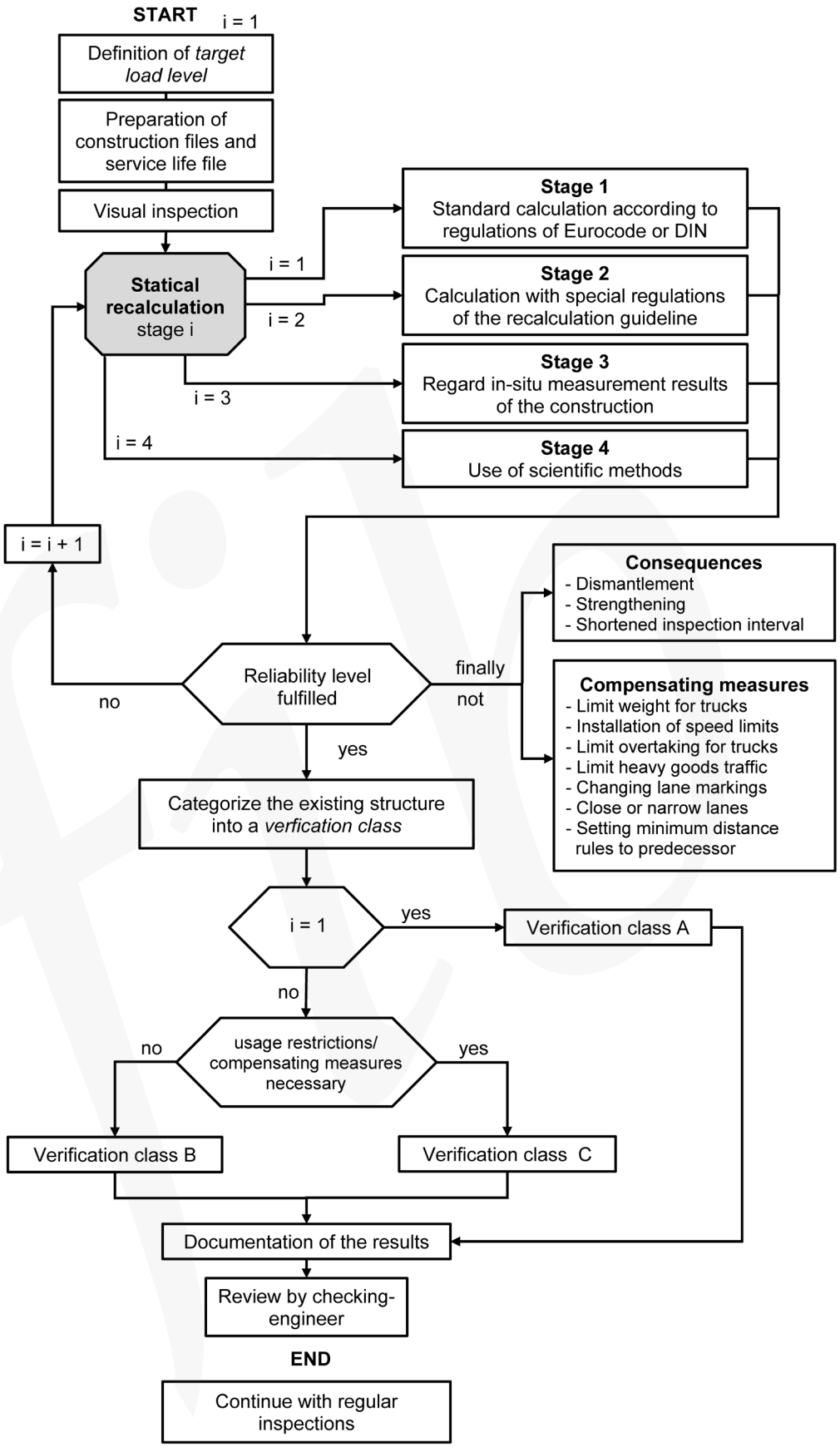


Fig. 5-12 Flow chart for a static recalculation of a bridge based on the on the recalculation guideline“ (statt„BEM-ING“).

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If the calculation approaches cannot provide the necessary reliability level, usage restrictions and compensatory measures must be selected:

- Limit of total weight for trucks;
- Limit of axle load for trucks;
- Installation of speed limits;
- No overtaking for trucks;
- No heavy goods traffic with permanent permission and subject to approval;
- Changing lane markings;
- Close or narrow lanes;
- Setting minimum distance rules for trucks to their predecessor during traffic jams.

To give a quick and standardised overview of the status of the recalculated structure, three different verification classes (German: Nachweisklassen) are introduced. They inform about the results of the examination, in case use restrictions for the future usage must be issued or modifications of the calculation were necessary, to successfully proof the target load level. For verification class A, both the load bearing capacity and the usability must be fulfilled without any restrictions and by only using the methods of stage 1. If modified calculation approaches are necessary, but no usage restrictions are required, the bridge can be classified with verification class B. In case of necessary usage restrictions, the structure is classified in verification class C.

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6 Case studies

This chapter contains a selection of case studies that show the implementation of the methods and tools that have been described in the previous chapters of this Bulletin, on actual large-scale structures.

The offered examples cover a wide range of types of concrete structures, mainly bridges under diverse damage scenarios induced by corrosion, fatigue, or extreme events such as floods, buildings under seismic excitation, but also applications on cultural heritage buildings and bridges, dams, harbours, wind turbines, and stadiums. Examples of both static and dynamic continuous monitoring systems (SHM) and periodic NDT investigations are reported, as outlined in Table 6-1.

The scope of the offered selection is to provide the main information related to the type and underlying motivation of the monitoring system, to the analysis procedures applied on the data acquired, which involve structural response and environmental/operational data, and finally on the PIs that are derived to assess the structural condition and performance. The case studies presented herein have been previously published and include a reference to relevant published material. If available, a link to the repository of data relevant to the case study is also reported.

Table 6-1 List of case studies.

Case study	Structural type	Investigation	Monitoring
6.1	bridge	Fatigue degradation	SHM
6.2	bridge	Corrosion of internal prestressing tendons	Dynamic/Static
6.3	bridge	Failure and damage detection of corroded external prestressing tendons	Dynamic
6.4	bridge	Operational and environmental variability	Dynamic
6.5	bridge	Localisation of settlement	Dynamic
6.6	bridge	Fatigue cracks	Static
6.7	bridge	Creep, shrinkage, relaxation, temperature effects	Static
6.8	bridge	Scour	Dynamic
6.9	bridge	Ground deformation	Dynamic/static
6.10	bridge	Reliability	NDT
6.11	building	Optimal sensor placement location	Dynamic
6.12	building	Seismic damage	Dynamic
6.13	Tunnels	Cracking, local failures	Static
6.14	Dam	Swelling	Dynamic
6.15	Dam	Swelling	Dynamic
6.16	BCH	Corrosion	DT and NDT
6.17	BCH	Cracks	Dynamic
6.18	BCH	Carbonation induced corrosion	NDT
6.19	Harbour	Residual strength, durability	NDT/DT
6.20	Wind turbine	Damage assessment, prediction of service life	Dynamic
6.21	Pavement/slab	Cracks	NDT (GPR)
6.22	Stadium	Serviceability	Dynamic

This chapter was written by Ballio, Chatzi (main author), Cigada, Darò, Elfgren, Gentile, Górski, Küttenbaum, Limongelli, Mancini, Oliveira, Rasol, Schmidt, Shan, Sousa, Torrent, Ueda, Verstrynge.

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6.1 Quantification of traffic and temperature effects on the fatigue safety of a reinforced-concrete bridge deck based on monitoring data



Fig. 6-1 60-Year-old road viaduct - Switzerland.

Contributors: Imane Bayane[1], Amol Mankar[2], Eugen Brühwiler[1], John Dalsgaard Sørensen[2].

General description: The case study is a 60-year-old road viaduct, constructed in 1957, located in Switzerland. The viaduct has a length of 195 m with a width of 12.7 m. It is composed of 8 reinforced-concrete slabs fixed in two steel girder beams. The slab has a thickness varying from 17 to 24 cm, and the steel girder has a height of 1.3 m.

SHM Motivation: The viaduct was instrumented with different monitoring systems to evaluate possible fatigue damage in RC slabs under traffic and environmental changes.

Damage monitoring: Two spans were instrumented with strain gauges, Linear Variable Differential Transformers (LVDTs) and thermocouples at mid-span of Span 2 and 4 out of 8 spans. A load test was performed after the instrumentation with a 5-axle truck having the legal limit load of 40t. Continuous monitoring was then performed, for one year, to capture the cyclic loading effect of traffic and temperature on the viaduct and estimate fatigue damage.

Analysis Procedures: Vehicles are identified from strain measurements using the peak over threshold approach. An inverse method is then developed to estimate the weight and the position of the identified vehicles by interpolating measured strain according to load test results. Probabilistic assessment is performed to capture variations and distributions of temperature and traffic properties. The structural behaviour of the instrumented slab is simulated with a calibrated FEM. The fatigue damage distribution is then estimated for steel reinforcement and concrete according to Miner's rule as a function of the weight and position of the identified vehicles.

Performance Indicators: The one-year monitoring data is used to identify the probabilistic distribution of strain, temperature and vehicle weights and positions. The combination of probabilistic assessment of monitoring data and numerical modelling of the instrumented slab provides the probabilistic distributions of stresses in the slab as a function of the weight and the position of vehicles. These probabilistic distributions are used to estimate the fatigue damage and its evolution in time as for steel reinforcement and concrete. The presented monitoring approach can be used as a base on how to perform monitoring to evaluate the fatigue safety of RC bridge slabs.

Acknowledgements: This work has been carried out under project INFRASTAR (Innovation and Networking for Fatigue and Reliability Analysis of Structures - Training for Assessment of Risk). The project INFRASTAR (infrastar.eu) has received funding from the European Union's Horizon 2020 research and innovation programme under the Marie Skłodowska-Curie grant agreement No 676139.

