Non-destructive evaluation of reinforced concrete structures
Related titles:

**Strengthening and rehabilitation of civil infrastructures using fibre-reinforced polymer (FRP) composites**  
(ISBN 978-1-84569-448-7)  
The book discusses the mechanical and in-service properties, the relevant manufacturing techniques and aspects related to externally bonded FRP composites to strengthen/rehabilitate/retrofit civil engineering structural materials. The book focuses on: mechanical properties of the FRP materials used; analysis and design of strengthening/rehabilitating/retrofitting beams and columns manufactured from reinforced concrete (RC), metallic and masonry materials; failure modes of strengthening systems; site preparation of the two adherend materials; durability issues; quality control, maintenance and repair of structural systems; case studies.

**Developments in the formulation and reinforcement of concrete**  
(ISBN 978-1-84569-63-6)  
Developments in the formulation and reinforcement of concrete are of great topical interest to the construction industry worldwide, with applications in high-rise, offshore, nuclear and bridge structures, and in pre-cast concrete. This authoritative book addresses in one source the current lack of information on the latest developments in the formulation and reinforcement of concrete. The book discusses the latest types of reinforced concrete and reinforcement and includes chapters on hot weather concreting, cold weather concreting and the use of recycled materials in concrete. It presents current research from leading innovators in the field.

**Failure, distress and repair of concrete structures**  
(ISBN 978-1-84569-408-1)  
Many concrete structures around the world have reached or exceeded their design life and are showing signs of deterioration. Any concrete structure which has deteriorated or has sustained damage is a potential hazard. Understanding and recognising failure mechanisms in concrete structures is a fundamental pre-requisite to determining the type of repair or whether a repair is feasible. *Failure, distress and repair of concrete structures* provides in-depth coverage of concrete deterioration and damage, as well as looking at the various repair technologies available. The first part of the book describes failure mechanisms in concrete including causes and types of failure. The second part examines the repair of concrete structures including methods, materials, standards and durability.

Details of these and other Woodhead Publishing materials books can be obtained by:

- visiting our web site at www.woodheadpublishing.com  
- contacting Customer Services (e-mail: sales@woodheadpublishing.com; fax: +44 (0) 1223 893694; tel.: +44 (0) 1223 891358 ext. 130; address: Woodhead Publishing Limited, Abington Hall, Granta Park, Great Abington, Cambridge CB21 6AH, UK)

If you would like to receive information on forthcoming titles, please send your address details to: Francis Dodds (address, tel. and fax as above; e-mail: francis.dodds@woodheadpublishing.com). Please confirm which subject areas you are interested in.
Non-destructive evaluation of reinforced concrete structures

Volume 2: Non-destructive testing methods

Edited by
Christiane Maierhofer, Hans-Wolf Reinhardt and Gerd Dobmann

© Woodhead Publishing Limited, 2010
Contents

Contributor contact details xv
Preface xxi

Part I Planning and implementing non-destructive testing of reinforced concrete structures 1

1 Planning a non-destructive test programme for reinforced concrete structures 3
C. Maierhofer, BAM Federal Institute for Materials Research and Testing, Germany

1.1 Introduction 3
1.2 Strategies for the application of non-destructive testing (NDT) methods 4
1.3 Overview of non-destructive testing (NDT) methods 6
1.4 Qualification/validation of methods 8
1.5 Sources of further information and advice 10
1.6 References 12

2 Non-destructive testing methods for building diagnosis – state of the art and future trends 14
C. Flohrer, HOCHTIEF Construction AG, Germany

2.1 Introduction 14
2.2 Tasks for building diagnosis 14
2.3 Efficient testing methods 15
2.4 Examples of the application of the testing methods 18
2.5 Future trends 28
2.6 References 29
<table>
<thead>
<tr>
<th>Chapter</th>
<th>Title</th>
<th>Pages</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>Development of automated non-destructive evaluation (NDE) systems for reinforced concrete structures and other applications</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>G. Dobmann and J. H. Kurz, Fraunhofer-IZFP, Germany; A. Taffe, BAM Federal Institute for Materials Research and Testing, Germany; D. Streicher, Joint Lab of Fraunhofer &amp; BAM, Germany</td>
<td></td>
</tr>
<tr>
<td>3.1</td>
<td>Introduction</td>
<td>30</td>
</tr>
<tr>
<td>3.2</td>
<td>The innovation cycles</td>
<td>31</td>
</tr>
<tr>
<td>3.3</td>
<td>Data acquisition, control and evaluation in automated multisensor systems</td>
<td>34</td>
</tr>
<tr>
<td>3.4</td>
<td>Case studies of successful innovations to automated systems in non-destructive testing (NDT) engineering</td>
<td>36</td>
</tr>
<tr>
<td>3.5</td>
<td>Non-destructive testing for structural engineering</td>
<td>46</td>
</tr>
<tr>
<td>3.6</td>
<td>Multiple-sensor data acquisition by the OSSCAR (On-Site SCAnneR) scanner</td>
<td>54</td>
</tr>
<tr>
<td>3.7</td>
<td>Conclusions</td>
<td>60</td>
</tr>
<tr>
<td>3.8</td>
<td>Acknowledgements</td>
<td>60</td>
</tr>
<tr>
<td>3.9</td>
<td>References</td>
<td>60</td>
</tr>
<tr>
<td>4</td>
<td>Structural health monitoring systems for reinforced concrete structures</td>
<td>63</td>
</tr>
<tr>
<td></td>
<td>W. R. Habel, BAM Federal Institute for Materials Research and Testing, Germany</td>
<td></td>
</tr>
<tr>
<td>4.1</td>
<td>Introduction</td>
<td>63</td>
</tr>
<tr>
<td>4.2</td>
<td>Demands on monitoring systems: monitoring capabilities</td>
<td>64</td>
</tr>
<tr>
<td>4.3</td>
<td>Innovative monitoring methods</td>
<td>67</td>
</tr>
<tr>
<td>4.4</td>
<td>Selected examples of effective and innovative monitoring technologies</td>
<td>74</td>
</tr>
<tr>
<td>4.5</td>
<td>Reliability of structural health monitoring (SHM) systems and standardization</td>
<td>87</td>
</tr>
<tr>
<td>4.6</td>
<td>Future trends</td>
<td>90</td>
</tr>
<tr>
<td>4.7</td>
<td>References</td>
<td>91</td>
</tr>
<tr>
<td>5</td>
<td>Combining the results of various non-destructive evaluation techniques for reinforced concrete: data fusion</td>
<td>95</td>
</tr>
<tr>
<td></td>
<td>C. Maierhofer, C. Kohl and J. Wöstmann, BAM Federal Institute for Materials Research and Testing, Germany</td>
<td></td>
</tr>
<tr>
<td>5.1</td>
<td>Introduction</td>
<td>95</td>
</tr>
<tr>
<td>5.2</td>
<td>Combination of non-destructive testing (NDT) and minor destructive testing (MDT) methods</td>
<td>96</td>
</tr>
<tr>
<td>5.3</td>
<td>Data fusion</td>
<td>98</td>
</tr>
<tr>
<td>Section</td>
<td>Title</td>
<td>Page</td>
</tr>
<tr>
<td>---------</td>
<td>-------</td>
<td>------</td>
</tr>
<tr>
<td>5.4</td>
<td>Fusion of radar data</td>
<td>100</td>
</tr>
<tr>
<td>5.5</td>
<td>Fusion of radar and ultrasonic data recorded along a beam of a box girder bridge</td>
<td>101</td>
</tr>
<tr>
<td>5.6</td>
<td>Fusion of radar and ultrasonic data at a cross beam inside a box girder bridge</td>
<td>103</td>
</tr>
<tr>
<td>5.7</td>
<td>Sources of further information and advice</td>
<td>105</td>
</tr>
<tr>
<td>5.8</td>
<td>Conclusions and future trends</td>
<td>105</td>
</tr>
<tr>
<td>5.9</td>
<td>Acknowledgements</td>
<td>106</td>
</tr>
<tr>
<td>5.10</td>
<td>References</td>
<td>106</td>
</tr>
<tr>
<td></td>
<td>Part II Individual non-destructive testing techniques</td>
<td>109</td>
</tr>
<tr>
<td>6</td>
<td>Wireless monitoring of reinforced concrete structures</td>
<td>111</td>
</tr>
<tr>
<td>6.1</td>
<td>Introduction</td>
<td>111</td>
</tr>
<tr>
<td>6.2</td>
<td>Basic principles of wireless monitoring</td>
<td>112</td>
</tr>
<tr>
<td>6.3</td>
<td>Definition of the monitoring task</td>
<td>114</td>
</tr>
<tr>
<td>6.4</td>
<td>Monitoring system design and assembly</td>
<td>116</td>
</tr>
<tr>
<td>6.5</td>
<td>Wireless monitoring systems in operation</td>
<td>119</td>
</tr>
<tr>
<td>6.6</td>
<td>Application of intelligent wireless monitoring</td>
<td>119</td>
</tr>
<tr>
<td>6.7</td>
<td>Conclusions and future trends</td>
<td>122</td>
</tr>
<tr>
<td>6.8</td>
<td>References</td>
<td>122</td>
</tr>
<tr>
<td>7</td>
<td>Non-destructive testing of concrete with electromagnetic and acoustic–elastic waves: data analysis</td>
<td>125</td>
</tr>
<tr>
<td>7.1</td>
<td>Introduction</td>
<td>125</td>
</tr>
<tr>
<td>7.2</td>
<td>Similarities and differences between seismic, ultrasonic and electromagnetic wave propagation and their implications on data processing</td>
<td>125</td>
</tr>
<tr>
<td>7.3</td>
<td>Standard data processing</td>
<td>127</td>
</tr>
<tr>
<td>7.4</td>
<td>Sophisticated data processing</td>
<td>136</td>
</tr>
<tr>
<td>7.5</td>
<td>Conclusions and future trends</td>
<td>142</td>
</tr>
<tr>
<td>7.6</td>
<td>References</td>
<td>142</td>
</tr>
<tr>
<td>8</td>
<td>Non-destructive testing of concrete with electromagnetic, acoustic and elastic waves: modelling and imaging</td>
<td>144</td>
</tr>
<tr>
<td>8.1</td>
<td>Introduction</td>
<td>144</td>
</tr>
<tr>
<td>8.2</td>
<td>Electromagnetic, acoustic and elastic waves</td>
<td>145</td>
</tr>
</tbody>
</table>
8.3 Numerical wave field modelling for acoustic, electromagnetic and elastic waves 151
8.4 Wave field inversion and imaging: acoustic waves 154
8.5 Wave field inversion: electromagnetic and elastic waves 158
8.6 Conclusions 159
8.7 References 161

9 Laser-induced breakdown spectroscopy (LIBS) for evaluation of reinforced concrete structures 163
G. Wilsch, BAM Federal Institute for Materials Research and Testing; A. Molkenthin, Specht, Kalleja + Partner GmbH, Germany
9.1 Introduction 163
9.2 Laser-induced breakdown spectroscopy (LIBS): fundamentals and measurement 164
9.3 Characterization of cement, mortar and concrete 167
9.4 Detection of specific elements: specific testing problems 173
9.5 Mobile set-up: on-site applications 180
9.6 Limitations and reliability 183
9.7 References 184

10 Acoustic emission (AE) evaluation of reinforced concrete structures 185
C. U. Grosse, Technical University of Munich, Germany
10.1 Introduction 185
10.2 Basics: parametric and signal-based acoustic emission (AE) analysis 187
10.3 Sensors and instruments 191
10.4 Source localization 193
10.5 Source mechanisms and moment tensor analysis 197
10.6 Applications 199
10.7 Limitations and accuracy 206
10.8 References 210

11 Magnetic flux leakage (MFL) for the non-destructive evaluation of pre-stressed concrete structures 215
G. Sawade, University of Stuttgart, Germany; H.-J. Krause, Forschungszentrum Jülich, Germany
11.1 Magnetic method for inspection of reinforced concrete structures 215
11.2 Description of equipment required 233
11.3 Examples of applications of the magnetic method on site 235
11.4 Perspective: recent developments of the magnetic method for inspection of reinforced concrete 239
11.5 Recommendations for the application of the magnetic flux leakage (MFL) method 240
11.6 References 241

12 Electrical resistivity for the evaluation of reinforced concrete structures 243
J.-F. Lataste, University of Bordeaux 1, France
12.1 Introduction 243
12.2 Physical principles and theory 244
12.3 Use of electrical resistivity 255
12.4 Other developments 264
12.5 Impedance spectroscopy 268
12.6 References 270

13 Capacimetry for the evaluation of reinforced concrete structures 276
X. Dérobert, LCPC, France
13.1 Physical principle and theory 276
13.2 Equipment 279
13.3 Calibration 280
13.4 Data acquisition and interpretation 280
13.5 Applications 281
13.6 Limitations and reliability 282
13.7 References 283

14 Techniques for measuring the corrosion rate (polarization resistance) and the corrosion potential of reinforced concrete structures 284
C. Andrade and I. Martínez, Instituto de Ciencias de la Construcción Eduardo Torroja (CSIC), Spain
14.1 Introduction 284
14.2 Principles 285
14.3 Measurement methods 293
14.4 How to interpret the measurements 303
14.5 Practical application 306
14.6 Monitoring systems 310
14.7 Future trends: new techniques 311
14.8 Conclusions 312
14.9 References 313

© Woodhead Publishing Limited, 2010
Contents

15 Ground penetrating radar for the evaluation of reinforced concrete structures
J. HUGENSCHEIDT, EMPA, Switzerland

15.1 Introduction to ground penetrating radar (GPR)
15.2 Physical principles and theory
15.3 Display formats for ground penetrating radar (GPR) data
15.4 Data processing and interpretation
15.5 Equipment
15.6 Limitations and reliability of ground penetrating radar (GPR)
15.7 Current and future trends
15.8 Symbols and constants
15.9 References

16 Radar tomography for evaluation of reinforced concrete structures
L. Zanzi, Politecnico di Milano, Italy

16.1 Introduction
16.2 Physical principles
16.3 Fundamental equations
16.4 Resolution
16.5 Equipment
16.6 Acquisition procedures
16.7 Data pre-processing
16.8 Data inversion
16.9 Artefacts
16.10 Interpretation of results
16.11 Examples
16.12 Hints on advanced algorithms
16.13 Conclusions
16.14 References

17 Active thermography for evaluation of reinforced concrete structures
C. MAIERHOFER, M. RÖLLIG and J. SCHLICHTEING, BAM Federal Institute for Materials Research and Testing, Germany

17.1 Introduction
17.2 Physical principle and theoretical background
17.3 State of the art
17.4 Experimental equipment and calibration
17.5 Data processing
17.6 Areas of applications
<table>
<thead>
<tr>
<th>Chapter</th>
<th>Title</th>
<th>Authors</th>
<th>Pages</th>
</tr>
</thead>
<tbody>
<tr>
<td>17.7</td>
<td>Future trends</td>
<td></td>
<td>396</td>
</tr>
<tr>
<td>17.8</td>
<td>Guidelines and sources of further information and advice</td>
<td></td>
<td>397</td>
</tr>
<tr>
<td>17.9</td>
<td>References</td>
<td></td>
<td>399</td>
</tr>
<tr>
<td>18</td>
<td>Nuclear magnetic resonance (NMR) imaging for evaluation of reinforced</td>
<td>B. Wolter, Fraunhofer IZFP, Germany</td>
<td>403</td>
</tr>
<tr>
<td></td>
<td>concrete structures</td>
<td></td>
<td></td>
</tr>
<tr>
<td>18.1</td>
<td>Introduction</td>
<td></td>
<td>403</td>
</tr>
<tr>
<td>18.2</td>
<td>Physical background</td>
<td></td>
<td>404</td>
</tr>
<tr>
<td>18.3</td>
<td>Nuclear magnetic resonance (NMR) hardware</td>
<td></td>
<td>407</td>
</tr>
<tr>
<td>18.4</td>
<td>Application possibilities</td>
<td></td>
<td>409</td>
</tr>
<tr>
<td>18.5</td>
<td>Reliability and limitations</td>
<td></td>
<td>413</td>
</tr>
<tr>
<td>18.6</td>
<td>Conclusions and future trends</td>
<td></td>
<td>414</td>
</tr>
<tr>
<td>18.7</td>
<td>References</td>
<td></td>
<td>415</td>
</tr>
<tr>
<td>19</td>
<td>Stress wave propagation for evaluation of reinforced concrete structures</td>
<td>S. Tesfamariam, The University of British Columbia, Canada; B. Martín-Pérez, University of Ottawa, Canada</td>
<td>417</td>
</tr>
<tr>
<td>19.1</td>
<td>Introduction</td>
<td></td>
<td>417</td>
</tr>
<tr>
<td>19.2</td>
<td>Stress wave propagation methods</td>
<td></td>
<td>419</td>
</tr>
<tr>
<td>19.3</td>
<td>Applications</td>
<td></td>
<td>424</td>
</tr>
<tr>
<td>19.4</td>
<td>Discussion and future trends</td>
<td></td>
<td>435</td>
</tr>
<tr>
<td>19.5</td>
<td>Conclusions</td>
<td></td>
<td>436</td>
</tr>
<tr>
<td>19.6</td>
<td>References</td>
<td></td>
<td>436</td>
</tr>
<tr>
<td>20</td>
<td>Surface wave techniques for evaluation of concrete structures</td>
<td>J. S. Popovics, University of Illinois, USA; O. Abraham, LCPC, France</td>
<td>441</td>
</tr>
<tr>
<td>20.1</td>
<td>Introduction</td>
<td></td>
<td>441</td>
</tr>
<tr>
<td>20.2</td>
<td>Basic principles of surface wave propagation</td>
<td></td>
<td>443</td>
</tr>
<tr>
<td>20.3</td>
<td>Signal processing and data presentation</td>
<td></td>
<td>449</td>
</tr>
<tr>
<td>20.4</td>
<td>Equipment</td>
<td></td>
<td>457</td>
</tr>
<tr>
<td>20.5</td>
<td>Field application of surface wave methods</td>
<td></td>
<td>459</td>
</tr>
<tr>
<td>20.6</td>
<td>References</td>
<td></td>
<td>460</td>
</tr>
<tr>
<td>21</td>
<td>Impact–echo techniques for evaluation of concrete structures</td>
<td>O. Abraham, LCPC, France; J. S. Popovics, University of Illinois, USA</td>
<td>466</td>
</tr>
<tr>
<td>21.1</td>
<td>History of the development of the method</td>
<td></td>
<td>466</td>
</tr>
<tr>
<td>21.2</td>
<td>Basic principles of the impact–echo method</td>
<td></td>
<td>467</td>
</tr>
</tbody>
</table>

© Woodhead Publishing Limited, 2010
# Contents

21.3 Data interpretation 469  
21.4 Numerical simulations 474  
21.5 Signal processing, data presentation and imaging 475  
21.6 Equipment 480  
21.7 Impact–echo method applications 481  
21.8 Future trends 484  
21.9 References 485  

22 Ultrasonic techniques for evaluation of reinforced concrete structures 490  
M. SCHICKERT, Institute of Materials Research and Testing (MFPA Weimar), Germany; M. KRAUSE, BAM Federal Institute for Materials Research and Testing, Germany  

22.1 Introduction 490  
22.2 Ultrasonic wave propagation in concrete 491  
22.3 Applications and requirements of ultrasonic non-destructive testing 499  
22.4 Transmission methods 500  
22.5 Imaging of concrete elements 503  
22.6 Future trends 521  
22.7 Sources of further information and advice 525  
22.8 References 526  

Part III Case studies 531  

23 Inspection of concrete retaining walls using ground penetrating radar (GPR): a case study 533  
J. HUGENSCHMIDT, EMPA, Switzerland  

23.1 Problem description 533  
23.2 Data acquisition 534  
23.3 Data processing 536  
23.4 Results 538  
23.5 Conclusions 542  
23.6 Reference 542  

24 Acoustic emission and impact–echo techniques for evaluation of reinforced concrete structures: a case study 543  
M. OHTSU, Kumamoto University, Japan  

24.1 Introduction 543  

© Woodhead Publishing Limited, 2010
Contributor contact details

(* = main contact)

Editors

Christiane Maierhofer*
BAM Federal Institute for
Materials Research and
Testing
Division VIII.4
Unter den Eichen 87
12205 Berlin
Germany
Email: christiane.maierhofer@bam.de

H. W. Reinhardt
Department of Construction
Materials
University of Stuttgart
Pfaffenwaldring 4
D-70569 Stuttgart
Germany
Email: reinhardt@iwb.uni-stuttgart.de

G. Dobmann
Fraunhofer-IZFP
Campus E 3 1
66123 Saarbrücken
Germany
Email: gerd.dobmann@izfp.fraunhofer.de

Chapter 1

Christiane Maierhofer
BAM Federal Institute for
Materials Research and Testing
Division VIII.4
Unter den Eichen 87
12205 Berlin
Germany
Email: christiane.maierhofer@bam.de

Chapter 2

Claus Flohrer
HOCHTIEF Construction AG
Farmstr. 91–97
64546 Mörfelden-Walldorf
Germany
Email: Claus.Flohrer@hochtief.de

Chapter 3

G. Dobmann* and J. H. Kurz
Fraunhofer-IZFP
Campus E 3 1
66123 Saarbrücken
Germany
Email: gerd.dobmann@izfp.fraunhofer.de
Chapter 7
Dr Karl-Josef Sandmeier
Zipser Straße 1
76227 Karlsruhe
Germany
Email: info@sandmeier-geo.de

Chapter 8
K. J. Langenberg*, K. Mayer and R. Marklein
Department of Electrical Engineering and Computer Science
University of Kassel
34109 Kassel
Germany
Email: langenberg@uni-kassel.de

Chapter 9
Gerd Wilsch*
BAM Federal Institute for Materials Research and Testing
Division VIII.2
Unter den Eichen 87
12205 Berlin
Germany
Email: Gerd.Wilsch@bam.de

Dr Andre Molkenthin
Specht, Kalleja + Partner GmbH
Reuchlinstr. 10–11
10553 Berlin
Germany
Email: molkenthin@skp-ingenieure.com

Chapter 4
Wolfgang R. Habel
BAM Federal Institute for Materials Research and Testing
Division VIII.1
Unter den Eichen 87
12205 Berlin
Germany
Email: wolfgang.habel@bam.de

Chapter 5
Christiane Maierhofer*, Ch. Kohl and J. Wöstmann
BAM Federal Institute for Materials Research and Testing
Division VIII.4
Unter den Eichen 87
12205 Berlin
Germany
Email: christiane.maierhofer@bam.de

Chapter 6
Dr Markus Krüger
MPA Universität Stuttgart
Pfaffenwaldring 2b
D-70569 Stuttgart
Germany
Email: markus.krueger@mpa.uni-stuttgart.de
Chapter 10
Prof. Dr Christian U. Grosse
Department of Non-destructive Testing
Centre for Building Materials
Technical University of Munich
Baumbachstraße 7
81245 München
Germany
Email: grosse@cbm.bv.tum.de

Chapter 11
Gottfried Sawade*
Material Testing Institute (MPA)
Universität Stuttgart
Pfaffenwaldring 4
D-70569 Stuttgart
Germany
Email: gottfried.sawade@mpa.unistuttgart.de

Hans-Joachim Krause
Forschungszentrum Jülich
Institut für Bio- und Nanosysteme
52425 Jülich
Germany
Email: h.-j.krause@fz-juelich.de

Chapter 12
Dr Jean-François Lataste
Université Bordeaux 1
Laboratoire GHYMAC
Avenue des facultés, Bâtiment B18
33400 Talence
France
Email: jf.lataste@ghymac.u-bordeaux1.fr

Chapter 13
Xavier Dérobert
LCPC
Route de Bouaye
BP4129
44341 Bouguenais
France
Email: xavier.dérobert@lcpc.fr

Chapter 14
Carmen Andrade and Isabel Martínez*
Instituto de Ciencias de la Construcción Eduardo Torroja (CSIC)
C/Serrano Galvache n° 4
28033 Madrid
Spain
Email: isabelms@ietcc.csic.es;
andrade@ietcc.csic.es

Chapter 15
Johannes Hugenschmidt
EMPA
Ueberlandstrasse 129
8600 Duebendorf
Switzerland
Email: Johannes.hugenschmidt@empa.ch

Chapter 16
Prof. Luigi Zanzi
Department of Structural Engineering
Politecnico di Milano
Piazza Leonardo da Vinci 32
20133 Milano
Italy
Email: luigi.zanzi@polimi.it
Chapter 17
Christiane Maierhofer*, Mathias Röllig and Joachim Schlichting
BAM Federal Institute for Materials Research and Testing
Unter den Eichen 87
12205 Berlin
Germany
Email: christiane.maierhofer@bam.de

Chapter 18
Dipl. Ing. Bernd Wolter
Fraunhofer Institute for Non-destructive Testing (IZFP)
Campus Building E 3.1
D-66123 Saarbrücken
Germany
Email: bernd.wolter@fraunhofer.de

Chapter 19
S. Tesfamariam*
School of Engineering
The University of British Columbia
3333 University Way
Kelowna, BC
V1V 1V7
Canada
Email: solomon.tesfamariam@ubc.ca

B. Martín-Pérez
Department of Civil Engineering
University of Ottawa
161 Louis Pasteur St, Room A018
P.O. Box 450, Stn A
Ottawa, ON
K1N 6N5
Canada
Email: bmartin@eng.uottawa.ca

Chapter 20
John S. Popovics*
The University of Illinois
205 N. Mathews Ave. MC 250
Urbana
IL 61801
USA
Email: johnpop@illinois.edu

Odile Abraham
LCPC
Route de Bouaye
BP4129
44341 Bouguenais
France
Email: odile.abraham@lcpc.fr

Chapter 21
Odile Abraham*
LCPC
Route de Bouaye
BP4129
44341 Bouguenais
France
Email: odile.abraham@lcpc.fr

John S. Popovics
The University of Illinois
205 N. Mathews Ave. MC 250
Urbana
IL 61801
USA
Email: johnpop@illinois.edu
Chapter 22
Martin Schickert*
Institute of Materials Research and Testing (MFPA Weimar) at the Bauhaus University Weimar Coudraystr. 4 99423 Weimar Germany Email: martin.schickert@mfpa.de

Dr rer. nat. Martin Krause
Federal Institute for Materials Research and Testing (BAM) Fachgruppe VIII.2 Unter den Eichen 87 12205 Berlin Germany Email: martin.krause@bam.de

Chapter 23
Johannes Hugenschmidt
EMPA Ueberlandstrasse 129 8600 Duebendorf Switzerland Email: Johannes.hugenschmidt@empa.ch

Chapter 24
Masayasu Ohtsu
Graduate School of Science and Technology Kumamoto University 2-39-1 Kurokami Kumamoto 860-8555 Japan Email: ohtsu@gpo.kumamoto-u.ac.jp

Chapter 25
Xavier Dérobert*
LCPC Route de Bouaye BP4129 44341 Bouguenais France Email: xavier.derobert@lcpc.fr

Bruno Berenger
LRPC Angers 23, av. de l’Amiral Chauvin BP 69 49136 Les Ponts de Cé Cédex France
The scientific and technological development of non-destructive testing (NDT) of materials is based on the interdisciplinary integration of a variety of different and complementary scientific and engineering methods. In addition to physics, material science is essential. The development of test systems requires additional handling technology and robotics, electronic hardware, computer science and software as well as mathematical algorithms for the numerical simulation.

The current state of research and development in the sub-disciplines determines which one takes over the leading role in systems engineering. In the past, the primary driver for NDT innovations came from physics. A significant step forward was achieved by introducing new types of sensor principles, for example in digital industrial radiology and x-ray computer tomography, in low-frequency electromagnetic testing, and in thermography. New trends in development are the integration of system functions in miniaturized digital circuits or by completely processing the inspection data on the software level, resulting in significant power savings and higher system reliability. More NDT applications are now possible in real time.

NDT methods are widely used in several industry branches. A variety of advanced NDT methods is available for metallic or composite materials. However, in civil engineering, NDT methods are still not established for regular inspections and worldwide only a few standardized procedures exist. Guidelines for NDT are currently applied only in special cases, mainly for damage assessment. In recent years, rapid, high-level progress was achieved in the development of technology, data analysis and reconstruction, automation and measurement strategies. Much knowledge and experience were gained and data acquisition was simplified. Therefore, the intention of this publication is to raise awareness within the civil engineering community about the availability, applicability, performance reliability, complexity and restrictions in understanding and application of NDT.

The following chapters cover a major part of the current knowledge and state-of-the-art in this field. This information is arranged as follows.
Volume 1 describes deterioration processes in reinforced concrete and related testing problems (Part I) and contains several conventional/standard testing methods (Part II) for the analysis of concrete components, internal structure and large structural elements. In Volume 2, strategies about planning and implementing NDT campaigns on reinforced concrete structures are outlined (Part I). This part is followed by chapters detailing the individual NDT methods. Part III of Volume 2 presents selected case studies.

Basic principles of the methods as well as practical applications are both addressed, although the emphasis varies between chapters. It should be mentioned that although several aspects have been considered by involving three editors from different fields of knowledge, this selection is not comprehensive. In order to achieve an entire and updated overview, the cited references and conference proceedings should be used.

The editors hope that this book will be a helpful tool for practitioners in applying the new technology, and can contribute to increased safety, reliability and efficiency of reinforced concrete infrastructure.

The editors would like to thank all contributors for their effort, without which the book would not have been possible. They also acknowledge gratefully the patience and continuing encouragement of the staff of Woodhead Publishing and the perfect production of the two volumes.

Christiane Maierhofer
Hans-Wolf Reinhardt
Gerd Dobmann
Planning a non-destructive test programme for reinforced concrete structures

C. MAIERHOFER, BAM Federal Institute for Materials Research and Testing, Germany

Abstract: To ensure the structural safety, durability and performance of infrastructure, an efficient system for early and regular structural assessment as well as for quality assurance during and after both new constructions and reconstructions is urgently required. Various non-destructive testing (NDT) methods and monitoring technologies are available to be applied in civil engineering. Their application is improved by partial automation of data recording and analysis. The results of on-site assessments allow quality assurance systems and methodologies for regular inspections to be improved by employing an efficient selection and planning of measures related to the different stages of assessment, to the type of structures and to the age, condition and loading of the concrete.

Key words: non-destructive testing, concrete, on-site assessment, method validation, quality assurance.

1.1 Introduction

To achieve and maintain a high level of structural safety, durability and performance of the infrastructure of a country, an efficient system for early and regular structural assessment is urgently required. Quality assurance during and after the construction of new structures and after reconstruction processes, and the characterisation of material properties and damage as a function of time and environmental influences is increasingly becoming a serious concern. Non-destructive testing (NDT) methods have an increasing potential to be part of such a system for the management of infrastructures.

The strategy to develop NDT methods in civil engineering is to modify existing technologies from other areas of application in several industry branches. Aircraft, nuclear facilities, chemical plants, transport systems, electronic devices and other safety-critical installations are tested regularly and non-destructively requiring fast and reliable testing technologies. Thus, NDT is highly advanced and a variety of methods is available for metallic or composite materials.
The development of NDT methods as well as their on-site applications in civil engineering is mainly performed in projects externally funded and in close co-operation with partners from industry, public administrations, universities and other research institutions. These projects concern, among other issues:

- basic research on the penetration of electromagnetic, sonic and ultrasonic waves through building materials (e.g. research group FOR384, funded by the German Research Society DFG),
- development of new NDT methods and system configurations for specific testing problems (such as radar, microwave applications, ultrasonics, impact–echo, active thermography, laser-induced breakdown spectroscopy (LIBS)) (e.g. *Development of an innovative ground penetrating radar system for fast and efficient monitoring of rail track substructure conditions, SAFERAIL*),
- combined and complementary application of NDT methods to infrastructures (e.g. *Sustainable bridges*),
- development of monitoring systems (e.g. *Smart monitoring of historic structures, SMOOHS*),
- pre-cast concrete pile monitoring from manufacture to after installation, *PILE-MON*,
- development of fast and on-line data processing and reconstruction software,
- development of software for numerical simulation,
- automation of NDT methods (on-site scanner systems) (e.g. *An automobile multi-sensor robot system for non-destructive diagnoses of reinforced concrete structures, Betoscan*),
- development of quality inspection systems,
- development of guidelines and recommendations (e.g. ASTM, DGZfP, see below).

Thus, in recent years, innovative NDT methods that can be used for the assessment of existing structures have become available for concrete structures. But these are still not highly established for regular inspections. Therefore, the intention of the following chapters is to raise awareness within the civil engineering community about the applicability, performance, availability, complexity and restrictions of NDT. For an efficient application of NDT methods, a comprehensive planning of the test campaign is needed. Potential initial approaches are discussed below.

### 1.2 Strategies for the application of non-destructive testing (NDT) methods

Irrespective of the structure under investigation, its condition and age, and the stage at which testing is performed (during/after construction, during...
service or regular maintenance, after damage and deterioration, before change of use, and before and after repair), the application of NDT methods is normally most useful at the beginning of a building assessment. With the application of e.g. a covermeter, radar or active thermography, large areas can be recorded in a short time enabling a fast overview about the position of reinforcement and tendon ducts, the presence of delaminations and voids, the presence and distribution of enhanced moisture, as well as about geometrical parameters. Selected smaller areas can be investigated afterwards with enhanced accuracy with further NDT, or with minor destructive or destructive techniques. Examples of these detailed investigations, which are described in other chapters in this book, include sampling, ultrasonic investigation of the grout condition in tendon ducts, location of cracks in tendon ducts using magnetic methods. In addition, NDT methods can be applied repeatedly over longer periods (monitoring).

The application of NDT methods is possible in all areas of a building that are accessible from the surface. Conventional destructive methods cannot be replaced completely, but the number of core samples taken can be reduced considerably.

Careful planning of a measurement campaign and a precise definition of the questions and the resulting testing problem(s) are essential for it to be successful. All available information about the object under investigation (such as plans, documentation, and previous investigations) should be collected. In particular, previous investigations of small samples of material can be helpful for the calibration of NDT methods (e.g. ultrasonic velocity). Also records and notes about enhanced loading, damage owing to external reasons and environmental hazards such as earthquakes, floods and storms are very useful. Testing methods are selected according to the testing problem, as are procedures for data analysis, visualisation and interpretation of data. It is necessary to define in detail the parameters to be determined and the required accuracy for an efficient application of the methods. Beyond this, the customer should think in advance about the decisions to be taken on the basis of the measurement results, and which additional measurements and actions are required afterwards.

The realisation of the measurements involves the following steps:

- preliminary inspection by those responsible,
- contacting NDT experts (contractors),
- common site inspection, including selection of areas to be investigated and photographs,
- specification of the testing problems, including requested accuracy and resolution,
- definition of time schedule and effort with costs,
• planning and performance of investigations with one or more NDT and minor destructive methods, complementary investigations with destructive testing (depending on position and size of areas to be investigated, systems for automatic data recording as described in chapter 3 should be applied),
• analysis of experimental data,
• interpretation and documentation of experimental results, and
• common discussion, comparison of the tasks, further investigations and measures.

In all steps during the performance of measurements, a close co-operation in a multidisciplinary working team with stakeholders, civil engineers, structural engineers, and NDT personnel is required.

1.3 Overview of non-destructive testing (NDT) methods

The aim of the application of NDT methods is not only to detect and classify defects and damage, but also to quantify them. This includes the determination of position and size (dimensions) as well as collecting further information about mechanical, physical and chemical properties. These data are required to analyse the origin of damage and deterioration processes, to plan repair and maintenance measures, to assess the load-bearing capacity and to predict durability and lifetime of the construction. Most of the NDT methods can be classified into acoustic, electromagnetic, and optical methods.

1.3.1 Acoustic methods

Acoustic methods are based on the propagation of sound waves and mainly vary in frequency bandwidth of the emitted and detected signals and thus in spatial resolution. For most of the acoustic methods, the propagation time of impulses is measured. For some methods, the intensity of the signal is analysed. The acoustic methods described in this chapter include ultrasonic, impact–echo, and acoustic emission techniques.

1.3.2 Electromagnetic methods

Electromagnetic methods, which are relevant for applications in civil engineering, are based on the propagation of electromagnetic waves in non-metallic (dielectric) materials. Some of these methods, e.g. radar, are based
on the transmission and reflection of very short electromagnetic impulses, which are emitted and detected by antennas. In a similar way to ultrasonic techniques, configurations in reflection and transmission are possible. Depending on the frequency bandwidth, the propagation velocity is highly influenced by the moisture content. At frequencies below 2 GHz, the absorption strongly depends on conductivity, i.e. the content of dissolved salt ions. Electromagnetic methods include radar, covermeter, capacitometry, electrical resistivity, potential field methods, and other microwave methods for moisture detection.

1.3.3 Optical methods

Optical methods to be applied for NDT in civil engineering can be classified into those methods that are used for topographical survey, mapping, and dimensional and deformation measurements, e.g. photogrammetry, laser scanner, laser vibrometer, Speckel interferometry, stereography; those methods that record direct images of the surface, e.g. digital photography, videoscopy, thermography (passive and active); and those methods that involve a spectral analysis of the object under investigation, e.g. LIBS.

All of the methods described above are imaging techniques:

- Digital photography, videoscopy and thermography are generating a direct image, which can be processed further.
- Lasers can be scanned rapidly enabling a fast recording of two-dimensional images.
- Data of impulse–echo methods, e.g. radar and ultrasonic and impact–echo techniques are usually displayed as:
  - A-scan, where the detected signal intensity is a function of time (depth);
  - B-scan, which is a collection of several A-scans recorded along a line perpendicular to the surface of a structure; or
  - C-scan, which is a slice or projection of a three-dimensional data set.
- Data recorded along a line, e.g. moisture content inside a borehole, or data recorded with the drilling resistance method, can be displayed as a one-dimensional plot. In addition, several parallel lines can be combined into a two-dimensional image.

A good overview of testing problems and how these can be analysed with different NDT methods, is given in Volume 1, chapter 2 and in chapter 2 of this volume. A more detailed description of the individual methods is given in Part II of this volume.
1.4 Qualification/validation of methods

1.4.1 Qualification of personnel

For the application of NDT methods, personnel should be fully trained and familiar with the equipment used. A qualification procedure is given in DIN EN 473. Additionally, knowledge and experience is required about the materials and structures to be investigated. For a successful measurement campaign, close co-operation is required between the personnel and the engineers responsible for the structure under test.

1.4.2 Validation of methods

Validation describes the process for checking a testing method or system to ensure that it fulfils its purposes concerning a defined testing problem (see e.g. DIN/ISO 17025). Thus, validation is confirming that the method satisfies the needs of the users or customers. If one of the NDT methods or systems is applied as an accredited method, a validation is essential, but also without accreditation, the basis of this standard may usefully be applied to NDT methods to ensure that faulty results and wrong interpretations may be avoided.

For validation, amongst others, the testing methods should be evaluated in relation to the given tasks and requirements concerning the following topics:

- uncertainty,
- limit of detection,
- selectivity,
- linearity,
- repetition accuracy, and
- robustness against external influences or interference errors.

From a variety of influencing factors, the most significant ones should be detected. If necessary, the influences should be determined experimentally by single tests. In several instances, material and structural properties are not known in advance or are very inhomogeneous, so that not all influences could be considered in advance. In either case, customer and expert should discuss possible influencing factors together in advance. An agreement should be made concerning the topics that are relevant for the planned campaign.

A further verification can be performed by comparing the experimental results with numerical or analytical modelled data. A combination of complementary methods or the applications of a reference method can increase the reliability of results.
### Table 1.1 International, European and national standards for non-destructive testing of concrete

<table>
<thead>
<tr>
<th>Organisation</th>
<th>Number</th>
<th>Year</th>
<th>Title</th>
</tr>
</thead>
<tbody>
<tr>
<td>CEN</td>
<td>EN 473</td>
<td>2008</td>
<td>Non-destructive testing – qualification and certification of NDT personnel – general principles</td>
</tr>
<tr>
<td></td>
<td>EN 12504-4</td>
<td>2004</td>
<td>Testing concrete. Determination of ultrasonic pulse velocity</td>
</tr>
<tr>
<td>ASTM</td>
<td>C1153-97</td>
<td>2003</td>
<td>Standard practice for location of wet insulation in roofing systems using infrared imaging</td>
</tr>
<tr>
<td></td>
<td>C1383-04</td>
<td></td>
<td>Standard test method for measuring the P-wave speed and the thickness of concrete plates using the impact–echo method</td>
</tr>
<tr>
<td></td>
<td>C597-02</td>
<td></td>
<td>Standard test method for pulse velocity through concrete</td>
</tr>
<tr>
<td></td>
<td>C876-09</td>
<td></td>
<td>Standard test method for half-cell potentials of uncoated reinforcing steel in concrete</td>
</tr>
<tr>
<td></td>
<td>D2950-09</td>
<td></td>
<td>Standard test method for density of bituminous concrete in place by nuclear methods</td>
</tr>
<tr>
<td></td>
<td>D4695-03</td>
<td>2008</td>
<td>Standard guide for general pavement deflection measurements</td>
</tr>
<tr>
<td></td>
<td>D4788-03</td>
<td></td>
<td>Standard test method for detecting delaminations in bridge decks using infrared thermography</td>
</tr>
<tr>
<td></td>
<td>D6087-08</td>
<td></td>
<td>Standard test method for evaluating asphalt-covered concrete bridge decks using ground penetrating radar</td>
</tr>
<tr>
<td></td>
<td>D6432-99</td>
<td>2005</td>
<td>Standard guide for using the surface ground penetrating radar method for subsurface investigation</td>
</tr>
<tr>
<td></td>
<td>D6639-01</td>
<td>2008</td>
<td>Standard guide for using the frequency domain electromagnetic method for subsurface investigations</td>
</tr>
<tr>
<td></td>
<td>E2128-01a</td>
<td></td>
<td>Standard guide for evaluating water leakage of building walls</td>
</tr>
<tr>
<td></td>
<td>E2270-05</td>
<td></td>
<td>Standard practice for periodic inspection of building facades for unsafe conditions</td>
</tr>
<tr>
<td></td>
<td>E494-05</td>
<td></td>
<td>Standard practice for measuring ultrasonic velocity in materials</td>
</tr>
<tr>
<td></td>
<td>E748-02</td>
<td>2008</td>
<td>Standard practices for thermal neutron radiography of materials</td>
</tr>
</tbody>
</table>
1.5 Sources of further information and advice

General publications about NDT methods in civil engineering can be found in references 9, 10, and 11. There are also two special journal issues about NDT testing of concrete structures. 12, 13

The proceedings of the following conferences give more details about recent applications of NDT in civil engineering:

- International symposium on non-destructive testing in civil engineering, Proceedings of 2009, Laboratoire Central des Ponts et Chaussees, Nantes, France.

There are only a few standards, guidelines and recommendations available concerning NDT of concrete. In Table 1.1, a selection of some international (ISO), European (CEN) and national standards (American Society for Testing and Materials (ASTM), British Standards Institution) is given.

Several guidelines and recommendations have been developed by the International Union of Laboratories and Experts in Construction Materials,
### Table 1.2 Guidelines and recommendations

<table>
<thead>
<tr>
<th>Organisation</th>
<th>Number</th>
<th>Year</th>
<th>Title</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Materials and Structures, 38(284), 907–911</td>
<td>2005</td>
<td>Recommendation of RILEM TC 189-NEC ‘Non-destructive evaluation of the concrete cover’: comparative test – Part II: Comparative test of ‘Covermeters’</td>
</tr>
<tr>
<td></td>
<td>Materials and Structures, 26(155), 43–49</td>
<td>1993</td>
<td>RILEM TC 43-CND Draft recommendation for in situ concrete strength determination by combined non-destructive methods</td>
</tr>
<tr>
<td>DGZfP</td>
<td>B2</td>
<td>1990</td>
<td>Guideline for the detection of reinforcement and covermeter for reinforced and prestressed concrete</td>
</tr>
<tr>
<td></td>
<td>B5</td>
<td>1993</td>
<td>Guideline about thermographic investigation of structures</td>
</tr>
<tr>
<td></td>
<td>B6</td>
<td>1995</td>
<td>Guideline about visual inspections and endoscopy as optical methods for non-destructive testing in civil engineering</td>
</tr>
</tbody>
</table>
Systems and Structures (RILEM) and the German Society for Non-Destructive Testing (DGZfP). The latter are currently only available in German. These documents are listed in Table 1.2. New guidelines are under development in the RILEM TC 207-INR Interpretation of NDT results and assessment of RC structures which is soon to be published.

1.6 References

5. ‘Pre-cast concrete pile monitoring from manufacture to after installation (PILE-MON)’, EC Project funded in the FP7.


Non-destructive testing methods for building diagnosis – state of the art and future trends

C. FLOHRER, HOCHTIEF Construction AG, Germany

Abstract: Non-destructive testing methods are indispensable in building analysis and are urgently required for the supervision and control of expenditure for preservation of buildings. Procedures and modern testing methods are described for building diagnosis and examples of applications are presented. The differences between manual large-scale applications and specific applications of high-quality technology such as radar and impact–echo are outlined.

Key words: non-destructive testing, concrete, reinforcement, radar, impact–echo.

2.1 Introduction

Building diagnosis, as a component of a strategy for long-term maintenance of value or for the assessment of the stability and durability of a building, only succeeds reliably, if the load-bearing parts can be evaluated. In large-scale building preservation measures or the reuse of buildings, the expenditure necessary is often underestimated for an area-wide stocktaking. Non-destructive testing (NDT) methods for building diagnosis are available. The expenditure necessary for application of the procedures and the efficiency in carrying out the task required are very different.

2.2 Tasks for building diagnosis

In construction, there are various applications for building diagnosis by NDT from regular building check-ups to the application of testing methods while repair work is in progress. One reason for the non-regular use may be the small number of standardized NDT methods in the civil engineering (NDT-CE) area. Apart from Schmidt’s hammer and the cover meter, few practically useable testing methods for civil engineering are available to planners and operators. Nevertheless, guidelines published by Deutsche Gesellschaft für Zerstörungsfreie Prüfung (DGZfP), the German society for non-destructive testing, exist for many other procedures and users of
NDT are well experienced with the application. In these standard methods, application possibilities and limits are given, basics are described, and case studies contribute to improved safety of application.

The parameters for assessment of load-carrying constructions within the scope of a building diagnosis that have to be investigated by non-destructive testing methods can be varied. In Table 2.1, typical tasks are summarized. Test results can be demanded locally for a single component or parts of it. Nevertheless, often information is expected on all components of the same type, so that a large-scale or widespread check becomes necessary. According to the expected test result, the available testing methods have to be used in different configurations. For instance, for information about the durability of a bridge with integrated displacement bodies, in which no water must stand, an entire check of all displacement bodies is necessary. This is not required if any possible water intrusion into the displacement bodies can be recognized by external signs. In contrast, for investigating the condition of compaction of tendons, it might be sufficient to examine these further only at known weak points. For such a low-scale test, the investigation intensity can be raised and the application of the testing methods and the test expenditure can be enhanced. The decision on which testing methods to use, has to be based on matching the cost of the test to the task and this has to be made by professional engineers for building diagnosis and for the application of NDT methods.

2.3 Efficient testing methods

Electromagnetic, acoustic or magnetic methods have improved over the past few years to the extent that difficult testing problems are now solvable in steel and pre-stressed concrete buildings or in other components and other materials. In Table 2.1 the available testing methods (single methods or combinations) are described alongside the problem addressed and the aim of the test. The advantages of the combination of the procedures are particularly found in the application of scanning systems. The common coordinates of the position allow a direct overlaying of the signals. Currently, the application of scanning procedures is limited to small-scale investigations to achieve economic efficiency because of the high measuring point density. Concrete cover meters have been improving from approximately 1990 so that an evaluation of the measuring signals recorded for the track is now possible. In later developments, the signals were no longer recorded along lines. A measuring grid was captured with multiple transducers. Then, the data were evaluated by displaying the reinforcement grid in an image. Nevertheless, the information content was not improved, because the resolution of these inductive measuring methods is limited by the depth and the distance between rebars. With these methods,
<table>
<thead>
<tr>
<th>Test problem</th>
<th>Test method</th>
<th>Aim of investigation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete compressive strength</td>
<td>Bouncing hammer (Schmidt hammer) and destructive testing of drilling cores</td>
<td>Categorization of supplied concrete into classes of compressive strength</td>
</tr>
<tr>
<td>Surface tensile strength</td>
<td>Test of tensile strength</td>
<td>Application of composite layers on old concrete surfaces</td>
</tr>
<tr>
<td>Concrete cover, determination of diameter of reinforcement</td>
<td>Cover meter, radar (deep reinforcement)</td>
<td>Assessment of the durability and the load-carrying capacity</td>
</tr>
<tr>
<td>Position and alignment of reinforcement</td>
<td>Cover meter, radar, radiography</td>
<td>Assessment of the durability and the load-carrying capacity</td>
</tr>
<tr>
<td>Detection of defects inside concrete, structural modifications</td>
<td>Radar, ultrasonic echo, impact–echo</td>
<td>Assessment of homogeneity of massive elements</td>
</tr>
<tr>
<td>Determination of the thickness of the structure, depth of installation parts or defects</td>
<td>Impact–echo, ultrasonic echo, radar</td>
<td>Unilaterally accessible structural elements, displacement bodies inside concrete, steel installation parts. Detection of insulating layers or dividing layers, multilayer components</td>
</tr>
<tr>
<td>Layer composition of wall and floor</td>
<td>Radar and further minor destructive testing (e.g. endoscopy)</td>
<td>For the large-scale stock-taking within the building diagnosis</td>
</tr>
<tr>
<td>Moisture content of the elements</td>
<td>Microwaves, radar, capacitive methods</td>
<td>Determination of the moisture content of elements and building materials</td>
</tr>
<tr>
<td>Location of tendons (lateral position, depth position)</td>
<td>radar</td>
<td>Reliable detection of tendons as a prestudy in advance of other investigations of the tendons or for repair work</td>
</tr>
<tr>
<td>Compaction faults inside tendons of post-tensioning</td>
<td>Ultrasonic echo</td>
<td>Contribution to the stability analysis of a prestressed concrete structure</td>
</tr>
<tr>
<td>Active corrosion of reinforcement</td>
<td>Potential difference method</td>
<td>Assessment of the durability and the stability</td>
</tr>
<tr>
<td>Cracks of tension wire cracks</td>
<td>Magnetic field method</td>
<td>Investigation of pre-stressed concrete elements with regard to possible cracks of tension wires</td>
</tr>
<tr>
<td>Glued laminated timber beams</td>
<td>Ultrasonic echo</td>
<td>Investigation of glulam beams with regard to structure or delaminations</td>
</tr>
</tbody>
</table>
usually only the cross-wise reinforcement grid close to the surface can be detected.

For the location of deep reinforcement, the radar method which has been used in Germany for approximately 15 years is required. In particular, for detecting objects in materials with higher permittivity surrounded by a medium with a lower permittivity, radar has advantages over methods based on the propagation of mechanical waves.\textsuperscript{3,4} By a combination of radar and the cover meter, several layers of reinforcement can be visualized and the position of tendons below the reinforcement can be differentiated.\textsuperscript{5} Both methods are still used today for these tasks. In most cases, it is necessary to locate reinforcement inside concrete on a large scale. Therefore, rail-controlled systems are too slow. Nevertheless, rail-controlled systems were used approximately 10 years ago for small subranges to raise the density of information. At that time, even the fast superposition of the measuring signals of different testing methods was not possible. Now, the radar method can be classified as a standard testing method for the detection of tendons in pre-stressed concrete. Owing to many disturbing signals, generated by the reinforcement bars, the interpretation of the measuring data still requires highly experienced operators. For the application of radar a guideline of the DGZfP is available in Germany.\textsuperscript{6} For standard applications, the manual measurement still has the advantage that the realization of the measurements can be better adapted to the surroundings terms. Furthermore, the measuring lines can be varied more easily. The closer the tendons lie (e.g. in bridge superstructures), the more it becomes necessary to scan local areas with a high density of measuring lines. Only then can the tendons be detected from the recorded data.

Impact–echo is to be classified as a standard procedure for the determination of the thickness of components made of reinforced concrete. The capture of flatly extended voids or internal layers with acoustically soft impedance is impossible with high reliability. A minimum distance to the test surface has to be considered. For the application of impact–echo, a guideline of the DGZfP is also available in Germany.\textsuperscript{7}

With ultrasonic echo, similar tasks can be solved with the same reliability as with impact–echo. Because of the arrangement of multiple transducers in arrays, ultrasonic echo has advantages in the detection of smaller, not flatly extended voids. With both methods no information is gained about the thickness of the void or the internal layer. The magnetic field method was improved to such an extent that tendons can be investigated with respect to cracks of the prestressing steel along rail-controlled measuring systems. In addition, the first applications of scanning measurements along large areas for examination of tendons close to surfaces have been reported for several bridges.\textsuperscript{8}
2.4 Examples of the application of the testing methods

2.4.1 Location of the reinforcement and measuring of the concrete cover with radar

In the course of the construction of a city railroad line, a tunnel was provided using a bridge cap construction (Fig. 2.1). The frame building with about 1450 m of total length consists of two overlapped constructed post walls and a block-for-block connected reinforced-concrete tunnel soffit.

On exposing the pile heads as well as after the earth excavation, i.e. on exposing the internal pile wall surfaces on one of the reinforced-concrete troughs to be inserted along a length of about 6 m (area 1 in Fig. 2.1), an eccentricity of the reinforcement cage became evident (Fig. 2.2). This eccentricity was evaluated as a divergence from the contract. According to the size and frequency of the eccentricity, it was calculated on a systematically lower concrete cover and a high amount of undercut of the demanded minimum value of the concrete cover of 5 cm.

The proof of the applicability of the radar method was made at first within the scope of test measurements. A SIR GSSI 20 system was used with a 1.5 GHz antenna. Because of the rough surface of the drilling pile only one measurement could occur in the time mode which, nevertheless, enables economic measurement of the concrete cover. Along vertical measuring lines (Figs 2.3 and 2.4) the depth position of the helical reinforcement was determined and compared by destructive measurements in the pile. The concrete cover could be proved with an accuracy of ±0.5 cm.

![Schematic of tunnel cross-section](image-url)
2.2 Exposed pile head, eccentric reinforcement.

2.3 Three vertical measuring lines (ML) per position.

Thus, from the interior of the tunnel, the concrete cover was measured without destruction at about 19000 measuring positions (Fig. 2.5). In drilling cores, the results recorded with radar were controlled randomly and the measurement tolerance was within ±0.5 cm.

In Fig. 2.6, the statistical distribution of 11533 measured concrete covers is shown. Therefore, the designed concrete cover of 10 cm with an average size of 13.4 cm was exceeded and only 1.05% of the measured concrete covers were smaller than (50 ± 5) mm. The 5% quantile also counts in the proof of the minimum concrete cover (DBV-Guideline Concrete Cover in 2002). Thus, the required concrete cover of 50 mm was proved for the
2.4 Radargrams of three different measuring lines.

2.5 Exposed pile wall area at the tunnel inside, prepared for the concrete cover measurement (up to four heights with three measuring positions each, with about 50 cm of measuring distance).
measured areas. Taking into account the extensive concrete cover measurements, the proof of a sufficient load-carrying capacity and use suitability of the frame load-bearing structure could be provided for the demanded laid life span of 100 years.

2.4.2 Other testing problems with radar

For the detection of metal-covered hollow bodies (e.g. displacement pipe in bridge slabs), the use of radar is suitable in a manual application. For instance, displacement bodies are located in the highway slabs of flat bridges with radar and afterwards they are checked by endoscopic investigations. For that purpose, the whole bridge soffit which leads, e.g. above a railroad, has to be examined in only a few hours while train traffic is blocked. If investigation platforms or cherry pickers are used for testing, the required flexibility excludes the application of rail-controlled systems or, in particular, of scanning systems. Because of the relatively low information level needed (lateral position, depth position or sloping position of the displacement pipes), no scanning procedure is required or economically beneficial (Fig. 2.7). Measurements of the same type are carried out, for instance, at displacement bodies produced by lost wooden formwork. The reflection of the radar wave at the interface between concrete and air gap (with low density and high wave speed) is relatively weak and more difficult to interpret than at the interface concrete/metal. If a statement about possible water filling of the hollow cavities is demanded, the radar method offers advantages because the difference is significant between the reflexion at the dry cavity and the reflection at the water-full hollow cavity.
For the large-area recognition and assessment of the similarity of the weight-bearing structures or superstructures (ceilings or walls) on top of further weight-bearing structures within stocktaking or building diagnosis the radar procedure can also be used. With radar, the resolution of the slab load-carrying system also works in underground superstructures, because the electromagnetic radar waves penetrate building material that only has one-sided accessibility. Thus, information about underlying layers is obtained. The radar method is also applicable for masonry investigations, because it can penetrate even thick masonry layers and visualize single layers. Furthermore, it can be used without direct coupling onto the surface of the structure. This is particularly important in the conservation of monuments and historic buildings (Fig. 2.8).
2.4.3 Determination of the wall thickness of a double-leaf wall with impact–echo

In a newly built cinema with connected multistorey car park, the partition between cinema and car park is a double-leaf wall with inside recumbent sound absorption. To accelerate the construction, the double-leaf wall was built in one operation and, as a result, the sound absorption was insufficiently fixed. During concreting, the sound absorption within the wall cross-section moved, so that locally, the planned wall thicknesses were considerably distorted (Fig. 2.9).

The purpose of the impact–echo measurements was to determine the wall thickness and the position of the insulation along the surface of the entire wall without destruction. Thus, the stability of the wall was checked statically.

The measurements were carried out at individual measuring points along a selected measuring grid of 50 cm (Fig. 2.10). The thickness of the wall was presented in a height profile (Fig. 2.11). As a result of the measurement and the static postcalculation, the stability of the wall could not be proved. Thus, the double-leaf wall had to be taken back. The condition of the wall (and the course of insulation) could be understood after demolition of the upper wall layer (Fig. 2.12).

2.9 Wall cross section of the double-leaf wall with distortion after concreting.
2.10 Measurement of the wall thickness with impact–echo.

2.11 Thickness of the wall (in cm; see Fig. 2.9) as a height profile checked with impact–echo.
2.4.4 Applications of the magnetic field method to the detection of cracks in pre-stressed steel rods

Pre-stressed concrete in the direct bond

Using the remanent-magnetic field method, cracks can be detected in pre-stressed steel rods, pre-stressed tension wires and pre-stressed steel rods inside tendon ducts (post-tensioning). The vehicle in which the magnetization unit and the probes for measuring the magnetic field are mounted is moved along rail systems (Fig. 2.13). An aim of the related investigation was the detection of expected cracks in pre-stressed steel rods in roof binders made of pre-stressed concrete of an industrial hall. The rods are
arranged in the lower flange of the binder as a package of eight rods in three layers lying one after the other. For testing bent rods, the rail systems must be adapted to the course. The rail system is tightened against the soffit of the binder, so that the magnetization of the straight arranged pre-stressed steel rod occurs while the vehicle crosses the concrete surface (along the rail system). The magnetization may be necessary several times before the remanent magnetic field is induced on the steel rod. Then, for measuring the remanent magnetic field the vehicle is moved again along the surface. For the pre-stressed concrete binders presented here, no cracks were located.

*Post-tensioned concrete*

The investigation of transverse tendons in highway bridge slabs was previously carried out using the same procedure of magnetization of single tendons and measurement of the magnetic field. The magnetization and measuring process occurred either through hand-controlled crossing of the tendons with the measuring instruments (Fig. 2.14) or again through rail-controlled crossing. Nevertheless, the investigation of a whole bridge is barely possible because of the high cost of time.

Therefore, a scanning measuring system which, in one measuring journey, can examine a whole lane for its full length was developed by the Technical University of Berlin. With this device, first the whole steel is magnetized by crossing in the area close to surfaces using a yoke magnet. Then the whole magnetic field is taken up in the area of the lane with a rotation scanner and is evaluated subsequently. The measurements can also be carried out on top of the asphalt surface, if the layer is only a few centimeters thick and if the underlying sealing contains no metal inserts.

2.14 Investigation of single transverse tendons with the magnetic field method by manual movement of the measuring instruments.
During the repair of an approximately 900 m-long pre-stressed concrete bridge, it was discovered that pre-stressed steel rods were strongly corroded and cracked in areas that had been opened randomly. The defects were all distributed in the area of the transition between highway and bridge cap, because the bridge shows a cross fall. Therefore, water accumulated in this area. The magnetic field method was used with the scanner and the treated area was examined with two crossings. The trace recorded with the scanner had a length of approximately 1700 m. Figure 2.15 shows the application of the scanner on the valley bridge. In the front part of the measuring instrument the yoke magnet with two big coils is inserted, while the rear carriage includes the rotation scanner for data recording. The captured data are evaluated and the result is shown as an image. Figure 2.16 shows the cracks of the tendons as an image represented as a plan view of the highway slab.

2.15 Application of the magnetic field scanner to a large valley bridge during the repair of the bridge.

2.16 Imaging of the cracks of two tendons in the plan view of the highway slab.
2.5 Future trends

The above-mentioned methods were used in the past as manual operations to solve the measuring tasks individually and to adapt the method specifically to each building and each structural element. Inevitably, the measuring point density is lower than with rail-controlled or scanning measuring systems. Nevertheless, the advantage of the manual application of the procedures is that the application of the testing methods is economical and thus acceptable to the customer. Provided that such methods or the further developed scanning procedures are within the scope of research and development activity, the customers will be interested. If the full cost of the measuring service has to be met, manual testing methods will find high acceptance. For the improvement of the efficiency of the testing methods, scanning systems that allow a high measuring point density with data recording are also used. An essential increase in resolution is expected from automated measuring systems that can grasp horizontal and vertical surfaces extensively and rapidly and can employ several sensors at the same time. Prototypes of automated, self-running scanning systems were developed and used as experimentally buildings for the localization compaction faults inside tendon ducts (Fig. 2.17). Using the synthetic aperture focusing technique (SAFT) algorithm, measuring data can be analyzed and reconstructed. Thus, the results can be presented as user-friendly 3D reconstructions.9,10

In addition to the 2D recording of measuring point with high point density, a superposition of the measuring data (data fusion) of different methods is possible by using scanning systems. This is an essential improvement for the localization of defects in tendon ducts with post-tensioning. Currently, for the detection of cracks in transverse tendons of bridges, a
yoke magnet can be used for impulse magnetization and the generated magnetic field is gathered by use of a rotation scanner. For some years already the development of self-running robots that can be equipped with different sensors has been directed towards investigating bridges and buildings. Currently, there is a requirement for the complete investigation of large multi-storey car parks. Large-scale investigations can also become necessary for structural elements not relevant for stability, e.g. if the use of office rooms is derogated. For several years, large areas of gypsum plaster have been detaching from the concrete base and there is the risk of the plaster falling. For early detection of delamination, or to be able to monitor changes in the size of the delaminations is a potential application of this testing.

2.6 References

5 Pöpel, M. and Flohrer, C.: Combination of a covermeter with a radar system – an improvement of radar application in civil engineering, International symposium in non-destructive testing in civil engineering, Berlin 1995, DGZfP
Development of automated non-destructive evaluation (NDE) systems for reinforced concrete structures and other applications

G. Dobmann and J. H. Kurz, Fraunhofer-IZFP, Germany; A. Taaffe, BAM Federal Institute for Materials Research and Testing, Germany; D. Streicher, Joint Lab of Fraunhofer & BAM, Germany

Abstract: The introduction of an innovative process for non-destructive testing is described in terms of its different phases and how the process is controlled in the Fraunhofer IZFP in order to obtain optimal results. Examples are discussed of applications by IZFP in various important and safety-relevant industrial sectors where the automated inspection systems were introduced. These case studies include the inspection of railway components such as the wheel sets of the German high-speed train and in-line pipe inspection by using intelligent pipeline inspection gauges. Complex inspection systems, both hardware and software, developed in partnership with the Federal Institute for Materials Research and Inspection (BAM), were applied to inspection tasks in civil engineering. Two automated systems based on a robot and on a flexible manipulation are presented. Both systems can be applied to diverse inspection technologies and to the data fusion of various types of non-destructive testing (NDT) data.

Key words: automated inspection systems, non-destructive testing, civil engineering, robots, data fusion, railway components.

3.1 Introduction

Researchers, especially those who come from research-driven institutions, often resemble the bee, flying from one flower to the next to find fresh nectar to sip. Under the constant demand to explore new topics they often neglect the most significant step in the innovation cycle (innovation: the successful exploitation of new ideas) that is the implementation of development results into a marketable product.

The Fraunhofer-Gesellschaft (FhG) as the main institution of applied contract research in Germany presents other demands on their institutes. The implementation of development results in products, mentioned above, has a significant role. Obviously, this last innovation phase, according to
European legislation, is no longer or only minimally supported by public-sector-funded research, and therefore requires more industrial participation. Consequently, the success of an institute in the Fraunhofer Society is measured by an indicator called $p$, (in percent), which is the ratio of the industrial income of an institute to the total income, and also by a measurement of satisfaction of industrial customers, resulting from the frequency of contracts by the same customer in a 5-year-long time period.

This chapter presents a discussion of the advantages of automation as a driver to new innovations in non-destructive testing (NDT) and tells a few Fraunhofer success stories in product innovations and NDT engineering in the industrial sectors of railway and pipeline safety where automated systems are a must to reliably inspect infrastructure. Then, the application of automation to a robot (BetoScan) and to a scanning (OSSCAR) NDT device for automated data-acquisition in the construction industry are discussed. These are based on multiple techniques combined with options for data fusion. The developments are products of a fruitful co-operation with the Federal Institute for Materials Research and Testing (BAM).

3.2 The innovation cycles

According to P. Höller, a former director of IZFP, a definition of applied research and development (R&D) for NDT is based on an innovation cycle composed of four phases:

1. identifying the results of basic research and self-performing of application-oriented basic research,
2. development,
3. application technology, and
4. implementation.

Höller regarded the first and second phase as a task to be sponsored to 100% by research funding of the public sector, therefore support from the German Research Foundation (DFG), the ministries of science and culture in the federal states, the program and departmental research of the federal ministries and the research programs of the European Union.

Up to the mid 1980s, in public R&D programs in Germany 100% funding was a rule, however this was then changed, and not only by European legislation and harmonization. Basic industrial research was then limited by a promotional rate not higher than 50% and R&D projects were merely required to produce a technological demonstration and not an industrial prototype. Since about 1982, national rules have asked for public/private partnership (industrial collaborative research). However, after R&D services of the industry were no longer tax-favored, Fraunhofer institutes in
partnership with industry were asked for self-financing of the remaining 50% funding requirements.

Research funding from the German Research Foundation (DFG) is oriented towards university researchers, whose overhead costs are small compared with those of research institutions such as Fraunhofer. Therefore, DFG funding of Fraunhofer institutes only was, and still is, covering the costs to about 45%. Currently, Fraunhofer institutes may only apply for DFG funding in combination with university chairs. Therefore, Fraunhofer needs to co-operate with R&D institutions such as universities that have greater access to basic funding, and also with institutions like BAM.

The contract research model of the Fraunhofer Society as a total is based on institutional funding from the Ministry of Education and Research (basic funding), industrial income, and income of public projects at a ratio of 2:2:1. The individual basic funding is given to the institutes according to an incentive model, dependent on success in business, i.e. dependent on the industrial success.

The institutes use their basic funding in three ways:

- One portion is used to subsidize the underfunded projects where the full-cost calculation is not covered.
- The second part is for the application-oriented, in-house basic research, which involves work on strategically important issues for the future. The benefit of that procedure is that the intellectual property rights stay with Fraunhofer.
- The final proportion of basic funding is used for risk management. When industrially oriented research is performed, higher risks are taken into account. In some instances, time and cost overruns require rework and the possibility of financial compensation.

Höller asked for a R&D policy where the third phase of innovation should also be strongly supported by the public sector, especially when it comes to the use of NDT in industry with safety-related components, because of the public interest. For R&D in NDT for nuclear safety this is still the funding procedure in Germany.

Under application technology, Höller understood the phase of the further development of a demonstrator into an industrial prototype, validated under real industrially oriented environmental conditions. However, here the commitment of the industry in collaborative partnership with R&D institutes is a strong requirement.

The meaning of the last phase, application, is the implementation of the prototype, which has to be tested and validated. This brings the user industry, if the product is convincing, to a 100% funding.

In this last phase, the financial risks, of course, are high because the industrial customers, with good reason, require the specifications to be met
and the adherence to a planned project schedule and cost plan. Often there is even a legal contract penalty in the case of exceeding agreed deadlines.

In general, it can not be assumed that the final phase will not fail, or that the result of the developments will not fulfil the expectations of the beginning or will not justify the invested capital of preliminary research by a return of investment. However, in that event, economically speaking, the investment into the jobs in R&D over the period of time of the development still remains.

Real satisfaction, however, is only gained if the implementation succeeds and the product or service is brought to a market success. In NDT, we do not require a large number of systems, compared with medical technology for instance; it is only a niche market. The automotive industry has introduced the concept of platforms when different models of cars over the years with changing customer tastes and exterior design are adapted to the same constructive characteristics of the chassis, the drive line and support of supply modules. The reason is cost savings. The more the developer is active in the niche market of NDT, modular systems as platforms are a must, in hardware as well as in software, in order to document a return of invest even in the case of small quantities.

Finally, it should be noted that the small- and medium-sized companies engaged in both the assembling process of the systems and the life-long system maintenance are required to be integrated into the innovation process. Skilled jobs are developed and 70–80% of the total costs are spent in integrated supplying SMEs.

Fraunhofer IZFP measures its success not only on the basis of economic returns, but also by asking for customer satisfaction. Satisfied customers are defined as those who ask for more than five following orders, and IZFP is proud to have in the overall industrial revenues per year on average 75% of satisfied customers.

The transition from a primarily research-oriented institute with scientific excellence into a profit center, successfully searching for industrial customers and with $p_t$-values of 55–60%, became possible owing to the strategic market-oriented direction.2

The scientific development of NDT of materials has its basis in the interdisciplinary integration of a variety of different and complementary scientific and engineering methods. Besides the physics primarily material science is asked for. The development of test systems require additional handling technology and robotics, electronic hardware, computer science and software as well as mathematical algorithms in the numerical simulation.

Depending on the current state of research and development in the sub-disciplines, either the one or the other takes over the leadership in system engineering. In recent years, the primary driver for the NDT innovations was physics. A significant step forward has been achieved by introducing
new types of sensor principles, for example in digital industrial radiology and X-ray computer tomography, in low-frequency electromagnetic testing and in thermography. New trends in development are the integration of system functions in miniaturized digital circuits or by completely processing the inspection data on the software level, resulting in significant power savings and higher system reliability. More NDT applications in real time are now possible. Within this context, automation is an important driver to integrate NDT into quality management and lifecycle management procedures for infrastructure. Increasingly the principles of combined methods and data fusion for enhanced interpretation have become popular. This is discussed in the next section.

3.3 Data acquisition, control and evaluation in automated multisensor systems

Data acquisition in multisensor systems has become an important approach for various research and application areas, where mainly non-invasive and non-destructive investigations are required. A simple and well known example in the field of medical sciences is that in medical diagnoses visual inspection is often combined with radiographic (X-rays) and ultrasonic investigations. However, this is a manual approach related to the expert system ‘medical doctor’ and performed stepwise. For technical applications, an automated multisensor approach is used where several sensors perform the measurements simultaneously. Examples can be found in geosciences especially in geophysics and in a variety of fields of engineering sciences using NDT methods. Figure 3.1 shows the principal parts of a multisensor approach that can be subdivided into three components: the combined sensors, the data acquisition, including reliable data control and data analysis, and the data-evaluation.

The scheme of tasks where multisensor approaches are required is always similar. Generally, multiple-parameter-dependent or even ambiguous problems have to be solved and this is usually not possible by using only one measurement method delivering one parameter. Therefore, various methods have to be applied and the combination of the results leads to a unique assessment. A technically combined approach reduces the inspection risk because the reliability of the inspection task increases and therefore the costs are reduced. Niese et al., for example, showed that for in-line pipeline inspection of gas pipes a combination of an electromagnetic acoustic transducer with the eddy current technique and magnetic flux leakage is able not only to measure the wall thickness of a component by evaluating ultrasonic testing (UT) data but also to determine the spatial location of detected metal loss in the wall, i.e. id or od position of corrosive wall thinning. UT
alone as an individual method in this instance is not able to deliver an unambiguous inspection result.

The data analysis of multisensor measurement systems offers several possibilities depending on the type and quality of collected data. The simplest but often most effective approach is a direct comparison of these results, in particular if the results are stored in inspection images.

An expert system, e.g. in the form of an inspector’s brain, is required to correlate the different analysis results and perform the assessment. Furthermore, if clear assessment structures are given, an algorithmic trained evaluation system can also be used for analysis. The next ‘higher level’ of data analysis is data fusion. Data fusion is generally defined as the use of techniques that combine data from multiple sources and gather that information in order to achieve inferences, which will be more efficient and potentially more accurate than if they were achieved by means of a single source. A variety of applications in the field of NDT already exists. Joint inversion or multi-objective optimization means the simultaneous minimization (or maximization) of several objective (target) functions, for instance by use of generic or evolutionary algorithms. Applied to NDT data, this means finding a model that explains several data sets at once. Various approaches especially in geophysics are already well known and applied. These can be divided into joint inversion approaches of data measuring the same physical property (e.g. travel time), and those approaches where the different physical properties of different methods are used. Then
it is essential to establish the relationships between the various physical properties.\textsuperscript{14}

Without discussing the details here, the use of a multisensor approach requires a highly flexible data acquisition system. It has to be guaranteed that any combination of the implemented sensors can be chosen. Furthermore, it should be possible that supplementary sensors can easily be implemented in the modular data acquisition system. In most instances this depends on an open protocol of the used digital interface (RS232, USB, Ethernet) provided by the manufacturer of the measurement device. Because today no national or international standards exist to define the documentation format for data exchange in automated NDT, this task is not particularly easy to solve; it requires the opening of the interfaces of the individual hardware of various industrial manufacturers who will not a priori support the same procedure.

However, for the applications described in the following, a unified format for all sensors was defined. The length of the data is also specified. The package number guarantees the correct composition and recognition of data if it is transmitted in several files.

Owing to the large inspection areas the amount of data collected in civil engineering applications can also be very large. However, the scanning velocities in these applications are usually not high compared with those in other fields of NDT. This leads to moderate data transfer rates.

The data is organized in a database. This simplifies the composition of several measurement areas and the analysis of recurrent inspections.

### 3.4 Case studies of successful innovations to automated systems in non-destructive testing (NDT) engineering

#### 3.4.1 Railway safety

\textit{AUROPA}

The activities of IZFP in the wheel-rail business had an initial and early scientific and technical highlight with the award to H.-J. Salzburger of the Dr. Hermann and Ellen Klaproth Foundation Award. He received the 1980 ‘Award for Innovation’ for the development of an in-motion-inspection NDT rig called AUROPA for the ICE high-speed train (Fig. 3.2).

It was shown that, for the detection and classification of surface-breaking cracks in the tread of the wheel rim surface, an ultrasonic surface wave (Rayleigh wave) can be used.\textsuperscript{15} Both reflections (pulse echo) as well as through-transmitted signals were evaluated. One advantage is that the wave runs along the circumference of the wheel more than once and that the
attenuation of the through-transmitted signals is correlated with the depth of surface-breaking crack-like indications (a 45 mm (length) × 3 mm (depth) saw-cut is applied for the sensitivity setting). The advantage for the application is the use of electromagnetically generated and received ultrasound (EMUS), which does not require coupling.

German Rail then asked IZFP to develop a demonstrator tested in the area of the local main station, by rolling with diesel locomotives along the transducers built into the rails. It quickly became evident that the design of the transducer housing had to be optimized with respect to wear resistance. German Rail brought in its own experts to optimize the transducer housing concept (Fig. 3.3). In the next phase of innovation, a prototype of the system was delivered by IZFP for validation by German Rail. It was integrated in the main station of the new built track for high-speed traffic (Fig. 3.4). The so-called Inter City (IC) train Experimental which was a prototype of the ICE1 generation, was tested from 1985 on this first high-speed line every night the wheels of the train were inspected (Fig. 3.5). Results of both the material deterioration by plastic deformation of the tread, as well as the reliability of the test procedure were collected before the ICE1 was taken into service. As a result, IZFP in partnership with the company Hegenscheid, translated the systems into practical applications, where the company combined the surface inspection system with two other modules (geometric flange monitoring, surface waviness measurement) which were marketed worldwide. IZFP as an extended workbench of Hegenscheid was a worldwide supplier and to date 30 modules are in application. They are used in Germany, Switzerland, France, Russia, USA, South Korea and China (China has at least 13 units, but not in combination with Hegenscheid modules).16
Depending on the complexity, the module price is around €140000–160000. Figure 3.2 shows the probe with its holder in top view and Fig. 3.3 shows the side view. In Fig. 3.4, the integration of two probes into the inspection rail can be seen and in Fig. 3.5 a high-speed train is seen shortly before the in-motion inspection in a workshop.

Automated ultrasonic rail wheel systems, AURA

German Rail (DB AG) in 1998 in Nuremberg, Germany, introduced the first system AURA, delivered together with partners, Fraunhofer TEG, Arxes, IMF and SOBATEC\textsuperscript{17,18} for examination of the high-speed train ICE
wheel sets. It was a 48-channel computer-integrated ultrasound device (called PCUS 40), equipped with piezoelectric probes to detect various oriented assumed faults (specification according to German Rail) in the rim and the wheel axles. In 1999, installations in Munich and Neumunster followed with similar systems and in 2000 in Delitzsch, Wittenberge and Paderborn with optimized systems. For the inspection of the tread surface in addition eight eddy current channels were included.

The AURA Paderborn was also equipped with a US-phased array module for the inspection of the solid axles. Each test system was able to communicate via an ISDN on-line diagnosis with Fraunhofer IZFP to assist customers when inspection problems occur. Examination data were automatically compiled into a special database, so that the test data could always be checked against a look-up table.

In 2004, there followed the installation of newer-generation AURA systems, including eddy current tread surface inspection, in Krefeld and Kaiserslautern (Figs 3.6 and 3.7, respectively). In the AURA system in Kaiserslautern, for freight trains, a wheel web inspection and inspection of the solid axles by ultrasonic phased array (shrink seat) were integrated. Furthermore, a variety of different wheel types was taken into account. The phased array system supplied by GE Inspection Technologies was integrated in the overall system.

To inspect the various ICE wheels occurring when introducing the ICE generation III for the AURA system in Krefeld (including drive wheels with holes in the wheel), automatically interchangeable transducer modules were introduced as a constructive innovation. In the first use of ultrasonic
hardware in Krefeld and Kaiserslautern, a newly developed and miniaturized 24-channel ultrasound industrial front-end was introduced, which was based on a redesign and adaptation that observed enhanced industrial specifications [A-Scan, TD-image, distance–amplitude–correction (DAC), etc.] partly from PCUS-40 and using an electronic hardware originally introduced for in-line pipeline inspection (see Section 3.4.2, Fig. 3.14).²⁰

Wheel set inspection is the quality concept of German Rail and it is based on recurrent in-service inspection (under-floor-inspection system UFPE after 250000 km in the wheel-installed state on ICE). At about 500000
Driven km this concept requires a re-profiling of the treads of the wheels, which, in turn, requires removal of fatigued material (plastic deformation and micro cracks). The wheel is then subjected to a 100% ultrasonic volume testing using AURA systems. The benefits to the railway are mainly the time savings, but also the proof of a complete examination and its documentation. A testing time of 2 h per wheel in conventional manual testing could be reduced to 5–10 min with the use of the automated systems.

**Under-floor inspection unit, UFPE**

For periodic inspections in the periods before reprofiling, German Rail, in intervals between 75 000 and 160 000 km uses ultrasound-based so-called under-floor-inspection units (UFPE), inspection being performed at the wheel sets in the installed state. To achieve the inspection, the ICE vehicle is hydraulically raised some millimeters, so that the wheel set can rotate and sensor carriers are applied to both wheels. In 2000 and 2001, three of such IZFP testing systems were introduced in Munich, Hamburg and Frankfurt. Because of spatial restrictions, the miniaturization of the probe carrier with 42 transducers was particularly important and the installation of ultrasonic front-end electronics was performed directly on the test system. As a special innovation, a Wi-Fi standard was introduced for communication and complicated cable lines could be saved. Figure 3.8 shows the CAD drawing of a new system with UFPE probes in situ and Fig. 3.9 shows a view in the Frankfurt maintenance workshop with a calibration wheel set.

![3.8 Under-floor inspection, CAD drawing.](image-url)
Ultrasonic wheel inspection systems, rail wheel inspection (RWI)

The IZFP together with the Fraunhofer-TEG introduced, from 2000 to 2004, fully automated ultrasonic wheel inspection systems (rail wheel inspection, RWI) at the request of manufacturers of wheels and wheel sets in the steel industry for a standardized production-assisted testing.

RWI installations were set up in Bochum, Ilsenburg (Fig. 3.10) and four more in Nishniy Tagil (Fig. 3.11), in the Ural region of Russia. The systems are 12- to 16-channel ultrasonic units with 2–8 channels in reserve.
The most innovative aspect of these systems is their high sensitivity, which is given by a calibration-setting circular disc reflector of 1 mm diameter. For installations in Russia, it must be emphasized that a handling time, unique in the world, of 60 s per wheel was required (testing time of approximately 40 s per wheel), an extreme specification demand that could be fulfilled only by using all possible human resources and also additional financial support. As a result, these innovative achievements were recognized and the engineers Götz (TEG), Montnacher (TEG), Kappes (IZFP) Rockstroh (IZFP) and Fatehi (Fraunhofer-IPK), were rewarded with the Fraunhofer Prize in 2002.

The AURA and under-floor inspection systems, depending on the required inspection specification, cost in Germany between approximately €1.2 million (without axle inspection) and approximately €1.65 million (AURA with axle inspection). The UFPE system costs approximately €1.4 million whereas for wheel production-assisted inspection systems (RWI) a price of approximately €2.8 million has to be taken into account, because of the much larger effort required in management and mechanics. Spare-part packages are charged separately according to customer requirements.

Prices for foreign customers are dependent on the inspection demands as well as on services required (e.g. provision of test and calibration wheels, training of personnel, documentation, and certification according local standards).

3.4.2 In-line pipeline-inspection gauges (PIG)

The IZFP specifically operates a policy of long-term industrial partnership with the company NDT Systems & Services, which was founded in 2001,
strongly supported by IZFP and Fraunhofer which is 25% share-holder. The company increased in size within 5 years to 120 employees, a measure of its success in the marketplace. The business areas of the company are automated production-assisted systems in the metal industry and service of pipelines. In both, in order to secure the future, broad-based R&D is required and IZFP is also engaged in this.

In addition, a co-operation based on the personal commitment of the CEO of the Company to the IZFP has been established, and it appears that such a commitment is an essential factor for success in partnership. O.-A. Barbian was, in the 1980s one of the executive leaders of IZFP responsible for the development of multichannel automated ultrasonic inspection systems for German nuclear technology. In 1981, Barbian and R. Neumann jointly received, for the development of the so-called ALOK inspection system, the then newly introduced Fraunhofer Prize for innovation in application technology. Over the years, the ALOK system was the standard test unit for examining primary circuit components in nuclear power plants in Germany. Barbian moved from Fraunhofer into industry to a company that was a supplier for heavy-plate inspection systems in the steel industry. Here, he organized the transfer and application of NDT innovations into the field of steel production and processing, e.g. software for the evaluation of ultrasound findings in heavy plates according to any given world-wide standards required by customers. After the acquisition of the company Pipetronix, Karlsruhe, Germany, by his employer, Barbian became the technical business officer in Pipetronix with the objective of integrating plant inspection activities into Pipetronix, which was operating in pipeline service.

The company had just very successfully introduced ultrasonic examination with piezoelectric acoustic transducers for the detection of wall-thinning corrosion by use of intelligent PIG, developed for oil pipeline inspection. Barbian initiated, together with IZFP as subcontractor, the world’s first development of an ultrasound-based crack-detection PIG, validated by TÜV Rheinland and successfully introduced to the market. For their share of that development, W. Bähr (IZFP), R. Neumann (IZFP) and H. Egner (TEG, Stuttgart) received the Fraunhofer Innovation Prize in 1998.