References: Imane Bayane, Amol Mankar, Eugen Brühwiler, John Dalsgaard Sørensen, (2019). Quantification of traffic and temperature effects on the fatigue safety of a RC bridge deck based on monitoring data. Engineering Structures, Volume 196, ISSN 0141-0296

https://doi.org/10.1016/j.engstruct.2019.109357

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6.2 Continuous SHM system of a diffusely corroded prestressed concrete operating viaduct

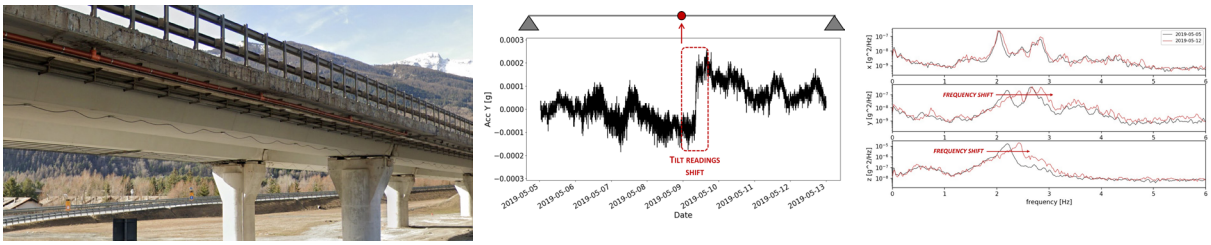


Fig. 6-2 Prestressed concrete bridge.

Contributors: Isabella Alovisi[1], Paola Darò[1], Dario La Mazza[1], Monica Longo[1], Giuseppe Mancini[1].

General description: The monitored structure is a prestressed concrete bridge built in the early 60s, composed by two independent carriageways, each having nine simply supported spans. The typical span is 35 m long with a cross section made by a prestressed concrete slab having constant height of 1.5 m.

SHM Motivation: Following an inspection, some cracks were found at the inner surface of the slab, due to the failure of a significant number of pre-stressing tendons caused by a diffuse corrosive phenomenon. Strengthening works were necessary; the solution chosen was the adoption of an external pre-stressing system. The SHM system installed in 2019 was developed for both short and long-term monitoring, with measurements, in fact, initiated prior to strengthening in order to collect a dataset that is representative of the damaged state and are still ongoing.

Data Description: Each span is equipped with a chain of five biaxial MEMS inclinometers and five triaxial MEMS accelerometers. Inclinometers have two measuring axes X and Y, corresponding respectively to the transversal and longitudinal direction of the deck, while accelerometers have three measuring axes X, Y and Z, where the X and Y axes align with the inclinometers and the Z-axis corresponds to the vertical direction. The system is completed by two gateways; one per carriageway and a power line communication to connect all the devices to the network.

Analysis Procedures: During the maintenance works, the clinometers raw data were analysed to find the instantaneous change of the deformed shape when the external pre-stressing was applied to the deck and to evaluate the plastic deflection recovery. As for accelerometers readings, vibration-based damage detection techniques were used to identify the changes in the dynamic response of the system: OMA techniques were used to identify mode-shapes vectors, Power Spectral Density (PSD) and Frequency Domain Decomposition (FDD) were used to evaluate the system natural frequencies shifts. A stable peak-picking analysis was performed to track the natural frequencies evolution in the long-term period. Non-Linear Finite Element Analysis (NLFEA) and a model updating procedure were used to simulate the bridge behaviour both in damaged and strengthened state. Static and dynamic load tests were performed to characterise the structural response before and after the strengthening works. The FE model was also used to simulate a set of possible damaged scenarios to calculate the thresholds in terms of plastic rotations and frequency shifts for the long-term monitoring.

Performance Indicators: Based on the collected data, the damage parameters (or DFs) identified for short and long-term monitoring are i) the average values of the tilt recordings to monitor the static behaviour and to check, if the structure is accumulating plastic deformation, and ii) the natural frequencies shift to evaluate the dynamic behaviour and to identify possible structural damage.

References: A. Cigada, F. Lucà, M. Malavisi, G. Mancini, 'Structural Health Monitoring of a Damaged Operating Bridge: A Supervised Learning Case Study', Dynamics of Civil Structures, Volume 2, 2021.

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6.3 SHM for damage detection of the external prestressing system of a highway bridge

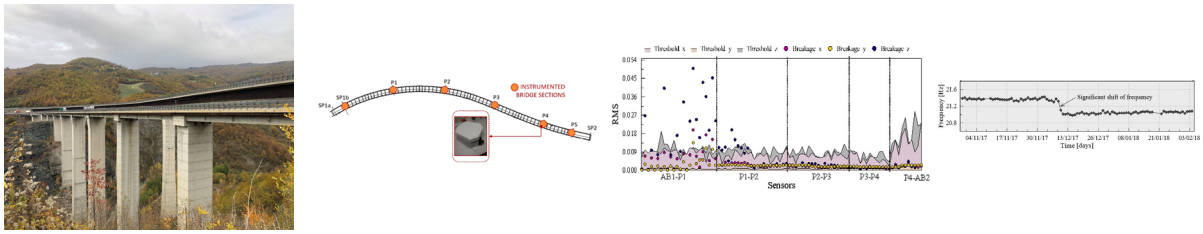


Fig. 6-3 Prestressed composite highway bridge.

Contributors: Isabella Alovisi[1], Paola Darò[1], Dario La Mazza[1], Monica Longo[1], Giuseppe Mancini[1].

General description: The monitored structure is a prestressed composite highway bridge opened to traffic in 2006. The structure is 624 m long and is characterised by a counterweight span of 30 m, 4 hyperstatic spans of 120 m, an hyperstatic span of 71 m, and an isostatic one of 43 m. The deck has a composite box girder section with concrete slabs and steel webs, while the prestressing system is made by both internal bonded and external unbonded tendons.

SHM Motivation: After a failure of an external tendon, the bridge was instrumented in 2017 with an extensive monitoring system for the real-time and long-term check of the tendons conditions, as well as to provide an efficient damage detection and early warning system. The monitoring system is still active.

Data Description: The external tendons are equipped with 88 MEMS tri-axial accelerometers, two per monitored tendon. Each device captures the vibration under traffic excitations along three orthogonal directions. In addition, environmental data, such as air temperature and RH are collected to remove the effects of these parameters on the measures. Each device has a microcontroller providing data sampling and elementary computing at sensor level. A Controller Area Network (CAN) BUS network is used to connect the accelerometers to two Internet of Things (IoT) gateways to send data to a dedicated cloud platform.

Analysis Procedures: An IoT cloud architecture has been developed to pre-process the data for improving network performance and rendering the system more effective. In the pre-processing phase at the gateway level, the acceleration data are filtered and the average, the Root Mean Square (RMS), and minimum/maximum values are calculated. Since the signals have a random nature, a frequency domain approach has been used to determine the natural frequencies using a PSD analysis in the cloud environment. A peak identification algorithm has been developed to monitor the power spectrum peaks trend over time. Since an evident inverse correlation with temperature was found, a linear regression analysis was used to eliminate its effect on the data.

Performance Indicators: The variation of the peaks over time proved to be an efficient long-term indicator of the dynamic behaviour and of possible structural damaging. Multiple strands failures have been correctly identified, and promptly communicated to the road operator to activate further maintenance procedures. In terms of semi-real time control parameters, as the traffic conditions can be considered invariant over time, trends in RMS values were used as DIs. Changes in the average values can indicate a variation in the cable position, while the combination between the average with the information coming from maximum and minimum values can be considered a sensor diagnostic indicator.

References: Cigada A., Lucà F., Malavisi M., Mancini G. (2021) A Damage Detection Strategy on Bridge External Tendons Through Long-Time Monitoring. In: Pakzad S. (eds) Dynamics of Civil Structures, Volume 2. Conference Proceedings of the Society for Experimental Mechanics Series. Springer, Cham Quantification of traffic and temperature effects on the fatigue safety of a reinforced-concrete bridge deck based on monitoring data. Engineering Structures, Volume 196, ISSN 0141-0296, https://doi.org/10.1016/j.engstruct.2019.109357

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6.4 Continuous monitoring of Chillon viaduct under environmental & operational variability conditions



Fig. 6-4 Chillon viaduct, Veytaux - Switzerland.

Contributors: Henar Martín-Sanz[1], Konstantinos Tatsis[1], Vasilis Dertimanis[1], Luis David Avendaño-Valencia[1], Eugen Bruhwiler[2] and Eleni Chatzi[1].

General description: Constructed in 1969, the parallel Chillon viaducts were strengthened in 2015 via a layer of Ultra High Performance Fibre Reinforced Concrete (UHPFRC) cast atop their decks.

SHM Motivation: In order to assess the efficacy of this rehabilitation solution, and further evaluate its long-term performance, a SHM campaign was implemented by ETH Zurich. Three months of monitoring data have been shared with the community, with more to follow, allowing to study the influence of Operational and Environmental Conditions on the modal parameters and to further infer PIs for assessing structural behaviour in terms of deterioration processes, such as fatigue accumulation and reliability analysis.

Data Description: A span of 98 m, in the central portion of the viaduct, has been instrumented with 11 accelerometers distributed along the span, as well as four strain gauges and environmental sensors (measuring temperature and humidity) installed at mid-span. The strain gauges were located directly on the rebar, aiming to capture fatigue effects in both the longitudinal and transverse direction.

Analysis Procedures: The gathered data was analysed by means of both an operational modal analysis (ERA-NeXT), as well as a non-stationary analysis (SP-TAR) to investigate environmental and operational influences. The bridge behaviour was further simulated via an updated shell-based FEM, set up in SAP2000. Polynomial Chaos Expansion (PCE) was used as a tool for extracting data driven PIs, which account for Environmental and Operational influences.

Performance Indicators: Relying on the data, different PIs were obtained to quantify the effects of UHPFRC strengthening. We distinguish the function of these PIs into a) purely data-driven, i.e., fatigue accumulation calculated from the installed strain gauges and damage detection by monitoring frequency evolution, and b) hybrid PIs, where a FE model is coupled to the data for the purpose of more intricate investigations, such as reliability analysis.

Web Link to Data Repository: https://zenodo.org/record/3234805#.XooIVogzZaR

Acknowledgements: The authors would like to acknowledge the Swiss National Science Foundation (SNSF), which has supported this research (project # 154060).

References: Martín-Sanz, H., Tatsis, K., Dertimanis, V. K., Avendaño-Valencia, L. D., Brühwiler, E., & Chatzi, E. (2020). Monitoring of the UHPFRC strengthened Chillon viaduct under environmental and operational variability. Structure and Infrastructure Engineering, 16(1), 138-168.

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6.5 Localisation of settlement in a prestressed bridge



Fig. 6-5 S101 Bridge, Reibersdorf - Austria.

Contributors: Pier Francesco Giordano[1], Maria Pina Limongelli[1], Helmut Wenzel[2].