Figure 3.12 shows one product of IZFP from this period of cooperation, the miniaturized AMUS-P unit, a 32-channel ultrasonic hardware (transmitter, receiver, low-noise pre-amplification, A/D-conversion, controller) with a volume of only 1 liter. Miniaturization is a necessity if for instance a 56 inch pipeline has to be inspected (Fig. 3.13). In this instance, 896 ultrasonic channels have to be integrated in the PIG, which constitutes a weight of 5000 kg and moves forward with a speed of 1.5–2 m s\(^{-1}\) in oil, which also acts as a coupling agent. The sensitivity setting for this type of tool allows...
the detection of external axial cracks. The calibration reflector is a saw-cut of 3 cm (length) × 1 mm (depth).

Currently, Barbian operates his own company and drives further developments with IZFP. Figure 3.14 shows the miniaturization of ultrasonic electronics, as for 10 inch PIGs it was necessary to save space. The five circuit boards shown with the AMUS unit in Fig. 3.12 were reduced to one single board of the same size (Fig. 3.14). Figure 3.15 shows an example of a crack detection PIG from the fleet of PIG of the company NDT Systems.
© Woodhead Publishing Limited, 2010

3.5 Non-destructive testing for structural engineering

The Federal Institute for Materials Research and Testing (BAM), together with the German Society for Non-destructive Testing (DGZfP), began in 1985 to evaluate the state of the art of NDT in construction engineering. D. Schnitger and G. Schickert of BAM, respectively a NDT expert and a structural engineer with a strong emphasis on using and advancing the application of NDT in the construction industry, organized the first
NDT Symposium for Civil Engineering in Berlin. The conference proceedings\textsuperscript{25} appeared in 1986 and documented the state of the art in the technology which led the National Science Foundation of the USA to initiate a conference on the topic in 1988.\textsuperscript{26} The meeting in Berlin was the beginning of the establishment of the technical committee of the DGZfP ‘NDT for Civil Engineering’ on 17 September 1986. Schickert was elected as chairman and H. Wiggenhauser (BAM) succeeded him from 26 April 1996 and holds this position to the present day.

It is recommended that all younger colleagues study the report\textsuperscript{25} in order to recognize the progress that NDT has brought to the construction industry in the past 20 years. As an example it should be pointed out that in 1985 it was still not possible to apply the ultrasonic pulse–echo technique for the examination of concrete constructions. Progress in knowledge since that time and the progress of technology have significantly enhanced the applications. However, there is a strong demand for cost-effectiveness and for mainly automated inspection systems.

BAM and Fraunhofer IZFP have, therefore, by a cooperation agreement, established a joint laboratory in the BAM as a project group under the responsibility of Wiggenhauser. The BAM brings in its expertise in construction engineering as well as the ability to perform application-oriented basic research in order to allocate basic knowledge to NDT in civil engineering. IZFP contributes mainly with its experience in the implementation of industrial inspection systems mentioned previously. The objective of the joint lab is ‘To better meet the market needs’.

In the following, two examples are discussed where BAM and IZFP, together with other research and industrial partners, in a program of the German Ministry of Economics and Technology which is specifically addressed to SMEs developing automated systems for construction engineering applications.

3.5.1 BetoScan: an automobile multisensor robot system for non-destructive diagnosis of reinforced concrete structures

A large number of parking structures and bridge decks are suffering from severe corrosion problems world-wide, mainly owing to the ingress of de-icing salts or marine attacks combined with insufficient concrete quality, causing the steel reinforcement to begin to corrode and crack, with spalling and losses in cross section leading finally to a reduced loading capacity of the whole structure.

To evaluate the condition of such structures adequately, extensive investigations are usually necessary. However, to save the costs for the required
works often only simple investigation programs are carried out, and these are not suitable as a basis for an adequate design of measures for maintenance, repair and protection of the structures. Subsequently, often improper measures are carried out leading finally to higher total costs than the complete investigations based on a reliable database of the structure. This situation has been the basis for a research and development project to develop a robotic system that is able to drive over large floors and measure the relevant parameters of the concrete surface simultaneously. The collected data is stored for each investigated point of the structure allowing complex evaluations of the data regarding the assessment of the condition, prognosis of the future state, design of measures for protection and repair as well as quality control.

A demonstrator system of the BetoScan system is actually under development. For this purpose three research establishments syndicated with seven small- and medium-sized enterprises (SME) and one industry partner (Table 3.1).

<table>
<thead>
<tr>
<th>Table 3.1 Project partners</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Category</strong></td>
</tr>
<tr>
<td>Research establishments</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>SME</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Industry partner</td>
</tr>
</tbody>
</table>

© Woodhead Publishing Limited, 2010
The demonstrator system

The BetoScan system consists of a mobile robot platform that is able to navigate quasi-autonomously over horizontal areas equipped with various sensors for non-destructive measurements (Fig. 3.16). At first, an orientation cruise is required where the robot is able to detect walls, piles and other barriers with its 270° scanner. From these detections, the user generates a digital environmental map. Within this map an inspection area has to be defined and an optimized meandric inspection run will be generated automatically. The ultrasonic sensors on the front allow the robot to detect even movable obstacles. On the back of the platform all sensors except the eddy current sensors are fixed. To avoid disturbances these latter sensors are fixed at the front side. With special attachments developed within the project, the sensors can be positioned in different ways. With this modular setup, it is possible to upgrade the system with further sensors.

The sensors are able to collect the data at a driving speed up to 0.1 m s\(^{-1}\), the robot itself can drive up to 1 m s\(^{-1}\). Thus, the system is able to investigate surfaces of some hundred square meters with one set of batteries. The accumulator pack at the top of the platform (Fig. 3.16, black boxes) provides enough energy for a minimum of 8 h.

3.16 The BetoScan robotic system.
All measured raw data is stored in a XML format along with the individual local positioning information of the platform. A computer in the platform stores all relevant data, which are then downloaded via WLAN to a notebook of the operator and organized into a database. The analysis software can be used to handle the data in the database in order to generate the maps on site.

The sensors

To investigate reinforced concrete structures, various commercially available sensors have been chosen to be integrated into the robotic system (Fig. 3.17). The integrated measuring instruments can be used further on as standard handhelds, so that manual collection of further data is still possible.

For concrete structures exposed to de-icing salts, the first step of a condition survey should in many cases be a potential mapping of the whole concrete surface. During recent years affordable instruments to measure, store and plot the electrochemical potentials have become commercially available. Within the project, Proceq’s instrument ‘Canin +’ is used in combination with copper/copper sulfate reference electrodes. In order to generate concrete cover depth maps of the whole surface, Proceq’s ‘Profometer 5+’ based on the eddy current method and MALÅ’s GPR ProEX system based on the radar method are attached to the system. Knowledge of the concrete cover depths can be very helpful in cases of assessing the condition
of structures in order to design an adequate repair measure or to control the application quality after replacement of the concrete cover by a repair mortar.

By means of the ultrasonic system ‘A1220 Monolith’ from ACSYS, the structure thickness can be determined over the whole surface. Additionally, it is possible to detect and to map voids in the concrete if additional information of e.g. the biggest aggregate diameters and the surface quality are available. One advantage of the use of this specific instrument is the possibility of connecting the ultrasonic sensor heads onto the concrete without a coupling gel, thus simplifying the automation of the measurement. As mentioned above, the ultrasonic sensor needs a direct connection to the concrete surface. Therefore, the robot platform has to stop while doing the measurement. A pneumatic pump in the frame of the attachment module presses the sensor onto the floor while the whole measuring procedure is carried out.

To investigate the relative moisture distribution of the areas near to the concrete surface, the microwave sensors ‘Moist PP’ and ‘Moist RP’ from HF-Sensor are used. These sensors do not need a direct connection to the concrete surface.

3.5.2 The driving modes

For automation of NDT methods, two driving modes are required. The first mode implies a continuous driving if only contactless measurements are carried out and the potential mapping with wheel electrodes is applied. The second, discontinuous mode, considers the measurements if the ultrasound ‘contact sensor’ is additionally used. In that instance the robot stops driving while the measurements are accomplished. Therefore this mode is more time consuming but still faster than a measurement by hand.

3.5.3 Features of the BetoScan system

The most outstanding advantage of the BetoScan system is the simultaneous measurement of all key parameters over the whole surface. Each parameter can be investigated with a corresponding sensor before it is saved with the appropriate co-ordinates in a database where all relevant information is collected. The operator can track the gained data at the same time via WLAN and can generate the initial maps on site. With this information further investigations can be planned, prepared and carried out without time delay.

As shown in Fig. 3.18 for every measurement point all relevant parameters are known, and thus combining all this information results in a more explicit knowledge of the current state of the structure.
The BetoScan system opens several new possibilities for the overall life cycle management of concrete structures. In summary, these are:

- diagnosis of ageing phenomena,
- life prediction,
- service life management,
- repair planning and protection of components, and
- quality management

### 3.5.4 Diagnosis of ageing phenomena

The diagnosis of reinforced concrete structures can, by means of the BetoScan system, be divided into four steps as shown in Fig. 3.19. As a result, the operator obtains all measurement values over the whole surface of the investigated area and a complete assessment can be carried out. One major advantage of the BetoScan system is the possibility of dividing the whole surface into zones with defined damage classes. Such classes could be:

- no damage owing to high concrete cover and good concrete quality,
- corrosion of the reinforcement owing to carbonation,
- chloride-induced corrosion of the reinforcement,
- cracking owing to corrosion,
- spalling owing to corrosion, and
- concrete damage owing to environmental factors such as frost.
3.5.5 Life prediction

Some of the measurement data can be used as a basis for a detailed evaluation of the future state of the structure. Suitable models for corrosion initiation (chloride ingress and carbonation of the concrete) are available and models for the propagation phase (corrosion of the reinforcement) are actually investigated in an extensive research project (www.dfg-for537.de.

When the concrete cover is known for the whole surface and reliable estimations for the speed of carbonation or the ingress of chlorides are worked out based on the data from the diagnosis, the time to corrosion or the time to reach a critical limit value can be calculated for each position of the surface. This allows the surface to be divided into zones with defined classes regarding the prognosis. A possible classification of the zones could be done as follows:

- no action required for the next 20 years,
- no action required for the next 10 years,
- no action required for the next 5 years, and
- immediate actions required.

These maps could be a useful basis for the service life management of reinforced concrete structures.
3.5.6 Service life management

A reliable prognosis of the condition and behavior of a structure is an important basis for an effective service life management. After finishing the building process, an initial diagnosis should be carried out with the BetoScan system to document the initial state of the structure over the whole surface as a basis for a first prognosis (birth certificate). From time to time, additional diagnoses allow an update of the prognosis leading to an increased reliability of the prognosis with time. By means of these repeated or cyclic diagnoses, an update and sharpening of the prognosis are possible. With the BetoScan system, the costs for a complete survey can be reduced and the database for a service life management can be improved economically.

3.5.7 Repair planning: protection of components

Based on the knowledge of all measurement data over the whole surface, a selection of suitable measures for protection and repair can be carried out adequately. Zones where the same repair methods could be applied can be carried out. As a result detailed maps for the design of suitable measures for protection and repair will be available.

3.5.8 Quality management

During and after the application of protection and repair measures, different key parameters can be determined for quality management purposes using the BetoScan system (e.g. documenting the concrete cover depths after the replacement by a repair mortar). Scanning the whole surface area allows a reliable statistical evaluation of the data and a detailed localization of critical zones.

3.6 Multiple-sensor data acquisition by the OSSCAR (On-Site SCAnneR) scanner

3.6.1 General remarks

The multisensor application presented in this section, in addition to the application of the BetoScan system discussed in the previous section, belongs to a project named OSSCAR (On-Site SCAnneR), an automated non-destructive investigation of bridges using a manipulator. The aim here is also to improve the data density and the data quality through sensor combinations. An additional benefit is time-saving by automated data acquisition which is much faster owing to parallel acquisition and therefore less expensive than the usual manual state of the art procedures.
Figure 3.20 shows the newly developed scanner system (principle CAD sketch in Fig. 3.20a and demonstrator in Fig. 3.20b). The sensors used here are: Profometer (eddy current testing for concrete coverage and rebar position), ACSYS A1220 (ultrasound for geometrical information about the component especially about tendon ducts and flaws therein) and MALÅ

3.20 (a) Sketch of scanner frame that consists of three beams containing all service lines and cables for energy, data and air; (b) photograph of the scanner together with all devices that can be used for automation (radar, MALÅ ProEx; ultrasonic, ACSYS A1220; eddy current, Proceq Profometer 5+).
ProEx (radar) for geometrical information of the component and evaluation of the reinforcement position. Two different sensor carriers can be used, one combining eddy current testing and radar, and the other having two ultrasound probes with a pneumatic contact control mounted one on top of the other.

In contrast to the BetoScan robot, the procedure for acquiring data here is different; a constant plane is scanned with the scanner and then the scanner device is moved. Afterwards, the data collected from the measurements on these planes are collated to give a coherent image of the region of interest. This leads to the application of data fusion and joint inversion concepts for reconstruction of the elements under investigation. Therefore, the data is handled in a general data cube format.

3.6.2 Application for bridge testing according to German Standard DIN 1076

Engineering structures in connection with federal highways are tested according to German Standard DIN 1076 at regular intervals. For complex damage patterns, it is necessary to obtain additional information by detailed analysis at the object level (OSA). Non-destructive testing methods are part of this further analysis. In Germany, as in many other western countries, the total amount of bridge deck area of prestressed and reinforced concrete structures is approximately 90%. Many of these bridges date from the 1960s and 1970s with typical damage of the prestressed steel caused by incomplete grouting of the tendon ducts, a low degree of reinforcement, and cracks in the coupling joints. For the condition assessment of such structures, reliable tools for non-destructive testing in civil engineering (NDT–CE) are needed. The OSSCAR system has been developed to gain a detailed insight of internal construction of prestressed concrete members. It is considered to be a helpful tool for a bridge inspector in combination with structural engineers to come to reliable decisions about the condition of bridges concerning stability, traffic safety and durability.

3.6.3 Development and features of the OSSCAR system

Within the integrated project OSSCAR funded by the German Ministry of Economics and Technology, a lightweight, flexible and easy-to-handle scanner was developed. The frame provides a testing area of 50 cm × 100 cm, and allows quick installation and easy changing of the measuring position (Fig. 3.20). Testing tasks to be carried out with the scanner are:

- imaging of the geometry,
- location, and measurement of depth and diameter of the multilayer reinforcement,
• location and measurement of depth of multilayer tendon ducts,
• quality assurance for complete grouting of tendon ducts, and
• location of grouting defects in existing constructions.

Owing to the small size, the OSSCAR system scanner fits in narrow corners and can be used even for columns. It is easy to transport and even suitable for access into a small manhole (less than 1 m × 1 m) in a bridge because the frame with its three beams is mounted directly on the structure to be measured. For non-destructive fixing, vacuum feet have been developed.

Intelligent combination of various methods along with advanced data processing optimizes the current capabilities of NDT–CE methods. The OSSCAR approach is the combination of radar, ultrasonic–echo and eddy-current methods using commercially available devices (Fig. 3.20b). As shown in Fig. 3.21a, radar is the method for the detection of metallic reflector but it has a limited penetration depth especially in young concrete or structures containing reinforcement. In these instances, the ultrasonic method allows greater analyzing depth for thickness measurement but with resolution limited to a single rebar. The eddy current method gives detailed information about the upper reinforcement layer such as precise concrete cover or bar diameter. Using three devices for automation (Fig. 3.21b–d) that are similar to devices on the market gives an engineering company that owns these devices the flexibility to use each device as a single hand-held device or as a scanner-driven automated method with the option of method combination.

High quality data as a result of automated testing in a dense and precise grid is the base for data processing and subsequent imaging of the results. Therefore, all-in-one software has been developed with the following features:

• control of the scanner (width of the measuring grid, meander or lines),
• automated data collection of each method in a 3D data cube with a global coordinate system,
• function container for best data processing (e.g. filter, reconstruction algorithm such as migration or SAFT), and
• 3D data browser for imaging the results allowing variable sections through the 3D data cube.

The software enables the scanner to be controlled in a separate window with the option of defining the starting point, the width of the measuring grid, and how the scanner axis (lines or meander) is moved as shown in the left window of Fig. 3.22b. The actual position of the scanner is displayed continuously. An additional window (Fig. 3.22b, right) allows data collection from radar and ultrasonic methods (A- and B-scan) to be controlled.
3.21 (a) Method combination of the OSSCAR system: radar, ultrasonic and eddy current with strengths and limitations. Each sensor adapted for the OSSCAR system: (b) radar, (c) ultrasonic echo, and (d) eddy current.
3.22 (a) Screen shot of the scanner software with (b) a separate window for control of the scanner and a window for controlling data collection of radar (top), ultrasound (middle) or eddy current (bottom).
3.7 Conclusions

For automated NDT to be applied in complex inspection tasks for the lifetime management of infrastructure requires the interdisciplinary cooperation of various engineering disciplines to overcome problems with the testing techniques, the scanning devices, the data acquisition and evaluation. Lessons learnt when developing inspection systems for enhancing safety and reliability in high-speed railway traffic and/or oil and gas transportation in transmission pipelines today also contribute to the development of market-accepted multiple-sensor data acquisition systems using NDT sensors in civil engineering applications where complex materials and material compounds are investigated. Three major parts have to interact for a working system: the sensors, the data acquisition and the data analysis. Regarding the data analysis, three different approaches can be applied: data correlation for a comparative analysis, data fusion to give added value from merging the data, and joint inversion to find a model that explains several data sets at once.

The multiple-sensor applications discussed here for construction engineering are designed for the investigation of reinforced concrete and tendon ducts. The advantage is that large surfaces can be investigated in short times and the measurements are of reproducible quality. This guarantees the data quality for recurrent inspections. However, the general concept of multisensor data acquisition and data analysis presented here is not limited to the field of civil engineering applications. Multisensor applications offer better inspection possibilities for complex material and constructions. In particular, for subordinate assessments reliable data sets including geometrical and material state information can be guaranteed. The data analysis of multisensor measurements still offers a variety of research capabilities and the simplification of sensor changes (hardware and software adaptations) is under current investigation.

3.8 Acknowledgements

Developments to OSSCAR and BetoScan are financially supported by the Federal German Ministry of Economics and Technology; furthermore, the contributions of all partners in these collaborative research projects are gratefully acknowledged.

3.9 References

9 Rosen Headquarters: Magnetic flux leakage (MFL) and ultrasonic testing (UT) for optimal detection of metal loss and pipeline wall features, 3R International, 46(2), 2007, 44–47.


Structural health monitoring systems for reinforced concrete structures

W. R. HABEL, BAM Federal Institute for Materials Research and Testing, Germany

Abstract: Online sensing of materials or of the condition of structures has become increasingly important in evaluating the overall behaviour of a complex structure. This task requires reliable structural health monitoring (SHM) systems. Because some simple measurement systems are often mistakenly called monitoring or SHM systems, a clear understanding of the intention and meaning of SHM systems is necessary. Details about the demands on SHM systems are given and their capabilities explained. A definition of a real SHM system is given. The main section focuses particularly on innovative monitoring methods based mainly on the increasingly used fibre-optic sensor technology. Typical examples are presented of long-gauge-length and short-gauge-length fibre-optic sensors for measurement of mechanical, chemical and physical quantities; selected examples of their use are also presented. Sensor systems to be used for long-term monitoring must be stable in their function and work reliably over a long period of time. Therefore, problems relating to reliability are considered, and recent relevant activities in standardization of innovative measurement methods are presented.

Key words: structural health monitoring, sensors, fibre-optic sensor, damage assessment, reinforced concrete.

4.1 Introduction

The use of destructive methods to obtain information about materials, the condition of structures, or structural behaviour under loading is no longer accepted. Online sensing, continuous or periodic, has become more and more important in evaluating the overall behaviour of a complex structure, preferably from the manufacturing process until the end of its service life. This task requires a system consisting of sensing elements, data transmission lines (alternatively wireless communication), data processing, and, it should not be forgotten, the translation of the measurement results into a form accessible to those who (whilst they may be technically competent) are not experts in such systems, e.g. by visualization in the form of a time-domain signal. Such systems integrated into a structural component are called...
structural health monitoring (SHM) systems. Many sensor systems currently being used on real-world structures are only simple monitoring systems, but are often termed SHM systems. According to our experience, experts from various disciplines (lightweight construction materials scientists, civil engineers, information scientists, and measurement technology experts) interpret and use the expression SHM differently. There is also a range of definitions and interpretations for the term SHM. Therefore, the structure of a monitoring system will be addressed in the next section.

Monitoring systems make use of the well-known conventional methods associated with non-destructive testing (NDT), such as principles based on strain, vibration, acoustics, electromagnetics, humidity, temperature and so on. Furthermore, new physical technologies, such as fibre-optic sensor technologies, form a new cluster of methods that can be exploited for SHM tasks. Therefore, monitoring examples based on innovative physical methods, such as fibre-optic sensor technology, will be presented in more detail.

4.2 Demands on monitoring systems: monitoring capabilities

SHM systems are often simplified as systems that provide information (on demand) about any significant change or damage occurring in a structure. Monitoring activity is usually driven by ongoing structural deterioration caused by environmental influences (temperature cycles, humidity, chemical attacks) or by static and dynamic loads. Macroscopic damage, such as cracks, delamination of stratified materials or mechanical spalling, can be (and still are) identified by visual inspection and ‘monitored’ by continuing observation. This only works efficiently if the relevant location can easily be accessed. If not, automatic data recording systems based on sensors which collect data are required.

SHM systems are urgently needed if the damage is generated by deterioration at a microscopic level. Such damage, caused by factors such as penetration of chloride or sulfate into concrete structures, carbonation of concrete, or timber decay, may progress very slowly, and only become observable when the damage to the structure is considerable and sometimes only repairable at great cost. SHM systems which allow the optimized detection of damage progress at both microscopic and macroscopic levels provide a comprehensive insight into the structure, without having to visit the structure or to take samples via destructive methods. For this purpose, monitoring systems based on miniaturized sensors and sensor technologies can be integrated and/or attached to structural components of various kinds of materials without disturbing the behaviour of those materials. In contrast to most non-destructive technologies, the elements of an SHM system
(sensitive elements including wiring and possibly data reduction) can be considered as integral parts of a structural component.

Thus, integrated SHM systems do not only consist of additional sensing components, the monitoring system has to be designed holistically to become an integral part of the whole structure to be monitored. This includes assessment tools and prognostic models of degradation because a combination of such models with monitoring results enables more precise predictions of the rate of structural degradation and of the realistic lifetime of the structure, and may inform decisions to extend service life. Other very important uses for SHM include the capability to supervise manufacturing processes of structural components, supervise transport and construction work, and to help arrive at the most economical maintenance decision.

Considering these approaches, any definition, taking other definitions into consideration, should include the topics and their interactions in the following:

Structural health monitoring (SHM) is the analysis, localization and recording of the loading and damaging conditions of a structure by materials-integrated or structure-integrated sensing devices that permit a prediction in such a way that NDT becomes an integral part of the structure and a material.

Another definition was proposed by Housner et al.:

SHM is the use of in situ nondestructive sensing and analysis of structural characteristics, including the structural response, for detecting changes that may indicate damage or degradation.

Both definitions include the analysis aspect. However, it is a big challenge to design an SHM system that includes all components to be used from data acquisition through to predictions of structural behaviour. With regard to the data required when designing a SHM system, there are four main steps in the process: acquisition, validation, analysis, and management. It should be noted that all aspects have to be considered equally because often emphasis is placed on the collection of data rather than on the management of what is usually a huge amount of accumulated data, and on the development and use of tools to make decisions or to provide prognoses. Important aspects of these process steps are:

- data acquisition; types/placement and application/number of sensors, type of network communication and power supply, type of data retrieval (continuous data stream or data packages at pre-determined intervals);
- data validation; long-term functionality of sensors (reliable application and stable sensitivity, ageing of sensor components), validity of collected
data, errors and losses during transmission, comparability and integra-
tion with models and their updates;
• data analysis; interpretation of data as related to models of structural
response, updating of baselines and thresholds, consideration of the
environmental situation, use in damage prognosis and estimation of
remaining lifetime; and
• data management; data storage, data retrieval, data mining, integration
of tools for communication with owners, authorities, and supervising
offices, and maintenance of the data management system.

Real SHM systems in structural components (according to the defi-
definitions above) would enable comprehensive information fl ow begin-
ing from the manufacture of the components until the end of service life. Depending on
the measurement task, and also often on the available budget, monitoring
with integrated sensors has to be performed more or less frequently and
with the equipment that an organization’s budget may allow in different
situations:

• during curing of materials; curing of epoxy resin-based materials, con-
crete hydration at very early stages;
• monitoring during construction period; identification of critical states
after construction steps;
• limited continuous monitoring for a few days; monitoring of structure
response to temperature or load variations;
• measurements during test load cycles;
• measurements before, during and after repair and refurbishment;
• long-term monitoring of safety-relevant structures (over years);
• extraordinary measurements after special events (thunderstorms, earth-
quakes, terrorist attacks).

In order for sensors to function stably for the long term, some technological
demands have to be considered.

• The sensor installation and/or application procedure must be matched
with the corresponding technological conditions (during manufacture
or service); that is, the sensor components must not be damaged during
construction work.
• The sensor system must not perturb the material to be monitored or
even initiate degradation or failure. This applies especially to the design
of cabling. Therefore, the sensing system design must be considered
during design of the structure.

The following overview of monitoring details is limited to aspects of mea-
measurement technology and should provide insight into recently developed
measurement technologies that are ideally suited to monitoring of reinforced concrete structures.

4.3 Innovative monitoring methods

When a sensor system is needed to perform monitoring, the favoured choices are conventional sensor technologies, such as resistive strain gauges, extensometers, LVDT (linear variable differential transformers comprising three coils and a movable magnetic core; the displacement being measured by a variation of the magnetic flux owing to the movable magnetic core) strain transducers, and load or force gauges. From NDT alone, a number of methods can also be used, such as ultrasonic sensors or piezoresistive strain sensors. A wide overview on sensing technologies for monitoring is given in volume 5, part 9 of Civil Engineering Application in the *Encyclopaedia of Structural Health Monitoring*.1

Within the past 10 years, the introduction of new measurement technologies based on optical methods has contributed to revolutionary innovations in monitoring to bridge the gap between conventional concepts and practical requirements. One of these new measurement technologies is fibre-optic sensor technology. Thousands of fibre-optic sensors have already been installed in different types of civil engineering structures to provide measurement data for monitoring or even for SHM systems. There are many dozens of companies worldwide that offer commercial fibre-optic sensor solutions for use in concrete structures. Apart from the sensor type, the sensor system (the sensing element, cabling and ingress/egress area at the very least) usually has to be adapted to the specific on-site conditions of application and operation. A number of examples of the use of fibre-optic sensors in civil engineering are described by Glisic and Inaudi.3

In the following section, a brief introduction is given to only the newest fibre-optic sensor technology. (Because conventional sensor principles are generally known and described frequently, they need not be described here.) An outline of selected recent examples of the monitoring of reinforced concrete structures is given.

4.3.1 High-performance sensors for deformation measurement

Concrete structures have vast dimensions and, usually, numerous sensors are required to monitor an extended structural component. Fibre-optic sensors are very well suited for this measurement task because several sensors or sensor sections can be designed along the naturally extended
optical fibre. In addition, the fibre itself can act as a sensor at any point. This special feature enables extended (distributed) sensors to be made using only one optical fibre with one lightweight optical lead cable.

From the user’s point of view, an essential criterion in the choice of an appropriate sensor type is the length of the region to be evaluated, i.e. the number of sensors required. Different sensor types, such as short-gauge-length or long-gauge-length (Fig. 4.1) sensors, can be used depending on whether local (point-like) measurement information or distributed measurements, such as measurement of strain profiles along a bridge girder, e.g. for the detection of cracks or other damage along an extended structural component, are needed. Distributed fibre sensors have the very desirable feature of being able to measure not only a physical quantity affecting the fibre (e.g. temperature and humidity), but also the position where the measurand is acting. The scan frequency of such sensors is limited to a few Hz or less.

The complement to long sensor fibres is short-gauge-length sensors. Such local fibre-optic sensors (sometimes known as point sensors) are, in the case of strain sensors, similar to resistive strain gauges, or to pressure or inclination sensors (tilt meters). Such local fibre sensors can also be used as pH sensors or humidity sensors (see next section).

Long-gauge-length fibre-optic sensors

This group of sensors can be divided into three categories: averaging (integral) sensors of gauge lengths greater than 50 cm, quasi-distributed sensors such as segmented sensor fibres, and fully distributed sensors. The best-known averaging fibre-optic extensometer is the SOFO (surveillance d’ouvrages par fibres optiques, a registered trademark of the Swiss Company SMARTEC SA for a long-term monitoring system based on low-coherence interferometry) strain sensor. It is based on low-coherence interferometry
which ensures not only high resolution (about 2 μm in deformation) but also repeatability of measurements, even if the measurement system is switched off. Also, if components such as connectors or cables have to be exchanged, zero-point data loss does not occur. This type of sensor provides referenced measurement data, can be embedded or surface-mounted, and is used for static and dynamic measurements with gauge lengths of up to several metres. Several thousand such sensors have been installed worldwide in/at all kinds of structures. One example of these SOFO sensors as an integral part of a complex monitoring system is their use in the new Berlin main station, Lehrter Bahnhof.4

When light propagates through an optical fibre, it is scattered by spontaneous fluctuations in the dielectric constant of the light-guiding material. There are molecular vibrations (called Raman scattering), propagating density fluctuations (stimulated Brillouin scattering), and non-propagating entropy fluctuations (Rayleigh scattering). Brillouin scattering depends on both temperature and strain, whereas Raman scattering only depends on temperature. Referring to these effects, external perturbations (the measurand) can then be scanned and evaluated by determining the intensity and the spectral composition of small parts of a backscattered light pulse. In order to measure only temperature influences (distribution) along a fibre, the Raman backscattering method is recommended. For instance, the measuring time for an 8 km long fibre with a spatial temperature resolution of 1 m is about 1 min; a better spatial resolution of 0.5 m or 0.25 m requires a longer integration time. (The specific term ‘spatial resolution’ must be explained: spatial resolution is specified for a fibre by the minimum distance between two step transitions of the measurand at 20 times or more the measurand resolution. It has to be distinguished from ‘measuring spatial resolution’, which means the minimum distance over which the system is able to indicate the value of the measurand within the specified uncertainty, and the ‘detection spatial resolution’, which expresses the minimum distance that generates results that are within 10% of the measurand transition amplitude. Further explanations can be found in the COST fibre-optic sensor guideline.5) The measuring range of the distributed temperature sensor is from −50°C up to 300°C, with a temperature resolution of less than 0.05 K. Because the Raman scattering depends only on physical constants and on parameters of the light transmitted into the fibre, absolute temperature values can be obtained directly from the backscattered signal. Owing to different qualities of the fibre material used, a calibration function must be determined before measurement starts. This method of distributed temperature measurement is used for continuous detection of leakage at pipelines and vessels6 as well as for mass concrete structures such as concrete gravity dams to monitor the temperature development after pouring and owing to the hydration reaction.7
Examples for quasi-distributed and distributed strain sensors based on Rayleigh and Brillouin backscattering are given in the next section.

**Short-gauge-length fibre-optic sensors**

The best-known short-gauge-length fibre-optic sensor is the fibre Bragg grating (FBG) sensor. Another short-gauge-length type is the Fabry-Perot interferometer (FPI) sensor. Both sensor types have gauge lengths of about 5 to 20 mm and can be used as static and dynamic strain sensors as well as for temperature, vibration and pressure measurements. They are commercially available; however, for some specific applications, modified types are used. Such sensors can be embedded into materials to measure deformations inside the material. The FBG and FPI sensors have significantly different characteristics. After a short technical description of FBG and FPI sensors, examples of application in the next sections will make these differences clear.

### 4.3.2 Fibre Bragg grating (FBG) sensors

For experimental stress analysis, the most highly developed common fibre-optic sensor is the fibre Bragg grating strain sensor. This sensor (grating) is located in an optical fibre; its diameter is about 200 μm, its length is of the order of 10 mm. The material in the grating area is modified by periodic alterations in the fibre core’s index of refraction. The grating is illuminated with white light and, according to the grating dimensions, only one sharp wavelength (so called Bragg wavelength $\lambda_B$) of the white light spectrum will be reflected. Figure 4.2 shows this behaviour schematically. The Bragg wave-
length $\lambda_B$ is shifted if the sensor is longitudinally deformed by mechanical or temperature influences. This shift will be recorded and evaluated. An important fact is that the sensor always measures strain deformation and temperature variations simultaneously if both exist. In such (typical) cases, a reference grating that only senses the temperature has to be used to separate the strain value from the temperature value. Using reliable data recording systems, strain and temperature resolutions of about 1 $\mu$m m$^{-1}$ and/or 0.1 K are possible. Less expensive devices allow resolving strains in the order of 10 $\mu$m m$^{-1}$ (or even more) and about 1 K. It must be pointed out that the accuracy of strain measurement can be reduced as a result of different levels of skill in application (see Section 4.5).

Depending on the use of FBG, there are three strain sensor designs: Bragg grating fibre, optical strain extensometer and FBG strain gauge (patch-like similar to resistive strain gauges). All of these sensors are commercially available. The application conditions define the choice of the engineering design.

The most convincing advantages of FBG sensors are outlined below.

- Their multiplexing potential: a certain number of gratings (up to 25, more typically up to 12) can be placed in the same optical fibre at different locations. They form a sensor chain and every sensor is tuned to reflect the Bragg wavelength (the measurement information) back at a different peak wavelength. This feature enables the measurement of strain profile along the sensor fibre using several FBG sensors.

- The very small size of the sensor element (diameter about 250 $\mu$m): such small sensor elements embedded into microstructures susceptible to perturbations enable measurements without perturbation of the measurement zone.

- The ability of line-neutral measurements: because the measurement response is intrinsically encoded in the deformation-induced wavelength shift of the reflected signal, it is not influenced by external perturbations arising from the leading fibre, connectors or devices (if calibrated). If the sensor was disconnected, analysis of the signal spectrum after re-connecting provides the absolute measurement information again. This means that there is no loss of the zero-point reference during long-term monitoring from system components.

FBG sensors should be embedded without any perturbing elements such as eyelets or patch-like supports which could deform the grating in an inchoate way. Two important limitations have to be taken into consideration, when they are directly embedded:

- the stiffness of the sensor fibre material (Young’s modulus $\varepsilon$ is about 70 GPa) constrains the material of the structure to be measured for
deformation and/or the sensing element does not see any deformation at very early stages; thus, there are no measurement results during the first period of hardening;
- transverse effects to the grating, e.g. transverse pressure, bending, and local strain gradients along the grating itself might influence (sometimes disable) the measurement signal.

In general, FBG sensor elements are only able to measure deformations of stiff or already hardened materials.

### 4.3.3 Fibre-optic Fabry–Perot interferometer (FPI) sensors

The fibre-optic Fabry–Perot interferometer sensor is another very tiny local fibre-optic sensor mainly used as a microstrain sensor. Its diameter can also be much less than 1 mm, with a length of between 5 mm and about 30 mm. Extrinsic fibre Fabry–Perot interferometer (EFPI) sensors can be designed to provide minimal reaction forces against deformation if necessary. This feature allows the measurement of deformations of soft or curing materials such as mortar, concrete or plaster in the interphase between rheology and solid state because such a sensor does not constrain the surrounding non-stiff matrix from deformation. Test measurements revealed that embedded EFPI sensors only require a force of about 150 μN to react to material deformation.

Figure 4.3 explains the structure of such a flexible EFPI sensor for materials monitoring. The sensor used for these investigations is formed by a silica glass capillary (tube) with a typical inner diameter of 127 μm. The length of the tube can be between 5 mm and about 30 mm. Bare (uncoated) optical fibres with cleaved facings are inserted into opposite ends of the tube in such a way that the fibre ends form a gap inside the tube. The dis-
tance between these ends (gap length) has to be adjusted to a few micrometres when tensile strain is expected, and to about 120 \( \mu \text{m} \) when contraction (shrinkage) is expected. The gap changes when the sliding fibre is moved owing to material deformation. During this change, interfering light variations can be recorded which provide a very high strain resolution, better than 0.1 \( \mu \text{m m}^{-1} \).

This type of sensor has been used to monitor and evaluate the shrinkage behaviour with regard to micro crack development of cementitious materials in the first few minutes after settlement. Other measurement methods will not yet be able to record any deformations in the first few minutes of curing. In this way, the mixture of specific building materials could be optimized (see next section).

Another application field takes advantage of the excellent dynamic characteristics of FPI sensors. These make them useful to measure acoustic emissions from concrete structures, e.g. by embeddable geophones, to monitor and assess the integrity of large concrete piles or to evaluate their load-bearing capacity (see next section).

There are a number of other fields of interest in concrete structure monitoring where such a sliding micro strain sensor will be beneficial. For instance, this sensor can also be used to measure strain in interface areas of layered concrete structures such as in liner-coated concrete components or in the bonding zone (and close to it) of concrete profiles reinforced by fabrics (textile-reinforced concrete).\(^8\)

Comparing these two short-gauge-length sensor types, in civil engineering, the question of which sensors to use often arises: FBG or FPI sensors? As a rule, one can say: if deformations in silicon or natural rubber material, in curing matrices at early ages or in fume-like materials with low resistance reaction have to be measured, the compliant-fibre FPI sensor has to be chosen. More details about how long after starting the hydration process the stiff fibre-optic sensors become able to read the deformation have been reported by Schuler \textit{et al.}\(^9\) In order to monitor stiff components with limited extensions from a few centimetres to a few ten centimetres in length, and with frequency requirements that are not too high (up to a few hundred Hz), FBG sensors are the best choice.

### 4.3.4 Fibre-optic point sensors

Another specific design version of a local fibre-optic sensor is one where a sensitive layer is arranged at the end face of the optical fibre or in a local area of the fibre coating. Such layers or membranes are sensitive to humidity, pressure and to chemical species, e.g. hydroxide ion concentration, and thus act as a sensor. In the next section, a fibre-optic pH and a fibre-optic humidity sensor will be presented.
4.4 Selected examples of effective and innovative monitoring technologies

A large number of papers, conference reports and book projects have been published describing all types of monitoring technologies. One recent relevant publication is the monograph ‘Structural health monitoring of civil infrastructure systems’ edited by Karbhari and Ansari,\textsuperscript{10} which reviews key developments in research, technologies and applications, discusses systems used to obtain and analyse data and sensor technologies, and assesses methods of sensing changes in structural performance.

In this section, the most important sources and conferences are cited. In addition, recently developed or published innovations in the use of fibre-optic sensors, and unusual examples dedicated to monitoring of reinforced concrete structures have been selected.

4.4.1 Long-gauge-length sensors

In order to monitor local compressive strain of concrete structures, piezoresistive cement-based strain sensors (PCSS) were embedded in concrete beams and columns to monitor the compressive strain under field conditions.\textsuperscript{11} A four-pole arrangement of an embedded gauze electrode is used to eliminate the contact resistance between electrodes and piezoresistive cement-based material. Thereby, the measurement accuracy of the output signal of PCSS will be improved. Both carbon fibre and carbon black are added into cement-based materials to improve the reproducibility of piezoresistivity in PCSS; temperature and humidity effects will be compensated for. Experiments confirmed a sensitivity of 0.0138% μm\textsuperscript{-1}, a resolution of 0.007 μm, a linearity of 4.25%, a repeatability of 4.36% and a hysteresis of 3.63%. Embedded PCSS provide self-sensing concrete components that allow the monitoring of the compressive strain in concrete components.

A quasi-distributed fibre-optic strain sensor is used in the extremely heavy steel strand anchors (4500 kN tensile force each) of the German Eder dam next to the city of Kassel. Usually, the conventional way to evaluate the bonding of the fixed length of anchors in difficult soil areas and/or during anchor suitability tests is by measuring resulting forces at the anchor head. This test method delivers integral information as to whether the introduced anchor forces will be transferred into the soil area. This measurement method does not provide any monitoring information about skin friction distribution along the steel anchor or about how much the anchor length is involved in the load bearing. Because monitoring of the 10 m long fixed area of the Eder rock anchors was mandatory, an aramid rod with embedded optical fibres was positioned in the centre of every tenth anchor. The measurement method is based on the optical time domain reflectom-
etry (OTDR) principle (originally a method for evaluating optical fibres and finding weak points or damaged areas along fibre communication lines). Several reflectors are positioned chain-like along the optical fibre in the aramid rod forming the measuring sections (Fig. 4.4). By measuring the travel time of a short pulse transmitted into the fibre and backscattered on the two reflectors of one section, the deformation of this section can be determined at definite locations along the fibre. The elongation (compression or contraction) of a measuring section $L_o$ changes the travel time $t_p$ of the pulse and can be calculated from $\Delta \varepsilon \sim \Delta t_p (c/2L_0n)$; where $c$ is the speed of light and $n$ the index of refraction in the fibre. Based on this relationship, variations in the average strain of the marked sections along the fibre can be estimated. Twice a year, the deformation profile in the fixed anchor area is interrogated by using an OTDR device and the anchor integrity can be evaluated.

The achievable deformation resolution can vary depending on the backscattering device. The picosecond OTDR device used in the Eder dam (Type OFM 20, Opto-Electronics Inc., Oakville, Ontario, Canada) enables a deformation resolution of about 150 $\mu$m (reproducible to about 500 $\mu$m) for a maximum of about 15 measurement sections per fibre. If, however, a nanosecond OFDR device is used, 10 $\mu$m deformation changes can be resolved for sensor fibre lengths up to 70 m; the same device enables a resolution of 30 $\mu$m deformations for lengths of up to 2 km. The upper value of 500 $\mu$m deformation resolution is definitely sufficient to recognize dangerous changes in materials or even the loss of bonding integrity. The high resolution of 10 $\mu$m can also be used to detect cracks along concrete components such as bridge decks, tunnels, and sluice chambers. Apart from the rather high cost of the OFDR measurement devices, the fibres are cheap and the inspection outlay can be slashed by online monitoring.

If deformation or cracks have to be monitored in extended concrete structures that require sensor lengths of more than 2000 m, distributed sensors which use the Brillouin backscattering method provide long-term monitoring.

4.4 Quasi-distributed fibre sensor based on backscattering signal evaluation.
monitoring capabilities over lengths of up to several kilometres. One example of a distributed Brillouin sensor specially used for crack monitoring was described by Glisic et al.\textsuperscript{13} In general, detection of very small cracks, e.g. a crack width of about 150 $\mu$m, is challenging because they are normally invisible to the detection system. For the Brillouin-based measurement method in particular, the system must be made sensitive to cracks occurring over lengths shorter than the spatial resolution of the device used. In the applied fibre itself, high stress concentration is generated at the point where the crack occurs. Therefore, a matched sensor installation has to be carried out to ensure that the fibre survives the stress concentrations. Such demands on crack detection were put into practice on the concrete bridge Götaälven-bron in Gothenburg, Sweden over the river Gota. The development of advanced algorithms allowed the detection of events that occur over lengths shorter than one half of the spatial resolution. An appropriate installation procedure has been developed to allow controlled strain redistribution over a length compatible with algorithm requirements and sensor mechanical properties. The commercial SMARTape sensor from SMARTEC Company, Switzerland was used as the sensor cable.

If very large crack widths appear and have to be monitored, glass sensor cables could just break. In this case, polymer optical sensor fibres are used to obtain information about crack initiation, crack development and finally crack monitoring. Polymer optical fibres survive strain of up to 450 000 $\mu$m m$^{-1}$ (45\%) and can therefore be exploited, e.g. for crack monitoring on concrete structures. The measurement method is based on the previously described OTDR technique. A pulse will be launched into the fibre, and the time interval between launching the pulse and the return of the backscattered light (pulse response time) is measured. It depends linearly on the distance of the scattering location initiated by a local or integral deformation. Because the level of the backscattered light increases at locations where strain is applied to the polymer optical fibre, this effect can be exploited for detection of cracks and deformation. Investigation has shown that the sensor exhibits good repeatability of the strain response; maximum variances are equivalent to 0.5\% strain.\textsuperscript{14} However, as mechanical relaxation plays a role in polymers for such large strains, some measurement inaccuracies have to be considered when long-term processes are to be monitored.

### 4.4.2 Short-gauge-length sensors

Many examples of FBG sensor use for monitoring purposes have been presented within the past few years. The conference proceedings of the biennial SHMII Conference of the International Society for Structural Health Monitoring of Intelligent Infrastructure (ISHMII),\textsuperscript{15} or the Optical
Fibre Sensor Conference OFS with the same biennial frequency, are recommended reading. Numerous examples are given in the books cited in this chapter.\textsuperscript{1,3,16} Owing to lack of space, only a few examples of sensing methods for monitoring especially developed for and dedicated to concrete structures are given.

An atypical example of sensor use is the installation of an FBG to measure physical quantities such as humidity, using a moisture sensitive coating. Such a fibre-optic relative humidity (RH) sensor specifically designed for structural health monitoring is based on a polymer-coated FBG. With the increase in moisture level, the polymer coating swells and thus applies strain to the FBG, causing a Bragg wavelength shift. This practical humidity sensor is available in two different designs with varying robustness.\textsuperscript{17} The small version consists of a stainless-steel tube about 10 to 15 cm long, and having a diameter of 1 mm. The Bragg grating is positioned inside the tube at one end. Multiple holes are drilled there to allow moisture to enter and to react with the coating.

Figure 4.5 shows the design of the RH sensor. In order to evaluate accuracy and reproducibility of the sensor as well as its functionality for long-term monitoring, several tests have been carried out against reference humidity conditions. The sensor shows reasonably good precision and reproducibility in the range from 11.3% RH to 95% RH. There is a slow

![Image of fibre-optic RH probe](image_url)

4.5 Fibre-optic RH probe: (a) packaged probe; (b) layout of the Bragg grating sensors inside the tube (the second FBG compensates for temperature influences). PI, polyimide.
response time of about 2 h; however, this is not a problem for structural monitoring applications. The close correlation with the reference data makes this embeddable probe an acceptable solution for RH monitoring in concrete or other building materials. Another more robust design for versatile installation conditions in concrete structures is under investigation.

Very small compliant fibre-optic FPI sensors allow monitoring of micro-deformation processes in cementitious matrices. For instance, durability and strength of high-performance concrete is often reduced owing to micro-cracks occurring as a result of restrained shrinkage and temperature deformations at early ages. Embedded compliant fibre-optic microstrain sensors with low resistance reaction are able to measure even slightest deformations of the material. Because such sensors are not only able to evaluate the microstructure behaviour of mortar or concrete from the beginning of the hydration reaction, but also allow the assessment of restraining effects of reinforcement steel, they can be used to optimize such materials within the required specifications. Critical zones within the microstructure, e.g. in the vicinity of aggregates or other inclusions, can be monitored and investigated systematically. This helps to enhance the durability of high-performance concretes by using properly adjusted combinations of cement type, water–cement (w/c) ratio, additives and admixtures.

Figure 4.6 shows the tubular EFPI sensor described above placed in a silicon mould before casting. Such samples are used in laboratories to

4.6 Silicon mould with EFPI sensor before casting.
monitor and optimize cementitious materials. The sensor dimensions for
the measurement tasks described here are as usual: length 10 mm, outer
diameter 0.35 mm. The leading fibre in Fig. 4.6 links the sensor to the
recording instrument. The glass tube itself and the fibre near the tube were
not additionally protected so that they were entirely enclosed by the cement
paste. In order to achieve reliable bonding between sensor and matrix as
soon as it sets with a minimum uncertainty, small ceramic eyelets (inner
diameter 0.3 mm, outer diameter 2 mm, thickness 0.25 mm) were fixed at
the end of the tube, and at the leading fibre just where it enters the tube.
The distance between the eyelets defines the gauge length of the sensor.
Thus, the sensor is able to measure deformation immediately after finishing
the casting. Figure 4.7 shows a micro-section of a fibre FPI sensor embed-
ded in cement mortar. It can easily be seen that the glass tube is entirely
enclosed by the cementitious matrix material. Extensive investigations have
confirmed that optical fibres embedded in the highly alkaline environment
of concrete are durable for years.\textsuperscript{18,19}

The monitoring range in terms of axial displacement depends on the
distance between both displacement measurement eyelets. Usually, it is in
a range from $-2500$ $\mu$m m$^{-1}$ to $+5000$ $\mu$m m$^{-1}$. The resolution is about
0.1 $\mu$m m$^{-1}$.

Figure 4.8 shows possible monitored data for cement paste with a $w/c$ of
0.3. The FPI sensors were placed in the middle of the mould. Deformations

4.7 Micro-section of a carbon-coated fibre Fabry–Perot interferometer
sensor embedded in cement mortar (grain size 0–4 mm, curing period
8 months).\textsuperscript{20} (The sliding optical fibre is positioned inside the glass
tube bonded to the cementitious matrix material.)
were simultaneously measured and referenced by a contactless measuring laser vibrometer. The measuring point for the laser beam was a steel plate at the front end of the specimen. The steel plate was anchored to the cement paste.

It is known that shrinkage of concrete is significantly lower than shrinkage of cement paste. The resulting deformation of reinforced concrete, or the difference in deformation of the cement matrix near to and at a distance from inclusions such as steel bar, cannot be easily estimated. In order to clarify the magnitude of the matrix deformations in the vicinity of inclusions, additional EFPI sensors were placed directly alongside a steel bar. EFPI sensors for comparison purposes were positioned in the same specimen at such positions where no restraining effects are expected. Figure 4.9 shows that the steel bar has a restraining influence on the deformation, particularly during the period of high deformation rate. The choice of different steel bars, ribbed and plain ones, with various diameters, allows variation of the steel-to-concrete bonding. Thus, the degree of perturbation of the cement matrix can be varied.

In order to estimate the reliability and repeatability of the results, measurements during the first few hours in particular have been directly validated by using computed tomography (x-ray tomographic technology). The described use of FPI sensors for monitoring in laboratories requires some expertise to embed the filigree sensors without damage. However, FPI sensors can also be used in real concrete components when they are attached to sensor bodies. The following example shows the use of robust fibre-optic microstrain sensors for assessment and long-term monitoring of large con-
Whenever the integrity and ultimate bearing capacity of large concrete piles in existing or newly constructed foundations have to be assessed, sensors are usually attached to the reachable part of the concrete component, such as the pile head. Propagating waves are then generated by an impact at the pile head. The sensors at the head record the acoustic emission (AE) signals from the concrete structure. The bearing capacity and the pile performance can then be estimated using the one-dimensional theory of wave propagation. For cohesive soil areas in particular, one can only make vague assumptions about pile–soil interaction as well as material and subsoil conditions. The commonly used instrumentation at the pile head often does not provide sufficient information. The performance analysis of large concrete piles can be improved by using high-resolution fibre-optic AE sensors based on Fabry–Perot technology. Fibre FPI sensor elements enable the recording of dynamic signals up to the range of several hundred kHz. This method provides more precise information about the pile response over the whole pile length, especially using dynamic pile test methods.

A special sensor body onto which the sensing element is attached has been developed. The design of the sensor body had to be matched to the concrete mixture (aggregate size) and to the dynamic loading conditions of the pile dimensions. Figure 4.10 shows the reinforcement cage of one 19 m long concrete pile in which the sensors were fixed before pouring the

4.9 The restraining effect of a steel bar measured by an EFPI sensor close to the steel bar (sensor distance 5 mm) (upper curve); the unrestrained deformation at the reference position (bottom curve). (Marginally measured deformation before setting can be explained by slight inclines of the sensors.)
The number of sensors and their locations were defined depending on the soil stratification. Two FPI sensors and three resistive strain gauges were attached to each sensor body for comparison purposes.

Figure 4.11a shows one of the sensor-equipped piles during erection; Fig. 4.11b shows the piles, after they have been driven into soil. All sensors survived the high impact energy during the driving process. This is essential for using structure-integrated sensors for long-term monitoring of piles after in situ concreting and/or driving in, and, finally, after loading.

Field testing of this new monitoring technology confirmed several important features.

- The fibre-optic AE sensor can be used for low-strain and high-strain dynamic pile tests as well as for static load tests, i.e. it provides data for all of the three different pile test methods with only one measuring system. (Previously, several measurement systems were used for different loading tests.)
- During pile integrity tests (PIT) in particular, fibre-optic FPI sensors record very small deformations inside the concrete structure and, using
Structural health monitoring systems

a number of embedded sensors distributed along the pile length at defined locations, PIT allows the improved determination of material properties by calculating the wave velocity of different pile sections separately.

Close correlation in the results for both measurement systems was achieved during field tests. More details have been given by Schallert et al.\textsuperscript{22,23} The sensor system is being transferred into a product for commercial use and should be available from mid-2010.\textsuperscript{24}

4.4.3 Fibre-optic point sensor

The final example concerns a sensor much in demand in steel-reinforced concrete structures: a corrosion warning sensor. Steel-reinforced concrete structures such as sewer pipes, cooling towers or rock anchors are often exposed to a wide variety of damaging influences. Aside from mechanical stress, corrosion of steel is one of the most prominent damaging processes in steel-reinforced concrete. It presents a safety risk to people and
Environment because failure can occur without prior indication. In addition to the concentrations of moisture and chloride ions, pH is a chemical parameter of major importance in health monitoring of steel-reinforced and pre-stressed concrete structures. The lifetime of steel-reinforced concrete structures depends heavily on their pH, as embedded steels in concrete structures are only passive at a pH higher than about 9. For this reason, long-term monitoring of pH in the range from pH 9 to 13 with a resolution of about 0.5 pH units is relevant for early detection of a potential corrosion condition.

Commercially available structure-integrated sensors for early detection of steel corrosion in concrete structures do not always sufficiently match the in situ requirements. Fibre-optic sensors are a promising technology for corrosion monitoring because they offer a large number of attractive features, such as small size, flexibility, geometric versatility, resistance in corrosive and hazardous environments, no signal interference owing to the presence of moisture, an in situ and non-destructive measurement method, and, last but not least, immunity against lightning strikes.

In order to draw benefit from these advantages, a concrete-embeddable, long-term stable fibre-optic pH sensor has been developed. The most challenging requirements to meet in its development concerned its long-term stability under strong alkaline conditions at pH 9–13 over a period of at least 25 years. The resulting sensor can be integrated in harsh environments and inaccessible places.

The new fibre-optic pH sensor consists of a pH-sensitive layer made of a pH indicator immobilized in a solid substrate. In order to overcome instability problems resulting from a decrease in the indicator concentration (owing to photodegradation or leaching out), and problems resulting from drifts of the light source intensity or bending of the leading optical fibres, a ratiometric absorption method is used. This method also ensures reliable results. It is based on use of the ratio between the intensity at two different wavelengths, e.g. at the maximum intensity points or at the isosbestic point. Such a ratio of intensities is not altered by external factors. The measurement principle, along with some more details, is described by Dantan et al.

Figure 4.12 shows the design of the sensor head. The diameter of the head is 8 mm; the sensitive layer is protected but sufficiently sensitive for contact with the concrete matrix. Extensive investigations confirmed that the sensing element must have direct contact with the concrete matrix to detect the pH changes. The sensitive membrane must not exceed a particular thickness because the ion diffusion would be hindered. In order to ensure a reliably stable thickness of membrane, a special powder-compacting tool for the manufacturing of pH-sensitive membranes was developed and is used.
The measurement resolution of the sensor for pH values between 9 and 12 is in the range from 0.1 to 0.6 pH units depending on the pH value. The highest resolution can be achieved in the middle of the measurement range (between 9.7 and 11). One particular condition is that the pH-sensitive membrane must not dry out. This requirement is mostly fulfilled by hydraulic engineering and geotechnical engineering and engineering geology. In order to prevent drying out before integration into concrete structures, the pH-sensitive membrane is protected by a small watertight topcoat (Fig. 4.13).

Suitability of this sensor has been successfully tested for 5 years (continuously since the beginning of summer 2005) in steel anchors installed in the harbour at Rostock (northern Germany). Figure 4.14 shows two of ten pH sensor prototypes that have been fixed on prefabricated anchor bodies (two in each anchor) before inserting them into the borehole. The topcoat (Fig. 4.13) was removed shortly before inserting the prefabricated anchor into

4.12 pH sensor head design; the head is removable and can be replaced. The colour of the sensitive membrane will change depending on the pH.

4.13 Water-filled topcoat to prevent dehydration of the pH-sensitive membrane. Photograph: Stump GmbH.
the borehole and before grouting. This procedure ensured that the membrane maintained its hydrophilic properties.

The fibre-optic pH sensor provides useful information about the pH value of the cementitious material. It monitors any changes in pH value which one would like to see for an early detection of potential danger of corrosion in the steel-reinforced concrete structure. The first commercial sensor probes became available in autumn 2009. Because multiple-sensor designs can be fabricated, the price of pH sensors can be kept low.

New sensor types are also available that detect corrosion in a different way from the approach described above. One approach is a corrosion sensor for concrete structures based on an iron coating that is sputtered on the end of a cleaved optical fibre. This iron coating is then packaged and placed inside the concrete structure. Light is sent to the fibre end and the reflected power is monitored. Once the environment becomes corrosive, owing to chloride penetration or carbonation, the iron coating is removed, and the reflected power shows a significant drop. This type of sensor was installed in a footbridge in the southern part of Hong Kong Island, adjacent to a beach and facing the sea. Under high tide conditions, the piers of the footbridge are partially submerged in seawater. Also, wind will carry chloride ions and moisture to the surfaces of the pier directly facing the sea. The footbridge was constructed about 30 years ago, and has shown significant deterioration such as cracking, concrete spalling and steel rusting. It was repaired several years ago but rust stains have re-appeared in some parts, indicating the recurrence of steel corrosion. The sensors have survived two typhoons within the monitoring period. Details of the sensor
design, fabrication, packaging, installation and data acquisition are described by Leung et al.\textsuperscript{28} The field trial demonstrated the robustness and the applicability of the corrosion sensor in an aggressive marine environment.

Another corrosion monitoring approach is to detect and predict the onset of steel corrosion in concrete with the SensCore system.\textsuperscript{29} It consists of sensors, which measure corrosion current, resistivity and temperature of data loggers, and a measurement hub that concentrates the data from several data loggers and transmits it to a central database, where it can be accessed by the authorized users. The sensors measure several parameters that are critical to evaluate the present and future risk of rebar corrosion in concrete. In particular, the corrosion current and the concrete humidity are measured at several depths between the concrete surface and the rebar depth, to analyse the progression of the corrosion front as well as to evaluate the performance of hydrophobic coatings or waterproof membranes. Combining these measurements, it becomes possible to follow the propagation of the conditions that allow initiation of corrosion, from the surface to the depth where the real rebars are installed. The sensors can simply be installed and transmit their data wirelessly to the measurement hub.

The SensCore system has been deployed on a number of structures, e.g. the I35 St Antony Falls Bridge in Minneapolis, USA, the St Gotthard street tunnel in Switzerland and the Colle Isarco Bridge in Italy. Details are described by Inaudi et al.\textsuperscript{29}

4.5 Reliability of structural health monitoring (SHM) systems and standardization

SHM systems have to work reliably and safely under the corresponding environmental conditions. For this purpose, not only must all components of a monitoring system be validated for use, but also the complex interactions in the monitoring system should be tested and validated under conditions close to reality. There have been outstanding activities in the development of SHM systems. Early applications to use and test long-term monitoring systems were developed many years ago for the assessment of the behaviour of the Berlin Westendbrücke (Westend bridge). A research project at the BAM Federal Institute for Materials Research and Testing dealt with the concept of a monitoring system for the assessment of existing structures, and its implementation. This activity enabled, in addition to its main purpose of appraisal of the bridge and standardization, the assessment of the functionality, stability and long-term reliability of long-term monitoring systems under real environmental conditions.\textsuperscript{30}
Another recent, comprehensively monitored bold structure is the Berlin Central Railway Station. Because of the problematic nature of soil in Berlin, being mostly sand, this famous building, made from concrete, steel and glass, needed a complex monitoring system to detect critical states. Such long-term monitoring systems must work reliably over a long period of time and withstand all operational and environmental influences. For this reason, accompanying model tests have been carried out. The monitoring concept included equipping two large concrete beam models with sensors, which were subjected to defined climate conditions (inside the testing laboratory and outside) as well as to current load situations (pre-stressing, bending, deflection). These accompanying long-term tests enabled an excellent evaluation and validation of the long-term stability of the sensors applied.

Usually, sensor elements or sensor probes containing diverse sensors are the weakest link in the chain of complex monitoring systems. Therefore, particular attention should be paid to the long-term stable function of these sensors, if new innovative sensors have been chosen for long-term monitoring. When using established conventional sensors such as resistive strain gauges, extensometers or accelerometers, guidelines and standards are available. Users interested in seeking specific performance information about innovative sensors are not always able to find reliable information about their features from standards or established guidelines. A number of issues are raised. Important reliability aspects for the use of fibre-optic sensors, exemplified by FBG sensors, are comprehensively discussed in a chapter of the *Encyclopaedia of Structural Health Monitoring*.

Behind the physical components of a SHM system are hidden a multitude of aspects and parameters which influence the performance (reliability) of the sensor signal and the stability of the system components. Most of the commercially available fibre-optic components (connectors, connecting cables, FBG elements, reading devices) are validated according to standard test procedures developed for data communication but not for sensing purposes. As a minimum, functional tests have been carried out according to each company’s own rules. The calibration curve for the sensing element and/or the completed (applied) sensor sample must be made available. Additional problems arise when a read-out device used to record a signal from an FBG sensor on-site has to be exchanged. The measurement uncertainty then often exceeds the acceptable level.

When using a sensor system for long-term monitoring, not only do the influences of temperature on all system components have to be considered but also the specification under overall climate conditions. Primarily, the component’s behaviour under reference climate conditions must be understood. The following list provides an example of some of the important technical features and characteristic quantities that have to be proven for deformation sensors at reference climate:
• optical characteristics (i.e. attenuation, reflection coefficient and pulse characteristics, e.g. for FBG),
• deformation sensitivity,
• transverse (lateral) sensitivity,
• ultimate deformability (axial strain, pressure, bending) of fibre sensing area,
• mechanical hysteresis,
• fatigue behaviour,
• influence of thermal and mechanical changes along the leading fibre of sensor signal,
• ultimate acceptable optical power at reference climate,

as well as in general terms:
• temperature resistance of the whole system,
• temperature characteristics of sensor material \( \alpha_{\text{Sensor}}(T), \alpha_{\text{Sensor}}(T) - \alpha_{\text{Material}}(T) \),
• temperature characteristics of optical signal \( \partial n/\partial T \), which can be considered as temperature-correlated strain in strain sensors,
• influence of local temperature changes on sensor signal,
• temperature characteristics of sensitivity of deformation \( \partial S_n/\partial T \); thermal hysteresis.

In order to be included in reliable monitoring systems, all components have to be validated according to standard ISO/IEC 17025:2000. Validated systems enable consulting engineers, suppliers and users to evaluate suitability and reliability of the measurement system for its specific use. The key characteristics: accuracy, long-term stability and reliable function under environmental influences, have to be proven by validation, depending on the client’s needs. Validation is always a balance between cost, risks and technical possibilities.

Procedures for validation of the sensor system components include:

• determination of sensor-related characteristics (measurement range, resolution, and sensitivity are the most important);
• estimation of measurement uncertainty, repeatability and reproducibility of data from components of the measurement system;
• proof of stability of the system components’ characteristics.

The first determining steps on the way to reliable use of innovative fibre-optic sensors have been taken with the development by expert committees of guidelines for the use of fibre-optic strain sensors. On the one hand, general fibre-optic sensor-related terms and important aspects such as functional parameters, response characteristics and quantities of random nature have been defined in a European expert committee within the COST 299
action. On the other hand, the first guideline for the use of FBG-based strain sensors with the title ‘Experimental stress analysis – optical strain sensor based on fibre Bragg grating; basics, characteristics and its testing’ has been developed by the Expert Committee 2.17 ‘Fibre optic sensors’ of the Association of German Engineers (VDI), and was published by Beuth. This VDI/VDE guideline 2660 covers the most important terms and proof test conditions for manufacturers and users alike. Because worldwide, standardized sensor and application procedures are needed, this guideline should be accepted as a basic document for other groups working on or interested in this standardization, on which further standards can be built. Both guidelines aim to enable better understanding of the characteristics and performance of fibre-optic sensors and help to enhance the stability and reliability of fibre-optic sensor systems.

The Association of German Engineers (VDI) has published a number of guidelines for supporting the industry with basic knowledge, suggestions and support for the efficient use and application of sensor technology. The most well-known sensor guidelines are VDI/VDE/GESA 2635, part 1 and part 2 for the use of resistive strain gauges. Similar to these guidelines are the guidelines ASTM E251 from the American Society for Testing and Materials as well as the NAS 942 from the National Standards Association.

The RILEM (Réunion Internationale des Laboratoires et Experts des Matériaux, Systèmes de Construction et Ouvrages) founded a new Technical Committee, TC OFS (Optical fibre sensors for civil engineering applications), in 2005 to follow the increased demand on fibre-optic sensors. The following deliverables are being produced:

- a state-of-the-art report on fibre-optic sensing,
- a document of case studies on the application of fibre-optic sensors in civil engineering,
- an application guideline for fibre-optic sensors,
- a website for easy access of information and continued update on the status of the technology.

4.6 **Future trends**

Amongst the established monitoring activities, additional activities in SHM will be focused more and more on non-mechanical features of structures. In steel-reinforced and pre-stressed concrete structures, monitoring of pH and of the (movable) chloride content is very important to recognize corrosion-promoting conditions. By being able to identify corrosion-promoting conditions, it can be estimated when corrosion will start at the steel-
reinforcing components. This problem has great importance in a large number of structures and environments, where visual inspection is difficult or impossible. The pH sensor probe as a basic module can be modified for different measurement tasks. For example, it can be designed crest-like and stepped, consisting of several tiny sensor probes so that the movement of the carbonation front in a concrete component can be observed.

New and innovative sensor principles will be adopted for monitoring concrete structures. For example, the tiny fibre-optic sensor probes for measurement of humidity in concrete will be used to measure and evaluate humidity conditions in critical areas of various types of structure. Such a sensor probe is temperature-compensated so that temperature variations do not interfere with the humidity response.

Fibre-optic sensors based on sapphire crystal fibres can be used for temperature and strain measurements at 1200 °C and above. This opportunity should be exploited in the future to evaluate the deformation behaviour of concrete structures in case of fire. However, some research is necessary to align such crystal-based fibre-optic sensors to the harsh concrete environment.

Future structural health monitoring systems will differ from conventional observation systems, which are based on a set of sensors and a flood of information over a long period of measurement. Monitoring systems will deliver complex signal information which has to be evaluated, filtered and compressed. Specific acquisition algorithms and classification techniques are required to get usable information about the amount and the location of damage until an assessment of the structure’s residual life can take place. This is especially important if wireless data transmission requires efficient data processing on a low power budget, e.g. for on-site use. Corresponding developments in this field are being made at several research centres worldwide. Such activities aim for the development of efficient methods for load monitoring, regulation and control, reliable sensor technologies including intelligent signal processing, wireless data communication, and innovative component design with integrated flexible sensor networks (sensor-web) based on highly integrated sensors such as MEMS (micro-electro-mechanical system).

4.7 References


35 http://www.vdi.de/7636.0.html?&L=1.
38 http://www.rilem.net/tcDetails.php?tc=OFS.
Combining the results of various non-destructive evaluation techniques for reinforced concrete: data fusion

C. MAIERHOFER, C. KOHL and J. WÖSTMANN, BAM Federal Institute for Materials Research and Testing, Germany

Abstract: For the investigation of various testing problems on reinforced concrete bridges, which are a major part of the ageing infrastructure, efficient combinations of non-destructive testing (NDT) methods are urgently required. Among these, data fusion has been developed rapidly during the past few years enabling an efficient understanding and interpretation of complementary and redundant data. It is demonstrated how the pixel-wise fusion of data sets recorded on-site by various methods along the same area of structural concrete elements (concrete bridges) enhances the information content in the fused data set. The varying approaches employed by data fusion algorithms are discussed. The results of these investigations show the high potential of reconstruction and data fusion for the improvement and simplification of the analysis and visualisation of large data sets measured with impulse–echo methods.

Key words: non-destructive testing, concrete, data fusion, radar, ultrasonic techniques.

5.1 Introduction

The comprehensive assessment of concrete structures should be based on non-destructive testing (NDT), minor destructive testing (MDT) and destructive testing methods for gathering information about the structural and material condition. Depending on the particular situation and problem, NDT and MDT are useful for a first survey of large areas at the beginning of investigation. It is then possible to investigate selected parts with higher precision. These techniques can also be applied for ongoing observations (monitoring) or can be used for quality assurance during construction and after repair interventions.

In recent years, commercial systems or specifically developed systems such as ground penetrating radar (GPR), ultrasonic, impact–echo, sonic, and acoustic emission techniques, magnetic methods, potential measurement
methods, laminography, and active thermography have been applied to NDT and MDT of concrete structures. In most cases, however, only one of these methods has been used, usually executing only single-point measurements or two-dimensional (2D) profiles. Often the results of these investigations were difficult to interpret and the data were analysed only qualitatively.

The interpretation as well as the reliability of NDT data can be enhanced by combination of the different methods. This can be achieved in two different ways:

- selected methods can be calibrated and validated with reference methods or
- distinct methods or different sensors of one method can be combined to deliver complementary as well as redundant information.

The ongoing automation of measurement procedures enhances the efficiency of tests and the accuracy of the positioning of sensors. This enables repeatable measurements to be obtained with different sensors at the same location and a subsequent data fusion of these results.

In data fusion, single data sets of the same volume can be merged by various mathematical operations into one fused data set containing all essential information for the user. Data fusion can be applied for data recorded with the same method but different sensor configurations (e.g. polarisation, frequency), and/or for complementary data sets as e.g. ultrasonic echo and radar to improve the information content of structures in the specific volume and to enlarge the overall depth range.

For the fusion of different data sets recorded in the same slice (2D) or volume (three-dimensional (3D)), various mathematical operations are available. Before data fusion, several data processing steps are required to adapt the data sets. An expansion of simple data fusion by the classification of positive and negative evidence following the principle of the Dempster–Shafer theory affords a space-resolved assessment of 3D data sets of single and fused data sets. This provides a quantitative comparison of the reliability of single methods.

In the following, examples of the application of combined NDT methods and of data fusion will be presented.

### 5.2 Combination of non-destructive testing (NDT) and minor destructive testing (MDT) methods

In Table 5.1, an overview of selected useful combinations of methods for the investigation of concrete structures is given, including references. Very often, radar and acoustic methods are combined because they are complementary related to the sensitivity to reinforcement (metal), moisture, and
### Table 5.1 List of useful combinations of methods for the investigation of concrete structures

<table>
<thead>
<tr>
<th>Combination of methods</th>
<th>Complementarities</th>
<th>Examples of testing problems</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Radar and acoustic methods</td>
<td>Radar is very sensitive to the detection of metal reinforcement and to moisture.</td>
<td>Location of rebars and ducts, location of voids inside ducts. Determination of the thickness</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>Ultrasonic waves are mainly reflected at air gaps and voids</td>
<td>of concrete layers and floors. Location of honeycombing</td>
<td></td>
</tr>
<tr>
<td>Radar and magnetic methods</td>
<td>Radar is sensitive to the detection of reinforcement. Magnetic methods detect</td>
<td>Location of cracks in prestressed tendon ducts</td>
<td>9</td>
</tr>
<tr>
<td></td>
<td>cracks of the reinforcement</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ultrasonics and impact–echo</td>
<td>Ultrasonics have a higher frequency, different kind of data analysis</td>
<td>Location of vaults in grouted tendon ducts</td>
<td>10</td>
</tr>
<tr>
<td>Radar and active thermography</td>
<td>Different penetration depths (thermography is surface sensitive, radar has a</td>
<td>Internal structure, location of voids near rebars: close to the surface with thermography,</td>
<td>11</td>
</tr>
<tr>
<td></td>
<td>higher penetration depth</td>
<td>deeper structures with radar</td>
<td></td>
</tr>
<tr>
<td>Radar and electrical and capacity</td>
<td>Radar is sensitive to reinforcement, moisture and salt content. Electrical and</td>
<td>Investigation of moisture and salt content, penetration depth, material properties</td>
<td>12</td>
</tr>
<tr>
<td>methods</td>
<td>capacity methods are sensitive to moisture and salt content</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Radar (ultrasonics/sonics) and videoscy</td>
<td>Enhanced depth resolution and visualisation of internal structure with endoscopy,</td>
<td>Internal structure, videoscopy can be used as a reference method, e.g. for the depth of</td>
<td>13</td>
</tr>
<tr>
<td></td>
<td>but only at selected local positions</td>
<td>rebars, small voids or for the internal inspection of tendon ducts</td>
<td></td>
</tr>
</tbody>
</table>
air gaps (voids). Another combination is active thermography with radar or acoustic methods, to achieve the advantages of different penetration depths. In most cases, radar is ideally suited for the location of reinforcement.

5.3 Data fusion

5.3.1 Terms/definitions

The combination of methods, the overlay of data, and data fusion are three different terms that are often applied in different ways describing dissimilar steps of data processing. In this chapter, the following definitions will be used:

- The combination of methods is the lowest level and means the comparison of different data sets, where the single data sets are not necessarily modified.
- For the overlay (or crossfade) of different data sets, e.g. of a thermogram and a digital photo, both data sets have to be transformed into one coordinate system (image registration), but no pixel mixing occurs and only the opacity of the different data sets is modified.
- Data fusion is defined as the processing, interpretation and use of data from different sources. Generally, a new data set is generated based on single data sets from different sensors or methods. This new data set includes more information than the single data sets regarding selected criteria, e.g. reflections from an inhomogeneity.

5.3.2 Pixel-level data fusion

As described by Krzysztofowicz, data fusion can be performed at various stages of data processing:

- fusion of raw data,
- fusion after data processing (e.g. filtering or reconstruction),
- fusion of probabilities of the single data sets.

In this chapter, data fusion is performed after signal processing on a pixel level applying various basic mathematical algorithms as shown in Fig. 5.1.

In this instance, for data fusion, radar and ultrasonic techniques have been combined, but further methods can be implemented. Data sets recorded in the same volume with one method and various configurations (e.g. sensors and polarisation) and with different methods have been fused applying various mathematical operations.

Before data fusion, several data-processing steps are required to adapt the data sets. For example, weighting of the single data sets is possible with...
Combining the results of various NDE techniques

constant, linear and non-linear functions, e.g. for considering different reliabilities of the methods. In addition, signals with low signal/noise ratio and/or dc offset can be filtered. If data have been recorded with different density of data points, missing or needless data points can be interpolated or deleted, respectively, if this is a constraint for the data-fusion algorithm.

In B-scans recorded either by radar (radargrams) or ultrasonic techniques, a small reflector appears as a hyperbola caused by the opening angle of transmitter and receiver. As these patterns are different for different sensors, a reconstruction before data fusion is advised. As described in chapter 8, reconstruction algorithms based on the synthetic aperture focusing technique (SAFT) can be applied, taking into account the final size of the antenna aperture. These create 2D or 3D data sets in which the reflectors are focused in their true positions. Furthermore the signal/noise ratio is improved. A pre-condition for the SAFT reconstruction is the knowledge of the velocity of propagation. The SAFT algorithms used for the measurements described in this chapter allow at present only calculations under the assumption of a constant velocity. If there is a velocity gradient with depth this can result in the wrong depth information being produced. Such a gradient can occur for example for inhomogeneous moisture distribution.

In this chapter, the radar data were reconstructed with the FT SAFT (Fourier transform SAFT) algorithm based on the Stolt migration. After processing, the various data sets can be added, subtracted, multiplied or divided. Additionally, it is possible to calculate the average of different data sets or to compare the data sets and include only the maximum amplitudes in the fused data. The decision about which data-fusion operation has to be used depends on the nature of the problem. For example, because it is only appropriate to add, subtract and divide complete data...
sets, the data sets must have the same size. The division and subtraction operations can be used to detect a temporal change in the object in different successive measurements. Incomplete data sets can be fused with the operation which only includes the maximum amplitudes. For the data presented here, in most cases only this last operation was used to enhance the advantages of the complementary information of the various configurations and methods.

A comparison and fusion of experimental data and theoretical data calculated by numerical modelling allows the solution of the ‘inverse problem’. Under certain conditions a quantification of voids, inhomogeneities and inclusions is possible.

5.4 Fusion of radar data

Below a concrete floor slab, the position of a strip foundation had to be determined. The thickness of the slab was unknown but assumed to be between 15 and 30 cm. Thus, in a first step, a radar trace with a length of 4.5 m was recorded with the 1.5 GHz antenna by using the survey wheel for positioning as displayed in Fig. 5.2. From these data, it is shown that the concrete cover of the first layer of reinforcement is at a depth of about 12 to 15 cm. A second layer of reinforcement can be detected at a depth of about 32 to 35 cm and it is assumed that the reflection from the bottom side of the slab is superimposed by these reflections.

At a lateral distance of about 120 to 130 cm, strong reflections can be seen just below the first layer of reinforcement, which appears at the

![Radargram (B-scan) of a trace with a length of 4.5 m recorded with the 1.5 GHz on top of a concrete floor slab for the location of strip foundations. The polarisation of the electric field was parallel to the crossed reinforcement for optimum detectability.](image)

© Woodhead Publishing Limited, 2010
assumed position of the strip foundations. A reflection from the bottom side of the strip foundation could not be detected. Thus, for a more detailed investigation, an area of about 1.65 m × 1.65 m above one of the strip foundations was analysed by using the scanner system together with the 1.5 GHz antenna. Two data sets were recorded with perpendicular directions of antenna movement and thus polarisation of the electric field. Both data sets were reconstructed and fused as described above by adding up of the amplitudes of the different data sets.

In Fig. 5.3, two reconstructed B-scans of traces parallel and perpendicular to the strip foundation are shown as well as a 3D view of the reconstructed and fused data sets after threshold suppression. In the B-scans, and especially in the 3D view, the strip foundation cannot be located directly, but the position is given by the additional reinforcement.

5.5 Fusion of radar and ultrasonic data recorded along a beam of a box girder bridge

For the investigation of the internal condition of tendon ducts, larger areas of a box girder concrete bridge were investigated in the frame of a comprehensive test programme. Radar and ultrasonic measurements were performed with an automated scanner system. In Fig. 5.4, the recording of radar data with the 1.5 GHz antenna is shown.
In the following, results of data recorded along an area of 10 m × 1.5 m are discussed. Figure 5.5 shows three C-scans, each from a depth of 14.5 cm with the following properties:

(a) is cut from a fused data set based on two reconstructed radar data sets which were recorded in two different polarisations of the linear dipole antenna with 1.5 GHz;
(b) is cut from a reconstructed ultrasonic data set, which was recorded with an ultrasonic transducer array (ACSYS) at 50 kHz;
(c) is cut from the fused data set, which was generated by summation of amplitudes.

In C-scan (a) recorded with radar, the tendon ducts on the left side can be detected more clearly than with ultrasonics (b). On the right side, which is the area of the coupling links, the tendon ducts can be observed with ultrasonics with a higher signal/noise ratio. Several reflections from further rebars are superimposing the signals from the tendon ducts recorded with radar. Ultrasonic waves are much less influenced by metal inclusions. Therefore, the fused data set in C-scan (c) is presenting both areas under optimum conditions.
Combining the results of various NDE techniques

5.6 Fusion of radar and ultrasonic data at a cross beam inside a box girder bridge

At one of the cross beams of a prestressed concrete bridge, a crack was recorded between the beam and the carriageway slab. Therefore, it was required to locate the exact position of the reinforcement inside the structure. Four areas were selected and radar and ultrasonic measurements were performed by using an automated scanner system. In the following, data recorded at the cross beam inside the box girder of the bridge are analysed. Radar measurements were carried out with the 1.5 GHz antenna using horizontal and vertical profiles with perpendicular polarisation each. Both data sets were reconstructed by means of FT SAFT separately; afterwards they were fused as described above. The ultrasonic data were recorded with a transducer array at 50 kHz (ACSYS) and were reconstructed by means of 3D SAFT. From the radar as well as from the ultrasonic data set, in Fig. 5.6 similar B-scans are presented. In the radar data, Fig. 5.6(a), the reinforcement bars at the surface can be detected very clearly whereas the reflection from an underneath tendon duct at a depth of approximately 25

5.5 (a) Radar C-scan at a depth of 14.5 cm. (b) Ultrasonic C-scan at a depth of 14.5 cm. (c) C-scan of the fused data set at the same depth.
to 30 cm appears only with low signal/noise ratio. In the ultrasonic B-scan Fig. 5.6(b), the rebars close to the surface cannot be resolved whereas the tendon duct is shown very clearly.

The fused radar data set and the ultrasonic data were fused by adding the amplitudes of each volume cell. It has to be considered that before data fusion the calibration of the velocity in each data set might be different depending on the on-site condition. Additionally, inhomogeneities inside the concrete might lead to different deviations of the velocity of electromagnetic waves and of acoustic waves, as both methods are based on different physical phenomena. As data fusion is only successful if the data set of one method is adjusted to the other one, here the data sets were adapted to achieve a common depth at the maximum amplitude of the reflection from the tendon duct in one specific point. In Fig. 5.7, the B-scans of the

\[\text{Data fusion results of ultrasonic and radar: (a) without velocity correction, (b) with velocity correction.}\]
Combining the results of various NDE techniques

fused data sets are shown before, Fig. 5.7(a), and after, Fig. 5.7(b), this correction of depth scale demonstrating an enhancement in spatial resolution.

5.7 Sources of further information and advice

There are two books available that are devoted to the concept of multisensor integration, data fusion and its applications to non-destructive testing.\textsuperscript{19,20} Basic knowledge related to data fusion is presented by Mitchell\textsuperscript{21} and mathematical techniques are described by Hall and McMullen.\textsuperscript{22}

More information about data fusion is presented on the Data Fusion Server (www.data-fusion.org). Several publications about NDT data fusion are available on the NDT Net (www.ndt.net).

5.8 Conclusions and future trends

In this chapter, the advantages of the combination of complementary NDT and MDT were discussed. Examples for pixel-wise data fusion of radar and ultrasonic data were presented. The fusion of radar data sets measured with two different directions of polarisation of the electric field vector leads to a result that is independent of the polarisation of the antennas. This is especially important if longish reflectors orientated in different directions (such as rebars and tendon ducts) have to be detected and visualised. The combination of reconstructed radar data sets with reconstructed ultrasonic echo data sets allow the compression of all important information in one data set and the compensation of the disadvantages of one method in comparison to another. A maximum of information is gained about those structures containing a high reinforcement density, tendon ducts and/or air voids and gaps. The tendons in the investigated building structures could be localised with the 1.5 GHz radar antenna at measurement depths up to 16 cm and with ultrasonic echo at measurement depths up to 40 cm. All the presented data used for data fusion were recorded with an automatic scanner system, which enhances the accuracy and reproducibility of sensor position and performs data acquisition more efficiently than manual data recording.

The fused data sets show that the combination and fusion of different sensors improve the reliability and efficiency of the investigations. The fused data can be presented with the same commercial visualisations programs as the single data sets. Before fusion, the physical principles, advantages and restrictions of each method must be known for a reliable interpretation of the fused data. Data processing steps before data fusion should be limited to those required for reducing the influence of sensor aperture, for aligning the resolution, and for conversion to a similar data format.
For the future, the actual pixel-level fusion algorithms will be expanded by evidence theories considering different reliabilities of various methods. Data from further methods (see Table 5.1) as well as from numerical modeling will be fused. On-site data fusion will be implemented into automated multisensory systems.

5.9 Acknowledgements

The investigations on the box girder bridge in Germany were realised in co-operation with the Federal Highway Research Institute in Germany and funded by the Federal Ministry of Transport, Building and Housing. Further parts of the work were funded by the German Research Council (Deutsche Forschungsgemeinschaft, DFG) via grant number FOR 384. The experimental work was supported by several members of the BAM Division VIII.2 and especially by M. Krause, F. Mielentz, B. Milmann and D. Streicher (ultrasonic echo 3D). The programs used for 3D-SAFT reconstruction were developed by W. Müller (Fraunhofer IZFP) and K. Mayer (University of Kassel).

5.10 References

4 KOHL, C. and STREICHER, D., ‘Results of reconstructed and fused NDT-data measured in the laboratory and on-site at bridges’, Cement and Concrete Composites, 28 (2006), 402–413.
9 KRAUSE, H.-J., WOLF, W., GLAAS, W., ZIMMERMANN, E., FALEY, M. I., SAWADE, G., MATTHEUS, R., NEUDERT, G., GAMPE, U., and KRIEGER, J., ‘SQUID array for mag-
Combining the results of various NDE techniques, Physica C: Superconductivity, 368 (2002), 91–95.