General description: The case study is the S101 bridge located in Reibersdorf, Austria. It consisted of a three-span, 56 m long, prestressed concrete bridge built in early 1960. The structure was artificially damaged for research purposes in 2008. After that, the bridge was demolished due to overall bad maintenance conditions and to allow the enlargement of underneath A1 Westautobahn.

SHM Motivation: The experimental campaign was carried out in the realm of the IRIS project to investigate damage identification methods using data recorded on a real structure. The settlement of one of the piers was simulated by cutting its base and progressively lowering the pier, and some tendons were cut to simulate prestress loss.

Data Description: The response of the bridge to ambient vibrations was recorded during 3 consecutive days by 15 three-directional FBA-23 Force Balanced Accelerometers (FBAs) from Kinemetrics deployed along its deck. Specifically, 14 accelerometers were placed on the west side of the deck while 1 accelerometer was positioned on the opposite side to detect torsional modes. During the vibration test, traffic was not allowed on the bridge while the roadway beneath was in use. The temperature was stable in the range of -3/-4°C during the experimental campaign.

Analysis Procedures: Vertical accelerations gathered by the sensors placed on the west side were used to compute several damage-sensitive features. The collected data were analysed using the Enhanced FDD technique to retrieve mode shapes and natural frequencies. Furthermore, ODSs were obtained from the power spectral densities of the responses.

Performance Indicators: Damage localisation was carried out applying the curvature method, the strain energy method, and the interpolation error method. The related Damage Sensitive Features (DFs) were formulated in terms of both mode shapes and ODSs. Thresholds of the DFs alerting on the location damage were defined based on the distribution of the DFs in the damaged configuration.

References: Wenzel H. IRIS - Industrial Safety and Life Cycle Engineering: Technologies, Standards, Applications. 2013. Giordano PF, Limongelli MP. Response-based time-invariant methods for damage localisation on a concrete bridge. Struct Concr 2020; 21:1254–71.

https://doi.org/10.1002/suco.202000013

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6.6 Temporary closing of Autio prestressed concrete bridge due to cracking, fatigue, and strain gauge drift



Fig. 6-6a Prestressed Concrete Bridge over Torne river, Autio - Sweden.

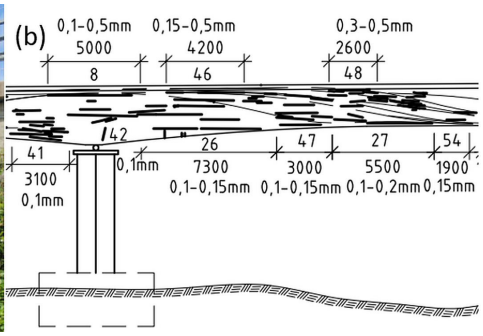


Fig. 6-6b Cracks on the outside of the West Beam of the Autio Bridge, September 15, 2020. Ca 10 m to the right is a section where mid-span and support cables are anchored.

Contributors: Angelica Agredo Chavez[1], Jaime Gonzalez[1], Mats Petersson[1], Erik Anderssson[1], Jens Häggström[2], Gabriel Sas[1], and Lennart Elfgren[1].

General description: Extensive cracking caused a temporary closure of a prestressed concrete bridge over Torne River at Autio in northern Sweden. Strain gauges on concrete drifted and a failure in fatigue was feared. The bridge was built in 1963 with BBRV cables. It comprises three spans and a total length of 134.8 m, see figure. Due to mining activities, the bridge carried trucks with a total weight of 90 tn in the period 2012-2014 and from 2019.

SHM Motivation: To secure the safety of the bridge when mining trucks were allowed to traverse, crack development was monitored, first manually and from 2020 onward via strain gauges and LVDTs. Cracks normally varied between 0.1 to 0.3 mm in width and grow in length with time, see figure.

Degradation. Parameters/Indicators: In November 2020, some of the strain gauges on the concrete showed alarming growth and the bridge was closed for traffic. The original strain gauges were compensated for temperature and the reason for the drift could not been clarified. Additional strain gauges were installed on vertical reinforcement bars. The new strain gauges did not indicate any growth in strain.

Methods used: An assessment of the fatigue capacity of the bridge was carried out, using the Palmgren-Miner hypothesis, and it was found that the fatigue capacity was sufficient. The bridge could be opened again for traffic after being closed for five weeks. The monitoring program and possible reasons for the drift in the strain gauges along with a fatigue assessment are presented in Agredo Chavez et al. (2021). Alternative methods for monitoring and model-based updating are also discussed.

Acknowledgements: The authors would like to acknowledge the Swedish Transport Administration, which has supported this research.

Reference: Agredo Chaves A, Gonzalez J, Andersson K, Leidzen J, Andersson E, Petersson M, Sas G, Elfgren L & Häggström J (2021). Cracking and Fatigue in Heavy Loaded Prestressed Concrete Bridge in Sweden. Proceedings IABSE Symposium Prague, 8 pp, Preprint.

http://diva-portal.org/smash

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[2] Trafikverket, Luleå, Sweden

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6.7 SHM of the short and long-term behaviour of a long-span prestressed concrete bridge

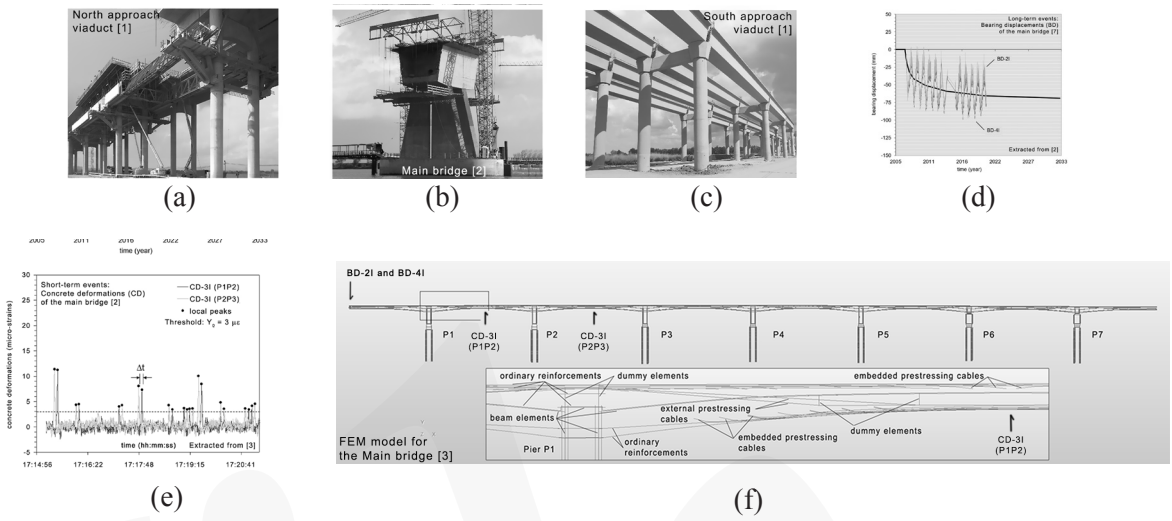


Fig. 6-7 The Lezíria Bridge, Lisbon - Portugal.

Contributors: Helder Sousa[1].

General description: The Lezíria Bridge, built in 2005–2007 nearby of the Portuguese capital, Lisbon, with a length of 11.67 m, is materialised by the: (a) north approach viaduct; (b) main bridge; (c) and south approach viaduct.

SHM Motivation: Due to the high socio-economic impact of such infrastructure, the monitoring project was part of the bridge design. SHM data is seen as valuable information for better management of the structure over its lifetime.

Degradation & Indicators: The monitoring system, with ca. 400 sensors, measures vertical and bearing displacements, concrete strains, rotations, accelerations, scour, and corrosion, which are installed in foundations, piers, and deck girder. Temperature and RH are also monitored. The objective is to track the short- and long-term performance of the structure considering ageing effects over time.

Methods: Advanced FE models were set by a rigorous scanning of all available information concerning properties of (a) concrete, (b) prestressing cables, (c) soil, (d) loading, and (e) geometry. Prediction of the (a) long-term behaviour was performed since construction, allied to investigations on (b) short-term behaviour due to expected in-situ traffic loads (based on Extreme Value Theory) and anticipation of damage scenarios (based on Bayesian statistics).

Thresholds: The FE model predictions are used as a baseline against the collected data, which has allowed monitoring the bridge performance over time on a comparative basis. Thresholds established at the design phase, for the different monitored parameters and locations, are used as alert levels.

References: Sousa, H., Félix, C., Bento, J., Figueiras, J. (2011). Design and implementation of a monitoring system applied to a long-span prestressed concrete bridge. Structural Concrete, 12(2), 82-93. Sousa, H. (2020). Advanced FE modelling supported by monitoring toward management of large civil infrastructures: The case study of Lezíria Bridge. Structural Concrete, 21(4), 1309-1320. Sousa, H., Costa, B., Henriques, A., Bento, J., Figueiras, J. A. (2016). Assessment of traffic load events and structural effects on road bridges based on strain measurements. J. of CE and Mgmt, 22(4), 457-469. Sousa, H., Rozsas, A., Slobbe, A., Courage, W. (2020). A novel pro-active approach towards SHM-based bridge management supported by FE analysis and Bayesian methods. Structure and Infrastructure Engineering, 16(2), 233-246.

[1] BRISA Group, Portugal

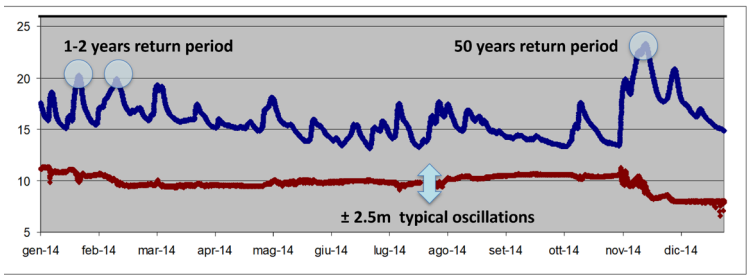
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6.8 Borgoforte river bridge: environmental monitoring for a decision-support system



(a)

Fig. 6-8a Borgoforte bridge over the Po River - Italy.



(b)

Fig. 6-8b: Collected data from the monitoring system on the bridge relevant to water depth, elevation of the riverbed.

Contributors: Francesco Ballio, Alfredo Cigada, Giulio Ballio, Silvio Franzetti, Gianluca Crotti[1].