Wireless monitoring of reinforced concrete structures

M. KRÜGER, University of Stuttgart, Germany

Abstract: The use of wireless sensor systems for monitoring concrete structures in civil engineering is reviewed. Applications of monitoring systems already in use are described and analysis methods that are applicable to improve informative values in terms of serviceability, safety and remaining lifetime are identified. In addition to the fundamental concepts of wireless monitoring, further requirements and design steps for wireless monitoring systems are discussed. For structural health monitoring, it is not sufficient to focus on hardware solutions alone. It is the data processing that defines the benefit for the user (e.g. remaining lifetime, load-bearing capacity, serviceability). Data reduction, data analysis and data interpretation methods are essential to enable engineers to use more precisely information for structural analysis and repair, lifetime prediction, and maintenance. Some approaches to distributed computing strategies are described with respect to the limitations of wireless monitoring systems and areas requiring further research are identified.

Key words: wireless monitoring, wireless sensors, distributed computing strategy, intelligent monitoring, concrete structures, structural health monitoring.

6.1 Introduction

Wireless monitoring systems are often thought to have several advantages compared with wired monitoring systems, for example easy installation, cost-effectiveness and autonomous operation over longer periods providing remote control and analysis features. Therefore, many research and development activities are ongoing with regard to wireless monitoring systems in application to civil engineering structures such as bridges (Kim et al. 2007, Meyer et al. 2007a) as well as to historic structures (Grosse et al. 2008c). At first glance, continuous monitoring with wireless sensor networks seems to be a perfect solution to get more detailed information about structures than from visual inspection only. However, wireless monitoring is often not straightforward if the monitoring task is more complex than just acquiring and transferring relatively simple data such as temperature or humidity every hour. For such simple tasks many competitive solutions with adequate
reliability in the form of data loggers, also partly equipped with wireless communication, are commercially available.

The situation becomes challenging if the desired monitoring is focused on acquiring and analysing data such as stress, strain, inclination, and salt and moisture content inside materials or even vibration or acoustic emissions caused by fracture processes that require higher sampling rates. A main problem in this context is the power supply (mainly primary batteries are used) so that the wireless monitoring hardware and software are subject to several restrictions. To remain cost-effective and practicable, a balance between the monitoring task with respect to the expected result from the monitoring and the time and effort to perform the continuous monitoring must be found. This is why wireless monitoring systems mostly have to be customized for the desired monitoring objective. Thus, structural health monitoring is also to be seen as an interdisciplinary engineering task, which requires much further research and development.

### 6.2 Basic principles of wireless monitoring

Most of the wireless sensor networks under development consist of several multi-sensor nodes, called motes, and a minimum of one base station, which also could have an integrated modem (e.g. GPRS/UMTS) for internet connection and remote control. The wireless data transfer is conducted by using several sensor network topologies. A relatively simple topology is the star topology in which one mote acts as a central mote that just communicates directly with surrounding motes. Although the communication protocol is simple, the size of such a network is limited by the transmitting range of the used transceiver (approximately 20 to 100 m for most available systems).

A more advanced topology is a mesh network (or multihop-network, Fig. 6.1) in which every mote communicates with direct neighbour motes.

![Principle sketch of a multihop network with sensor clustering.](image-url)
Such networks are configured dynamically and also could reconfigure automatically if certain motes fail. With respect to power consumption, network robustness, and the possibility of building up big meshes, multihop-networks seem to be the best choice for monitoring large structures.

The motes are the main components of a wireless monitoring system. There are various tasks that a sensor mote has to perform: to collect and digitize data from different sensors, to store sensor data, to analyse data with simple algorithms, to send and receive selective and relevant data to and from other nodes as well as the central unit, and to work for an adequate time period without a wired power supply. Therefore, a sensor mote consists of a central processing unit (CPU) and/or digital signal processor (DSP) with sufficient memory, a low power radio, an aligned analogue-to-digital conversion module (ADC), a power supply and one or more diverse sensors. There are many different wireless sensors that have been developed by several researchers world-wide to be used for structural health monitoring (SHM). A comprehensive review of available wireless sensing units is given by Lynch and Loh (2006) that show the state-of-the-art at that time. However, many shortcomings especially with respect to reliability are obvious. The biggest problem is still the conflict between power consumption, storage capacity and system bandwidth. The system bandwidth is mainly restricted by the limited wireless communication throughput. That is why multihop network algorithms and mote clustering is the subject of the recent research (Gao 2005, Krüger et al. 2005, Lynch 2004, Lynch and Loh 2006, Ruiz-Sandoval 2004, and Wang 2007). Another drawback is the lack of adequate sensors especially with respect to sensitivity, reliability and robustness as well as their integration into a mote (Nagayama and Spencer 2007).

Although numerous commercial smart sensors are also available together with some application software from different companies (Dust Networks, Microstrain, Millenial Net, Sensametrics, Sensicast, and Testo), most of these sensor networks are just wireless data acquisition systems that only transmit measured raw data to a central base station for further processing. Further on most of the systems do not fulfil the requirements with respect to robustness, long-term stability or sensor reliability.

Because most available wireless monitoring systems suffer from shortcomings partly indicated above, further improvement of hardware and software is in the focus of many developers and researchers.

Figure 6.2 shows an example of an actual development. It shows a wireless sensor mote equipped with low-power microcontroller (TI MSP430), wireless transceiver (CC2420), primary batteries (Li-SOCl₂ type) and several interfaces (analog input, I²C, SPI) for attaching various types of internal or external sensors for multiple sensing. The hardware is optimized to work under harsh environmental conditions such as those found in struc-
tural health monitoring, and it supports several ultra-low power modes. Various types of sensors could be attached to the wireless mote simultaneously, that is various microelectromechanical systems (MEMS) sensors with digital output, e.g. for the acquisition of data on acceleration, temperature, humidity, inclination, or solar radiation. Additionally analog sensors such as resistive strain gauges or piezobased vibration sensors could be attached by using especially developed electric circuits for the signal conditioning. Owing to its modular concept, customization and optimization for specific monitoring objectives is supported.

Based on the principle flow-chart shown in Fig. 6.3, some of the most important aspects of customizing and using wireless technologies for structural health monitoring are discussed in the following sections.

6.3 Definition of the monitoring task

The definition of the monitoring objective at a specific structure is the first aspect an engineer has to deal with. This definition is based on the technical language of civil engineering and does not include constraints in terms of the measurement technology in detail. In most cases, continuous monitoring is aimed at:

- better understanding the behaviour of the structure in different exposure conditions and loads (performance monitoring),
- observing, characterizing, understanding or better predicting deterioration processes (structural health monitoring with the focus on maintenance), and/or
- safety reasons (health monitoring with the focus on safety and serviceability).
Depending on the monitoring task, various information should be collected before the monitoring system is designed and assembled. If structural problems are of interest, basic information might be the results from structural calculations, structural dimensions, material properties, or definition of weak points. In addition, the duration of the monitoring is of utmost importance because of the power restrictions that wireless sensor networks have. Starting from such elemental information, the monitoring concept should include information on:

6.3 Principle flow-chart for applying wireless structural health monitoring.
• accuracy, sensitivity and robustness of required sensors,
• environmental conditions and other exposures that might either influence the structure or material itself or, which is as important, might affect the reliability of the sensors or of the used measurement technologies.

One example of problematic environmental conditions is the measurement of the strain of materials like concrete or wood. Although the measurement of strain could be made seemingly easy by simply attaching a strain gauge to the surface, the actual total value of strain of a material such as concrete or wood is a time-dependent function mainly driven by temperature, moisture content and applied stress. In addition to that, the resistance of a strain gauge as well as the data acquisition unit is affected by temperature so that the precise determination of absolute strain has to take account of many different possible sources of error. Hence, the determination of absolute strain in changing environmental conditions requires in most cases the simultaneous acquisition and compensation of temperature and humidity, which makes it rather complex.

Accessibility of the structure with respect to sensor mounting is also an important site-specific factor that must be defined before an appropriate monitoring system could be chosen.

Finally, background information about any models that might be useful to analyse the monitoring data should be collected to avoid missing measurands and data during the monitoring, but also to keep the amount of data that has to be collected as small as needed.

6.4 Monitoring system design and assembly

For most desired monitoring objectives, system design and assembly is a task for measurement technology experts. This is mainly owing to the high degree of customization, the large variety of available and suitable sensors as well as to the further hardware and software restrictions wireless monitoring systems show. If the monitoring objective is simply the acquisition of air temperature and humidity, there are still some competitive commercial systems available that are readily applicable. However, such simple ‘environmental’ monitoring tasks do not effectively meet the terminology of structural monitoring. As shown in Fig. 6.3, system design and assembly could be broken down into three main steps: the data acquisition concept, the data analysis and distributed computing concept, and hardware selection. The determination of principles of data acquisition and handling is the first step of system design, whereas principles of data evaluation and inter-
pretation are included in the data analysis and distributed computing concept.

This differentiation is made because for the second step the desired monitoring objectives come to the fore, i.e. mainly analysis of the data by adequate and individual methods to be implemented into the software, in particular, the application software for achieving the monitoring objectives. The data acquisition concept is more generic, because it defines requirements for choosing the most appropriate hardware and software with respect to monitoring technology limitations that are already quite well known, such as limited bandwidth and wireless data transmitting range, limited calculation power and storage capacity, and limited power.

There is a correlation between the distributed computing concept and the data acquisition concept in which middleware plays a decisive role. Middleware is software that connects software components or applications. In the context of the multiprocessing in wireless sensor networks, middleware provides interoperability between the distributed applications running on each mote and on the base station. The middleware services are manifold. One basic function of middleware is the provision of reliable communication in combination with ad hoc communication and automatic reconfiguration of the network topology. The most important aspects in this context are reliable data transfer and time synchronization. Much research is ongoing to provide solutions for reliable data transfer and it is often mentioned that a certain loss of data has to be accepted if wireless communication is used and power consumption plays an additional major role. The data loss rate varies with the system setup and is also strongly influenced by the environment. However, the argumentation of accepting a certain amount of data loss is not clear. Some data loss might be acceptable for some applications, but if an important event has to be transmitted that was defined to be critical (e.g. alarm generation in case of structural failure), undefined packet loss rate is unacceptable, especially with respect to a proposed safety index. This means that the effect of data loss must be considered in the monitoring task and reliable communication protocols suitable for sending data with a defined allowable data loss rate must be used. The communication protocols itself could vary according to the significance of the data to be transmitted. As an example, event-based data acquisition and transmission requires more reliable protocols than discrete monitoring. In this context, a general problem is that data loss rate is strongly influenced by the environmental conditions. It is noticed in several applications that sometimes the data loss could be 100% for periods of several minutes up to hours or days and on the other hand the data loss rate could be almost zero for another period in the same system without having made any changes.
Time synchronization is another important function of middleware. Time synchronization is mainly needed for two purposes. One is the communication between the motes, which must be synchronized, because multiple motes can cause packet collision if they try to send data packets at the same time. A second purpose is the time synchronization of the measured data. Some monitoring tasks require high accuracy in time synchronization, for example the modal analysis of a structure or acoustic emission analysis techniques to analyse fracture processes. This aspect is often neglected, although protocols for adequate time synchronization have been investigated and developed by many different persons. Time synchronization with an accuracy of up to $10 \mu s$ between two motes is found to be obtainable, e.g. by using flooding time synchronization protocols (Maroti et al. 2004, Meyer et al. 2007a).

In addition to the provision of basic functionality for distributed computing, middleware may also contribute directly by providing services for collaborative distributed data processing. Owing to power considerations it is recommended that the data transfer through the radio module is minimized, which means that signal processing and data analysis should be done in a mote as far as possible. Simple aggregation functions such as MIN, MAX, and COUNT can be used for certain types of data in order to further minimize the amount of transmitted data. Advanced methods, which are partly presented below, also use regressive models that could reduce effectively the amount of data and also provide information with respect to any changes.

In addition to the local signal processing techniques running on a single mote, the information useful within mote clusters can be aggregated, further analysed in intermediate nodes and forwarded as needed in compound packets that could save a lot of energy. Such data aggregation and data reduction methods are also needed to minimize the data traffic that is essential if the monitoring system consists of a high amount of motes. First analysing and storing a set of data in a mote and then sending it consecutively through the radio module at specific time intervals, events or on request will also improve the reliability of data, because data transfer could be specifically controlled.

The desired analysis of data acquired within motes or mote clusters is also called ‘Distributed Computing Strategy’ (DCS), a field in which research activities are promising (Gao 2005, Meyer et al. 2007b). Some examples of DCS are given below. However, distributed computing strategies are software applications for very specific monitoring objectives for which reason adaptability and modularity of the methods play a major role in terms of making the system assembly feasible.

In conclusion, for practical monitoring system assembly, the monitoring objectives, the monitoring concepts and the system limitations both of
hardware and of software are closely connected. All these aspects must be considered for which structural health monitoring with wireless sensor networks is becoming an interdisciplinary engineering task.

6.5 Wireless monitoring systems in operation

Although the installation of a monitoring system is not very complicated, the initial operation of the monitoring system has to be made with care. Sensors must be attached correctly, then configured and tested, or even calibrated, to avoid unrecognized malfunction. With respect to the desired distributed computing strategy, reference data acquisition in combination with a data reliability check and model adjustment is crucial. Based on reference data and the implemented data analysis models, alarm levels or other thresholds should be defined so that user notification or interaction is supported. This is not only required with respect to the desired monitoring results, but also owing to potential malfunction of the monitoring system (e.g. sensor or mote failure, discharged battery) and the need for permanent self-monitoring. Although wireless monitoring is expected to operate autonomously for most of the time, system maintenance and user interaction is still needed. This is also true owing the fact that the overall analysis and interpretation of the monitored data always require expert knowledge and further decision making.

For convenience, in particular with respect to making changes and adjustments in the application software running on the motes, system maintenance should be assisted by remote control. This consists mainly of revision of the upper software layer and the object specific settings, e.g. model updating, data analysis algorithm updating, modification of alarm levels.

6.6 Application of intelligent wireless monitoring

As discussed above, wireless monitoring should be more than just acquiring diverse measurands at different locations on a structure and then storing it in a database. If the monitoring task and the expected result are well-considered, immediate processing of the data is recommended to avoid collecting large amounts of senseless data no one will look at afterwards. If such immediate data processing is considered, wireless monitoring becomes intelligent and of direct practical use. As already discussed, the distributed computing strategies, which include data analysis and reduction algorithms are of utmost importance. Some examples are briefly described in the following. Autoregressive models may be used to identify inter-relationship of different physical values in time series and to predict future behaviour. Therefore, they are well suited for identifying influences
e.g. from seasonal weather conditions, repeated loading or other phenomena that occur periodically. Autoregressive models are used by many researchers to fit measured time series data to stationary data that define an initial status, while, at the same time, not knowing which changes might occur in future. During further monitoring, this data is used to detect significant changes and to characterize these changes with respect to the monitoring objectives.

Much research is ongoing in monitoring structures with wireless sensors by analysing natural frequencies and their changes, which might give some information on possible damage (Gao 2005, Gao and Spencer 2007, Lynch 2002, Meyer et al. 2007b, Nagayama and Spencer 2007, Ruiz-Sandoval 2004, Wang 2007). A distributed computing strategy proposed by Gao (2005) shows a complete calculation procedure in the field of vibration analysis, i.e. natural excitation technique (NExT), eigensystem realization algorithm (ERA) or damage locating vector method (DLV). Nagayama and Spencer realized the implementation of these analysis procedures into the Imote2, a wireless mote equipped with an x-scale processor, a processor that has relatively high computational power and large memory compared with other available wireless sensor devices. The algorithms they employed on the wireless motes include resampling and synchronization of the measured data, correlation function estimations (cross-spectral density estimation, IFFT, averaging) and the numerical calculations (NExT, ERA and DLV) for damage identification. Although some parts of the calculations were made on each mote separately, the damage estimation calculation was executed in the cluster heads only. In laboratory tests at a truss structure equipped with a wireless network consisting of 19 motes and three cluster heads they showed that the total calculation time for running the analysis including debugging was approximately 20 minutes (at 100 MHz processor speed). Nagayama and Spencer also showed that the calculation time could be reduced to only a few minutes by optimizing the analysis procedure. This challenging example shows that data analysis in terms of a distributed computing strategy is quite promising but nevertheless is also limited.

Another computing strategy for wireless sensor networks with respect to natural frequencies was investigated by Meyer et al. (2007b). They implemented a stepwise procedure on an ultra low-power microcontroller (MSP430) starting with band pass filtering of the recorded data. For the estimation of the natural frequencies they used a simple discrete time autoregressive (AR) model with only two parameters. In a repetitive procedure with further averaging at the end of the procedure natural frequencies were estimated with sufficient accuracy. In field tests on a cable-stayed bridge, the natural frequencies of the cables were monitored with that technique successfully over a longer period (Meyer et al. 2007a).
The above-mentioned strategies are time-discrete concepts at which the data recording and data analysis is determined by the duty cycle for a particular time. The aspect of time-discrete monitoring becomes problematic if a critical short event occurs during the time the monitoring system is in sleep mode, thus it is not capable of recognizing this event. For such purposes, event-based monitoring becomes obligatory. Event-based monitoring is useful if temporary loads or other influences stress the structure, e.g. trains, trucks, wind, snow or rain, earthquakes or structural failure itself. That means that an object-specific event triggers the measurement progress. If such an event-based monitoring is chosen, it has to be considered that every measured value has to get its time stamp, because in this connection time is a parameter to be measured.

Some examples of event-based monitoring concepts have been reported by several researchers. A case study on which event-based monitoring was successfully tested was the detection of a train crossing the bridge (Krüger and Grosse 2008). The task was to measure the dynamic strain of steel girders while a train crossed a bridge. The train detection was conducted by using a MEMS vibration sensor on each mote that could be configured by software to trigger the system. The MEMS sensor provides a vibration detection mode while using little power. If a train crosses the bridge the vibration is recognized by the MEMS sensor, which then wakes up the microcontroller from sleep mode by using the interrupt function. After that the measurement procedure is started within a few milliseconds. The procedure was used to acquire dynamic strain data during the train crossing with a sampling rate of 100 samples s⁻¹ using resistive strain gauges. The collected data was first stored temporarily inside the mote and was transmitted to the base station consecutively after the train passed. This procedure was necessary to reduce the data loss rate.

One of the most challenging examples of event-based monitoring is the acoustic emission analysis, which is useful to detect and also to localize fracture processes. Qualitative acoustic emission analysis techniques often require very sensitive sensors and high-speed data acquisition systems, because the full wave forms are analysed. Owing to the hardware and software restrictions it is obvious that only certain quantitative acoustic emission analysis techniques could be implemented into a wireless sensor network. In terms of the acoustic emission analysis hit rate (relevant acoustic events per second) determination, beamforming techniques for localizing acoustic events as well as signal characterization and classification techniques were investigated and possible solutions for both hardware and software were discussed (Grosse et al. 2007, 2008a and 2008b, Krüger et al. 2006 and 2007a, 2007b, Meyer et al. 2007b). Although not all the mentioned concepts have fully been implemented into a mote and further
investigations are necessary, the concepts of acoustic emission data analysis in wireless sensor networks are promising.

6.7 Conclusions and future trends

Wireless sensor networks using intelligent data processing could enormously reduce the costs for structural health monitoring to just a few percentage of a conventional cabled monitoring system. This will increase their range of applications and thus more detailed information could be obtained from the structural behaviour as well as the actual condition of the building structure. This will enable engineers to use more precisely information for the structural analysis and repair as well as lifetime prediction. For that reason, first prototypes of wireless monitoring systems and promising distributed computing strategies were developed. The purpose of intelligent distributed computing strategies is manifold. They reduce the amount of data and thus become indispensable with respect to the restrictions of wireless sensor networks. In addition, they provide a certain interpretation of the raw data and are useful to identify the interrelationship between diverse influences that might cause deterioration of the monitored structure. By determination of alarm levels and implementation of permanent self-monitoring, intelligent monitoring systems could operate autonomously for most of the time. However, a lot of work has still to be done. Reliability, especially with respect to long-term monitoring, is still challenging and the high complexity of customizing and assembling monitoring systems is in contrast to their easy handling and usability. Thus, more practicable modular concepts must be developed so that structural health monitoring will be intrinsically more efficient and this will help to reduce maintenance costs.

6.8 References


7

Non-destructive testing of concrete with electromagnetic and acoustic–elastic waves: data analysis

K.-J. Sandmeier, Sandmeier Scientific Software, Germany

Abstract: The similarities of and differences between the data-processing techniques used for seismic and electromagnetic wave propagation data, will be summarized. The different acquisition modes and the resultant implications for data processing are listed. Some main standard processing techniques are shown and applied to different electromagnetic reflection data. Some more sophisticated processing tools such as deconvolution and migration are described in detail. A simulation method is used in order to examine the effectiveness of the methods.

Key words: data processing, ground penetrating radar (GPR), seismic wave, electromagnetic wave, filtering, migration, reflection, diffraction, deconvolution, noise.

7.1 Introduction

The physical base for electromagnetic and seismic wave data is very different. Nevertheless often the same data-processing techniques are used for both data types.

In this chapter, the similarities and section differences of seismic and electromagnetic wave propagation will be summarized (Section 7.2). The various acquisition modes and the resultant implications for the data processing are listed. In Section 7.3 some of the main standard processing techniques are outlined and applied to different electromagnetic reflection data. In Section 7.4, more sophisticated processing tools such as deconvolution and migration are investigated. A simulation method is used in order to examine the effectiveness of the methods.

7.2 Similarities and differences between seismic, ultrasonic and electromagnetic wave propagation and their implications on data processing

The principal goal of data processing is to present an image that can be optimally interpreted. Seismic, ultrasonic and electromagnetic data are all
based on wave propagation and at first glance the data seem to be very similar. The physical basis of seismic and ultrasonic wave propagation is the same: the elastodynamic wave equation. These waves differ significantly in the omitted signal frequency. The electromagnetic wave data are based on the Maxwell equations. Under several assumptions (constant magnetic permeability, frequency independent parameters), the electromagnetic wave equations can be transformed into one equation. A similar approach is given for the seismic case, when only considering the compressional modules and the pressure, which leads to the acoustic wave equation.

These two equations exhibit a very similar form. As the conditions for their derivation often nearly hold true in reality, it seems to be a good approximation to apply similar data-processing techniques to all three types of wave data.

The main differences are in the different range of the relevant physical parameters and in the different source types. Whereas the compressional and shear wave velocity mainly dominate seismic and ultrasonic wave propagation, the dielectric constant and electrical conductivity are the dominant parameters that control electromagnetic wave propagation. The other parameters normally do not play a very significant role. The spatial distribution of these parameters underground is the main control of the necessary processing steps.

Because of the similarities between electromagnetic and elastic reflection data, the following processing description will be restricted to the case of the electromagnetic wave propagation. The two main acquisition modes are the reflection and the transmission mode. Whereas for the reflection mode, the source and receiver are located within the same acquisition plane or even at the same place, for the transmission mode, the source and receiver normally are situated on opposite planes in order to acquire transmitted waves. Each data type requires a different data-processing technique. Whereas within the transmission mode the detection of the first arrival is of most importance, the complete wavefield will be interpreted for the reflection mode. As the transmission mode normally only needs a few data-processing steps that must be used very carefully, the following chapter will be restricted to the data processing of reflection data.

There exist many different geometry configurations of the source and the receiver(s). Seismic data and sometimes also electromagnetic data will often be acquired using a receiver array. Such an array configuration needs some pre-processing such as normal moveout correction (NMO correction) and stacking. The NMO correction is necessary in order to remove the influence of the offset between source and receiver. For electromagnetic data the most common geometry configuration is the use of source and
receiver at or nearly at the same place, the so-called zero offsets (ZO) section. Here, we only consider ZO ground penetrating radar (GPR) data but many of the methods described can also be used for the other source–receiver configurations. The main processing steps are introduced and applied on some example data but no mathematical details of the methods are given. More detailed consideration of these mathematical aspects has been reported for seismic (Yilmaz, 1987, Mari et al., 1999, Kanasewich, 1981, Sheriff et al., 1995) and for GPR data (Daniels, 2007).

7.3 Standard data processing

The main purposes of data processing are:

- to increase the signal to noise ratio (e.g. stacking, bandpass filtering, and averaging),
- to remove system-induced irregularities (e.g. background removal and static correction), and
- to correct geometrical effects caused by the data acquisition (e.g. migration).

Data processing can be classified as following:

- A-scan processing, where the filter acts on each trace independently and
- B-scan processing, where the filter will be applied on the complete B-scan (2D) and involves all traces or a part of them. If a complete equi-distant 3D dataset exists the processing may not be restricted to one direction but may include data acquired within the complete xy plane.

The mathematical background of the following standard data-processing methods is quite straightforward. The main problems consist in the best possible adaptation of the methods to the type of data used and in the best possible use of the necessary parameters.

7.3.1 Dewowing and standard bandpass filtering

Many GPR data show a significant very low frequency component either owing to inductive phenomena or to possible instrumentation restrictions. This low frequency range must be removed before applying any other digital filter algorithms. This can be done in many different ways. A simple dewow filter acts within the time domain. A running mean value is calculated for each value of each trace. This running mean is subtracted from the central point. As a filter parameter, the time range for the calculation of the running mean value must be entered, and this should be set to about one or two principal periods. A possible static shift will also be removed using this filter.
Alternatives to the dewow filter may be a high-pass bandpass filter working either within the frequency or time domain or a simple subtract dc-shift filter if only a constant value is to be removed.

Figure 7.1 shows an example of the dewowing process. The original data are shown in Fig. 7.1a, Fig. 7.1b shows the data after filtering using the dewow filter with a filter length of 25 ns, and Fig. 7.1c after filtering using a bandpass filter working within the frequency domain with cut-off frequencies of 5 and 150 MHz, and an adequate tapering window.

The bandpass filter does not affect the signal shape whereas the simple dewow filter slightly changes the signal owing to its non-symmetrical shape. Small precursors may occur which must be neglected when applying the next processing step, the removal of the time base shift. The bandpass filter may also be used in order to get rid of high-frequency or monochromatic noise. Seismic data can be similarly handled.

7.3.2 Time-base shift correction

The time base of GPR measurements is not exactly fixed and it may exhibit a significant drift owing to a temperature difference between the instrument electronics (especially owing to the avalanche transistor effect) and the air temperature, or to damaged cables. Such a drift causes misalignment of the reflections. A well-corrected time base is also very important for the interpretation of a 3D dataset especially when looking
at the timeslices (C-scans) as different time bases may significantly destroy the coherent character of the reflecting elements. Figure 7.2 shows an example (raw data in Fig. 7.2a and time-base corrected data in Fig. 7.2b). Seismic data may also show those irregularities and they can be compensated for in a similar way.

7.3.3 Time zero

An exact definition of time zero is nearly impossible. It is not a constant value but depends on the surface material type and the antenna set up configuration (Fig. 7.3).

Figure 7.4 shows a comparison between the simple correction to the first onset (open circles) and the dynamic correction (filled squares) under consideration of the source receiver distance and the velocity of the medium (Mayer, 1994). The reference builds the dynamic correction based on the source receiver distance, the size of the air gap and the velocities of air and the medium therefore taking into account the refraction of the omitted waves. If the velocity contrast is sufficiently high (more than 2:1, Fig. 7.4a) the error using the simple static correction is small enough to ensure a sufficient accuracy for the complete time range.
7.3 A standard transmitter receiver configuration and possible ray paths.

7.4 Comparison of the errors for the time zero correction when using a static correction (open circles) and a dynamic correction (filled squares) for (a) a high-velocity and (b) a low-velocity contrast (Mayer, 1994). The errors (y axis) are plotted against the investigation depth $d_2$ (x axis).

An automatic and stable static correction may be done either on the first negative, first zero crossing or first positive peak. A correction to the first break position (the very first onset that stands out against the background noise) might be the best solution but this may be unstable over time owing to variations in the electromagnetic properties of the underground, i.e., the subsoil, in the near-field of the antenna. An alternative may be an automatic correction to the first zero-crossing and then performing a static shift to the first arrivals with a signal length of the first peak. In either case, the picking
of the reflections or diffractions and the subsequent depth calculation must take into account the chosen time zero method.

Owing to its different source coupling directly to the ground, seismic data must be handled a little bit differently. Here, a dynamic correction may lead to better results depending on the geometric settings of the source and the receiver(s).

7.3.4 Time varying gain

The waves lose significant energy while travelling through the subsurface owing to spherical divergence and intrinsic and scattering attenuation. Therefore, these energy losses have to be compensated. Several conditions have to be adhered to: the time series must have a zero mean value, otherwise a significant dc offset especially at later times may occur, and the noise level for longer times should be as small as possible.

It is possible to enter a manual gain value or to use a continuous gain function (Fig. 7.5). When manual gain values are applied, rapid changes in the gain values should be avoided because these may introduce unwanted artificial wavelets. It is strongly recommended to use the same gain function for all profiles that are to be interpreted together. This also holds true for 3D data especially when looking at timeslices.

![Graphs showing time varying gain](image)

7.5 (a) Before and (b) after application of a continuous gain function.
Spherical divergence and intrinsic and scattering attenuation also occur during seismic wave propagation. Therefore, the same methods can be used in order to compensate for the seismic losses.

7.3.5 Clutter reduction, background removal

GPR data are often contaminated by clutter. The clutter mainly consists of the GPR system noise, ground bounce, soil roughness scattering, and reflection signals from external anomalies. The clutter mostly appears as nearly horizontal and periodic ringing. Clutter reduction is therefore one of the most important challenges as particularly deeper or weak events are often completely masked by this clutter.

The GPR system-based coherent noise ringing can be easily eliminated using a simple background removal (subtraction of an average trace) filter as the statistical properties of the clutter have only weak variations along the distance axis (Fig. 7.6).

The situation is much more complicated if the statistical properties of the clutter vary along the distance axis owing to different ground coupling and/or subsurface scattering. In this instance, more sophisticated methods must
be used. Two-dimensional filters such as the fk-filter or Radon transform, predictive and deterministic deconvolution or eigenimage-processing techniques are the ones most used (Kim et al., 2007). For all these methods, the definition of the filter parameters must be adapted to the individual situation in order to keep horizontal events, i.e. the reflections with no significant lateral changes, but to guarantee a high performance of ringing elimination. In particular, deconvolution and eigenimage processing require careful setting of the entered parameters. A good compromise between ease of use and performance may often be given by the fk-filter or the subtracting mean in combination with a notch filter if the noise exhibits monochromatic characteristics.

Figure 7.7 shows a comparison of the raw data, which include both system-induced coherent clutter and incoherent ringing owing to the subsurface conditions. The background removal fails in the elimination of the incoherent clutter but both the fk-filter and the subtracting mean within a moving trace interval seem to be a sufficient approach. In addition, the background removal filter introduces some artefacts resulting from stronger ringing within some ranges, e.g. between 160 and 200 m, which has been smoothed over the complete distance range. Seismic data also often show clutter noise and it can be handled in a similar way.

![Comparison of methods for incoherent clutter reduction: (a) the original data, (b) the background removal filtered, (c) the application of the fk-filter and (d) the use of the subtracting average filter.](image-url)
7.3.6 Profile energy balancing

A major problem is the often non-uniform energy feeding-in along a 2D profile or for different 2D profiles to be interpreted together (e.g. creating time slices). The causes may be varying ground conditions, equipment changes, use of multi-antenna systems or differences in the field acquisition. Whereas variations along the profile caused mainly by different coupling conditions can be quite easily compensated for using a trace normalization or a gain function in profile direction, more sophisticated methods must be used when dealing with 3D data especially when the interpretation is based on time slices.

Figures 7.8 and 7.9 show an example of a multi-grid survey using a double-antennae system, each antenna having different energy characteristics. Figure 7.8 shows two parallel 2D raw profiles (0.05 m increment) indicating the different energy content. Figure 7.9 shows two time slices. In Fig. 7.9a, the time slice is based on the raw data, whereas in Fig. 7.9b the time slice has had an energy compensation applied first. The horizontal stripes in the Fig. 7.9a time slice are caused by the different energy characteristics of the two antennae used. They may lead to misinterpretation.

Depending on the equipment used and the prevailing ground conditions, often not only a uniform factor but a time-varying curve must be deter-
mined for compensation of the different energy input. This may lead to an amplitude decrease in the reflections that is only present in some profiles (e.g. pipes which are orientated parallel to the acquired profiles). Therefore, such a compensation must be used very accurately.

The same holds true for seismic data, but, owing to the different source coupling, the energy input may be very different. In addition, if using a receiver array, the different transmission characteristics of the individual receiver elements must be corrected. Equivalent methods as described above can be used.

7.9 Time slice for a 3D grid survey using a double-antennae system: (a) no energy compensation and (b) an energy compensation has previously been applied to the various 2D profiles that comprise the complete 3D dataset.
7.4 Sophisticated data processing

Deconvolution and migration are the basis of the main sophisticated data-processing methods, which are based on more complicated mathematical models. As with the standard data processing, the theory is not outlined here, but the preconditions, the profits and the possible problems caused by its use are discussed.

7.4.1 Deconvolution/shaping

A major problem in signal interpretation is the lack of resolution of overlapping events owing to the reverberation character of the signal. The main purpose of the deconvolution is to invert the convolution process of the medium impulse response and the outgoing signal. The ideal outcome of the filter is again the medium impulse response. Although this ideal cannot usually be achieved, many different methods have been developed for different preconditions. One filter is called the Wiener filter (Wiener, 1949), and this filter minimizes the differences between the output and the desired result. Other methods involve a direct inverse filter. In general, deconvolution techniques are not very well suited for GPR applications as the main preconditions such as minimal phase and lag time zeros are normally not satisfied. In many instances, predictive deconvolution techniques or wavelet shaping (Robinson, 1983) may lead to a better result. A suitable filter strongly depends on the characteristics of the signal.

Figure 7.10 shows an application of a wave-shaping filter. The wave-shaping filter allows the characteristic waveform of the profile to be con-

![Figure 7.10 Application of a wavelet-shaping filter: (a) raw data, (b) the filtered data, (c) respective traces of the datasets.](image-url)
verted to a new desired one. It is clear that, after application of the shaping filter, all signals exhibit a much clearer and sharper form leading to a better resolution in the time direction. The precondition is that a characteristic waveform can be found to fit over the complete time–distance range. Owing to different coupling conditions and waveform changes during the propagation, this precondition is only rarely satisfied. The spiking filter is a special type of wave-shaping filter that is designed to compress as much as possible of the original wavelet into a spike (uniform frequency distribution).

The main goal of predictive deconvolution is the suppression of multiples. The desired output is a time-advanced version of the input signal. To suppress multiples, one has to choose a lag corresponding to the two-way-traveltime of the multiple. In the following, synthetic radargrams have been created using a forward finite difference time domain (FDTD) method, (Yee, 1966) in order to examine the effectiveness of a predictive deconvolution method for multiple and ghost removal. A similar approach for generating seismograms based on the elastic wave equation was first established by Kelly et al. (1976).

Figure 7.11a shows an example with strong multiples coming from a very near-surface interface with very high-velocity contrast. The predictive deconvolution used yields reasonably good results (Fig. 7.11b). The reverberating character of the signal could be reduced to a sharp signal with only two maxima both for the primary onset and the reflections and therefore the distinguishability has been significantly improved.

Some further investigations have been done on introducing a ghost by bringing in a strong reflector above the receiver line. The amplitudes of the ghost were varied. Whereas the weak ghost was almost eliminated using the

![Figure 7.11](image-url)
predictive deconvolution, no good results were achieved for a ghost with amplitudes similar to those of the primary onset (Fig. 7.12). After applying predictive deconvolution, the ghost is still visible although with smaller amplitudes (Fig. 7.12b). In addition, the deconvolution process produces multiples.

In summary, it can be concluded that the predictive deconvolution method used for this investigation was able to reduce multiples and ghosts. The signals presented within the synthetic seismograms are minimal phase. The effectiveness of the deconvolution will be lower if this does not hold true or if reverberations are also present. Therefore, in reality, the results of the predictive deconvolution may be significantly poorer.

Seismic data show many similarities, although the source signal characteristics (need for minimal phase and zero lag time) may be sometimes more suitable for the application of the deconvolution techniques. In both instances, for GPR and seismic data, the methods always require intensive adaptation of the filter parameters and a manual check for possible artefacts.

7.4.2 Migration

The waves coming to the receiver will be acquired vertically along the acquisition line and therefore do not represent the correct positions of small-scale diffractors or sloped reflectors. The goal of the migration [some methods are also known under the name synthetic aperture focusing technique (SAFT, Mayer et al., 1990)] is the downward continuation of the acquired wavefield to their origin. The basis of this wavefield continuation
is a given depth velocity model. During the migration process, diffractions will be concentrated and dipped layers will be moved to their correct place. The precondition for the migration is a good understanding of the underground velocity field.

Standard migration is carried out for 2D data. Fast algorithms exist for a constant velocity field (e.g. fk-migration; Stolt, 1978) but the more sophisticated methods such as the finite difference approximation of the one way equation can also be applied on standard PCs with reasonable computer-time consumption. Figure 7.13 shows a synthetic example (FDTD method) including diffractions and steep reflections. The model shown in Fig. 7.13a serves as the basis of a forward simulation of a ZO section (Fig. 7.13b). The small elements within the first layer produce diffractions that interfere with each other and this does not permit individual identification. The trough at about 10 m is characterized within the ZO section by two diffractions and a reflection from the bottom. It is evident (Fig. 7.13c) that the structure elements were shifted to their real location only after migration and the diffractions were concentrated so that they were distinguished from each other.

Migration is most useful if time slices (C-scans) are produced for subsequent interpretation. Figure 7.14 shows both the raw data (Fig. 7.14a) and the migrated data (Fig. 7.14b). Again, the strong energy concentration within the migrated data is evident and it leads to a much better ability to discriminate between the individual elements.

A 3D migration is useful for a coarse 3D data grid with equal spacing in the x and y directions. Owing to the large amount of computer time needed for a complete 3D migration, methods with a constant migration velocity (e.g. Kirchhoff migration) are used by default.

7.13 (a) A model including small diffractors and steep reflectors, (b) the ZO simulation, (c) the migrated ZO section.
Again, the seismic data show an equivalent behaviour. The same migration techniques are used. As seismic data are often acquired using a receiver array, pre-stack migration techniques are very common if a good understanding of the velocity field is available. Such a technique includes stacking, migration and time–depth conversion within one step (Yilmaz, 1987).

7.4.3 Frequency–wavenumber (fk) filter

Unwanted reflections from the borders of the investigation medium are often characterized by a distinct slope that corresponds to medium or air velocity. Those structures can be easily removed using a multichannel filter. The most popular filter is the so-called frequency–wavenumber (fk) filter which works within the frequency wavenumber range. Figure 7.15 shows the raw data (Fig. 7.15a) including a distinct side reflection and the filtered
It must be remembered that other elements showing the same slope are also removed using such a filter. In most instances, tapering must be used in order to avoid artefacts.

### 7.4.4 Time–depth conversion

The final step in the data processing is the time–depth conversion. Similarly to migration, the time–depth conversion needs a good understanding of the velocity field of the medium. There are several ways to get information about the velocities:

- from the curvature of existing diffractions within normal zero offset profiles;
- from common midpoint (CMP) measurements if a layered medium is given (for this special arrangement the source and the receiver will be continuously moved from a midpoint to both sides using the same increments);
- from borehole measurement and subsequent comparison of these data within the zero offset data; and
- from previous reported studies if the underground material is known.

The easiest way is the use of a constant velocity. This does not affect the shape of the signals within the radargram. If using a time varying or even...
time–distance varying velocity distribution for the time–depth conversion, the signal shape may be significantly changed. This must be taken into account in the subsequent interpretation.

If no pre-stack depth migration has been used seismic data may be considered correspondingly.

### 7.5 Conclusions and future trends

The main processing algorithms for GPR and seismic reflection data have been introduced but the entire field of signal processing techniques could not be covered. The main focus has been on the description of the problem and on the examination of the effectiveness. It was shown that the choice of the method and of parameters is of most importance and it is recommended to always check the process steps and to assure their validity.

Standard processing techniques such as dewowing, bandpass filtering or time varying gain are quite well established and do not need any fundamental further developments. Deconvolution, migration and clutter reduction techniques are under continual further development and the near future will surely yield new interesting findings.

The focus has been on 2D data (B-scans). Currently, the processing of 3D data is mainly done for the individual 2D cuts in building up the complete 3D dataset. Some 3D examples were shown, such as energy balancing and migration. In the future, owing to developments within data acquisition, more coarse 3D data will be available and therefore the needs for complete 3D processing techniques will increase. In addition, image processing for the time slices will become more important.

It has been shown that simulation is a good tool for examining the validity and effectiveness of some processing tools. The FDTD method for example allows radargrams or seismograms to be simulated for very complex subsurface models and it may help in understanding the behaviour of some tricky processing techniques.

### 7.6 References


MAYER, V., 1994: Probleme der quantitativen Interpretation von Radardaten; Diploma thesis at the Geophysical Institute, University Karlsruhe.


Non-destructive testing of concrete with electromagnetic, acoustic and elastic waves: modelling and imaging

K. J. LANGENBERG, K. MAYER and R. MARKLEIN, University of Kassel, Germany

Abstract: The theory of acoustic, electromagnetic and elastic wave fields in terms of the underlying physics, resulting plane and spherical waves, and scattered field representations is discussed with respect to common aspects leading to a unified derivation of modelling and imaging algorithms. The latter require a linearization of the inverse scattering problem. Applications referring to the location and assessment of tendon ducts in concrete are presented that include synthetic as well as experimental data, culminating in a real-life example for data taken from a bridge.

Key words: non-destructive testing, concrete, electromagnetic wave, acoustic wave, elastic wave.

8.1 Introduction

The problem to be considered is the location and assessment of tendon ducts in concrete as a specific task in non-destructive testing of concrete. As an example, Fig. 8.1 shows the making of a test specimen: a tendon duct was placed below a steel reinforcement and the picture was taken while the concrete is poured in. Concrete principally allows for the propagation of electromagnetic as well as acoustic and elastic waves, and, hence, both wave modes can be utilized to solve the task of locating the duct below the reinforcement, and the latter one to assess its integrity, i.e. identifying grouting defects. Electromagnetic waves face the problem of being shielded by the steel grid whereas elastic waves are strongly affected by the inhomogeneity of the concrete composition. Hence, it is advisable to investigate the wave propagation in terms of a parametric study applying numerical codes to solve the respective underlying wave equations; the similarity of the latter then suggests the formulation of a unified theory of inversion and imaging.

This concept is presented without equations; the corresponding mathematics are given in references 1 and 2 (in English and German, respectively).
8.2 Electromagnetic, acoustic and elastic waves

8.2.1 Underlying physics

Electromagnetic waves

It was James Clerk Maxwell who introduced the so-called displacement current into the theory of electric and magnetic fields unifying it to a theory of electromagnetism. Ampère’s law stated that a purely solenoidal magnetic field is created by electric currents, the sources of this field, yet Maxwell, guided by mechanical similarities, claimed that another current, the displacement current, which was a physically real quantity in electrically polarizable materials, should also contribute to the magnetic solenoidal field and he quantified it as the time variation of an electric quantity, the dielectric displacement, nowadays called the electric flux density. The resulting equation is the Maxwell equation. It is complemented by Faraday’s law of induction stating that a time variation of a magnetic quantity, the magnetic induction (now called the magnetic flux density), creates a purely solenoidal electric field. Both equations are now known as Maxwell’s equations of electromagnetism; they exhibit a symmetric structure, because time variations of flux densities are related to spatial variations of field strengths. They can be made even more symmetric if a physically non-existing magnetic current is introduced as a complementary source in Faraday’s law that simplifies certain mathematical derivations considerably. It is our physical understanding that currents are nothing but moving charges, i.e. the time variations of the latter are the sources of currents in terms of continuity relations; again, for magnetic currents this is physically fictive. So-called
compatibility equations follow from Maxwell’s equations and from the continuity equations relating the sources of the flux densities to the electric and magnetic charges. It is worth noting that it is the displacement current in Maxwell’s equations that allows for electromagnetic waves as fundamental solutions. From a mathematical viewpoint Maxwell’s equations are partial differential equations; yet, with the help of some famous theorems, Gauss’ and Stoke’s theorems, they can be transformed into a set of integral relations that prove very useful in the formulation of the Finite Integration Technique as the unified technique for numerical simulations (see Acoustic waves below).

Elastic waves

The sources for elastic waves in solids are mechanical forces as well as deformations. Forces appear in Newton’s law of momentum conservation relating the time variation of the linear momentum of an infinitesimally small material volume to the inherent forces applied by neighbouring volumes and by the externally applied source forces. It was Cauchy who interpreted the inherent forces as sources of mechanical stresses, hence, the Newton–Cauchy equation of motion states that the time variation of the momentum density, the motion of a material volume, is equal to the source density of stresses plus the external force density. The equation of motion is complemented by the deformation rate equation that defines the time variation of the deformation of a material volume in terms of the spatial variation – the gradient – of the velocity of the volume (particle velocity) plus the externally applied deformation rate as source term.

The governing equations of elastodynamics bear a striking similarity to Maxwell’s equations: time variations – first order time derivatives – of physical fields are related to spatial variations of complementary fields plus sources. The difference is in the mathematical operations describing the spatial variations, both Maxwell equations exhibit a curl operator, whereas the equations of elastodynamics exhibit a divergence operator (Newton-Cauchy equation) and a gradient operator (deformation rate equation).

Acoustic waves

Acoustic waves propagate in liquids or gases, yet they are often an appropriate approximation for elastic wave modes, in particular for the pressure wave mode. They can either be understood from basic physics or by application of the governing equations of elastodynamics assuming that the stress (tensor) is proportional to a scalar pressure, namely a single number (with a physical dimension), whereas the stress tensor is composed of six scalar numbers (originally nine, reduced to six owing to physical symmetry prop-
Thus, the source density of the stress tensor in Newton–Cauchy’s equation is reduced to the gradient of the pressure. The deformation rate equation reduces to the dilatation rate equation because the (tensor) deformation is replaced by a scalar dilatation, the same being true for the deformation/dilatation rate source term.

Therefore, our wave family now consists of three members, but before we can take advantage of this with respect to non-destructive testing we have to complete the three systems of governing equations by constitutive equations expressing the detailed physical properties of the material under consideration.

8.2.2 Constitutive equations

Constitutive equations do not follow from the governing equations by mere calculus, they have to be based on the physics of matter without violating the governing equations and their immediate consequences such as conservation of momentum and energy together with physical principles such as causality. In the present context we concentrate on the simplest versions of constitutive relations implying linearity, time invariance, instantaneous and local reaction as well as isotropy.

Electromagnetic waves are special insofar as they propagate in vacuum without any host material; nevertheless, even then, the flux densities have to be connected to the field strengths through the definition of proportionality factors, the electric field constant between the electric field strength and the electric flux density, and the magnetic field strength between the magnetic field strength and the magnetic flux density; these two constants match the physical dimensions of fluxes and fields. In materials with the above stated properties the field constants are multiplied by (scalar) factors, the (electric) permittivity and the (magnetic) permeability; these two material parameters may depend on the position within the material, i.e. the material may be inhomogeneous.

For elastic and acoustic waves, the momentum density is simply related to the particle velocity through the (scalar) mass density. When the material does not sustain shear stresses, the acoustic approximation holds, and the proportionality between pressure and dilatation introduces the (eventually inhomogeneous) compressibility.

However, in the more general example of linear elasticity, a relation between the stress and the deformation has to be established and this takes the form of a generalized Hooke’s law introducing the compliance tensor of rank four. Owing to the symmetry of the stress and deformation tensors (by virtue of definition and angular momentum conservation) and as a consequence of energy conservation, the compliance tensor (for instantaneously reacting, i.e. lossless materials) has at most 21 independent entries.
A further reduction to only two parameters, the Lamé constants, is observed for isotropic materials; they compose the stiffness tensor as the inverse of the compliance tensor.

8.2.3 Fundamental waves

Homogeneous materials

Waves are solutions of wave equations, and these can be derived by inserting the governing equations into each other paying attention to the respective constitutive equations. The simplest one – since it is scalar – is the wave equation for the acoustic pressure, which involves a material-dependent ‘wave operator’ consisting of a combination of spatial second derivatives with second time derivatives operating on the pressure and containing the sources, force and dilatation rate, on the right-hand side as inhomogeneity. The same is true for the respective equation for the particle velocity.

The wave equation for electromagnetic waves involves a similar wave operator with the appropriate material parameters; the sources on the right-hand side are composed of electric and magnetic currents.

As stated in the next subsection, the constitutive equations of elastodynamics require three material parameters, the mass density and the two Lamé constants, that have to enter the wave operator for the particle velocity. The connection with acoustic waves suggests a combination of an acoustic pressure operator combined with a purely elastic shear operator, and, this turns out to be true.

Plane waves

Solutions of the wave equations even exist for zero sources, i.e. for homogeneous wave equations; plane waves as special representatives of the set of solutions deserve special interest for two reasons: They give physical insight into the fundamental properties of waves (even though not physically realizable) and they serve as building blocks for the mathematical representation of electromagnetic aperture antennas and ultrasonic piezoelectric transducers through appropriate superposition.

Waves propagate through space with increasing time and with a characteristic speed $c$ that is calculated from the material parameters in the constitutive equations; therefore, an impulse which is observed at some point $x_0$ as a function of time will be found at $x_0 + ct$ when the time $t$ has passed: plane waves come from infinity and travel to infinity. The material parameters are different for the three wave types and, therefore, the speeds differ, so that, because the elastodynamic wave equation contains a pressure and
a shear term, we observe two elastic wave modes, pressure and shear waves, with different speeds $c_P$ and $c_S$, respectively, where $c_P$ is always greater than $c_S$ (in steel nearly twice as large). In an infinitely large homogeneous material, these two wave modes travel independently.

**Plane waves** are characterized by a constant amplitude on an infinitely large plane perpendicular to the propagation direction, therefore they can only ‘exist’ in an infinitely large material; the same impulse as a function of time is observed on the whole plane.

We already mentioned that acoustic pressure is a scalar quantity, yet the particle velocity of elastic waves is a vector, i.e. it has a magnitude and a direction, and three scalar numbers are needed to describe it mathematically, resulting in a characterization of pressure and shear waves according to their polarization. Plane pressure waves exhibit a direction of the particle velocity in the direction of polarization, i.e. they are longitudinally polarized, whereas plane shear waves are transversely polarized, i.e. orthogonally to the propagation direction, but this direction is not uniquely defined.

Physically, the electric field strength is defined as a force on an electric charge, and therefore, field strengths (as well as flux densities) are vectors too. We find that plane electromagnetic waves are transversely polarized in both the electric and the magnetic field strength.

If the infinite space is composed of two homogeneous half-spaces with different material parameters and separated by a plane boundary, plane waves coming from one of these half-spaces will be reflected, transmitted and, for pressure and shear waves, they will be generally mode-converted into each other. However, such a plane boundary enables two shear wave modes to be distinguished. Assuming there is another arbitrarily oriented infinite plane orthogonal to the half-space plane (as in Fig. 8.2), the so-called plane of incidence, then two orthogonal transverse polarizations can be uniquely defined: shear-horizontal (SH) plane waves, which are orthogonally polarized with regard to the plane of incidence, and shear-vertical (SV) plane waves, which are polarized in the plane of incidence, and both are orthogonal to each other and travel independently. Furthermore, plane pressure waves (P), whose polarization is also in the plane of incidence but orthogonal to the shear-vertical polarization, are only mode-converted into plane shear-vertical waves, and plane shear-vertical waves are only mode-converted into plane pressure waves, neither one is coupled to shear-horizontal plane waves, and the same is true for the reflection and transmission of shear-horizontal plane waves. Reflection, transmission and mode conversion coefficients for plane waves only depend on the respective material parameters and the angle of incidence, they do not depend on the impulse shape under concern (in lossless materials).
Spherical waves

The governing equations of waves exhibit sources that account for the generation of the respective waves. It is intuitively clear that waves travel away from their sources in all directions; hence, they can no longer be represented by plane waves. In order to establish their structure we concentrate on the simplest sources that could be imagined, i.e. point-sources; spatially extended sources, such as antennas and transducers, could then be synthesized by superposition of point-sources. Among all source terms in the governing equations there is only one scalar term: the dilatation rate in the acoustic equations. If that one is assumed to be point-like, i.e. the force density is zero, the resulting wave field is a scalar spherical pressure wave travelling in all spatial directions with the same amplitude and the same impulse shape (it is actually the time derivative of the impulse shape applied to the point-like dilatation); this wave field is called the (scalar) Green function and it plays a fundamental role in the mathematical representation of antenna and transducer fields, of wave fields scattered by inhomogeneities, i.e. defects, and in the formulation of imaging algorithms.

If we consider a point-like force in the acoustic pressure wave equation, the dilatation assumed to be zero, the radiated wave is again a spherical wave, but the amplitude now varies with propagation direction; nevertheless, it can be calculated from the scalar Green function through a spatial derivative (the gradient, actually). Spherical acoustic pressure waves are still longitudinally polarized.

The angular amplitude variation of spherical electromagnetic waves is given by the scalar Green function itself plus a double gradient operating on the scalar Green function; therefore, they are no longer strictly transverse, they exhibit a longitudinal field component in a region ‘close’ to the point source thus defining a near- and a far-field region; in the latter one,
the spherical wave behaves locally like a plane wave. Spherical elastic waves are still observed as pressure and shear waves with respective wave speeds, but, as in the electromagnetic case, their polarization is plane-wave-like only in the far-field.

The wave fields from extended sources, such as antennas and transducers, are obtained from superimposing Green function spherical waves weighted by the spatial distribution of the source terms. In the respective far-fields an appropriate approximation leaves us with the radial dependence of a spherical wave originating from the ‘centre of gravity’ of the source distribution multiplied by a function of observation angles only, the so-called ‘radiation pattern’. The synthesis of desired radiation patterns is the key task in designing antennas and transducers.

**Inhomogeneous materials**

Tendon ducts with or without grouting defects are (strong) inhomogeneities in an embedding material (concrete) and, as such, they act as scatterers for an incident wave field: they are sources of a scattered field. Thus, in a mathematical representation of the scattered field, the scatterers can be replaced by equivalent or secondary sources (the sources of the incident field being considered as primary sources) representing their geometry and material composition, which enter the respective integrals of point source superposition. These integrals are subsequently bound for an inversion, i.e. to deduce the properties of the scatterer from the knowledge of the scattered (and incident) field via imaging algorithms.

### 8.3 Numerical wave field modelling for acoustic, electromagnetic and elastic waves

In order to synthesize scattered wave fields numerically, we have to solve the underlying governing equations for given sources and given material inhomogeneities. Standard methods to achieve this are available, for instance finite elements, finite differences and the finite integration technique (FIT). The latter was originally developed for electromagnetic fields with a subsequent extension to acoustic and elastic waves (AFIT for acoustic waves and EFIT for elastic waves). FIT now provides a unified approach for all three wave fields under concern. Thus, the governing equations in differential form are transformed into integral forms that are bound for discretization on staggered grids, one of those following the discretization of the material inhomogeneity. For time, an explicit time integration scheme is chosen, which results in wave field movies as the output of the schemes.

Figure 8.3 shows single subsequent snap-shots of such a movie for two-dimensional acoustic waves. A spatially constant pressure distribution is
prescribed as an impulsive source within a finite aperture indicated by the black bar in the figure. The impulse is modelled as an RC2 pulse that is composed of two oscillations of a certain frequency with an amplitude modulation according to a raised cosine; such an impulse models a real transducer impulse sufficiently well. The first and second snap-shot of Fig. 8.3 tells us that the finite aperture radiates a nearly plane wave of finite extent accompanied by circular cylindrical wave fronts emanating from the aperture edges. The snap-shots to follow display the development of wave fronts scattered by a ‘crack’ parallel to the surface; these are composed of a reflected wave and circular cylindrical edge waves.

Figure 8.4 shows an example for three-dimensional electromagnetic waves: A tendon duct is located in homogeneous cement and is partly covered by a steel reinforcement grid (Fig. 8.1); it is illuminated by a plane wave with a Gaussian pulse time dependence. The polarization of the electric field strength is parallel to the tendon duct axis. The two snap-shots in Fig. 8.4 are two-dimensional slices out of the three-dimensional wave field chosen orthogonally to the tendon duct, first from the region without reinforcement and second below the reinforcement. It can be seen that the actual mesh size of the reinforcement grid clearly allows for a penetration of the electromagnetic wave creating a distinct scattered wave travelling back to the surface thus carrying the information of the tendon duct location to an appropriate receiving antenna. Such simulations allow for parametric studies to investigate the influence of various mesh sizes on ground probing radar measurements.

As demonstrated, the utilization of electromagnetic waves may be limited by reinforcement grids, whereas for elastic waves it is the inhomogeneity
8.4 (a) Computer model of a concrete specimen and snapshots of the propagation of electromagnetic waves: (b) without reinforcement (plane A); (c) with reinforcement (plane B).

8.5 Snap-shots of a movie for two-dimensional elastic wave scattering at a crack parallel to the surface.

of the concrete. The elastic counterpart of Fig. 8.3 is shown in Figs 8.5 and 8.6. First of all, we recognize that the aperture wave field is enriched by circular cylindrical shear waves originating from the aperture edges together with so-called lateral waves connecting these wave fronts to the pressure wave fronts on the surface. However, a plane aperture shear wave, as present in the pressure wave front, is missing owing to the zero value of the pertinent Green function orthogonal to the aperture. In addition to the pressure and shear wave fronts, Rayleigh surface waves emanate from both
aperture edges. The wave fronts scattered by the crack again exhibit both pressure and shear waves.

In Fig. 8.6, we get an impression of the influence of the inhomogeneity of concrete according to a specific distribution of gravel of different size, shape and material composition embedded in a cement matrix. EFIT allows for modelling of gravel in terms of a statistical distribution of ellipsoids (in two spatial dimensions) and spheroids (in three spatial dimensions) of varying axis ratios. Each one of these pebbles acts as a scatterer for elastic waves resulting in some diffusion of the originally sharp incident pressure wave front; an additional shear wave is no longer visible. This diffusion becomes more and more pronounced with time resulting in a weak scattered wave that barely reaches the surface. Nevertheless, as previously demonstrated,\textsuperscript{1,2,9,10} there is still sufficient information available to be focused as an image of the scatterer.

8.4 Wave field inversion and imaging: acoustic waves

8.4.1 Heuristic inversion: synthetic aperture focusing technique (SAFT)

From the fundamental knowledge of wave propagation as outlined in the previous sections, it is rather simple to establish an imaging algorithm heuristically. Nevertheless, to prove and extend it mathematically it has to be embedded in the mathematical framework of inverse scattering.
A point-like scatterer when illuminated, for instance, by the plane impulsive aperture pressure wave of a transducer, radiates, in the acoustic approximation, a spherical pressure wave. If this wave field is recorded on a plane measurement surface, one obtains B-scan data, i.e. ultrasonic data as a function of two orthogonal (cartesian) scan coordinates and time. If the excitation impulse is of infinitely short duration, the data are non-zero only on a rotationally symmetric hyperboloid as can be directly computed from the scalar Green function. Therefore, from the principle of superposition of point-scatterers for a spatially extended scatterer, it is readily concluded that the respective data consist of a superposition of hyperboloids. An inversion of these data in terms of the voxel driven approach goes as follows: a region of interest is defined where the scatterer resides that is then discretized into voxels (pixels in two spatial dimensions), and for each voxel a pertinent hyperboloid is calculated and the data existing on that particular hyperboloid are accumulated and stored in that voxel, i.e. they are focused to the voxel; hence, this imaging technique is called synthetic aperture focusing technique (SAFT). A processing alternative, the A-scan approach, exists if the backpropagation principle is exploited: owing to the knowledge of the Green function a particular data point from a chosen A-scan can only be reached from point-like sources on a half-sphere in the material with a radius proportional to the propagation time, hence, it is backpropagated and equally distributed on that half-sphere. Adding all backpropagated data yields the same image of the scatterer as the voxel driven approach. If the excitation impulse is band-limited, as shown for the RC2 pulse in Figs 8.3 and 8.5, the image of the scatterer is blurred accordingly; a tool for better interpretation consists of the application of either a Hilbert transform of the image data along the depth coordinate and taking the magnitude of the resulting complex-valued data or the simultaneous backpropagation of timely Hilbert-transformed data as a complementary imaginary part of the data with subsequent calculation of the magnitude.

SAFT can be and has been implemented in two different data recording versions, the pulse-echo and pitch-catch version. In the former, the transmitting and receiving transducer coincide and in the latter a spatially fixed transmitting transducer and a scanning receiving transducer are utilized.

8.4.2 Wave field inversion as a nonlinear problem

It is important to know whether SAFT is an explicit solution of the inverse scattering problem, or if any approximations and/or constraints are involved, i.e. whether SAFT delivers an ‘image’ or a ‘reconstruction’ of the scatterer? To answer this question an inverse scattering theory has to be established and it readily turns out that an exact inversion of a wave field has to cope with a serious nonlinearity. As previously seen, we replaced the
scatterer by secondary or equivalent sources. Therefore, if, for instance, the scatterer is nearly infinitely electrically conducting (an appropriate model for a tendon duct), it follows from Maxwell’s equations that the electric field strength must always be orthogonal to its surface. However, an incident field coming from an arbitrary antenna with an arbitrary polarization ‘does not know’ that there is a scatterer to which its field strength has to be perpendicular; it therefore induces sources, in this instance electric currents, on the surface that radiate a scattered field; these currents are induced in such a way that the total field, the sum of the incident and the scattered field, is orthogonal to the surface. Hence, the secondary sources depend not only on the known incident field but also on the still-to-be-calculated scattered field. The scattering process resembles a feedback loop in circuit theory, it is nonlinear with regard to the scatterer geometry and material composition and so is its inversion.11

Embedding SAFT in an inverse scattering theory reveals that it relies on an inherent linearization of the secondary sources; they are approximated to depend on the incident field only, and this is quantified as the Born and/or Kirchhoff approximations.12 Therefore, SAFT delivers in general an image of the scatterer and not a reconstruction.

8.4.3 Processing alternatives to SAFT: FT-SAFT

For three-dimensional imaging in particular it is rather tough to program SAFT efficiently, hence, processing alternatives are required. In the frequency domain (transition from the time to the frequency domain is via a Fourier transform), the superposition of incident and scattered wave fields in terms of spherical waves weighted by primary and secondary sources turns out to be, from a mathematical point of view, a three-dimensional convolution integral. These types of integrals can be transformed by a three-dimensional spatial Fourier transform into a product of the respective transforms of sources and Green functions. Hence, in the spatial spectral domain, we encounter a simple algebraic relation that could be instantaneously inverted for the spectral sources. Concentrating on the scattered field, as it is the one that is available as measurements, according to the above, its three-dimensional spatial spectrum is related to the three-dimensional spatial spectrum of the secondary sources, representing the scatterer, multiplied by the spectrum of the Green function which is analytically known. However, unfortunately, we cannot readily compute it because it is only available on a two-dimensional surface, the third coordinate is ‘missing’. When the measurement is a plane, as it usually is, we can at least take a two-dimensional spatial Fourier transform with regard to the scan-coordinates; this leaves us with a remaining integral on the right-hand side of our ‘inversion equation’ that contains two-dimensional spatial spectra of the
secondary source and the Green function. The ‘trick’ is now to exploit the particular mathematical expression of the two-dimensionally transformed Green function defining a third Fourier variable relating it to the two-dimensional Fourier variables and frequency. This results in a mapping of the two-dimensionally transformed data onto half-sphere surfaces in the three-dimensional Fourier space of the source whose mid-points and radii are determined by parameters of the incident field, namely angle of incidence and frequency.

Up to this point everything is exact. However, for an inversion, the resulting mapping is not sufficient because we have Fourier data only on a surface, an inversion would yield zero (describing this surface mathematically in terms of distributional analysis an inversion yields the generalized holographic image of the secondary sources\textsuperscript{13}). However, because of the parametrization of the half-spheres in terms of the incident field, we could ‘play’ with these parameters to exploit, for instance, frequency in order to map different frequencies of our (impulsive) data onto half-spheres of different radii, thus covering a finite region in three-dimensional Fourier space. In order to utilize the result, we have to make sure that the different frequency spectra of the secondary source correspond to the same physical source, and this is only true if we linearize the inversion assuming that the Born (for weak scatterers) or the Kirchhoff approximation (for strong scatterers) hold. Then, our data mapped Fourier space is bound for an inversion, the result is an image of the scatterer obtained with the frequency diversity version of multi-bistatic FT-SAFT, ‘FT’ standing for Fourier transform. ‘Bistatic’ (pitch-catch) comes from the terminology used by the radar community, it stands for a fixed incidence angle of the incident field and a fixed, but different position of the receiver, ‘multi-bistatic’ refers to a many bistatic positions of the receiver on a measurement plane.

Instead of using frequency as a diversity parameter, we could also rely on the angle of incidence obtaining a multi-bistatic angular diversity image of the scatterer, which may be different from the previous one, because a different region of Fourier space is covered that way.

A popular scanning technique in ultrasonic testing is the impulse–echo technique where transmitting and receiving transducers coincide and are scanned simultaneously. An FT-SAFT version can also be derived for this technique if the linearizing approximation is introduced right from the beginning; however, frequency diversity is the only pulse–echo diversity.

It is interesting to note that SAFT (in the time domain) can be analytically derived from the frequency diversity FT-SAFT versions above abiding by further approximations\textsuperscript{12,13} SAFT and FT-SAFT have been applied with considerable success for both electromagnetic and ultrasonic material testing,\textsuperscript{14} and the results have even been compared with the ones obtained from nonlinear algorithms where no approximations are involved;\textsuperscript{15} it
turned out that SAFT and especially FT-SAFT work particularly well for most applications.

8.5 Wave field inversion: electromagnetic and elastic waves

In the previous section, we elaborated upon imaging algorithms from an acoustic viewpoint but mentioned applications in ultrasonic elastodynamics and electromagnetics. However, these wave fields are no longer scalar as in acoustics. In the simplest case, one could take a single component of the underlying vector wave fields or the output voltage of an antenna or a transducer as scalar data. On the other hand one might think of deriving analytic inversion formulas from the explicit knowledge of the Green functions (tensors) of electrodynamics and elastodynamics. In the spatial Fourier domain, the wave field superposition formulas for electromagnetic and elastodynamic scattered fields differ only in a tensor multiplication of the vector secondary sources (electric currents and mechanical forces) involving the spatial Fourier variables, frequency and the incident field direction. Unfortunately, within the necessary linearizing approximations in order to apply any kind of diversity, these tensors cannot be inverted because they have zero determinant. However, certain projections of the vector data onto known parameters of the incident field, direction of propagation and polarization, yield orthogonal scalar components of the vector secondary sources. Thus, the complete available information on the secondary sources can be extracted from measured data upon linearization. In the elastodynamic case, one can go even further to separate pressure and shear mode information in the data and process them separately to obtain pressure and shear images from either pressure or shear excitation; complementary information on the scatterer is thus obtained.

Below we give two examples for electromagnetic vector wave and elastodynamic mode separation imaging, both refer to tendon duct imaging in concrete. In Fig. 8.7 we return to the geometry of Fig. 8.1, three-dimensional synthetic data are obtained with the FIT implementation in the commercially obtainable \( \mu \)-wave studio. In Fig. 8.7a and b, the polarization projected image of the tendon duct surface and the steel reinforcement for two different incident field polarizations are shown. In Fig. 8.7c and d the propagation vector projected images are displayed, and, in Fig. 8.7e all four images are superimposed. The result is convincing.

Figure 8.8a shows the cross-section geometry of a tendon duct embedded in homogeneous concrete filled with homogeneous cement. Figure 8.8b shows the incident elastic pressure wave as computed with the two-
8.7 Imaging of electromagnetic vector wave data gained by FIT calculation of the computer model in Fig. 8.4: (a) hh-polarization, (b) vv-polarization, (c) hv-polarization, (d) vh-polarization, (e) characteristic function of the scattering object by using all data.

Using the similarity of the underlying governing equations, we have evaluated a unified theory of modelling and imaging with acoustic, electromagnetic and elastic waves that can be applied to specific problems in non-destructive testing of concrete. Additional examples have been presented previously.9,10
Two-dimensional elastic wave field modelling and inversion: (a) geometry of the specimen, (b) wave front snap-shot, (c) and (d) B-scan data for the data component normal and tangential to the surface (horizontal axis, time; vertical axis, scan coordinate), and (e) and (f) (P ⇒ P)-image, (P ⇒ S)-image.
8.7 References


Laser-induced breakdown spectroscopy (LIBS) for evaluation of reinforced concrete structures

G. WILSCH, BAM Federal Institute for Materials Research and Testing; A. MOLKENTHIN, Specht, Kalleja + Partner GmbH, Germany

Abstract: The potential of laser-induced breakdown spectroscopy (LIBS) for the chemical analysis of building materials is discussed in this chapter. LIBS is applied directly on the sample surface with minimal sample preparation. The results are obtained immediately after the measurement. Special applications are demonstrated and experiences of on-site use are given. The limitations and reliability of the method are discussed.

Key words: laser-induced breakdown spectroscopy, LIBS, reinforced concrete, chlorine, on-site application.

9.1 Introduction

Laser-induced breakdown spectroscopy (LIBS) is a method for the elemental analysis of optically accessible surfaces. A pulsed laser is used to vaporize a small amount of the sample surface. The initiated plasma radiation is then analysed by measuring an optical spectrum. For quantitative results, a calibration is necessary. LIBS is the only technology which can detect all chemical species in all environments. It can measure directly on the sample surface without any extensive sample preparation. The LIBS investigation is a multielement method, i.e., within one measurement, information about major and minor (trace) elements is obtained. The LIBS technique has been investigated over the past few decades for application in process control, waste sorting, and geological, archaeological and medical applications (Miziolek et al., 2006).

The main advantage of LIBS application in the investigation of building materials lies in the direct measurement on the surface of the solid sample. From cores, depth profiles are available. The results are obtained directly after the measurement. The set-up is suitable for on-site measurements. Within one measurement, major elements (Ca, Si, Al, Mg, Fe) and minor elements (Cl, S, Na, K) are evaluated. The spatial resolution of measure-
ments is in the millimetre range. Owing to a statistical number of measurements (up to 100 measurements per second or more) the heterogeneity of the material is taken into account and the effects of sampling are excluded. By moving the sample in a plane perpendicular to the laser beam or scanning the surface by laser, the spatial distribution of elements is obtained.

This chapter describes the potential and applications of LIBS for the investigation of building materials. In the next section, the fundamental aspects, illustrates a typical measurement set-up and summarizes the basic principles. The following section exemplifies the special precautions necessary for LIBS application on heterogeneous building materials such as cement, mortar and concrete. Then, the detection of specific elements for the evaluation of specific testing problems is given. In the penultimate section, the potential of a portable LIBS device for on-site testing and on-site quality control of repair work is given, and in the last section a reflection on limitations of the method and reliability of results is presented.

9.2 Laser-induced breakdown spectroscopy (LIBS): fundamentals and measurement

A typical LIBS set-up used for the investigation of building materials is shown in Fig. 9.1. The radiation of a pulsed NdYAG laser (\(\lambda = 1064\) nm, pulse rate 10 Hz, energy per pulse 100 to 400 mJ) is focused to vaporize a small amount of the surface under investigation (some micrograms). Owing to the high power density in the focus area, plasma ignites and radiates for some microseconds (Fig. 9.2a). The plasma radiation is guided to a spectrometer system, which breaks the radiation down. At the exit slit of the spectrometer the light intensities are detected by a CCD camera. The focal length of the lens is 500 mm to minimize the influence of surface roughness. The volume around the plasma can be purged with air or helium to remove dust and enhance the results for the detection of some elements. The detection of chlorine and sulfur is more sensitive if helium is used as process gas. The sample under investigation can be moved in a plane perpendicular to the laser beam to measure the spatial distribution of elements on the surface.

The CCD detects light intensities according to wavelength (Fig. 9.3). Each element has typical peaks in the spectrum caused by the inner structure of the atom. There are databases available which give the wavelength position of these peaks (Ralchenko et al., 2009). Thus, an assignment of peaks to elements is possible. The intensity of the radiation at a single peak contains information about the relative content of this element in the evaporated volume. There are several types of detectors, including single-peak detec-
tors (photodiodes or channeltrons), various types of CCD cameras and complex echelle spectrometers. The choice of the detector is closely influenced by the number of elements to be detected, the limit of detection and the financial budget.

Figure 9.2b shows a concrete specimen with traces of LIBS measurements. The specimen was scanned in lines 2 mm apart with a resolution of one measurement per millimetre for each line. Owing to the measurement, the heterogeneity of the material is recorded and results can be correlated to the cement content. The scanning of the 28 mm × 41 mm specimen takes some minutes and the results are obtained directly after the measurement.

The possible output of the measurement is the distribution of a specific element on the surface under investigation or, if the measurements are averaged per line, a gradient of element content.

Owing to the multielement ability of the method the measurement of trace elements as well as the differentiation between cement and aggregates is possible. As a result, the content of the trace element can be given in correlation to the cement content. The LIBS measurement displays

9.1 A typical LIBS set-up. A Nd:YAG pulse laser (λ = 1064 nm, 300 mJ per pulse, 10 ns pulse width) is focused on the sample surface for plasma ignition. A spectrograph (200–1000 nm, f = 150 mm, resolution 0.08 nm) with attached CCD camera (1024 pixel × 1024 pixel, 16 bit) is used for wavelength-related recording of plasma intensity. The specimen is moveable in a plane perpendicular to the laser beam.
9.2 (a) Plasma ignition on a hardened cement paste surface,
(b) concrete sample with traces of LIBS measurement.
statistical information about the specimen and brings out the heterogeneity of the building material.

As shown in Fig. 9.3, during measurement information for more than one element is available. With a more complex detector (echelle spectrometer) up to 40 lines can be detected simultaneously.

The LIBS set-up can be made portable. First experiences of on-site applications are given in section 9.5.

The data-evaluation process needs some normalization procedures to eliminate interferences such as fluctuations of pulse energy, surface properties or dust in the beam path. There are programs that allow fast and automated data evaluation in only a few minutes. Nevertheless, currently a well-skilled engineer is necessary to carry out LIBS measurements.

### 9.3 Characterization of cement, mortar and concrete

Cement is a mineral, finely ground, hydraulic binder which is used to make concrete and mortar. Depending on the specific application, different types of cement are chosen on the basis of their composition.

Concrete and mortar concrete are materials that are also known as the five-component system, and they consist of bonding agents, aggregates and water, as well as concrete additives, which, in turn, break down into conditioners and admixtures.

One of the major differences between concrete and uniform materials, such as metals or liquids, therefore lies in the heterogeneity of the construc-
This heterogeneous composition also gives rise to different physical and chemical surface properties for the laser excitation. The particle size is a determining factor, primarily in terms of nanometres and millimetres. If the laser beam is focused on the surface of the mineral solid matter with intensity $I_{\text{Beam}}$ then the fractions $A \times I$ will be absorbed by it; $R \times I$ reflects and $T \times I$ transmits:

$$I_{\text{Beam}} = I(A + R + T) \quad [9.1]$$

where $A + R + T = 1$.

The incident light energy is absorbed by the construction material in such a short time that the excitation cannot be distributed over the solid but is instead converted to heat at an explosive rate in the irradiation zone only. The absorption ($A$) of concrete at a wavelength of 1000 nm is 55%. The transmission is negligible and the reflected part is 45%.

The sudden rise in temperature and the plasma expansion generate a secondary shock wave that is absorbed and muted to a sonic pressure wave on penetrating the solid, bringing about an erosion of the material. At the same time, the materials combined in the multicomponent concrete construction material have different ablation and evaporation rates.

The matrix surrounding a cement of conventional strength is heavily eroded whereas the stone particles are subject to far less erosion. Figure 9.5 shows the ablation crater (measured by a confocal laser scanning microscope) in the hardened cement paste compared with the cement mortar with stone particles up to 2 mm in diameter.
Views taken with the scanning electron microscope (Fig. 9.6) clearly show that the high plasma temperatures also cause the mineralogical structures on the crater walls to melt. The different ablation action of the material components does not, on its own, yield any adequate distinction of the vaporized parts within the focal spot.

In order to go one step further to this stage, there was a need for differentiators based on the specific elements that make up the composition of the concrete components. This spawned the idea of differentiating the excited proportion of solid particles by a specific set of criteria suitable for evaluating the radiation between the matrix and the stone.

Three fundamental options were found in terms of differentiation criteria specific to the process:

9.5 Visualization of ablation crater in (a) hardened cement paste and (b) cement mortar with aggregates up to 2 mm in size, measured with a confocal laser scanning microscope.
• the ratio of intensity of calcium to oxygen;
• the ternary relationship between the intensity levels of the elements, namely silicon, aluminium, iron, calcium and magnesium; and
• the distribution of hydrogen on the dried sample surface.

The calcium to oxygen ratio enables the normalized intensities of both elements to be compared in one spectrum. A raised Ca/O ratio can justify the assumption of a high vaporized proportion of binder in the focal spot. By contrast, low Ca/O ratios indicate a high proportion of grain. This criterion can also be applied to calcareous aggregate, as illustrated in Fig. 9.7. The specific calcium fraction is similar to that in cement but there is a higher proportion of oxygen in these stones which brings the Ca/O ratio below the cement level. This allows a clear separation between binder matrix and mineral aggregate (Fig. 9.8a) in a road surfacing concrete specimen. In Fig. 9.8b the sodium distribution in the binder matrix of that specimen is shown. The detected mineral aggregate that contains sodium is marked by black colour to avoid superposition with the wanted sodium distribution in the binder.

Figure 9.9 contrasts the intensity ratios of selected stones to cement particles in quantitative terms to show that the ternary relationship between the Si, Ca+Mg and Al+Fe fractions provides a suitable indicator for differentiation. Mapping these elements shows their concentrations and thus yields the second differentiation criterion. For the evaluation of ion transport processes in concrete, silicon and aluminium remain stable, even if solvent is applied to the solid matter, and the amount of dissolved calcium...
is negligible in relation to the total concentration, therefore this ternary relationship is retained between the elements.

The distribution of the elements in hydrogen provided a third basis for the differentiation criterion. As exemplified by the concrete sample in Fig. 9.10, the intensity distribution of the hydrogen line H at 656.3 nm over the surface allows a clear differentiation of structures. The mineral aggregates with low H fractions separate from the concentrations of hydrogen in the binder matrix. This differentiation is made possible by the chemically and physically bonded cement stone water, with noteworthy examples of this being the hydration phases.
9.8 (a) Calcium distribution in siliceous aggregate and (b) sodium distribution imaging, reproducing a Ca 825 nm/O 844 nm ratio of ≥0.22 (black = aggregate with Ca/O < 0.22) in a sample of road surfacing concrete.
9.4 Detection of specific elements: specific testing problems

9.4.1 Chlorine: deterioration of reinforced concrete

The strength and durability of buildings made of reinforced and pre-stressed concrete are conditional upon the measures taken to prevent the steel embedded in the concrete from being corroded by the alkalinity of the interstitial water. This protective mechanism can be undone by two processes. One of these processes is the migration of chlorine ions. Regulations have been in place for some years to set minimum levels for the direct addition of chlorine compounds to freshly mixed concrete. Currently, it is chloride contamination that is more likely to be the source of major damage, having arisen as a result of retroactive ion penetration, especially through de-icing agents. The bonding agent, the pore channels, and the moisture in the building materials affect the transfer, diffusion, and attachment of the dissolved chlorides.

The chlorine concentration in the cement or concrete can be determined by consulting the emission line at 837.6 nm (Wilsch et al., 2005) in the visible spectrum. Other CI lines are available in theory but they do not deliver the required sensitivity in practice for the quantity of material requiring detec-
9.10 Hydrogen distribution in a concrete sample (a) via evaluation of H 656.3 nm used to differentiate the matrix from the mineral aggregate on a concrete sample (b).

tion. Alternatively, as corroborated by the latest findings in sub-atmospheric pressures, adequate results can be achieved with the visible-UV emission line at 134.7 nm.

The chlorine line at 837.6 nm, needs to be classified as outside the sensitive range, despite the use of process gases like helium and argon. Substantial increases in the element quantity bring about only minor changes in the normalized intensity. Constant, reproducible measurement conditions are therefore essential.

Nevertheless, it was possible to reliably detect concentrations of greater than 0.1 mass % in cement in the repeat measurements, i.e. below the legally
specified corrosion-inducing chloride content of 0.4 and 0.5 mass % in cement. This concentration can be classified as fair and acceptable for building practice purposes.

It is now possible, on the basis of this distribution, to show the specific Cl gradient plotted against the depth (Fig. 9.11) and its distribution in the building material (Fig. 9.12). Heavily shaded parts indicate high chlorine concentrations.

9.4.2 Sulfur: concrete corrosion

The chemical action of sulfur in the form of sulfates or sulfides, is to attack concrete surfaces causing damage to, or breakdown of, the cement matrix. Considerable bulking is a potential problem with sulfates, e.g. owing to the volume increase through ettringite formation, hence the feared destruction of the cement structure.

One fact of particular note for sulfides is the corrosive action of sulfuric acid on sewage and wastewater treatment buildings.

The detection of sulfur by means of LIBS proves challenging insofar as, firstly, sulfur is already used in the bonding agent as a sulfate transporter and, secondly, there are precious few emission lines for sulfur identification in the visible range. One line which has proved suitable is the S line at 921.30 nm (Weritz et al., 2007).

The zones where sulfuric acid penetrates a concrete test piece can be shown with LIBS imaging, Fig. 9.13. It is evident that the sulfur is mainly absorbed at the edges. Exact localization of the areas contaminated by
sulfur helps in assessing the depth to which excavation of concrete is necessary.

9.4.3 Alkali metals: alkali silica reaction

The term ‘alkali silica reaction’ is used to describe the process whereby the siliceous minerals in some aggregates react with the soluble alkaline pore fluid elements (potassium, sodium) to form a calcium alkali silicate gel which readily absorb water. In unfavourable conditions, the volume of this gel increases to such an extent over time that the pressure gives rise to localized swelling. This pressure exceeds the tensile strength of the concrete and damages the structure. Outward signs of this are crazing, efflorescence and pop-out. Added road salt with alkali cations can intensify the reaction.

Figure 9.14 illustrates the change in alkali concentrations in a hardened cement paste which has been exposed to a 1 M CaCl₂ solution. There is clear evidence of a reduction in the soluble potassium and sodium fractions...
which have migrated into the solution. There is also a decrease in the calcium content as the solution has far lower Ca concentrations than the sample.

It can be concluded after further tests that there is an interaction between the alkali elements, with observations focusing primarily on relationships between Na, K and Li. This allows several basic assumptions:
The alkalis and chlorides in the liquid migrate into the building material at different rates. There is a retardation effect.

- The water front precedes the chloride front. This is followed by the alkalis which act at different rates within their sub-group.
- While the ions in the liquid migrate into the building material, ions are released from the building material and fill spaces left by the liquid.
- There is a displacement struggle as the elements supplant each other. For example, lithium penetrating from outside most notably dispels the potassium fractions in the cement.

The latter assumption is of immense importance for the destructive process of the alkali silica reaction. Lithium can have a restorative effect and minimize or stop this reaction (Thomas et al., 2008). The reason why this less alkaline element should have such a curative effect has remained a mystery to this day.

Another striking additional insight in the applied LIBS measurements was that LiNO₃ solutions penetrated further into the solid matter than the LiOH solutions. Thus, LiNO₃ impregnation produces maximum Li fractions of 0.3 mass % in the solid matter. Once this value is reached, the Li front migrates deeper into the sample (Fig. 9.15). This appears to define an absorption and loading capacity which is not exceeded.

9.4.4 Hydrogen: heterogeneity and moisture

In building work, the construction materials are exposed to atmospheric effects, the majority of which are extreme. If measurements are taken on rain-soaked buildings, therefore, or on buildings in harsh freezing conditions then this can affect the measurements. It was therefore necessary to determine whether a soaked building material supplied identical results to
Another point of uncertainty was whether cold and hot surfaces react alike to the laser beam. Wet samples show an increase in the total intensities in the observed spectra when compared with dry samples which have reached their moisture content equilibrium in interior air conditions. The background level is basically raised with wet samples (Fig. 9.16) and changes the

9.15 Interaction between potassium, sodium and lithium on addition of LiNO₃.

9.16 Change in entire spectrum with wet building material surfaces as opposed to dry surfaces (hardened cement paste).

a dry one. Another point of uncertainty was whether cold and hot surfaces react alike to the laser beam.
signal-to-underground ratio. Possible reasons include the changed absorption properties of the solid surface and a higher braking radiation resulting from the evaporation of the water. It was also apparent, however, that the peak amplitudes of the characteristic spectral lines were more marked.