General description: A bathymetric survey carried out in 2000 after an intense flood along the Borgoforte bridge over the Po River in Italy, indicated the presence of a scour hole with a depth of about 15 m. Three of the four piers in the riverbed were stabilised. No structural reinforcement was applied to the last pier, situated close to the bank, where the bed level is higher and apparently more stable. A real-time risk assessment system for this pier was designed and installed as a non-structural alternative to consolidations works.

SHM Motivation: Real-time data from the monitoring system aid the bridge managers to decide whether the bridge should be partially or totally closed to traffic in the case that environmental conditions (river, wind) indicate that the structure may reach a SLS. The monitoring system collects data relevant to water depth (from which also velocity can be deduced), elevation of the riverbed close to the pier (scour depth), and wind velocity, enabling the evaluation of the environmental actions on the structure. Cameras qualitatively control the accumulation of debris in front of the piers.

Methods used: Measured data allow for evaluating environmental actions on the structure (wind and water drag, including the effect of debris accumulation) and for defining the condition of the foundations. Such quantities represent inputs for a structural model of the bridge which, in turns, indicates proximity to limiting conditions. The lower figure reports the measured water and bed elevation during 2014.

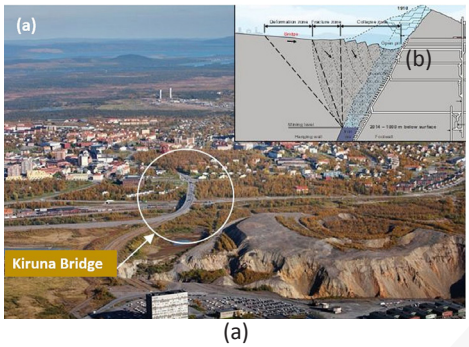
Thresholds for the decision support system: The objective of the SHM is to support decisions for bridge closure. Data from the SHM system are used as initial conditions for (simple) models forecasting the short-term evolution of the environmental condition and, consequently, of the structural state of the bridge. Thresholds are fixed by comparing the expected evolution of the environmental parameters and the time requested by the closure procedure (including human evaluation of the data and the actions to divert traffic).

References: Ballio F, Ballio G, Franzetti S, Crotti G, Solari G. (2018) "Actions monitoring as an alternative to structural rehabilitation: Case study of a river bridge.", Structural Control Health Monitoring, 2018, Vol. 25, n. 11, e2250. doi:10.1002/stc.2250

[1] Department of Civil and Environmental Engineering, Politecnico di Milano, francesco.ballio@polimi.it

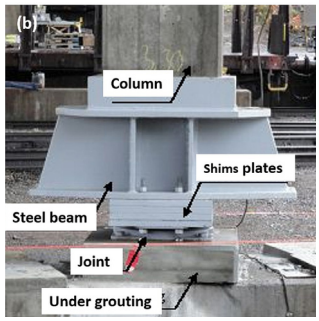
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6.9 Decision making due to ground deformations for the Kiruna Mine prestressed concrete bridge



(a)

Fig. 6-9a-b Large ground deformations caused by mining activities. The mine is expanding towards the bridge (encircled).



(c)

Fig. 6-9c Device to enable vertical and horizontal adjustment in the bottom of the columns. Shims can be inserted to counteract varying ground settlements for different columns.

Contributors: Bagge N1,[2], Enoksson O1,[3], Sas G1 & Elfgren, L[1].

description: A 55-year-old, 121.5 m long, five span, prestressed concrete bridge was situated in the deformation zone close to an iron mine in Kiruna in northern Sweden, see Figure.

SHM Monitoring: For tackling the risk of uneven ground deformations, the bridge was analysed and monitored. Results and measures are presented to ascertain the robustness of the structure.

Parameters/Indicators: The conducted analysis indicates that the bridge could sustain 24 mm in uneven horizontal and 83 mm in uneven vertical displacement of the two supports of a span. To be able to sustain larger deformations, the columns of the bridge were provided with joints, where shims could be inserted to counteract the settlements. To accomplish this, each one of the 18 columns of the bridge was unloaded via aid of provisional steel supports. The column was then cut and a new foot was mounted (see Figure). This enabled the lifting of each individual column with two jacks, when needed, and to adjust its height by inserting or taking away shim plates, Bagge et al. (2015).

Methods used: The deformations of the bridge and the surrounding ground were monitored. The eigenmodes of the bridge were studied with accelerometers and by analysis with FEMs. Comparison indicated good agreement between the model and the actual bridge, with calculated eigenfrequencies of 2.17, 4.15 and 4.67 Hz, for the first transversal, vertical and torsional modes, respectively. Measurements during winter resulted in higher values due to increased stiffness caused by frozen materials. The bridge was decommissioned in 2016 and then tested to failure to calibrate various assessment methods.

References: Bagge N, Nilimaa J, Enochsson O, Sabourova N, Grip N, Emborg M, Lundmark T, Tu Y & Elfgren L (2015). Protecting a five span prestressed bridge against ground deformations. In: IABSE Conference Geneva 2015: Structural Engineering: Providing Solutions to Global Challenges. Int. Ass. for Bridge and Structural Engineering; 2015. p. 255–62. http://ltu.diva-portal.org/smash/get/diva2:1008206/FULLTEXT01.pdf.

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[2] WSP Bridge & Hydraulic Design, Göteborg,

[3] Trafikverket, Luleå, Sweden

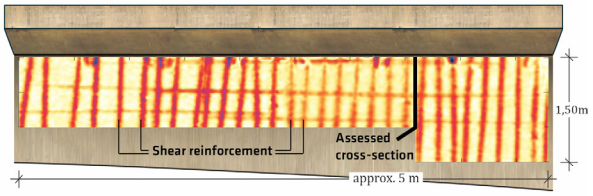
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6.10 Reliability assessment of a prestressed concrete bridge using geometrical NDT-results



(a)

Fig. 6-10a View of the prestressed concrete bridge, constructed in 1965.



(b)

Fig. 6-10b Detection of the reinforcement using a scanner system mounted on the web surface.

Contributors: Stefan Küttenbaum[1], Thomas Braml[2], Stefan Maack[1].

General description: The investigated prestressed concrete bridge was constructed in 1965 and has been dismantled in the meantime. Various performed non-destructive investigations could be evaluated during the demolition. The total length was 133 meters. The cross-section was a single-cell hollow box.

NDT Motivation: In preparation of the bridge dismantlement as a part of the extension of the overpassed federal highway, a structural reliability analysis had to be performed for the individual construction states. The proof of the shear reinforcement in ULS has been found to be decisive. Sensitivity analyses revealed, as expected, that conceivable deviations between the nominal spacing of the reinforcement specified in drawings and the actual spacing would significantly influence the structural reliability. GPR measurements were carried out to verify the information available prior to any testing and to explicitly consider the measured spacing in reliability assessment. In this way, biases in the basic variable representing the spacing, that might be computationally adverse, and uncertainties could be reduced.

Data Description: The measurements were performed automated using a scanner system mounted on the web surface in the area around the cross-section to be assessed. The system was equipped with a 2 GHz antenna, which has been moved on horizontal measuring lines vertically spaced 2 cm. The effort of the testing on an area of about 5 m² was less than one day. The recorded data consists of time signals.

Analysis Procedures: The gathered data was filtered and converted into spatially resolved volumes using the Synthetic Aperture Focusing Technique, considering the propagation velocity of the electromagnetic waves inside the investigated concrete member. The position of the reinforcement (which is mounted almost vertically) in the longitudinal bridge direction was picked on several horizontal lines, based on the volume elements with the maximum amplitude in the area of the indications of the respective steel bars. Based on this, the average stirrup spacing around the cross-section to be assessed could be derived.

NDT-supported assessment: The data quality was evaluated by measurement uncertainty calculations. This ensures the comparability of the measurement results and establishes confidence in measurement. The measurement result has been transferred into a testing-based basic variable representing the stirrup spacing considering the measurement, modelling, model, and statistical uncertainty. In this way, the level of approximation could be increased, and a more realistic reliability assessment performed. The increase in reliability after including the NDT results has been found to be individually $\Delta\beta \approx 1.60 - 0.65 = +0.95$ (ULS shear). A study on the NDT-supported SLS assessment can be found in the references.

References: Küttenbaum, S.; Feistkorn, S.; Braml, T.; et al. (2021) Methods to Quantify the Utility of NDT in Bridge Reassessment. In: Rizzo P., Milazzo A. (eds) European Workshop on Structural Health Monitoring 2020. Lecture Notes in Civil Eng., vol 127. Springer, Cham.Küttenbaum, S.; Braml, T.; et al. (2021) Reliability assessment of existing structures using results of nondestructive testing. Structural Concrete. 1–21. DOI 10.1002/suco.202100226

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[2] Universität der Bundeswehr München, Inst. for Structural Engineering, Germany

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6.11 Optimal sensor placement on a building for seismic SHM



Fig. 6-11 The Robert A. Millikan Library, Pasadena - United States.

Contributors: Maria Pina Limongelli[1].

General description: The Robert A. Millikan Library is a nine-story RC building located on the campus of the California Institute of Technology in Pasadena, California. Lateral resistance is provided by a central shear core (elevator shaft), by two shear walls in the North-South direction and moment resisting frames along the East-West direction.

SHM Motivation: The building is monitored by USGS (United States Geological Survey) to study its performance under seismic excitation and to use it as a full-scale test structure for the development and testing of structural health and performance monitoring methodologies. Several earthquakes have been recorded and data recorded during the Yorba Linda, CA earthquake (ML 4.8) have been used to validate the method for optimal sensors locations.

Data Description: The building is equipped with a permanent dense array of 36 uniaxial triggered strong motion sensors. Three acceleration horizontals components of the acceleration are recorded on each of the eleventh floors and three vertical accelerations on the basement. In addition, one Community Seismic Network accelerometer is deployed at each floor and a tri-axial episensor is installed at the 9th floor.

Analysis Procedures: Modal parameters of the building (see figure) have been estimated via a parametric data-driven frequency domain modal identification technique, able to account for the effect of a bidirectional excitation. The identified parameters have been used to select the optimal location of a limited number of sensors. The goal of the optimisation is to record responses that contain mainly the contribution of the lower modes and to use them to reconstruct the response at locations not equipped with sensors. This allows filtering out the contributions of the higher modes that are affected by higher uncertainty.