At the Cl line 837.6 nm, it was possible to draw a qualitative reference to the moisture content. The signal sent by the soaked samples was attenuated in intensity compared with the dry samples.

At the K (769.9 nm), Li (670.8 nm) and Mg (880.7 nm) emission lines, the wetness of the sample has only a slight effect on the intensity at material temperatures ranging from 0 to 85 °C. As the analyte concentration in the samples rises, however, this effect dies out.

By contrast, the temperature of the building material has virtually no effect on the measurement. One reason for this may be that the excited laser beam is already very hot (15000 °C) so that the initial temperature of the material is probably insignificant.

9.5 Mobile set-up: on-site applications

A mobile LIBS set-up with fast on-line results is helpful for the engineer to estimate the condition of concrete structures and for quality assurance during concrete repair work. It ensures for example that just the contaminated concrete is removed, as much as necessary and no more.

Potential applications are parking garages or bridge decks with chloride-contaminated concrete. During repair of sewage treatment plants the sulfur content of concrete structures will be evaluated. In the following, the set-up will be described. The application of the set-up for on-site evaluation of chloride-contaminated concrete structures in a parking garage is shown and the results and future trends are evaluated.

The schematic set-up of a mobile LIBS system for on-site investigations of building materials is shown in Fig. 9.17. On a tripod a pan-tilt unit carries the laser and the measuring head including shielding. The control and detection unit is arranged in a special box. The system is designed for horizontal and vertical operation (Fig. 9.18). To maintain laser safety regulations, a shield covers the laser beam path. For the measurement, the pan-tilt unit moves the laser beam over the surface under investigation (maximum area 15 cm × 15 cm). The plasma radiation is delivered to an optical fibre and analysed by a portable detection unit. The measurement of an area of 15 cm × 15 cm takes about 5 min. The system can be transported in two special containers in the boot of a normal van. The time for system set-up is about 15 min.

The system is designed for measuring chlorine, sulfur or alkali metals. The content of chloride or sulfate is calculated from the measured chlorine
9.17 Schematic diagram of mobile LIBS set-up.

9.18 (a) Horizontal and (b) vertical measuring mode of mobile LIBS.
or sulfur intensity. An automated data assessment program is integrated in the system.

The practical application of a mobile LIBS system for on-site evaluation of the chloride content in a parking garage in vertical operation is shown in Fig. 9.19. During the first on-site test in parking garages the performance of the mobile LIBS system was proved. The equipment is portable and set-up of the system takes less than 15 min. The chlorine, sodium and potassium content in different positions could be evaluated.

9.19 On-site use of mobile LIBS: (a) set-up and (b) plasma (indicated by direction of arrow) inside shielding on concrete surface.
Currently, the mobile system gives a detection limit for chlorine of 1%. However, further development should improve this detection limit to at least 0.4%. The measurements were performed during ongoing concrete removal near the position of measurement.

### 9.6 Limitations and reliability

Atomic spectroscopy methods are relative, therefore calibration must be undertaken to establish the correlation between the analyte concentration and the test signal. Experiments can be conducted on samples with defined and staggered concentrations of elements in order to establish these relations. The staggered samples with known concentrations yield characteristic calibration functions which take the following general form:

\[
I_x = E w_x + b \tag{9.2}
\]

where \(I_x\) is the normalized intensity, \(w_x\) is the concentration in mass percentage, \(E\) is the sensitivity and \(b\) is a constant. These calibration functions of the first degree apply in a linear dynamic range. Qualitative results at lower concentrations are subject to the limit of detection (LOD) whereas quantitative results are subject to the limit of quantitation (LOQ). The slope of the function curve within the linear working range denotes the sensitivity \(E\). If the function leaves the linear working range on account of excessively high concentrations then it flattens out into a non-linear saturation region (Fig. 9.20). This is primarily the result of an overload in the spectroscopic detection system.

**Fig. 9.20** Characteristics and sections of calibration function.
In-house tests (Molkenthin, 2008) with hardened cement paste samples supplied the calibration functions for the main chemical elements contained in the cement matrix and the mineral aggregate (Table 9.1). These limits were obtained on in-house measuring equipment and will be able to be defined far more clearly in the near future with the advances being made in the development of equipment.

### Table 9.1 Process limits in hardened cement paste, specific to the emission lines

<table>
<thead>
<tr>
<th>Emission line (nm)</th>
<th>Limit of detection (mass %)</th>
<th>Detectability limit (mass %)</th>
<th>Limit of quantitation (mass %)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Na 819.48</td>
<td>0.023</td>
<td>0.046</td>
<td>0.07</td>
</tr>
<tr>
<td>K 769.90</td>
<td>0.036</td>
<td>0.072</td>
<td>0.11</td>
</tr>
<tr>
<td>Li 670.79</td>
<td>0.01</td>
<td>0.02</td>
<td>0.03</td>
</tr>
<tr>
<td>Mg 880.67</td>
<td>0.10</td>
<td>0.21</td>
<td>0.32</td>
</tr>
<tr>
<td>Cl 837.59</td>
<td>0.08</td>
<td>0.16</td>
<td>0.2</td>
</tr>
</tbody>
</table>

### 9.7 References


THOMAS, M.D.A., HOOPER, R., STOKES, D. Use of lithium-containing compounds to control expansion in concrete due to alkali-silica-reaktion, 11th ICAAR in Quebec 2000, Tagungsband 2, S. 783–792.
Acoustic emission (AE) evaluation of reinforced concrete structures

C. U. Grosse, Technical University of Munich, Germany

Abstract: The acoustic emission technique is one of only a few methods that are able to monitor and characterize failure in materials as it is occurring. Proper AE applications need a deep understanding of the basics as well as reliable sensors, and recording and data processing techniques that can take account of all boundary conditions (such as material effects, geometry of specimen or structure, and noise conditions), and a careful setup. These basic principles are described and several measurement examples and applications are represented to demonstrate the advantages and disadvantages of this technique.

Key words: acoustic emission, signal localization, fracture mechanics, reinforced concrete, structural health monitoring.

10.1 Introduction

The acoustic emission technique (AET) is different from other non-destructive testing (NDT) methods, because it is usually applied during loading, whereas most other NDT techniques are applied before or after loading of a structure. A structure or structural part is usually tested under a working load without any additional load and the AET is able to detect a failure inside this structure at a very early stage, long before it completely fails.

A more important distinction between the various NDT techniques results from the way the technique is applied. The ultrasound method, for example, is able to detect the geometric shape of a defect in a specimen using an artificially generated source signal and a receiver, whereas the AET detects the elastic waves radiated by a growing fracture. Therefore, the AET should be considered to be a ‘passive’ NDT, because it usually identifies defects as they develop during the test, the elastic waves radiated by the growing defect are recorded. These characteristic features of the AE method result in advantages as well as disadvantages.

An advantage of AE techniques, compared with other NDT, is that damage processes in materials being tested can be observed during the entire load history, without any disturbance to the specimen. Ultrasonic analysis techniques, for instance, have to be applied in conjunction with
scanning techniques to detect a defect. They usually require stopping the loading of a structure. In contrast, AET under favorable conditions require only a few sensors to be able to monitor the AE activity of a structure, provided there are sufficiently strong signals to cross a threshold called the trigger level. The sensors can be fixed to the surface of the specimen for the duration of the test and do not have to be moved for scanning the whole structure point by point. Access to both sides of an object, which is necessary for all through-transmission methods, is not required in AET.

A disadvantage of the AET is that a particular signal is not perfectly reproducible owing to the nature of the signal source, e.g. the sudden and sometimes random formation of a crack. Although specimens of the same shape and same material properties should cause similar AE activities under load, this is not always the case. Materials with scattered inhomogeneities of a particular dimension, such as concrete, will not give similar AE results if the wavelength of the signals is of a similar size as the inhomogeneities. This is one of the reasons why it is useful to compare the results of AET with other testing methods, for example using a visual inspection of the surface or ultrasound methods, x-ray or radar.

Another point addresses the energy released by an acoustic emission. Signals, in particular those used as precursors of failure, are usually several magnitudes smaller than signals used in ultrasonic techniques. This requires much more sensitive sensors as well as reliable amplifiers and pre-amplifiers. Problems related to this are the influence of ambient noise, the attenuation of signals and the resultant low signal-to-noise ratio. It requires sophisticated data processing techniques to detect acoustic emissions, to localize them and to apply other advanced techniques or inversions.

A reliable analysis of acoustic emission signals and the interpretation of the data in material testing are very often only possible in instances where the signals have been localized successfully. Signal localization is the basis of most AET based on signal recordings. Before the localization topic is dealt with, however, a short characterization of the way acoustic emissions are recorded is required. Knowing how signals are recorded is essential in understanding the AET in general, and also provides insights into interpreting the results.

The sources of acoustic emissions can have widely varying characteristics owing to significant differences in the source signals. These differences become more pronounced with use of non-resonant transducers and after separating signals from noise, which can arise from artificial or natural sources with origins inside or outside the tested object.

Continuous emissions, produced for instance during metal cutting or by friction in rotating bearings (Miller and McIntire, 1987), show very different signal characteristics to the burst signals caused by the spontaneous release of energy during cracking (Fig. 10.1). Monitoring of continuous acoustic
emissions can be used to control the operation of machines, although it is often difficult to localize the source of the emission.

### 10.2 Basics: parametric and signal-based acoustic emission (AE) analysis

AET can be divided into two main groups: parameter-based (conventional) and signal-based (quantitative) techniques. Both approaches are currently applied successfully, and it is necessary to understand their differences. If AE events are recorded with one or more sensors, such that a set of parameters are extracted from the signal and later stored but the signal itself is not stored, the procedure is usually referred to as a parameter-based (or conventional) AET. The signals are completely described by the set of parameters and storing this relatively small amount of parameter data consumes less time and storage space than if entire waveforms are stored. Some typical AE parameters extracted by conventional AE equipment are the maximum peak-to-peak amplitude, the arrival time (defined as the first crossing of a given amplitude threshold), the rise time (defined as the duration between the arrival time and the time where the maximum amplitude is recorded) and the duration (defined by the last crossing of a given amplitude threshold) (ASTM E610, 1982; DIN EN 1330-9, 2008). A typical signal measured (but not stored) in AE analysis using the conventional approach is given in Fig. 10.2.

10.1 Example of burst signals (top) compared with a continuous emission of acoustic waves (bottom) (Grosse, 1996).
Using the so-called quantitative AET, as many signals as possible are recorded and stored, along with their waveforms, which are converted from analogue-to-digital (A/D) signals. A more comprehensive (and time-consuming) analysis of the data is possible using this approach, but usually only in a post-processing environment and not in real-time. The term ‘quantitative’, as it is used in AET, was introduced by several authors (Sachse and Kim, 1987; Scruby, 1985) to compare this technique of waveform analysis with the conventional parameter-based techniques. Furthermore, the two different approaches are more related to the way the information is stored, although the terms ‘parameter-based’ or ‘signal-based’ AET are used with preference in this text.

When discussing the advantages and disadvantages of these two approaches it is important to keep the specific application in mind. This is true for the simple counting of the number of AE occurrences in a material under load by a single sensor, as well as for arrays of many sensors, used for localizing and analyzing data in the time and frequency domain.

10.2.1 Parameter-based acoustic emission techniques (AET)

The main advantages of the conventional AET are the high recording and data storing speeds that facilitate fast visualization of the data. This makes the technique very economical. In contrast, when an entire signal waveform recorded by several sensors is stored (signal-based approaches), the recording system shuts down for a short period (called the delay or dead time) while
the information is being stored. This can result in a loss of information. Storing only some parameters reduces this delay time significantly.

Conversely, reducing a complicated signal to only a few parameters can be a significant limitation, and can sometimes be downright misleading. In practical applications, it can be difficult to discriminate an AE signal from noise (e.g. caused by electronic pulses) after the signal has been reduced to a few parameters. This is especially true when resonant sensors (see following section) are used because such sensors result in the signal differences being further hidden. The extraction of simple parameters characterizing the signal can also be difficult when broadband sensors are used. An added complexity is that in many experiments the parameters of AE signals are strongly related to the material and the geometry of the structure. Various wave modes related to compressional, shear or surface waves, as well as reflections contribute to the shape of the signal enhancing its complexity.

The latest measurement devices are able to extract AE parameters while automatically localizing the signals. With these devices, it is usually not possible to control the algorithms used for this procedure, or to influence the accuracy of localization. Several hundreds, or even thousands, of events can be parameterized and localized by these black-box devices per minute. However, even for situations where a large number of AE signals have to be handled and the ‘conventional’ approach of AE testing is the chosen method, it is highly recommended that some AE signal waveforms, selected at random, are recorded and inspected to assess whether the monitoring system is working properly and to check the noise conditions.

10.2.2 Signal-based acoustic emission techniques (AET)

If a signal-based approach is used for example in a laboratory environment, the waveforms recorded by the sensors (preferably broadband sensors) need to be analyzed. A suite of analysis procedures exists to evaluate fracture parameters. The first step in the analysis is usually the 3D localization of the rupture. More advanced analyses may then be applied, for instance, source mechanism calculations.

One of the biggest advantages of signal-based AET is the capability of signal-to-noise discrimination based on waveforms, because the waveforms are still available after the measurement and not deleted as is the case usually in parameter-based applications. Additionally, it is possible to apply different signal analysis methods using post-processing software. This software might include classification algorithms or different filtering techniques to enhance the signal-to-noise ratio and thus help to extract information about the material properties.

The reliability of the data interpretation can be improved significantly if signal-based methods are used. The drawback of signal-based approaches
is that usually a smaller number of events can be recorded. Although the time for immediate parameter extraction is saved, a large number of signals have to be stored digitally. To reduce the amount of data not related to the material failure, sophisticated trigger algorithms (such as reference band or slew rate) have to be applied. On the one hand, this may appear to add an artificial interference. However, one has to recognize that parameter-based methods usually use an arbitrarily set threshold level to trigger the parameter extraction process and this can also cause artifacts. On the other hand, not all the AE signals can be recorded because of dissipation and geometrical spreading effects that absorb many of the weak signals before they reach the surface of the structure.

The question of which of the parameter or signal-based AE techniques are more useful does not have a definite answer. The attributes of the different methods are summarized in Table 10.1. As stated, the differences

<table>
<thead>
<tr>
<th>Parameter-based AET</th>
<th>Signal-based AET</th>
</tr>
</thead>
<tbody>
<tr>
<td>Failure detection</td>
<td></td>
</tr>
<tr>
<td>Large-scale structures</td>
<td>Small-scale structures</td>
</tr>
<tr>
<td>Localization</td>
<td></td>
</tr>
<tr>
<td>1D (zonal)</td>
<td>Requires many sensors</td>
</tr>
<tr>
<td>2D (planar)</td>
<td>Minimum three sensors</td>
</tr>
<tr>
<td>3D</td>
<td>Minimum four sensors</td>
</tr>
<tr>
<td>Fast real-time data analysis</td>
<td>Requires PC with memory</td>
</tr>
<tr>
<td>Statistical analysis</td>
<td></td>
</tr>
<tr>
<td>Requires PC with memory</td>
<td>Requires PC with memory</td>
</tr>
<tr>
<td>Analysis of:</td>
<td></td>
</tr>
<tr>
<td>• amplitudes</td>
<td>Only statistical analysis</td>
</tr>
<tr>
<td>• frequencies</td>
<td>Requires broadband sensors</td>
</tr>
<tr>
<td>• waveforms</td>
<td>Sampling frequency &gt;1 MHz</td>
</tr>
<tr>
<td>Fracture analysis:</td>
<td></td>
</tr>
<tr>
<td>• fault-plane orientation</td>
<td>Minimum six sensors in the farfield</td>
</tr>
<tr>
<td>• fault-plane size</td>
<td>Distributed sensors</td>
</tr>
<tr>
<td>• fault-plane energy</td>
<td>Moment tensor inversion</td>
</tr>
<tr>
<td>• fracture mode (I, II, III, mixed)</td>
<td>Moment tensor inversion</td>
</tr>
</tbody>
</table>
between the methods are vanishing as technology improves. However, current trends definitely favor the signal-based techniques rather than others. The decision of which technique should be applied is often also a matter of data storage capacity and electronic components and is limited also by the financial constraints of a project.

### 10.3 Sensors and instruments

AET depend on the characteristics of all parts of the equipment used to record and analyze acoustic emissions and probably most of all on the type of sensor used.

#### 10.3.1 Acoustic emission (AE) sensors

Almost exclusively, the sensors used to analyze acoustic emissions are those that exploit the piezoelectric effect of lead zirconate titanate (PZT). Although piezoelectric sensors and their design are described in numerous books and papers (e.g. Hykes et al., 1992; Kino, 1987; Krautkrämer and Krautkrämer, 1986), some characteristics play an influential role in AE measurements and need to be highlighted. These features are important for the sensitive recording of acoustic emissions (sensitivity) and the broadband analysis of the signals with reference to fracture mechanics (frequency).

To enhance the detection radius of piezoelectric sensors to AE signals, they are usually operated in resonance, i.e. the signals are recorded within a small frequency range owing to the frequency characteristics of the transducer. The disadvantage is that an analysis of the frequencies present in the signal is of no value, because these frequencies are always the same. Very well damped sensors, such as those used for vibration analysis, are operated outside of their resonant frequency allowing broadband analyses to be performed, but are usually less sensitive to acoustic emission signals. Progress in the development of the theory of AE has led to the need for high sensitivity, wideband displacement sensors that have a flat frequency response (i.e. the sensor gives the same response over a wide frequency range). There are many papers dealing with a solution of this problem. For many years a NIST (National Institute for Standards and Technology) conical transducer developed by Proctor (1982, 1986) out of a Standard Reference Material (SRM) and mass-backed (600 g) was used as a reference for AE measurements. Several new approaches have explored other transducer materials out of polyvinylidene fluoride (PVDF) or copolymers (Bar-Cohen et al., 1996; Hamstad, 1994, 1997; Hamstad and Fortunko, 2005) as well as embedded sensors (Glaser et al., 1998). Recently, a new high-fidelity sensor was developed by Glaser that is based on the NIST-type sensor concept combining it with even higher sensitivity and robustness.
However, most sensors used currently in AE applications for concrete are manufactured in a more traditional way, showing either a resonant behavior or several particular resonances. These sensors, which are called multi-resonance transducers, have a higher sensitivity than sensors with a backward mass used outside of their resonance frequency. Such sensors should not, however, be considered as (true) broadband and it is essential to know their frequency response function. Otherwise, signal characteristics from the source are not distinguishable from artifacts introduced by incorrect knowledge of the frequency response. A calibration of the sensors’ frequency response, as well as understanding of the direction sensitivity, is important for many applications of AET.

10.3.2 Acoustic emission (AE) recording devices

A typical AE recording device consists of the analogue-to-digital converter, amplifiers and/or pre-amplifiers as well as storing facilities. For the A/D-converter part, transient recorders can do a good job. However, commercially available AE devices differ from transient recorders. Some are also able to record AE events but have to be treated as black-box devices.

Piezoelectric sensors transform displacement into a voltage and amplifiers are usually used to magnify the AE signals. Because cables from the sensor to the (primary) amplifier are subject to electromagnetic noise, specially coated short cables are used. Pre-amplifiers with state-of-the-art transistors are used to minimize the amount of electronic noise. Amplifiers with a flat response in the frequency range of interest are best. If available, transducers with integrated preamplifiers for an appropriate frequency band are often desirable. Another issue is the astonishing dynamic range of AE signals. The broad dynamic range requires gain-ranging amplifiers (storing the signal in analogue into a buffer for best amplification adjustment before digitization), but they are usually not yet available concerning commercial AE equipment.

With regard to the data acquisition system (DAS), there are two main problems concerning the A/D conversion and the triggering. Fast A/D units have to be used to ensure that a large number of events are recorded, usually there are A/D converters for each channel of the recording unit. Anti-aliasing (low pass) filters are required so that the signal can be properly transformed to the frequency domain by means of Shannon’s theorem (Rikitake et al., 1987).

If possible, applying different triggering conditions can reduce the amount of noise that is recorded. Simple threshold triggers are usually not adequate for many purposes in AET. More sophisticated techniques such as slew-rate, slope or reference band triggers are more appropriate. In modern computer-based systems, the time used to convert a signal from analogue to digital,
and to store it to a hard disk, is in the range of microseconds depending on the signal length. There are still many areas, where AE equipment can be improved. It is important to keep up-to-date about the recent developments in AE recording technology.

10.4 **Source localization**

Quantitative methods in AET rely on localization techniques to determine the co-ordinates of the emission source, to image cracks in the material. The method used to localize AE events depends on the geometry of the object being tested, and whether the resolution is required in one, two or three dimensions. Although a detailed description of these methods can be found in the literature (Grosse and Ohtsu, 2008), basic concepts, in particular using full waveforms, are described here to understand signal-based AE procedures.

10.4.1 Zone and planar location methods

The simplest way to locate the source of AE is the so-called zone location method. The exact source co-ordinates are not determined, but the defect is located within a radius of the sensor’s sensitivity range (or in the case of a plate-like structure, within a zone).

The zone location method is frequently used to monitor large structures such as buildings, pipes, and vessels. Sensors are distributed over a wide area on the surface of the structure and/or concentrated at the most critical locations. To enhance the detection radius, resonant transducers, which are more sensitive than broadband sensors, are the first choice. The failure must be within the detection range of the sensor to allow the AE to be detected. If AE is recorded by a particular sensor, the expert should inspect the vicinity next to this sensor for leaks or cracks. For one-dimensional objects (such as rods and pipes) a linear location method can do a good job in locating the source of the acoustic emission. This problem can be solved with planar location techniques using two sensor information.

A planar localization technique can be applied to two-dimensional structures, where the thickness is small compared with the length of the object and source co-ordinates are only required in two directions. Simple planar localization of cracks in a plate is shown in Fig. 10.3. Although only three unknowns (the two source co-ordinates and the time of origin) have to be determined, recordings from only three sensors are sufficient. Since compressional waves are used, the equations applied to calculate the source locations are similar to those of 3D localization techniques.

When the waves have wavelengths shorter than the thickness of the structure, plate or Lamb waves have to be used and their group velocities...
considered. The 2D method to locate AE sources is usually applied when the accuracy of zonal technique is inadequate for the application. Two-dimensional localization methods in civil engineering are often used to monitor large structures, such as bridges (Kapphahn et al., 1993). However, array techniques (as described later) can alternatively be used without the disadvantage of a spread distribution of sensors.

10.4.2 Basics of 3D localization

Several authors have applied 3D localization methods to AE data in civil engineering (Berthelot and Robert, 1987; Grosse, 1996; Köppel and Grosse, 2000; Labusz et al., 1988). The technique is similar to those applied in seismology, where earthquake hypocentres are determined using the arrival times of earthquake waves recorded at multiple seismometers. A commonly used method (the so-called ‘Geiger’ method) is based on the ideas of the German seismologist Ludwig Geiger (1910) and uses the arrival times of the first and second arrivals, the compressional (or P) and shear (or S) waves. The compressional waves travel at a higher velocity than the shear waves, so arrive at the sensors first, followed by the shear waves. This concept is well illustrated by the seismogram in Fig. 10.4, where the arrival time of the P wave \( t_P \) is less than that of the S wave \( t_S \).

The seismological algorithms can be adapted to the requirements in material testing and enable the study of different specimen geometries by considering the number of transducers and their position around the object. For the determination of the AE hypocenter (source) using the arrival times, the problem is exactly determined when four arrival times are used to calculate the four unknowns: three co-ordinates and the source time of an event. When more than four arrival times are available, the problem is over-determined and the calculation is performed using an iteration algorithm, where the co-ordinates are estimated by minimizing the errors of the
unknown parameters (Buland, 1976; Salamon and Wiebols, 1975). The more recordings, the more over-determined the system, and the more reliable the evaluation will be.

In seismology, it is common to use both P and S waves, because of the clear arrival times of shear waves (Fig. 10.4). However, in AET of small structures, the P waves are followed closely by S waves, resulting in the onset time (or first arrival, $t_S$) of the S wave being hidden in the wave train following the P wave.

Software to perform AE source localization should be able to calculate the standard deviations of the hypocenter and the source time, the weight of a single station, the number of iterations to be used and the residual of each channel with respect to the determined best arrival time.

An example of the onset time detection using an eight channel recording of an AE is given in Fig. 10.5. The equations used generally assume that the material is homogeneous and isotropic, and that the AE source resembles a point source. If this is not the case (e.g. for an anisotropic material like wood), the computational approach has to be modified. A different approach to localize AE sources by array techniques is described in the application section later.

The graphical representation of AE data that has been localized in 3D is a very important aspect of AE analysis. There are several ways to visually represent the data. A common approach is to draw projections of the $x/y$, $y/z$ or $x/z$ planes. The representation in a 3D co-ordinate system, as shown in Fig. 10.6, can lead to useful insights when the graphical data is animated, such features being implemented in most modern AE data analysis software. Representing 4D data (three co-ordinates of the sources and the time

10.4 Seismograms recorded by a triaxial sensor in the underground mining environment showing clear P and S wave arrivals (Grosse and Linzer, 2008).
10.5 Example of the onset time extraction for 3D localization of AE (Grosse and Reinhardt, 1999).

10.6 Example of the 3D visualization of AE data (zoomed to 50 mm³) along with the energy emitted by each acoustic emission as represented by the radius of the sphere. The dots on the bottom are the projections of the spheres to the x/y plane (Grosse et al., 2003).
of occurrence of all events) using print media, however, is difficult. To overcome this problem a record of the chronology of the AE events can be shown by simply numbering the source points. Additionally, the influence of heterogeneities (voids, reinforcement) can be evaluated (Grosse et al., 1995) via the analysis of the arrival time residuals at sensors in the shadow zone, which is the zone where the reinforcement bar is in between source and receiver.

Figure 10.6 shows an example of the AE localizations during a pull-out test of a steel reinforcement bar, in a concrete cube having a side length of 200 mm. The bond between steel and concrete is limited to a length of 40 mm, in the middle of the cube. The large spheres indicate the position of the 3D AE localization in the cube, and the small points at the bottom are projections on the $x/y$ plane. As expected, most of the AE sources are located near the ribbed portion of the bar, which is the area of largest load amplitude in the cube, resulting in local crushing at the concrete bar interface owing to compression by the steel ribs. The energy of each acoustic emission event emitted is represented by the radius of the spheres (Grosse, 2000).

Other aspects dealing with limitations and accuracy of AE localization are addressed in Section 10.7.

10.5 Source mechanisms and moment tensor analysis

Fracture types are of interest when studying problems of fracture mechanics and in understanding and classifying the way a material fails. Various terms are used to describe the cracking behavior. In the following section the terms ‘opening crack’ and ‘mode I’ are synonymous, as well as ‘mode II’ and ‘mode III’ for shear cracks, with forces parallel to the crack (in-plane shear) or forces perpendicular to the crack (out-of-plane shear). In seismology, shear dislocations are described by a double couple (DC) source because the DC force representation allows simplification of some mathematics.

Inversion methods are used to determine the fracture type and orientation of a rupture (fault), as well as the seismic moment, which describes the rupture area that is related to the released energy from the waveforms of the recorded AE events. As illustrated in Fig. 10.7, the failure of a brittle specimen (left) is accompanied by a sudden release of energy in the form of acoustic waves. Using an inversion algorithm, in combination with 3D localization, a fault plane solution can be determined (right) that enables the analysis of the fracture process in the material. Another more comprehensive method of fracture analysis is the application of moment tensor inversion methods. In this section, some examples of simple inversion techniques are given.
There are several ways to determine the crack type and orientation of AE sources. One way is to use the polarities of initial P-wave pulses – this is known as the first motion technique. The distribution of the two senses of the wave polarity around the focus is determined by the radiation pattern of the source. Using the distribution of the polarities, it is possible to estimate the orientation of the nodal planes (where no displacement takes place) and thus the mechanism of the source. However, it is important to bear in mind that, owing to the symmetry of the radiation pattern, two orthogonal planes can be fitted. These planes are often referred to as the ‘fault plane’ and the ‘auxiliary plane’.

Positive polarities would be measured at all sensors, in the case of an opening crack (mode I). For a shear fracture, the polarity of the P-wave onset changes from positive (upward deflection of waveform) to negative (downward deflection) according to the position of the sensor relative to the source and the shear planes (Fig. 10.8). These two examples assume that the sensors have been calibrated properly, so that a positive deflection of the signal indicates movement away from the source (i.e. compression).

If the radiation pattern of the source is to be analyzed, it is important that the pattern is sampled adequately over the focal sphere. This implies that many transducers are used, providing good coverage of the focal sphere, i.e. a good distribution of sensors over all angles with respect to the fault plane. A minimum of 23 sensors is required to uniquely characterize the mechanism (Lockner, 1993). For a smaller set of sensors, moment tensor inversion is more suitable (see later) to estimate the failure mecha-
nisms. Unfortunately, it is not possible to quantify the deviation from a pure shear dislocation and to determine isotropic components of the source with this first motion technique.

An alternative and more comprehensive procedure is the inversion onto the moment tensor of a fracture that is the source of an acoustic emission. The seismic moment tensor consists of a set of point sources that describe the source mechanism and allow the slip planes (nodal planes) to be determined. The moment tensor is a useful concept because it can be used to completely describe the radiation pattern and the strength of the source. However, because of its complexity and instrumentation prerequisites, the range of applications of this technique is limited to laboratory experiments only, in spite of the fact that in other fields this technique is routinely used. Currently, no applications are reported dealing with real civil engineering structures but test results have been reported (Grosse and Ohtsu, 2008).

### 10.6 Applications

The differences described above between parameter- and signal-based AET have a big influence on the setup and data processing of applications. While signal-based techniques are generally speaking more appropriate for laboratory applications, field applications are usually based on parameter-based techniques. Several field applications are described in chapter 24. Therefore, only two examples, one of a laboratory test and one of a field test, are described in the following section.

10.8 Radiation pattern of a vertically oriented shear crack showing variation of polarities and amplitudes with angle from the source (Grosse et al., 2003).
10.6.1 Example of laboratory applications

To generate controlled tensile failure in concrete, a splitting test according to (DIN 1048-5, 1991) using an unreinforced concrete cube (edge length 200 mm) was performed. Compressive load was applied using two parallel steel edges, one on the top and the other on the bottom of the specimen (Fig. 10.9a). Applying a load to the steel edges causes tensile cracks parallel to the edges. A controlled opening of the tensile cracks is necessary to obtain as many acoustic emissions during the evolution of the crack as possible and to enhance the significance of the AE analysis. Therefore, the crack opening was measured by two linear displacement sensors (linear variable differential transformer, LVDT), one on each side of the cube. The mean crack opening acted as the control parameter for the loading device.

Acoustic emissions were registered by eight piezoelectric ultrasonic sensors and recorded by a transient recorder (Grosse and Finck, 2006). Additionally, piston displacement, load, and crack opening were recorded. Results of the mechanical test data and the acoustic emission rate over the test period have been reported (Finck et al., 2003; Reinhardt et al., 2005). An overview of the results of the moment tensor inversion is presented in the form of the radiation patterns in stereographic projections (Fig. 10.9b). The decomposition of the moment tensors is revealing the percentile contributions of isotropic, DC and compensated linear vector dipole (CLVD) mechanisms (Aki and Richards, 2002). Significant positive isotropic components and a mixture of DC and CLVD mechanisms are evident.

10.9 (a) Set-up of the splitting test; (b) topography of the crack surface and the radiation patterns of selected events from the two clusters. Both radiation patterns reveal positive (dark) amplitudes parallel to the mean tensile stress and some negative (light gray) amplitudes parallel to the z axis owing to compressive stress (Finck et al., 2003).
Subsequent to the experiment, the specimen was ground in a stepwise manner, to digitize the run of the crack through the various sections. Thereby, the topography of the crack could be evaluated. The results of the localization of the acoustic emissions (black dots) and the inner crack surface (reticule) are plotted in Fig. 10.9b. As a result, the radiation pattern of both events is plotted as concentric spheres around the epicenter. Tension (dark) is dominant in the source representation being more or less parallel to the y axis and therefore parallel to the mean tensile stress and perpendicular to the load direction. A small compressive component almost parallel to the z axis is indicated by negative amplitudes (light gray).

The results corroborate the assumption of mode I failure with a major tensile crack parallel to the steel edges from which load was applied. The mean tensile stress axis is parallel to the y axis, which is also the preferred direction of positive amplitudes. The investigations reveal tensile fracturing as the dominant cause of failure. However, variations of the tension axes and deviations from pure tensile failure are observed. Inhomogenities, such as aggregates, generate undulations of the crack surface and can be responsible for an enhanced complexity of microcracking. Corresponding shear stresses would explain the significant DC components.

The results from this experiment were used to compare the localization results with the actual crack topography. After a stepwise grinding of the specimen, the 3D locations of the hypocenters were compared with the visible crack pattern. In Fig. 10.10a, the projections are shown of the AE events onto the x/y, y/z and x/z planes giving information also about their time-dependent evolution by using a gray scale for the AE locations. In Fig. 10.10b, the event pattern is compared with the crack visible in one of the ground slices. A close correlation between both is evident.

10.6.2 Example of field applications

Currently, AE data analysis techniques are often not appropriate for the requirements of structural health monitoring (SHM) of large structures in civil engineering. This is in particular true considering the length and large number of cables for AE applications usual necessary to monitor structures. On the other hand, wireless AE recording techniques (Glaser, 2004; Grosse, 2008) becomes more and more applicable since sensors become smaller and proper wireless data transmission techniques are available. However, the use of wireless networks for AE recording using conventional triangulation localization is still not appropriate owing to the very exact time synchronization needed between multiple sensors, and power consumption issues. To unleash the power of the acoustic emission technique on large, extended structures, recording and local analysis techniques need better algorithms to handle and reduce the immense amount of data generated. Most
problems can be solved by utilizing a new concept called acoustic emission array processing to locally reduce data to information. Array processing of AE data is a powerful method of concentrating many tens of thousands of data points recorded from one AE event into a single value, the azimuthal direction of the source relative to the known orientation of the sensor. If this is the information needed by the operator, the method is an obvious boon. In fact, a bridge is extremely seismically noisy, so that any AE signal recorded will have a low $S/N$ ratio. In AE field applications, there is actually little information present in a recorded acoustic emission signal beyond its directional relation to a sensor array.

Similarly to phased-array signal processing techniques developed for other non-destructive evaluation (NDE) methods, this technique adapts beam-forming tools developed for passive sonar and seismological applications for use in AE source localization and signal discrimination analyses.

10.10 (a) History of the occurrence of AE events: darker spots represent events occurring at a later load stage than lighter ones; (b) localizations of the hypocenter of AE events (white spots) compared with the crack formation observed after grinding (Finck, 2005).
It has been used extensively in radar (Haykin, 1985), sonar (Carter, 1981), and exploratory seismology (Justice, 1985; Kelly, 1967), and has been utilized as a tool for non-invasive testing techniques for spacecraft (Holland et al., 2006), pipelines and pressure vessels (Luo and Rose, 2007; Santoni et al., 2007), and medical applications (Kim et al., 2006). It has also been used for active damage detection in civil engineering materials (Azar and Wooh, 1999; Sundararaman et al., 2005), but it has not been applied to the method of acoustic emission (McLaskey et al., 2008).

For beam-forming applications, the user must assume that the wave field is relatively constant normal to the direction of propagation of waves incident upon the array (Dudgeon, 1977). AE sources are usually considered point sources, so the ‘delayed replica’ assumption is only valid if the distance between the source and receivers is large compared to the distance between neighboring sensors (easy to insure). The relatively delayed signals can be combined (or stacked) to form an array output with improved $S/N$.

Instead of using P wave picking algorithms, this method uses the energy-rich Rayleigh wave (Kelly, 1967) and a small array of four to eight sensors. Instead of using a distributed array, the beam-forming AE method relies on a small array of sensors spaced closely enough that, in the frequency range of interest (less than 50 kHz), all sensors will detect AE waves that have propagated along similar paths, and have been affected by similar attenuation and scattering. In the beam-forming AE method, the direction of arrival of the AE waves can be determined simply from the relative time delays of individual acoustic emission signals.

For the reported tests, arrays of eight, low frequency (50 kHz) resonant-type AE sensors were set into an approximately 250 mm diameter circle. This size easily allows the array to be serviced by a single AE mote, and only the back azimuth needs to be sent back through the network. Relative timing between each array sensor can easily be kept to 1 μs within a single mote. The assumption that the structure is plate-like, so the depth of source is a higher order term, is valid for the bridge-deck structures monitored. In general, beam-forming can accurately locate the AE source to within five to ten degrees of the actual azimuth (McLaskey et al., 2008). As expected, better quality sensors and denser arrays gave more accurate results, but not significantly better for most field applications.

To demonstrate array processing procedures, the equipment was installed for measurements of strain and AE during static loading of a large prestressed reinforced-concrete bridge deck model (Fig. 10.11) at the Technical University of Braunschweig, Germany, and at a smaller steel-reinforced concrete structure at the University of Stuttgart. Since both structures are subjected to little ambient noise, the influence of larger traffic noise was studied at the smaller structure (Grosse et al., 2007a). However, only the results obtained at the model bridge are discussed below.
The maximum detection radius of source to sensor array was investigated using standard ASTM E 976-99 test sources (break of a pencil lead). At a maximum, signals could be measured with usable signal-to-noise ratio at a radius of 10 m. In practice, noisy AE signals at a radius of 4.10 m (small source) and 6.90 m for a strong source produced by a forklift truck on top of the structure or a car were usable. This relatively long sensor distance indicates that beam-forming AE source location method can monitor a useful area of a bridge deck, for example.

The model bridge deck was loaded downward, with some small eccentricity to the right of the center (Fig. 10.12 top). The recorded AE waveforms from simulated damage were subdivided according to their signal-to-noise ratio into ‘category 1’ (good), ‘category 2’ (moderate) and ‘category N’ (possibly noise). The classification criteria to assign the various signals into their proper groups were obtained by training the Strintzis K-means algorithm (1999), one of the most widely used (Charalampidis, 2005; Ruspini, 1969). By comparing the incoming energy levels of the signals from the different sets over time, it is evident that most of the energy in the category 1 type signals will arrive earlier than in the other two categories. Incoming power can be compared with an assumed constant power influx that would result in the same energy for a given observation length of $N$ samples. Mathematically this is stated as:

$$u_i(n) = \sum_{l=1}^{l=n} s_i^2(l) - \frac{n}{N} \sum_{l=1}^{l=N} s_i^2(l)$$  \[10.1\]

where $s_i(n)$ denotes the $n$th signal sample at sensor $i$ and $N$ denotes the number of samples observed. The function $u(n)$ is thus a measure of the
incoming power in relation to a constant power influx. Because of the large variations in energy of the recorded signals, all waveforms were normalized according to energy in both property spaces (the sampling interval of all waveforms was 1 μs). Effectively this made the term to the right of the minus sign in equation [10.1] redundant. Figure 10.13 shows some examples of the $S/N$ categorized signals, $s(n)$, and their location on the deck relative to the AE array (indicated by the arrow in Fig. 10.12, top) as well as typical $u(n)$ functions, normalized according to energy content (smoothed). More information is given in (Grosse et al., 2007b).

In the lower right of Fig. 10.12, the results of the beam steering localization are presented. In the beam-former used here, the delays are computed for an assumed direction of arrival for all apparent velocities of the incoming wave and the corresponding signals are delayed according to the computations performed. When the true direction of arrival (back-azimuth) of the incoming wave matches the assumed one, the signals add coherently and a maximum in energy is obtained. If the computed delays are denoted by $\Delta \tau_{ic}$ the output of the delay-and-sum beam-former can be stated mathematically in continuous time as:
where $s_i(n)$ denotes the $n$th signal sample at sensor $i$ and $N_s$ denotes the number of sensors in the array and $y_c(t)$ is the beam formed according to a reference point $c$. Figure 10.12 shows that a better $S/N$ results in a more accurate localization of the AE events. The back-azimuth of category 1 beams vary in a range of 30° and category 2 in a range of 45° whereas category $N$ signals are more or less uncorrelated. More information about the results of these experiments was published by Grosse et al. (2007b).

$y_c(t) = \sum_{i=1}^{N_s} s_i(t - \Delta \tau_{ic})$  \[10.2\]

10.7 Limitations and accuracy

It is not possible to address in detail all issues related to limitations regarding AET. Two examples of problems occurring during statistical parameter-based analyses of AE data are given. The Kaiser effect, which was first investigated by Wilhelm Kaiser in 1950, describes the phenomenon where
A material under load emits acoustic waves only after a primary load level is exceeded. During reloading, these materials behave elastically before the previous maximum load level is reached. If the Kaiser effect is permanent for these materials, little or no AE will be recorded before the previous maximum stress level is achieved. The effect is illustrated in Fig. 10.14 in an experiment where a concrete cube, subject to compression, was tested under a cyclic load. The figure shows the AE rate versus time, and the applied load versus time. The described effect camouflages pre-existing cracks if they develop before AE recording.

In a second example, eight sensors were coupled to the surface of a concrete cube (100 mm side length) having a centrally placed reinforcement bar of 16 mm diameter with limited bond (Fig. 10.15). The bond length was limited to 40 mm (5 ribs) to minimize the number of sources producing acoustic emissions owing to local damage. During a pull-out test, the bar was pulled downwards, whereas acoustic emissions were recorded at all eight sensors. The results of the force and slip measurements for tests with different load histories (monotonically increasing displacements, cyclic loads and long term loads) were summarized by Balázs et al. (1996); the automatically extracted peak amplitudes of the burst signals versus time in the form of histograms represent a statistical evaluation of acoustic emissions (Fig. 10.15a). Unfortunately, this procedure does not consider the relative location of the sensors and the sources, and this can lead to false interpretations of the data. For example, in Fig. 10.15a the peak amplitudes of the first AE events, which are related to fractures in the upper region of the specimen (Fig. 10.15b and c), are higher for channel numbers 7 and 8, whereas channels 1 to 6 suggest later

10.14 Example of the Kaiser effect in a cyclically loaded concrete specimen. Thick black lines represents the AE activity, thin lines the load, and the dashed lines indicate the Kaiser effect (Grosse and Linzer, 2008).
events to be the ones with higher amplitudes. The reason for this is that sensors 7 and 8 are located closer to the cracks in the upper region that is more active at the beginning of load, whereas the other sensors represent the evolving cracks at later stages. This shows clearly that a histogram graph of acoustic emission data is sometimes more dependent on the location of the sensors, than on the failure of the material.
10.7.1 AE localization accuracy

The accuracy of AE localization is limited by several factors, such as the signal-to-noise ratio and the sampling interval of the digital acquisition system equipment. If the sampling frequency is set to 1 MHz for each channel, for example, the accuracy of the arrival times is limited to 2 μs or less. In concrete, the localization error for laboratory experiments using currently available equipment and triangulation methods is usually between 1 mm and 1 m, depending on the size of the tested structure and the distance of the sources to the sensors. Providing error information about location calculations is essential to interpret the results. Without this information, there is no way to assess the reliability of localization. An example is given in Fig. 10.16 showing data from the experiment described in Fig. 10.6.