Performance Indicators: Several seismic response parameters can be calculated from the structural response that was reconstructed using signals recorded by sensors deployed at the derived optimal locations. Namely, storey maximum absolute acceleration, displacement, shear and interstorey drift.

References: Limongelli M.P., (2003), “A new procedure to estimate seismic response parameters from a limited number of records” European Earthquake Engineering, 2, pp.45-55. Limongelli M.P., (2003),"Optimal location of sensors for reconstruction of seismic responses through spline function interpolation". Earthquake Engineering and Structural Dynamics, 32, pp. 1055-1074.

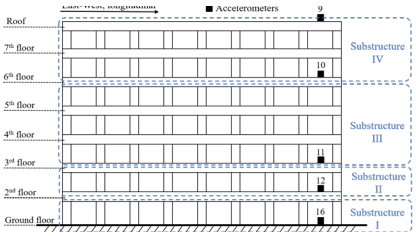
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6.12 SHM and data-driven seismic damage evaluation of a seven-story RC building



(a)



(b)

Sub-structure	Floor	Damage description	Data-driven damage index
I	First floor	Cracks between bricks short column cracks with width of 0.5-1.0 cm	0.28
II	Second floor	Diagonal cracks along columns with width of 0.5 cm cracks through the beams with width<1 cm	0.33
III	Third to fifth floor	"X" shear cracks in joints with width <0.5 cm beam and column cracks with width 0.5-1.0 cm	0.46
IV	Sixth to roof floor	No visible damage	0.16

(c)

Fig. 6-12a Seven-story RC building - United States.

Fig. 6-12b Illustration of four substructures.

Fig. 6-12c Damage evaluation result.

Contributors: Xilin Lu[1], Jiazeng Shan[1]*, Hanqing Zhang[1].

General description: An instrumented seven-story RC building, located in Van Nuys, California is presented to illustrate a health monitoring algorithm and damage evaluation methodology for seismic evaluation. The building is essentially rectangular in plan and instrumented by the California Strong Motion Instrumentation Program (CSMIP) with 16 channels. Five accelerometers measured the response in the longitudinal (E–W) direction at the ground, second, third, and sixth floors and the roof.

SHM Motivation: The accelerometers were instrumented prior to the 1971 San Fernando earthquake and updated to a 16-channel recording system to know more about structural seismic performance.

Data Description: Two earthquake events are considered, the 1992 Big Bear with Peak Ground Acceleration (PGA) of 0.02g and the 1994 Northridge with PGA of 0.47g. The building was expected to respond elastically with little damage during the Big Bear earthquake and was severely damaged by the Northridge earthquake. The measured longitudinal accelerations during the earthquakes were utilised in this case study.

Analysis Procedures: According to the incomplete measurement condition, a simplified four-degrees-of-freedom model presents monitoring substructures for the seven-story real-world structure. By adopting the decentralised model, the assumed inter-storey stiffness and damping parameters can be identified by the differential evolutionary technique under the Big Bear earthquake, while the drift response can be estimated via use of a Kalman filtering technique under the Northridge earthquake. The inter-storey restoring force can be calculated to generate the hysteresis curve.

Performance indicators: The proposed data-driven damage evaluation method, which combines both peak term corresponding to the peak deformation and the cumulative term related to the response difference between the linear and nonlinear model can be then calculated. The evaluation results show that the damage index value nears zero in the substructure IV, which shows no visible damage after the earthquake. An increasing value of damage index from substructure I to substructure III refers to more serious damage appears in the 3rd to 5th floor, even though the moderate damage also exist on the first and second floor. The described data-driven damage evaluation method is shown to adequately assess structural condition and is proven applicable for real-world studies.

References: Shan, J., Zhang, H., Ouyang, Y., & Shi, W. (2020). Data-driven damage tracking and hysteresis evaluation of earthquake-excited structures with test validation. Engineering Structures, 207, 110214.

[1] Tongji University, Shanghai, China, jzshan@tongji.edu.cn

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6.13 SHM of two operating tunnels subjected to a landslide

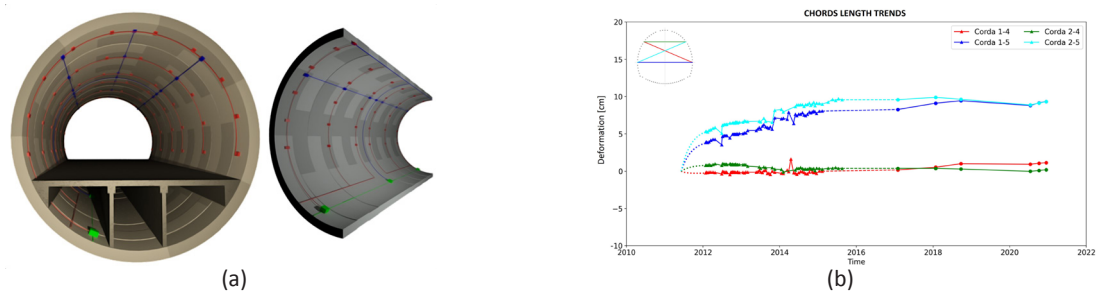


Fig. 6-13a-b Reinforced Concrete (RC) twin-tube tunnels - Central Italy.

Contributors: Isabella Alovisi[1], Paola Darò[1], Dario La Mazza[1], Monica Longo[1], Giuseppe Mancini[1].

General description: The monitored structures are two reinforced concrete twin-tube tunnels in Central Italy. The first one, 3.9 km long, was built with conventional tunnelling method between 2008 and 2015. The second one, 2.6 km long, was excavated by Tunnel Boring Machine (TBM) and built with precast segments between 2012 and 2015.

SHM Motivation: During the construction phases of both structures, several phenomena like RC cracking, local failure of precast segments, secondary non-structural elements collapse occurred. The main cause was found in an active landslide crossing the tunnels. Strengthening works were necessary and distinct solutions were chosen depending on the construction method and the entity of damage. In addition, since 2016, a SHM system was developed and installed for long-term monitoring. Currently the SHM is still active and used to check the structural behaviour of the tunnels.

Data Description Globally, more than 1400 sensors have been installed, for a total of 108 and 49 monitored cross sections respectively in the two tunnels and about 3.5km of monitored segments. Each section is instrumented with a chain of 9 monoaxial MEMS inclinometers. The sensors layout allows to detect the tunnel cross-section deformation under landslide movements or due to other anomalous events. Four gateways serve each tunnel, equally divided between the two carriageways.

Analysis Procedures The analysis flow is articulated in steps ranging from a sensor level to a wider network level that involves the analysis of each cross sections. At the sensor level, inclinometer raw data are preliminarily processed to highlight the average data trend and to remove environmental influences on acquired measurements. At sectional level, the structural behaviour is simulated by means of nonlinear FE models, including frame elements having NL fibre hinges and staged construction analyses, to reproduce the state of the structure at the beginning of the monitoring and its evolution over the monitored period. Each model is calibrated on the available data (deformations, doorstopper tests, flat jack tests, cracking patterns, geotechnical analyses). Based on the rotation recorded by the 9 sensors of the chain during the monitoring campaign, models are developed to evaluate the structural health status in terms of relative displacements and stress-strain behaviour and further define alarm thresholds.

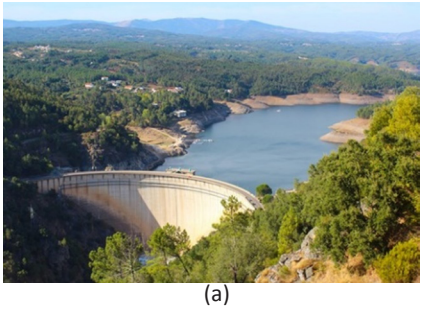
Performance Indicators: Based on the collected data, the parameters identified for long-term monitoring are the chord length trends, the velocity of their variation and the status of fibre hinges in terms of stress-strain behaviour to check if the structure is accumulating a plastic deformation and to identify any possible structural damage.

References: G. Mancini, P. Darò, I. Alovisi et al. (2018), Un sistema competitivo per il monitoraggio e la diagnostica della salute strutturale basato sul cloud, Structural Magazine, 10.12917/STRU215.02

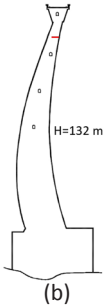
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6.14 SHM of Cabril dam: arch dam with cracking at the crest level

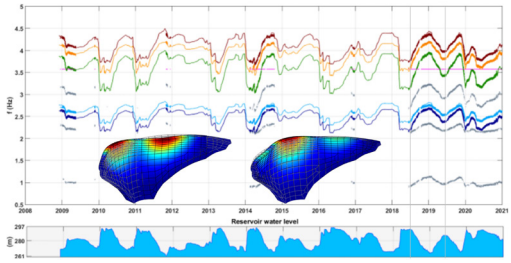


(a)



(b)

Fig. 6-14a-b Cabril dam - Portugal.



(c)

Fig. 6-14c Evolution of identified frequencies over time vs computed frequencies.

Contributors: Sérgio Oliveira[1], Ilídio Ferreira[2], André Alegre[3].

General description: The monitored dam is the highest arch dam in Portugal, Cabril dam, 132 m high. Under operation since 1954, its central cantilever has a maximum thickness of 20 m at the base and a minimum of 4.5 m, seven meters below the crest. At the crest. the thickness is 7 m.

SHM Motivation: Due to the higher crest thickness, significant horizontal cracking occurred at the upper part of the dam, from downstream to upstream face, during the first filling of the reservoir. Also, concrete swelling has been detected in the late 90's. Nevertheless, the dam's behaviour is globally considered normal to this day. A pioneer Seismic and SHM system was designed and installed in 2008. Both, hardware components for vibrations measurement and software modules for vibrations analysis and numerical modelling were developed. The goal was to implement a system with high dynamic range, capable of a continuous and accurate measurement of the dam's dynamic response for ambient/operation excitations and for seismic loads of different magnitudes. The system is expected to be capable to detect structural changes eventually induced by slow damage progression due to the swelling process evolution or induced by a sudden damage increase due to an exceptional occurrence like a strong earthquake.

Data Description: Radial accelerations are measured with 16 uniaxial accelerometers (±0.25g): 9 in the crest gallery and 7 in the gallery below the cracked zone. In the top of central section (crest gallery) is also located a triaxial accelerometer (± 1g). Two other triaxial accelerometers are used in order to measure the seismic accelerations in the dam-rock interface, near both banks.

Analysis Procedures: The software used for monitoring data analysis includes a module for automatic modal identification, using the FDD Singular Value Decomposition (SVD) method, and a novel technique for automatic peak selection, based on a shift smoothed sky-line method and using a clustering technique for distinguish the different modal frequencies; and ii) automatic earthquake detection, based on maxima analysis. Therefore, it is possible to study the dynamic behaviour of the dams over time and to perform comparisons with results from numerical modelling.

Performance Indicators: Variations in natural frequencies due to variations in water level can be identified with good precision and small structural changes over time, due to concrete deterioration can be detected through the correlation with the identified temporal evolution of the main natural frequencies and mode shapes. Structural effects caused by significant seismic events could also be detected through the comparison of the dam dynamic performance under ambient vibrations before and after an earthquake.

References: Oliveira, S., & Alegre, A. (2019). Seismic and Structural Health Monitoring of Dams in Portugal. In M.P. Limongelli & M. Çelebi (Eds.), Seismic Structural Health Monitoring (pp. 87–113). Springer International Publishing. https://doi.org/10.1007/978-3-030-13976-6_4.

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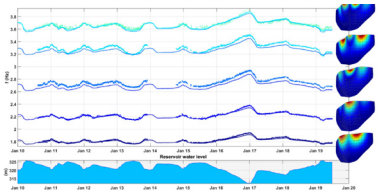
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6.15 SHM of Cahora Bassa dam: a 170 m high arch dam in Mozambique, built in the early 70s



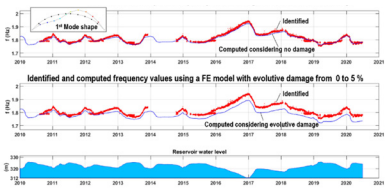
(a)

Fig. 6-15a-b-c Cahora Bassa dam - Mozambique.



(d)

Fig. 6-15d Evolution of identified frequencies over time vs computed frequencies.



(e)

Fig. 6-15e Vibration-based damage detection. Analysis of the 1st natural frequency evolution over time.

Contributors: Sérgio Oliveira[1], Ezequiel Carvalho[2], Paulo Mendes[3], André Alegre[4].

General description: Cahora Bassa dam was built in late 1974 on the Zambezi River. It is a thin double curvature arch dam, 170 m high, with a crest about 300 m long. The central cantilever thickness ranges from 23 m at the base and 4 m at the crest.

SHM Motivation: A few years after the dam's construction a concrete swelling process was detected (in the 1980's) and a typical cracking pattern can be observed at the crest surface. In 2010, Hydroelectric of Cahora Bassa (HCB) installed a SHM system for continuously measure accelerations in dam body and in the dam-foundation interface. The system was designed for the characterisation of the dam dynamic behaviour under operational/ambient excitation and seismic loads. Appropriate software is used to automatically process, manage and analyse the recorded data as well as software for the automatic comparison between the measured data and numerical results from FEMs.

Data Description: Radial accelerations are measured with 10 uniaxial accelerometers in the crest gallery. Three other triaxial accelerometers are used in order to measure the seismic accelerations in the dam-rock interface, near both banks and at the base of the central section.

Analysis Procedures: The evolution over time of the estimated natural frequencies for the first five modes is compared with the computed frequencies considering the actual water levels (the correspondent computed mode shapes are shown). A good agreement between modal identification and numerical modelling results was reached.

Performance Indicators: The time evolution curve for the 1st natural frequency that was calculated with a FEM considering no damage corresponds very well to the frequency values identified for the period 2010-2020. This fact means that the existing deterioration phenomena due to swelling are not significantly affecting the structural health of the dam. On the other hand, it was found that considering an FE model with evolutionary damage, the calculated frequency values will end up diverging from the identified frequency values.

References: Oliveira, S. & Alegre, A., "Vibrations in large dams. Monitoring and modelling", ICOLD 26th Congress and 86th Annual Meeting, Vienna, Austria, 2018.

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6.16 Heritage Case Study: combining NDT & core sampling to assess the state of conservation of an early 20th century RC heritage building in view of adaptive re-use

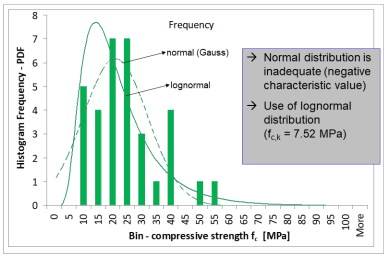


(a)



(b)

Fig. 6-16a-b RC heritage building in Belgium and sampling by core drilling.



(c)

Fig. 6-16c Gathered data translated into compressive strength values.



(d)

Fig. 6-16d Retrofitting by application of externally bonded reinforcement.

Contributors: Els Verstrynge[1], Luc Schueremans[1], Kris Brosens[2].

General description: Constructed in 1902, the building known as the "fire mills" is one of the first industrial buildings in Belgium in RC. It operated as a grain mill for about fifty years, after which it was neglected, leading to serious degradation. Early 21st century, a rehabilitation project converted the industrial heritage building into offices. This case illustrates the complexity in gathering data from an historical building for rehabilitation design.

Motivation: To support the design calculations required for the adaptive re-use of the building, information was gathered regarding the (degraded) material properties and layout of the RC structure. Because the building is listed, the number of DT had to be minimised, and therefore a combination of NDT and core sampling was performed to assess the concrete.

Data Description: Inspection and phenolphthalein testing revealed water infiltrations, overall carbonation-induced corrosion of the reinforcement, with serious concrete cracking and spalling. In addition, shear cracking was caused by low concrete strength and insufficient shear reinforcement. The concrete strength was determined using a combination of i) compressive tests on drilled cores (DT), ii) ultra-sonic testing (NDT), and iii) sclerometer testing (NDT), with core samples taken at 33 locations on different floors.

Analysis Procedures: The gathered data were translated into a compressive strength value for each of the applied measurement techniques. In order to account for the reliability of each test, weighting factors were applied: 0.6 for the most reliable test being compression testing of drilled cores, 0.25 for the ultrasonic tests, and only 0.15 for the sclerometer testing. The latter was considered unreliable as the concrete surface hardness measurement was influenced by a hard cement plaster, typical for early RC buildings. Finally, the characteristic strength was calculated from the weighted values, assuming a lognormal distribution, resulting in a very low value of 7.52 MPa that was subsequently used for the retrofitting design, including the design and application of externally bonded reinforcement (see picture).

References: Figeys W., Schueremans L., Van Gemert D., Brosens K., (2006). Preservation of the RC-monument 'Vuurmolen' (1902) at Overijse, Belgium, in: Proceedings of 2nd Int. *fib* Conference, Napels, 5-8 June 2006. Schueremans, L., Verstrynge, E., Figeys, W. (2011). Externally bonded reinforcement of concrete stuctures with FRP - pre-investigations and research perspectives. In Gieler, R. (Ed.), WTA Seminar Vol. 35. WTA Kolloquium. Fulda, 11 March 2011 (pp. 51-76) WTA-publications.

[1] Department of Civil Engineering, KU Leuven, Leuven, Belgium, els.verstrynge@kuleuven.be

[2] Triconsult NV, Lummen, Belgium, Kris.brosens@triconsult.be

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6.17 Cultural Heritage: Vibration-based SHM of the Brivio Bridge (1917)



Fig. 6-17 The Brivio bridge, Milan - Italy.

Contributors: Carmelo Gentile[1].

General description: The Brivio bridge, completed in 1917, is 132 m long and consists of three RC tied arches spanning about 44 m each. The bridge crosses the Adda river about 50 km North-East of Milan. The deck of each arch bridge is 9.05 m wide, for two traffic lanes and two pedestrian walkways, and consists of a cast in place RC slab supported by two longitudinal beams that are hung from each arch with 16 RC hangers.

SHM Motivation: Notwithstanding the satisfactory performance exhibited by the bridge in recent proof load tests and repeated dynamic tests, the age of the infrastructure along with the continuous increase of traffic and the presence of some cracks suggested the installation of a continuous dynamic monitoring system, including 8 seismometers for each span and 1 temperature sensor. The high sensitivity (78 V/m/s-1) seismometers are installed on the upstream side of the bridge only, taking advantage of the structure symmetry. Each group of 8 sensors is wired to a 24-bit digitiser and the digitisers are connected to one Universal Mobile Telecommunications System (UMTS) modem for data transfer. Every hour, a binary file for each sensor is created, and it is sent to Politecnico di Milano for data processing.

Data Description: The structural response (in terms of velocity) to ambient and operational excitations is recorded at a sampling frequency of 100 Hz, and datasets of 3600 s are continuously collected.

Analysis Procedures: Every hour, the collected data are automatically processed through a multi-level procedure developed in the Matlab framework and involving the following steps: (a) preliminary pre-processing of the raw data; (b) data analysis to extract the maximum and the RMS value of each time series; (c) low-pass filtering and down-sampling of data; (d) automated identification of modal parameters; (e) removal of environmental/operational effects from the natural frequencies.

Performance Indicators: Overall, seven vibration modes are identified for each span, with the natural frequencies increasing with decreased temperature, whereas no remarkable changes of mode shapes are detected so far. The onset of damage is identified by using both the changes in mode shapes and the cleansed natural frequencies (within the framework of classic novelty analysis).

Acknowledgements: The support of Lombardy Region is gratefully acknowledged.

References: Borlenghi, P., Gentile, C., & Zonno G. (2021). Monitoring reinforced concrete arch bridges with operational modal analysis. In: Proceedings 1st Conference of the European Association on Quality Control of Bridges and Structures, EUROSTRUCT 2021, Padova, 29 August - 1 September 2021.

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6.18 NDT-based prediction of carbonation-induced corrosion of Tokyo's National Museum of Western Art



Fig. 6-18a NMWA in the background with Rodin's sculpture 'Burghers of Calais' in the foreground.

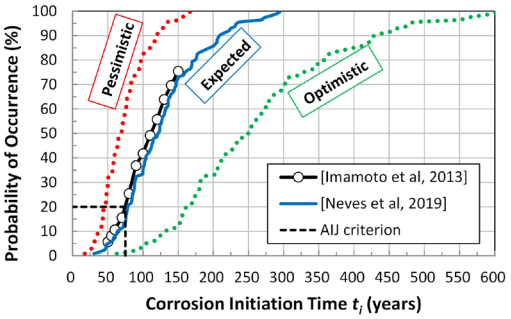


Fig. 6-18b Probabilistic prediction of corrosion initiation time.

Contributors: Kei-ichi Imamoto[1] and Roberto Torrent[2].

General description: The case study refers to Tokyo's National Museum of Western Art (NMWA), that houses a valuable collection of paintings and sculptures from renown western artists. It is the only building in Japan designed by the famous Swiss-French architect Le Corbusier and has been recognised by UNESCO as a World Heritage Site (Fig. N.1).

SHM Motivation: In 2009, when the building had reached 50 years of age, a visual inspection revealed incipient signs of steel corrosion in some parts of the exterior concrete beams, attributed to carbonation. Hence, a detailed investigation was commissioned, described in detail in the references.

Analysis Procedures: The coefficient of air-permeability kT was measured on 111 points of the external beams, from which the carbonation coefficient Kc (mm/year½) was estimated by two approaches: a) Calibration with carbonation measurement on 3 small cores (Ø20 x 50 mm); b) Analysis of data from several structures in Japan, Portugal and Switzerland.

Parallel determination of the cover thickness d (GPR technique) on the same 111 points allowed a probabilistic prediction of the corrosion initiation time, shown in Fig. N.2 for both approaches. Approach b) yields predictions under Pessimistic, Expected and Optimistic scenarios. Interestingly, a small probability of corrosion initiation is predicted at 50 years, consistent with the observed damage at that age. According to the recommendations of the Architecture Institute of Japan (AIJ), an intervention is due when the probability of corrosion reaches 20%, which corresponds to 75 years (i.e., 25 years later), see black broken line in Fig. N.2.

References: Imamoto, K., Tanaka, A. and Kanematsu, M. (2013). "Non-destructive assessment of concrete durability of the National Museum of Western Art in Japan". CONSEC'13, Nanjing, China, 23-25 Sept., 1335-1344. Neves, R., Torrent, R. and Imamoto, K. (2018). "Residual service life of carbonated structures based on site non-destructive tests". Cem. & Concr. Res., v109, 10–18.

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[2] Materials Advanced Services Ltd., Argentina and Switzerland, torrent.concrete@gmail.com

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6.19 Assessment of the residual strength and durability of the prestressed wharf Hermann du Pasquier of Le Havre harbour

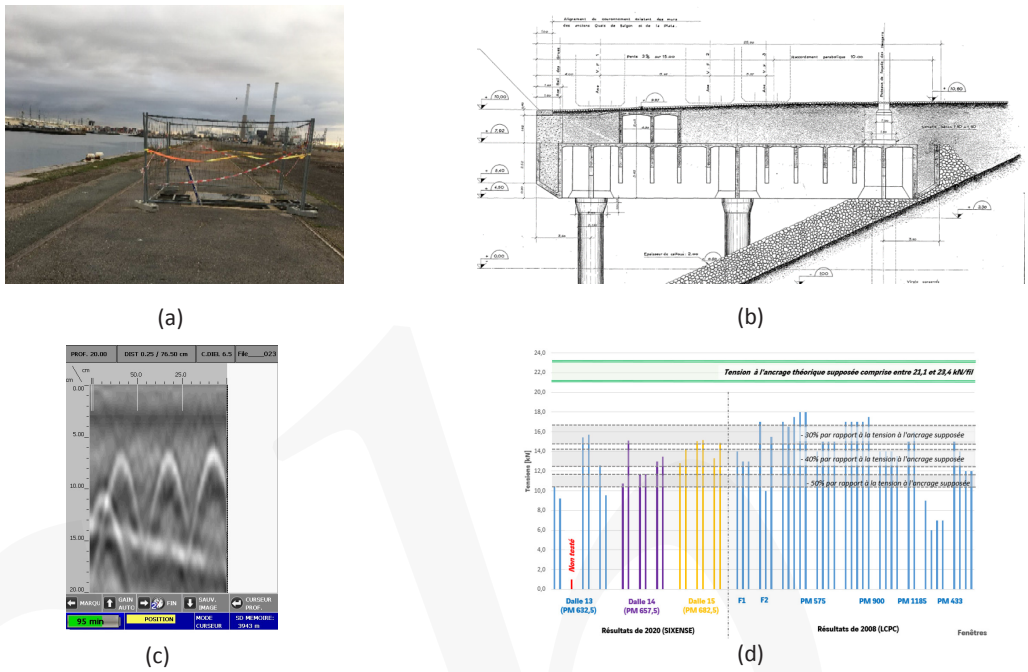


Fig. 6-19a-b-c-d The wharf Hermann du Pasquier, Le Havre, France.

Contributors: Bruno Godart[1], Franziska Schmidt[1].

General description: After the destruction of Le Havre harbour during WW2, the wharf Hermann du Pasquier has been built between 1949 and 1952. It is a self-standing installation, 1200 meter long, with a prestressed concrete deck supported by 3 piers of 1.50 m diameter in the transverse direction. The deck itself is composed of prefabricated square sections of 25 meters in length.

SHM and testing Motivation: In 2021, part of the wharf is undergoing a change of use. As the residual strength and life expectancy of the existing structure is uncertain, this part will be reconstructed. This is an opportunity for the harbour manager to apply NDT and DT methods to assess the whole structure.

Data Description: The wharf comprises a concrete structure prestressed in 3 directions (longitudinally, transversally and vertically on piers), the residual tension in the tendons has been measured. Samples from the wires and the injection grout are analysed. The reinforced steel ducts have been detected by radar to position the openings to access the tendons. On the other hand, the concrete has been investigated by taking samples and performing lab tests, namely determination of free and total chlorides, complete mineralogical analysis, carbonation depth measurement and measurement of Young's modulus and compressive strength.

Analysis Procedures: The gathered data has been analysed in terms of structural residual strength (tension in the tendons) and durability.

Performance Indicators: The durability indicators are those offered in the reference source. Prestressing Forces are compared against those listed in the design documents.

References: Conception des bétons pour une durée de vie donnée des ouvrages. Maîtrise de la durabilité vis-à-vis de la corrosion des armatures et de l'alcali-réaction. Etat de l'art et Guide pour la mise en oeuvre d'une approche performantielle et predictive sur la base d'indicateurs de durabilité, AFGC, 2004. Available online: https://www.afgc.asso.fr/app/uploads/2007/10/Indicateurs-durabilite.pdf.

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6.20 Wind turbine monitoring under environmental & operational variability



Fig. 6-20 Aventa wind turbin, Winterthur - Switzerland.

Contributors: Imad Abdallah[1], Konstantinos Tatsis[1], Vasilis Dertimanis[1], Charilaos Mylonas, Dominik Werne, and Eleni Chatzi[1].

General description: This case study involves a wind turbine of 6.5 kW rated power, with a rotor diameter of 12.9 m and a hub height is 18 m. The tower is a tubular steel-RC structure, supported on concrete foundation, while the blades are made of glass fibre with a tubular steel main-spar. The turbine is regulated via a variable-speed and variable pitch control system.

SHM Motivation: Designed and commissioned in 2002, the Aventa wind turbine in Winterthur is soon reaching its end of design lifetime. In order to assess data-driven techniques for Remaining Useful Life (RUUL) estimation, a SHM campaign was implemented by ETH Zurich. The monitoring campaign started in June 2019 and is still ongoing. In addition, the setup is used as a research platform on topics such as system identification, operational modal analysis, faults/damage detection and classification. We analyse the influence of operational and environmental conditions on the modal parameters and to further infer PIs for assessing structural behaviour in terms of deterioration processes.

Data Description: The tower and nacelle have been instrumented with 11 accelerometers distributed along the length of the tower, nacelle main frame, main bearing and generator. In additional we are collecting operational performance data (SCADA), namely: wind speed, nacelle yaw orientation, rotor RPM, power output and turbine status. Two full-bridge strain gauges is in the planning to be installed on the concrete tower base in the summer of 2021.

Analysis Procedures: The gathered data will be analysed to track the modal parameters by means of an automated operational modal analysis, as well as non-stationary analysis (e.g., PCE-SP-TARMA) to investigate abnormal/faulty turbine characteristics under varying environmental and operational influences. Furthermore, an aeroelastic model of the wind turbine is developed in the open-source simulator OpenFAST in order to create hybrid models for better prediction of remaining useful lifetime.

Acknowledgements: The instrumentation campaign has been supported by the BRIDGE Discovery Program of the Swiss National Science Foundation and Innosuisse, project number 40B2 - 0_187087. We further acknowledge the support of the European Research Council via the ERC Starting Grant WINDMIL (ERC-2015-StG #679843) on the topic of Smart Monitoring, Inspection and Life-Cycle Assessment of Wind Turbines.

Available online: https://zenodo.org/record/4972789#.YMsuovn7Sbg.

[1] Department of Civil, Environmental and Geomatic Engineering, ETH Zürich, Switzerland, chatzi@ibk.baug.ethz.ch

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6.21 Monitoring early cracks in the cement concrete pavement using GPR

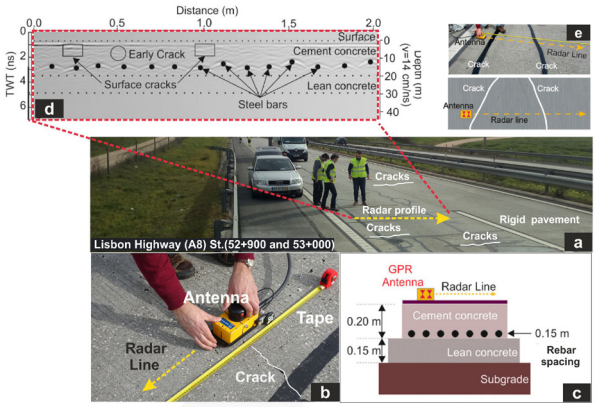


Fig. 6-21 Monitoring early cracks in the cement concrete pavement using GPR.

Contributors: Mezgeen Rasol[1].

General description: This case study describes a cement concrete pavement section located on the A8 highway in Portugal (Figure. a). Structural layers of the cement concrete pavement consist of different layers including RC concrete layer (see Figure. b and c).

SHM Motivation: The monitoring of cracks in cement concrete is a key to further understanding the overall performance of such critical infrastructures. GPR signal, in particular an increase in the amplitude is a PIs of the existence of early cracking damage due to internal fractures and weather conditions before fully propagation of cracks to the concrete surface.

Data Description: Two radar profiles, 2 and 3 meters crossing the main highway lane and a visible crack in the cement concrete was noticeable. Radar data was acquired with a high resolution 2.3 GHz centre frequency shielded antenna in two different orientations; antenna dipoles are perpendicular and parallel to the radar line. Profiles 1 presents a 1 ns time window (TwT) as indication for the concrete pavement surface, aiming to detect damage such as early cracks and possible damage such as layer settlement, and rebars in the cement concrete.

Analysis Procedures: Four stages of work plan are presented; controlled laboratory samples, real scale case study in Portugal, common post-processing methodologies (e.g., background removal, static correction and gain) and computational modelling based on Finite-Difference Time-Domain (FDTD) method is elaborated to validate measurements from laboratory samples and filed cases studies. Data interpretation highlighting the existence of early cracks before propagation to the surface of concrete (see Figure d and e). In addition, a line of steel rebar (not straight line) is detected, which could be influenced by the ground settlements.

Damage Indicators (DIs): An increase in amplitude on the concrete surface, and quantifying amplitude against crack dimensions are PIs of the existence of cracks, and early cracks to further provide valuable information as a supporting knowledge for decision-making owners. Hence dielectric constants of cracks (empty with air) and concrete are 1.0 and (5.0-8.5), respectively.

Acknowledgements: This research has been funded by the Spanish MINECO and FEDER projects referenced as CGL2011-23621, CGL2015-65913 -P, and COST Action CA15202 (SARCOS).

References: Rasol. M (2021). Development of New GPR Methodologies for Soil and Cement Concrete Pavement Assessment. PhD Thesis. Polytechnic University of Catalonia (UPC).

https://doi.org/10.13140/RG.2.2.32976.74242

[1] The Urban and Civil Engineering Testing and Modelling Laboratory (EMGCU), Department of Materials and Structures (MAST), Université Gustave Eiffel, mezgeen.rasol@univ-eiffel.fr

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6.22 Structural health monitoring of the Mezza stadium in Milano

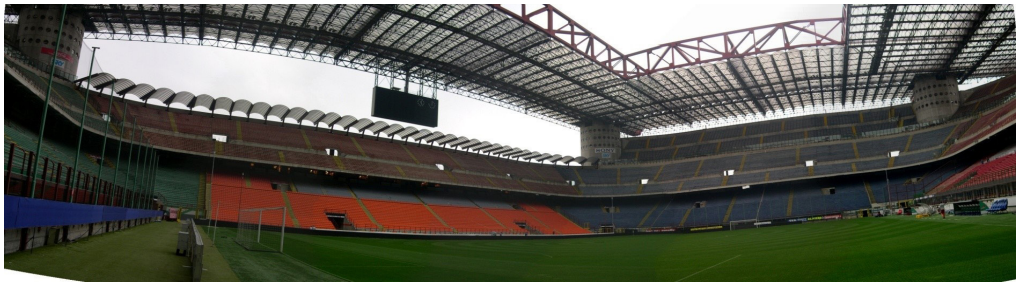


Fig. 6-22 The San Siro Stadium (Mezza stadium), Milano - Italy.

Contributors: Alfredo Cigada[1], Simone Turrisi[1], Emanuele Zappa[1].

General description: The San Siro Stadium (formerly Mezza) is 90-years-old; it is recognised as an icon in sports architecture. It can host up to 80'000 people on its three tiers. It has been the home of many important sports events but also of several concerts, religious events and more.

SHM Motivation: Stadia are critical infrastructures for which safety and proper maintenance form major issues. During events people tend to move, or bounce, producing a high level, unmeasurable excitation. Lively structures tend to move and peak values are linked to eventual discomfort, panic or even damage. The same monitoring system can be used when the stadium is empty, under low level (linear) wide band random excitation, to assess the structure health and help driving maintenance strategies.

Data Description: A sensor fusion approach has been adopted, with different strategies for each tier: concrete ageing is considered for the lower stands, vibration sensors for the cantilever structures of the second tier, vibration sensors and clinometers for the main post tensioned beams and the grandstands of the third tier and the roof. Displacement sensors check the thermal expansion of the main steel beams supporting the roof. The roof health is measured by 37 clinometers, a weather station and some crack sensors complete the set; the grand total is 200 measurement points. A local network is supervised by a central unit managing two separate networks: the first sending data to a cloud and allowing for remote control and the second periodically polling ten distributed FPGA based units, directly connected to the sensors. A metrological validation is offered by periodic surveying measurements.

Analysis Procedures: The number of sensors over each sub-structure does not allow for a real modal analysis: RMS, mean, maximum and minimum values every second provide prompt warnings in case data show anomalies. Short-term and long-term time and frequency evaluations are performed automatically. Many DFs are being implemented to have an effective early warning, ranging from PCA to co-integration, from AR models to auto-encoders.

Performance Indicators: Each sensor has a data past history, lasting 18 years. Limits to provide alarms have been fixed according to different strategies for the events and the general maintenance requirements. During events, a first alarm is issued when vibrations exceed 80% of the past maximum; a second limit is provided by a digital twin which has been tuned and updated by means of a modal analysis. As for maintenance, the daily and seasonal changes are strictly controlled to check a constant and steady structure "breath". Data are completed by corrosion sensors and topographic surveys.

Web Link to Data Repository: Data are sent to a private repository owned by the Municipality of Milano (owner of the stadium) and accessible only by registered users.

Acknowledgements: The authors would like to acknowledge the Municipality of Milano that supported this research.

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Annex

In recent years, automated optical, remote sensing and vision-based technologies have gained ground as an advanced means to visual inspection (typically applied periodically) owing to the extremely dynamically developing sectors dealing with remote sensing and image analysis. Table A-1, drawn from the work of Omar and Nehdi[269], summarises some of the main technologies for remote sensing, along with their strengths and limitations. Image acquisition can further be achieved based on conventional, high resolution or InfraRed (IR) cameras and video recorders installed on, or outside, the structure. Recently, remote robotic platforms, drones (Unmanned Aerial Vehicles – UAVs or Unmanned Aerial Systems – UAS), robots and cell phones can be used as mobile supports of this type of sensing devices. Processing of vision-based data for damage detection typically relies on the use of ML or DL-based image processing schemes, with Convolutional Neural Networks (CNNs) perhaps being a dominant approach to this end.

Methods developed to this end usually compare the referential image of the intact structure with images registering discrepancies of different types, often incorporating segmentation, template matching or edge detection schemes[270]. These discrepancies constitute the DIs, allowing for the detection of cracking, concrete spalling, corrosion stains, delamination of reinforcement and other damage effects. These techniques can be turned robust via extension of available databases with visual information from damaged specimens using radar, 3D laser or IR cameras (see Table A-2).

Table A-1 Summary of wave-based monitoring systems of RC bridges (reused from another study[269]).

Technique	Advantages & Limitations
Robotic Total Station[271]	+ High precision, automation and can measure the three-dimensional coordinates - Cannot complete multi-target tasks in a short time - Due to its low measuring frequency, the requirement of dynamic measurement cannot be satisfied
GPS Satellite-Surveying[272]	+ Calculate displacement based on measuring the coordinate changes in one, two or three dimensions + Applicable to all-weather continuous monitoring - Low measurement rate, high cost and requires highly skilled professionals
Motion Detection Cameras[273]	+ Measure bridge displacement through analysing the series of photo frames - Require a certain amount of light to achieve accurate results - Accuracy may be compromised due to susceptibility to vibrations from wind and vehicle loading when mounted on bridge
Digital Image Cross-Correlation[274]	+ Optical measurement technique for collecting displacements + Rapid testing of in-service bridges through digital photography - Accuracy of deflection measurement should be verified
Radar Sensors[275]	+ Wireless and mounted sensors with relatively low cost - Bridge motion can be extracted from the reflected signals of the radar waves - Signal demodulation and data processing are difficult in the presence of noise
Laser Doppler Vibrometer[276]	+ Measure the vibration and displacement of bridges - Provide accurate results but should be placed on the ground underneath the bridge and cannot be left unattended. Large device size - Not suitable for long-term continuous monitoring due to its fixed position

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Terrestrial Laser Scan[277]	+ Can rapidly build complex, irregular 3D visualisation scene models - Low monitoring accuracy: the post-processing of massive data is complicated
Laser Projection Sensing[278]	+ High image acquisition frequency and able to reflect the variation of structural dynamic displacement completely + A feasible method to monitor bridge displacement - Requires long-distance laser devices and high-resolution cameras

Table A-2 Summary of optical and vision-based monitoring systems of RC bridges (reused from another study[269])

Technique	Measurands	Applications
Impact Echo (IE)	Mechanical or seismic stress wave propagation method (acoustic) Short-duration stress pulse usually introduced using mechanical impact The compression waves will reflect at interfaces where there is change in acoustic impedance Complete reflection occurs at concrete-air interface Monitoring the frequency arrival of reflected waves at a nearby location	Detection of defects: delamination, cracks, voids, and honeycombing Detection of overlay debonding on decks with overlays Evaluate concrete elastic modulus
Ultrasonic Pulse Echo (UPE)	Ultrasonic stress wave propagation method (acoustic) Transducer emits short pulse, high amplitude of ultrasonic waves The compression and shear waves will reflect where there is a change in acoustic impedance Complete reflection of ultrasonic waves at concrete-air interface Measuring the transit time of the exited ultrasonic pulse	Detection of defects: delamination, cracks, voids, and honey combing Deck thickness measurements Debonding of reinforcement bars
Half-Cell Potential (HCP)	Electrochemical method Measures potential difference between steel reinforcement and standard reference electrode using a voltmeter	Identifies probability of active corrosion of steel reinforcement at time of testing
Ground Penetrating Radar (GPR)	Electromagnetic wave propagation method Electromagnetic pulses are emitted via a transmitter antenna The higher the frequency of antenna, the better the resolution but the lower the depth of penetration Reflection at interface with materials of different electric properties (electrical conductivity and relative dielectric permittivity)	Evaluation of deck thickness Measurement of thickness of concrete cover Description of concrete as a corrosive environment Locating steel reinforcement
Infrared Thermography (IRT)	Electromagnetic surface radiation in the InfraRed (IR) region Utilises the radiated heat energy to characterise subsurface conditions Anomalies interrupt heat transfer and result in surface temperature differentials	Detection of defects: delamination, disintegration, cracks, and voids Detection of layers and overlay debonding

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i Section 1.4 was prepared with the support of the project No. 22-00774S MAPAB.

ii Section 4.2.2 was prepared with the support of the project No. 22-00774S MAPAB.

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ISBN 978-2-88394-172-4
ISSN: 1562-3610

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