10.7.2 Accuracy picking signal onset times

Another factor affecting the accuracy of AE calculations is the onset time determination. During a routine test usually hundreds, or even thousands, of events are recorded, and automatic picking algorithms to extract the
arrival times at each sensor are essential. Using a simple threshold algorithm is not sufficient in many instances. Under good signal-to-noise conditions, the accuracy of automatic algorithms should be in the range of several data samples.

In applications where a detailed analysis of failure is required, the need for high localization precision is combined with a high AE event rate, and more advanced onset picking algorithms should be used (Kurz et al., 2005). The main trends are outlined in Landis et al. (1992), Zang et al. (1998), Grosse and Reinhardt (1999) and Grosse (2000).

10.8 References

balázs g l, grosse c u, koch r, reinhardt h w (1996), ‘Damage accumulation on deformed steel bar to concrete interaction detected by acoustic emission technique’, Mag. Conc. Res. 48, 311–320.

bar-cohen y, xue t, lih s s (1996), ‘Polymer piezoelectric transducers for ultrasonic NDE’, NDT.net 1(9), 7.


finck f, yamanouchi m, reinhardt h w, grosse c u (2003), ‘Evaluation of mode I failure of concrete in a splitting test using acoustic emission technique’, Int. J. Fract. 124, 139–152.


11
Magnetic flux leakage (MFL) for the non-destructive evaluation of pre-stressed concrete structures

G. SAWADE, University of Stuttgart, Germany; H.-J. KRAUSE, Forschungszentrum Jülich, Germany

Abstract: The magnetic flux leakage (MFL) measurement method is a non-destructive and contactless method for the inspection of the tendons of pre-stressed concrete structures. This chapter describes the basics of the MFL as well as the procedures of the signal analysis. Examples of on-site investigations are also presented.

Key words: magnetic flux leakage, pre-stressed concrete, non-destructive evaluation.

11.1 Magnetic method for inspection of reinforced concrete structures

11.1.1 Aim of the magnetic flux leakage (MFL) measurement method for the detection of ruptures of pre-stressed tendons

Pre-stressed concrete structures are manufactured using high-strength steels. These pre-stressed steels are arranged inside a duct (post-tensioned concrete) or directly inside the concrete with immediate bonding. High-strength steel (in particular, quenched and subsequently tempered steel) is at risk from hydrogen-induced stress corrosion (Isecke 1990). As result of this corrosion, brittle (low-ductility) ruptures of the steel occur. The accumulation of such ruptures in the cross-section can lead to a sudden collapse of the structure without any visible warning signs such as cracks in the concrete.

An example is the Ynys y Gwas bridge in Great Britain which collapsed in 1985 after 32 years in service, a short time after a regular visual inspection in which no evidence for any damage was discovered (Podolny 1992). Another example is the partial collapse of the Congress Hall in Berlin in 1980. Nürnbergber (1992) described the mechanism of the hydrogen-induced stress corrosion of pre-stressed steels in detail. The main difficulty in
assessing the condition of pre-stressed concrete structures is that failure of
the pre-stressed members may occur as a sudden collapse without any
advance notice such as concrete cracks or large deformations. This follows
from the fact that the concrete retains its pre-stressing up to a certain
number of broken steels (loss of cross-section of the tendons).

Therefore, it is important to detect the ruptures before the critical number
of cracked steels is reached. For detection of tendon ruptures in post-
tensioned concrete structures such as pre-stressed beams of bridges or
building roofs, the magnetic flux leakage (MFL) technique is the only non-
destructive testing method available.

The MFL method has been used on site for more than a decade to inspect
pre-stressed tendons. In most cases, measured rupture signals have been
confirmed to originate from tendon cracks by visual inspection after opening
the concrete. Thus, the utilisation of the technique has probably helped to
avoid a number of fatal collapses of structures. However, for the application
of the MFL technique, detailed knowledge of the possibilities and the limi-
tations of crack detection are of great importance.

11.1.2 Physical basis of the magnetic flux leakage
measurement method

The MFL was introduced by Kusenberger (1981) for investigations of steels
in pre-stressed concrete members. Later, the MFL method was developed
to allow its use on-site (Hillemeier et al. 1989, Ghorbanpoor 1998, Krause
Scheel and Hillemeier 2003, Sawade et al. 1995).

The MFL method is a magnetostatic measurement technique. In contrast
to ultrasonic and electromagnetic methods, the concrete has no influence
on the measurements, except in the case of ferromagnetic aggregates of the
concrete.

The MFL method can be explained as follows. The pre-stressed member
under consideration is magnetised by an exciting magnetic field \( H_0(p,x-x_0) \),
which is, in general, generated by a moveable yoke magnet as shown in Fig.
11.1. \( x_0(t) \) denotes the actual position of the yoke magnet, \( p(t) \) describes the
pole strength of the yoke magnet. The exciting field generates a magnetisa-
tion \( M \) in the tendon. Then the leakage field \( \vec{H}_s \) is obtained from the mag-
netic potential \( \psi \) as described by:

\[
\psi = -\frac{1}{4\pi} \int \frac{\text{div}\vec{M}}{r} \, dV, \quad \vec{H}_s = -\text{grad}\psi
\]  

[11.1]

where \( r \) is the distance between the position of the magnetisation and the
measurement point.
It can be seen from equation [11.1] that, only for non-homogeneous magnetisation \( M \), a magnetic leakage field is generated. Ruptures of the steels cause local disturbances in the distribution of the magnetisation, which produces a typical magnetic signal.

It should be pointed out that the term ‘magnetisation’ has two meanings. It denotes the process of applying of an exciting magnetic field to a member as well as the property magnetisation \( M \) that is the response of the ferromagnetic material to a magnetic field \( H \).

Normally in ferritic steels there is a non-linear and hysteretic relation between the magnetisation and the exciting magnetic field. The hysteretic behaviour of ferromagnetic material can be described in terms of the rate of the whole magnetic field \( H = H_0 + H_s \).

Figure 11.2 shows the measured magnetic curve (thick line) of a normal pre-stressing steel in comparison to the model (thin line).

For the forward modelling of the MFL (Sawade 1996) the reinforcement of concrete members can be considered to be an assembly of several discrete wire-shaped elements, Fig. 11.3. Each element has a certain diameter
The direction of the magnetisation of the element can be assumed to be parallel to the element axis. The magnetisation of each element is controlled by the magnetic field at its gauss-point (middle of the axis). The magnetic leakage field at all gauss-points as well as the magnetisation of all $N$ elements can be written as the vectors $\mathbf{H}$ and $\mathbf{M}$. Then equation [11.1] can be rewritten as:

$$
\mathbf{H}_s = \mathbf{K} \cdot \mathbf{M} \quad [11.4]
$$

$$
\mathbf{H}_s^T = [H_{s1}, \ldots, H_{sN}] \quad [11.5]
$$

11.2 Magnetisation curve of a typical pre-stressing steel (St 145/160, diameter 12.2 mm) numerical model with: $M_S = 15000$ A cm$^{-1}$; $\xi = 0.06$ cm A$^{-1}$; $\alpha = 0.06$; $\gamma = 1$, $H$ and $M$ in A cm$^{-1}$ ($M_S$, saturation magnetisation; $M_R$, remanent magnetisation).

11.3 Modelling of MFL: interaction between two wire-shaped elements.
and

\[ M^T = [M_1, \ldots, M_N], \quad [11.6] \]

where \( K \) is the operator of the ‘magnetic interaction’ of the elements. The coefficients \( K_{ij} \) (contribution of the element \( j \) to the gauss-point \( i \)) can be calculated using the integral term of equation [11.1]. This leads to exact coefficients of the transfer matrix. By combining equations [11.2], [11.3] and [11.4] the following are obtained:

\[ \Delta H_s = [1 - K \cdot \chi]^{-1}K \cdot \chi \cdot \Delta H_0 \quad [11.7] \]

and

\[ \Delta M = \chi \cdot [\Delta H_s + \Delta H_0] \quad [11.8] \]

where \( 1 \) is the identity matrix, \( \Delta \) denotes the rate, and \( \chi \) contains the differential susceptibility of all \( N \) elements according to equation [11.3]:

\[
\chi =
\begin{bmatrix}
\chi_1 & 0 & \cdots & 0 \\
0 & \chi_2 & 0 & \cdots \\
\vdots & \vdots & \ddots & \vdots \\
0 & \cdots & \cdots & \chi_N \\
\end{bmatrix}
\quad [11.9]
\]

From equations [11.7] and [11.8] the magnetisation of all ferromagnetic elements depending on the progress of the exciting field can be obtained (only the interaction between elements that contain ferromagnetic material have to be considered). By using the exact formula for the calculation of the transfer matrix according to equation [11.1], the boundary conditions at infinite distances are fulfilled automatically.

The exciting magnetic field of the yoke magnet can be approximately described by the following analytical expression (Sawade 1996):

\[ H_0 = -p \cdot \text{grad}[1/r_1 - 1/r_2] \quad [11.10] \]

where

\[ r_{1,2} = \sqrt{(x - x_0(t) \mp L/2)^2 + (y - y_0)^2 + (z - z_{\text{off}})^2} \quad [11.11] \]

and \( x_0 \) and \( y_0 \) denote the actual position of the middle of the yoke magnet, \( z_{\text{off}} \) is used as a parameter for the fitting of real exciting fields, \( L \) describes the length of the yoke magnet, and \( p \) describes the pole strength of the yoke magnet. Strictly speaking, equation [11.10] is valid only for a yoke magnet without an iron core. Using equations [11.2] to [11.11], the MFL can be simulated with satisfactory results (Fig. 11.3) This model works for a qualitative understanding of the influence of the important parameters like size of
the yoke magnet, the distance and diameter of the reinforcement bars and the magnetisation scheme.

To conduct the magnetic leakage measurements, a probe containing the magnetisation device (an electrically controlled yoke magnet) and the magnetic field sensors is moved along the pre-stressed tendons (Fig. 11.1). Local disturbances of the magnetisation owing to ruptures or reduction of the cross-section of a longitudinally arranged rod cause typical magnetic leakage signals (Fig. 11.4). The shape and amount of these rupture signals depend on the length $L$ of the yoke, the strength of the applied exciting field, the cross-section and the distance of the tendon and the mild reinforcement. Also, the velocity of the probe during the magnetisation has an influence on the magnitude of the rupture signals. A very slow and constant velocity of the probe is advantageous to reach the maximum magnetisation of the tendons. In practice, velocities in the range of 10–40 cm s$^{-1}$ are suitable.

Magnetic flux leakage measurements can be performed during the impact of the exciting field (active field, AF, measurement) or as a residual field (RF) measurement after a defined sequence of magnetisations.

In practice, the following magnetisation (with excitation current $I$) procedure has proven to be suitable. Table 11.1 lists the sequence of magnetisation and measurement scans which have to be performed in order to magnetise and demagnetise the tendons and the rebars. This scheme contains 13 scans of the probe with various magnitudes (currents $I$ of the yoke magnet) of the exciting field.

Figure 11.5 shows the magnetic leakage field signals obtained at one single broken rod using yoke magnets with $L = 46$ and $L = 24$ cm. In

11.4 Axial component of the magnetic leakage field of a cold drawn single wire with a rupture at $x = 100$ cm (diameter = 7 mm, crack width 1 mm, probe–wire distance 12 cm, yoke length 46 cm, $H_0 = 80$ A cm$^{-1}$), (a) measurement in the active field, (b) residual field measurement after switching off the probe at $x = 200$ cm. $M_0 = 15 \, 000$ A cm$^{-1}$; $\alpha = 0.06$; $\xi = 0.06$ cm A$^{-1}$; $\gamma = 1$. 

© Woodhead Publishing Limited, 2010
Table 11.1 Typical measurement scheme for magnetisation of a member (Sawade 2001)

<table>
<thead>
<tr>
<th>No.</th>
<th>Forward scan(^1)</th>
<th>Backward scan</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>4</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>6</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>8</td>
<td>8</td>
</tr>
<tr>
<td>6</td>
<td>0</td>
<td>8</td>
</tr>
<tr>
<td>7</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>8</td>
<td>6</td>
<td>0</td>
</tr>
<tr>
<td>9</td>
<td>4</td>
<td>0</td>
</tr>
<tr>
<td>10</td>
<td>2</td>
<td>0</td>
</tr>
<tr>
<td>11</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>12</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>13</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

\(^{1}\)Forward scans 2–5 and 8–11 are measurements in the active field (AF). The measurements at forward scans 1, 6, 7, 12 and 13 are performed as residual field measurements (RF).

11.5 Magnetic leakage signal (axial component) measured at a single broken rod (diameter 12.2 mm, probe–tendon distance 9 cm, crack opening 1 mm), procedure according to Table 11.1.
Fig. 11.6, the magnetic signals obtained at one broken and five additionally intact rods are shown. The maximum of the exciting field at scan no. 5 was about 80 A cm\(^{-1}\) \((L = 46 \text{ cm})\) and 50 A cm\(^{-1}\) \((L = 24 \text{ cm})\), respectively.

It can be seen that the final rupture signal, which is obtained at scan 13, becomes more clear in comparison to the previous scans. This is because the distribution of the magnetisation of the intact parts of the steels becomes quite homogeneous. For intact steels that are arranged in the vicinity of the ruptured steel, the amplitude of the rupture signal becomes considerably reduced, particularly for residual field measurements; the intact adjacent steel is said to act as a ‘magnetic shielding’, Fig. 11.7 (Sawade and Krause 2009).

Particularly important are the tendons, which contain several single prestressing steels. Therefore investigations were carried out with idealised tendons (Fig. 11.8), which contain 16 single cold drawn steels (diameter 7 mm). Thereby, the number and position of ruptures was varied. The ruptures were simulated by saw slice. The crack opening width was 1 mm.

The tests were performed according to the measurement scheme described in Table 11.1 using the large yoke \((L = 46 \text{ cm})\). Before the beginning of the scan sequence the ‘tendon’ was fully demagnetised.

11.6 Magnetic leakage signal (axial component) measured at one single broken rod (diameter 12.2 mm, probe–tendon sensor distance 9 cm, crack opening 1 mm) and five intact rods \((D = 12.2 \text{ mm})\), procedure according to Table 11.1.
The amplitude of the rupture signal can be expressed by the following empiric formula that depends on the distance $z$, the number $N$ (of total 16) of broken steels and their position inside the bundle.

$$H_{x \text{ max}} = C z^{-a} N^b$$  \[11.12\]

From the measured amplitudes from Table 11.2 the parameters $a, b$ and $C$ were obtained as given in Table 11.3.
Table 11.2 Amplitudes of rupture signals of a single tendon

<table>
<thead>
<tr>
<th>Ruptures</th>
<th>Amplitude $H_{\text{r, max}}$ (A cm$^{-1}$)</th>
<th>Ruptures</th>
<th>Amplitude (A cm$^{-1}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number</td>
<td>Row</td>
<td>$z$ (cm)</td>
<td>RF$^1$</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>6.5</td>
<td>0.69</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>11.5</td>
<td>0.25</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>16.5</td>
<td>0.12</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>21.5</td>
<td>0.05</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>6.5</td>
<td>1.20</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>11.5</td>
<td>0.50</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>16.5</td>
<td>0.22</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>21.5</td>
<td>0.08</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>6.5</td>
<td>3.00</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>11.5</td>
<td>1.20</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>16.5</td>
<td>0.60</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>21.5</td>
<td>0.21</td>
</tr>
<tr>
<td>8</td>
<td>1 + 2</td>
<td>6.5</td>
<td>5.50</td>
</tr>
<tr>
<td>8</td>
<td>1 + 2</td>
<td>11.5</td>
<td>2.60</td>
</tr>
<tr>
<td>8</td>
<td>1 + 2</td>
<td>16.5</td>
<td>1.20</td>
</tr>
<tr>
<td>8</td>
<td>1 + 2</td>
<td>21.5</td>
<td>0.60</td>
</tr>
</tbody>
</table>

$^1$Residual field measurement (scan 13).
$^2$Measurement at the active field (scan 5).

Table 11.3 Empirical parameter for estimation of the rupture amplitudes

<table>
<thead>
<tr>
<th></th>
<th>RF, row 1</th>
<th>RF, row 3</th>
<th>AF, row 1</th>
<th>AF, row 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C$</td>
<td>41.67</td>
<td>3.95</td>
<td>357.08</td>
<td>272.94</td>
</tr>
<tr>
<td>$a$</td>
<td>2.15</td>
<td>1.76</td>
<td>2.79</td>
<td>2.93</td>
</tr>
<tr>
<td>$b$</td>
<td>1.11</td>
<td>1.35</td>
<td>1.18</td>
<td>1.27</td>
</tr>
</tbody>
</table>

Equation [11.12] is a rough approximation for the estimation of the rupture amplitudes. The parameters $a$, $b$ and $C$ depend on the type of tendon (diameter and numbers of the steels) and the type of probe, in particular the size of the yoke magnet. The influence of the number $N$ of broken steels is nearly linear. The AF amplitudes are about 2–3 times bigger than the RF amplitudes.

For the RF, the exponent of the distance $z$ is about 2. However, in the case of AF this exponent is close to 3.

Furthermore, the influence of the position of the ruptures in the tendon can be seen. The RF ruptures, which are inside of the tendons, have considerably smaller amplitude than the ruptures that are arranged outside. Indeed, it is the steels close to surface that are mostly endangered in practice. Therefore, one can expect that the possible ruptures in a tendon will
be found near to the concrete surface. The influence of the numbers $N$ of the ruptures is nearly linear.

11.1.3 Analysis for the evaluation of measured signals

The MFL signals are affected not only by ruptures of the tendons but mainly by the mild steel reinforcement bars (rebars), which are located at a shorter distance to the probe than the tendons. To suppress these unwanted signals, several techniques for measurement and analysis have been developed (Scheel 1997, Sawade 2001). After a suitable sequence of magnetisations (according to Table 11.1), the relation between signals caused by cracks in tendons and the disturbance signals caused by rebars can be improved. Taking into account the scheme of Table 11.1, the magnetisation at the backward scans 6 and 12 causes an inversion of the magnetic signals of the transversely arranged rebars. Therefore, the signal portion of the rebars can be suppressed by adding the measurements $6 + 7$ and $12 + 13$.

Furthermore, the signal portion caused by the rebars can be reduced from the measured signal $H_{\text{meas}}(x)$ by filtering. For this purpose, the signal for the rebar can be approximated by an analytical expression or by a measured test signal. The following formula describes the axial component of the residual field of $N_B$ transversely arranged rebars (Sawade 2001):

$$H_p(x) = \sum_n p_n f_{B_x}(x - x_n) \quad [11.13]$$

where the axial component is:

$$f_{B_x} = \frac{1000r}{(t^2 + z_B^2)^{3/2}} \quad [11.14]$$

and the normal component is:

$$f_{B_z} = \frac{1000}{(t^2 + z_B^2)^{3/2}} \quad [11.15]$$

where $z_B$ denotes an empirical parameter, depending on the concrete cover of the rebars.

The unknown parameters $p_i$ are calculated using the best-fit method. Then the following linear system of equations has to be solved (Sawade 2001):

$$\int [H_{\text{meas},x}(x) - \sum_{n=1}^{N_g} p_n f_{B_x}(x - x_n)] f_{B_x}(x - x_{n_0}) \, dx = 0 \quad [11.16]$$

where $n_0 = 1, \ldots, N_B$. 
The exact positions $x_n$ of the rebars can also be obtained from the residual measurements 5 or 6 as the positions where its first derivative exhibits a local extremum. The integrals in equations [11.13] to [11.16] must be calculated as sums. By using the gradients in equations [11.13] to [11.15], the signal portions with higher spatial frequencies have more influence than the signal portions with lower spatial frequencies. In the ideal case, the low-frequency signal portions of the deeper arranged tendon have no influence on the parameters $p_n$. Then the remaining signal $H_{R,x}(x)$ contains essentially only the information of the longitudinal reinforcement and the tendons:

$$H_{R,x}(x) = H_{mx}(x) - H_{B,x}(x) \quad [11.17]$$

The remaining signal is compared and evaluated with the typical rupture signal $H_T$ by means of the local correlation method (Kusenberger 1981, Sawade 1996):

$$H_{T_1}(x) = p_t(x_0) \frac{1}{\left[(x-x_0)^2 + z_0^2\right]^{3/2}} \quad [11.18]$$

and

$$H_{T_2}(x) = p_t(x_0) \frac{(x-x_0)z_0}{\left[(x-x_0)^2 + z_0^2\right]^{5/2}} \quad [11.19]$$

The depth $z_0$ can be considered as a fitting parameter. It does not have to agree with the actual distance probe tendon. The local correlation coefficient $r$ describes the similarity of the shape of the remaining signal $H_R$ with the rupture signal $H_T$ inside the interval $(x-h,x+h)$, where $x$ is the position of the rupture (Kusenberger 1981, Ghorbanpoor 1998, Sawade 1996).

$$r(x) = \frac{\int_{-h}^{h} H_T(x+t)H_t(t)dt - 2h\bar{H}_T(x)\bar{H}_t}{\int_{-h}^{h} [H_T(x+t) - \bar{H}_T(x)]^2 dt \int_{-h}^{h} [H_t(t) - \bar{H}_t]^2 dt} \quad [11.20]$$

where

$$\bar{H}_T(x) = \frac{1}{2h} \int_{-h}^{h} H_T(x+t)dt \quad [11.21]$$

and

$$\bar{H}_t = \frac{1}{2h} \int_{-h}^{h} H_t(t)dt \quad [11.22]$$

where $h$ is the correlation length.
The rupture amplitude $p_r$ is the amplitude of the rupture signal inside the interval $(x - h, x + h)$ (Sawade et al. 1995, Sawade 1996, 2001). The local amplitude $p_r(x)$ of the rupture signal and the offset $a(x)$ is calculated by solving the following best-fit problem:

$$\psi(x) = \int_{-h}^{+h} [H_R(x-r) - p_r(x)H_I(t) - a(x)]^2 dt \rightarrow \text{Minimum} \quad [11.23]$$

where

$$\frac{\partial}{\partial p_r(x)} \psi(x) = 0 \quad [11.24]$$

and

$$\frac{\partial}{\partial a(x)} \psi(x) = 0 \quad [11.25]$$

As result, for every point $x$, the local correlation coefficient $r(x)$ and the strength $p_r(x)$ of the rupture signal are obtained. At the last step of the analysis, we consider the function:

$$p_{fr}(x) = \begin{cases} r(x)p_r(x) & \text{for } r(x) \geq r_{\min} \\ 0 & \text{else} \end{cases} \quad [11.26]$$

Ruptures are likely at positions where the function $p_{fr}$ has a local maximum. The above described method for the signal analysis is demonstrated in Fig. 11.9 a–h as an example of a RF measurement. The analysis was performed using the parameters $z_0 = 10$ cm, $h = 20$ cm and $r_{\min} = 0.7$. These parameters are particularly suitable for the signal analysis. Significant rupture signals can be seen at $x = 140$ and 220 cm. The $r$ values correspond to signal amplitudes of 0.4 and 0.3 A cm$^{-1}$, respectively.

In principle, this analysis method can also be performed using active field measurements. Because the shape of an AF rupture signal is similar to the signal of a stirrup signal, the removal of the signals of the rebars is not so efficient as it is for the RF measurement. Further, for the AF measurement the linear superposition principle of the leakage field signals does not apply.

For examination of this analysis method, synthetic rupture signals were added to measured signals, which are obtained at on-site investigations. These signals were analysed and the filtered rupture signal was subsequently compared with the original synthetic rupture signal. Figure 11.10 shows the comparison between a given synthetic rupture signal, which was added to the measured signal and the filtered signal.

From Fig. 11.10, it can be seen that the amplitude of the filtered rupture signal at $x = 140$ cm amounts to about 50% of the given synthetic rupture
11.9 Signal analysis. (a) RF measurement (scan 15); (b) detection of the position of the stirrups; (c) calculated signal portion of the stirrups; (d) calculated (filtered) signal; (e) filtered signal; (f) local correlation \( r(x) \); (g) local amplitude of the rupture signal \( p_r(x) \); (h) rupture amplitude \( r_p(x) \) \( (r > 0.7 \text{ and } p_r > 0) \).

11.10 Comparison between a synthetic rupture and the filtered RF signal.
signal. This finding was confirmed by many numerical experiments and laboratory investigations. In general, one can expect, that, for the usual reinforcements, the amplitude of a filtered rupture signal is half as large as the real signal. Furthermore, to be detectable, the real rupture signal must be about 50% of the signal of the rebars. Only for very regularly arranged rebars can this limit be reduced to about 25%.

In the last step of the analysis, the question concerning the amount of the damage (loss of cross-section) arises. As demonstrated in 11.1.2, the distance probe tendon and the position of the rupture inside the tendon considerably influence the amplitude of the rupture signal.

However, the crack width opening has only a small influence. If the rupture amplitude is known (by performing the signal analysis), one must choose the probe–tendon distance and the position of the possible rupture. For this example (Fig. 11.9 and 11.10) the followings findings are made:

at \( x = 140 \text{ cm} \), the amplitude of the filtered rupture signal is about 0.4 A cm\(^{-1}\),
actual 0.8 A cm\(^{-1}\);

at \( x = 220 \text{ cm} \), the amplitude of the filtered rupture signal is about 0.3 A cm\(^{-1}\),
expected 0.6 A cm\(^{-1}\).

With a probe–tendon distance of about 15 cm and the assumption that the ruptures are arranged near the surface, the number of broken steels obtained from equation [11.12] and Table 11.3 (RF row 1) is \( N \approx 4 \). In fact at this position, five steels (out of 12) were broken.

11.1.4 Limit of detection

The MFL works as a non-destructive and contactless method. As a result, signals that contain possible portions of a rupture signal are obtained. For the evaluation, one must compare the measured signals with characteristic rupture signals as described above. The existence of ruptures can be assumed if a high correlation with characteristic rupture signals is found. Therefore, the MFL provides only a statement concerning the likeliness of ruptures.

For the application of the MFL ascertaining the limit of detection is very important. In addition, it is necessary to determine the maximum size of the possible loss of cross-section even for an inconspicuous signal. Laboratory tests with similar structures of the reinforcement are the best method for the calibration of the rupture signals. Indeed, this requires much effort and is possible only in special cases.

First, it is necessary to make a distinction between a ‘first investigation’ and subsequent investigations of a pre-stressed member. The first investigation serves for the determination of the actual state of the member. Subsequent measurements allow the assessment of the development of the damage (newly generated ruptures) over a longer period.
For first investigations, we discern between the following cases:

1. Pre-stressed members with immediate bond with a lower portion of additional mild reinforcement.
2. Post-tensioned members with single-rod tendons (diameter in the range of 30 mm).
3. Post-tensioned members with tendons consisting of bundles of several wires or steels.

The MFL investigation in cases 1 and 2 is very simple compared with that in case 3.

In particular, for prefabricated ceiling beams, which contain only a few pre-stressed wires (cross-section about 20–40 mm$^2$) as longitudinal reinforcement, even the detection of larger incipient cracks was possible. Because members with immediate bond usually have a smaller content of mild reinforcement and a concrete cover up to 10 cm, single ruptures down to the second layer of pre-stressed wires can be detected safely. Also, for post-tensioned members with single-rod tendons, the detection of a rupture up to a concrete cover of 10–15 cm is unproblematic.

Case 3 proves to be much more complicated than both other cases. In order to measure the detectable loss of cross-section (number of ruptures) of the tendon, laboratory tests using the sample shown in Fig. 11.11 were performed. This arrangement allows the variation of the number and positioning of ruptures of the pre-stressed steels. The ruptures were generated

![Arrangement for laboratory tests (Ramrath 1998).](image-url)
by saw-cutting with a gap of 1 mm. Owing to the small influence of the usual metallic duct on the rupture signal, the specimen did not contain a duct. The longitudinal distance of the stirrups was about 25 cm. The tests were performed with consideration of the procedure according to Table 11.1.

From the tests, the following conclusions can be drawn:

- The minimum number of ruptures in row 1 of dummy 1 that can be detected with a high reliability was 2.
- The minimum number of ruptures in rows 2 to 4 of dummy 1 that can be detected with a high reliability was 4.
- Only the fully broken dummy 2 (16 ruptures) could be recognised. Therefore, the partially damaged tendons of the second layer of tendons cannot be recognised.
- The obtained amplitudes of the rupture signals amount to only half of the values according to equation \([11.12]\). This reducing of the rupture amplitudes results from the filtering process and the influence of the longitudinal reinforcement.

However, in general, the pre-stressed concrete members on site have a more complicated structure. For example, the distance of the stirrups is not regular, or there are more longitudinally arranged mild reinforcements and additional irregular ferromagnetic components. This leads to a larger reduction in the sensitivity of the MFL. Several on-site tests for evaluation of the MFL were performed (Krieger 2000, Mietz and Fischer 2005). In these tests, either a specimen with defined damage was prepared, or the specimen was demounted after the MFL tests. The test team was informed about the position and amount of the damage only after the ending of the investigations. In Fig. 11.12, the RF signals of a test specimen and the related signal analysis are shown (Sawade and Krause 2010). The tendon under consideration contained 12 cold drawn steels (diameter 12.5 mm) inside a metallic duct (thickness 0.3 mm, diameter 100 mm). The centre of the duct was 15 cm behind the concrete surface. Furthermore, the sample contained mild longitudinal and transverse rebars. The average longitudinal distance between the stirrups (diameter 8 mm) was about 15–25 cm.

From Fig. 11.12, it can be seen that ruptures from about 25% loss of cross-section can be detected. If the RF signals are used for the analysis, ruptures that are localised at the beginning or the end of the scan length (area of the yoke magnet) are not detectable. The obtained rupture signal amplitudes are considerably smaller than the amplitudes expected for a single tendon.

In a comprehensive project of the Bundesanstalt für Materialprüfung und Forschung (BAM), the capability of the MFL technique was tested (Mietz and Fischer 2005). For this purpose, two girders removed from an older bridge were investigated. After the non-destructive investigation of the girders, the tendons (with the ducts) were removed from the girders.
and the single tendons were once more investigated by means of MFL. After this, the tendons were opened and the single wires were inspected in order to verify the non-destructive testing results.

A large amount of damage (four wires with a double cut, two wires with a single cut) produced by saw cutting during demolition could be clearly detected. In addition to this heavily damaged tendon, the investigated girders had two other tendons with a smaller amount of damage (single strands with open ends, which were distributed over a length of 1–2 m (Mietz and Fischer 2005). In these instances, no unambiguous flaw signals could be found. Also, single cracks and incipient cracks could not be found.

The rupture amplitude at the critical point ($x = 90$) was about 0.8 A cm$^{-1}$. From equation [11.12] and assuming a probe–tendon distance of $z = 10$ cm, there are $N \approx 2$ broken strands. In fact, there were six cut strands. This result shows, that the rupture amplitudes obtained at the normally pre-stressed members are considerably reduced compared with the amplitudes obtained at a single tendon.

In summary, the detection limit of MFL for a first investigation of a post-tensioned girder with tendons manufactured by several wires can be estimated to be about 25–35% of the cross-section of the cracked strands.
For repeated investigations the situation becomes much improved. The signal analysis can be performed by the simple subtraction of the signals and the filtering of the stirrups is cancelled. The subtracted signal is evaluated only for the newly generated rupture signals. Figure 11.13 shows RF signals (scan No. 12, Table 11.1) before and after cracking of a pre-stressed steel of a concrete specimen, which are obtained at a long-term test. One of the six pre-stressed steels was treated by an electrochemical polarisation in combination with rhodanide and hydrochloric acid in order to generate hydrogen-induced stress corrosion. The signal of the rupture can be seen very clearly from the difference signal. It can be expected that for repeated MFL the cracking of single steels can be recognised as demonstrated in Fig. 11.14 (Sawade and Krause 2010).

11.2 Description of equipment required

The following facilities are necessary for performing MFL measurements:

- suitable magnet sensors for the magnetic scanning,
- a device for flexible magnetisation (electrically driven yoke magnet with adjustable power supply),
- a device for controlled movement of the magnet and the sensors (propulsion and a rail system).

If measurements in the active field are to be carried out, the sensors should be mounted at the centre of the yoke magnet.
In Fig. 11.15, the assembly of a large probe (yoke 46 cm) is shown. The sensors for measuring the magnetic field are implemented using Hall generators. In addition, a newly developed giant magnetoresistance sensor (GMR) operating in a feedback scheme such as SQUID sensors can be
used. The coils for the magnetisation contain about 1000 turns. Figure 11.16 show the axial component and normal magnetic field components $H_x$, $H_y$, and $H_z$, respectively, for an exciting current of 8 A for a distance of 10 cm.

Based on these results, the probe should have the following properties. For performing measurements in the active field, the effective range of the sensors must be up to 100–200 A cm$^{-1}$ (without compensation) or 10–20 A cm$^{-1}$ (with compensation). If only residual field measurements are carried out, the effective range is about 10 A cm$^{-1}$. The resolution of the sensors should be smaller than 0.02 A cm$^{-1}$. The yoke magnet should be able to generate a magnetic field of about 50–80 A cm$^{-1}$ at a distance of 10 cm.

11.3 Examples of applications of the magnetic method on site

The MFL technique has been used for more than a decade for the investigation of pre-stressed members. The first application of MFL in Germany was the investigation of pre-stressed ceiling beams, which are often used in cowsheds (Fig. 11.17).

Pre-stressed concrete ceiling beams, as shown in Fig. 11.13, were often used in the 1960s. These beams had pre-stressed wires with only a low amount of mild reinforcement. Frequently, the pre-stressed steels used
Non-destructive evaluation of reinforced concrete structures (cross-section 20–40 mm²) were prone to hydrogen-induced stress corrosion. After some collapses, many ceiling beams were investigated. Figure 11.18 shows the magnetic RF signals for a heavily damaged and an intact beam.

Because of the small concrete cover of these beams (about 2–4 cm) and the smaller amount of mild reinforcement, it was possible to detect larger incipient cracks. As a further example, the investigation of a post-tensioned concrete with single-rod tendons is shown, Fig. 11.19. The measurements were performed using a moveable platform. Spot drilling of the metallic ducts damaged many tendons, and thus, road salt reached the pre-stressed steels and caused corrosion. At the time of the investigation, some small concrete cracks were visible, and there was the question of whether some steels were broken. Figure 11.20 shows the MFL signal. All five sensors measure the same rupture signal at \( x = 120 \) cm.
11.19 View of the MPA equipment for investigation of tendons from below.

11.20 (a) Damaged post-tensioned concrete with single-rod tendons; (b) MFL signal (AF, active field measurement): axial component of five sensors (transversely arranged with a distance of 4 cm). Probe–tendon distance about 8–10 cm.
From these signals, it was concluded that two neighbouring single rod tendons, which were arranged with a transverse distance of about 15 cm, were broken. Therefore, at this position, the concrete lost the pre-stressing and its safety was not ensured. Owing to the large diameter of the rods of about 32 mm, the amplitude of the two ruptures is very large compared with the amplitude of the signals of the rebars. Therefore, the ruptures can be seen very easily. These findings were later confirmed by opening this girder. Because of the large amplitude of the rupture signals, the detailed signal analysis as described above seems not to be necessary. As a result of this investigation, all girders of the bridge have been removed in order to erect a new construction.

The final example shows the filtered RF signal of a partially damaged tendon (two broken steels out of a total of 40 steels, diameter 7 mm) in the deck of a highway bridge (Fig. 11.21).

Owing to the smaller concrete cover of 4 cm the rupture signal at 160 cm can be seen (after the signal processing) relatively well. The rupture signals at 230 and 255 cm are influenced by near-surface arranged mild reinforcement. From the actual probe–tendon distance of 6–7 cm as well as the signal amplitude of 1.2 A cm\(^{-1}\) the number of broken steels can be obtained using equation [11.12] to be about 1–2. In reality, two steels were broken at this

![Graphs of RF signals and rupture amplitudes](image)

11.21 (a) RF signals of sensors 1–5 (filtered) \(H_x\) in A cm\(^{-1}\) and (b) corresponding rupture amplitudes in A cm\(^2\), \(r_{\text{min}} > 0.7\), \(z_0 = 10\) cm, \(l = 20\) cm.

© Woodhead Publishing Limited, 2010
position. This damage was caused by an insufficient concrete cover in combination with the attack of road salt. In this bridge no other places with rupture signals were ascertained, so that no other actions (except the repair of this area) were necessary.

11.4 Perspective: recent developments of the magnetic method for inspection of reinforced concrete

It can be concluded that, in real-life situations (a complicated or irregular arrangement of the mild steel reinforcement and a narrow distance between the tendons), the detection of ruptures of single ruptures in normal tendons cannot be detected reliably.

If one is only interested to know the possible growth of damage of pre-stressed members during a certain monitoring period, a simple comparison of MFL signals over time suffices. This repetitive measurement technique is a simple method. Because of the high resolution and reproducibility of the magnetic measurements, small changes of state of the reinforcements can be detected.

The advantage of this ‘difference method’ is the fact that no sophisticated signal analysis has to be performed, provided that the equipment is positioned very accurately and the exciting procedure is always done in same way according to the protocol.

Pre-stressed concrete bridge decks contain a large number of transversely arranged tendons. For the rapid magnetic investigation of these tendons, new equipment was developed at the Technische Universität Berlin (Hillemeier and Walther 2008). The special features are, firstly, the long (about 3 m) yoke magnet for the magnetic excitation of the tendons, and, secondly, in place of a sensor array with a large number of independent sensors, a rotating sensor head with only a few sensors (in z direction, normal to the surface) is used. The superposition of the straight and the rotating movement guarantees, that the scanned area enfolds the (residual) field of the whole tendon.

With a fast-controlled hydraulic, the system is always kept at a constant distance from the surface. The magnetic field can be obtained as a ‘magnetic image’ in Cartesian co-ordinates. In addition, the software includes a self-calibration algorithm. This system was successfully tested in the laboratory and on-site, Fig. 11.22.

Figure 11.23 shows the result of an on-site test. In one day, the system measured an area of 6.4 m × 185 m. Every transverse tendon of the bridge deck was found and the result was that no tendon had a rupture.

In a new project, radar and magnetic techniques were combined to the investigation of highway plates in bridges (Krause et al. 2007). Using radar,
the humidity and the salinisation of the (near to surface) concrete become detectable. These properties were deduced from the real and imaginary part of the permittivity of the concrete. The depths of the rebars were obtained independently of the radar measurements by magnetic measurement. For this purpose, the RF signals of the rebars are evaluated. The concrete has no influence on the magnetic signals.

The velocity of the radar waves (or the real part of the permittivity) follows from a comparison of the depth of the rebars and the arrival time of the radar signals. Taking into account the already obtained results, the radar–magnetic method appears to be sufficient for the rapid scanning of larger areas of bridge and park decks that also have an asphalt layer.

### 11.5 Recommendations for the application of the magnetic flux leakage (MFL) method

The magnetic measurements require careful preparation. All available plans of the structure should be analysed in detail beforehand. Depending on the access possibilities to the locations of interest on the structure, suit-
able support equipment such as lifting ramps, scaffolds and so forth should be available. These conditions also determine the personnel required. Two persons should be considered the minimum.

For repetitive measurements, very precise repositioning of the measuring system of the order of millimetres is essential. Therefore, care should be taken that appropriate markers are applied to the structure. Depending on the situation on the structure, a small magnetic yoke probe with a weight of approximately 10 kg may be sufficient. However, structures with deep-lying tendons may require the use of a larger probe with a weight of 60 kg. The weight of the scanning system is additional to that. Scanning velocities (for the normal probe) should be chosen in the range of 10 to 40 cm s\(^{-1}\). With a measuring protocol including 8 to 15 forward and backward scans, this means that testing 100 m of tendons is a realistic estimate for the maximum achievement of one working day. The signal processing and evaluation should be performed using software as described above and the data interpretation has to be performed by experienced staff.

The MFL method permits a relatively quick non-destructive scan of pre-stressed concrete members. Visual inspection of the steels by opening the concrete and the tendon duct should be done by experienced staff, however, to confirm the findings.

11.6 References

krause h-j, glaas w, zimmermann e, faley m i, sawade g, mattheus r, neudert g, gampe u, krieger j, ‘SQUID array for magnetic inspection of prestressed concrete bridges’, Physica C, 2002, 368(1–4), 91–95.
krause h-j, rath e, dumat f, sawade g, ‘Radar-Magnet-Betontest’, Beton- und Stahlbetonbau, 2007, 102, 825 (in German).


Abstract: An explanation is presented of the electrical resistivity of concrete, a parameter function of material properties, such as porosity and composition, and material conditions, such as moisture and alteration. The influences of intrinsic and external parameters are described. Various measurement techniques, their methodology, objectives, advantages and limits are proposed and new developments or original uses of the electrical resistivity method are presented. The impedance technique is considered as an alternative electrical technique.

Key words: electrical resistivity, non-destructive testing, conduction, resistivity, reinforced concrete.

12.1 Introduction

Electrical resistivity measurement is a technique adapted from the field of geophysics and from the study and classification of natural rocks. Resistivity is a geometry-independent property related to the intrinsic characteristics of a material. Techniques aim at the electrical characterisation of concrete components, such as fly ash and silica fume, and their proportions, such as the water/cement (w/c) ratio. However, electrical resistivity (ρ) also varies in relation to concrete storage conditions (Table 12.1).

The electrical resistivity of concrete is a parameter of prime importance in the degradation process of concrete structures. Resistivity is a measure of the ability of an electrical current to flow within a material, and is thus an indicator of the material’s transfer properties (Baroghel-Bouny, 2004). For concrete, it is used as a measure of its ability to resist chloride ingress (ASTM C1202-97). The method is used in laboratory tests to characterise concretes or to compare them, and can be used in further on-site evaluation (TC RILEM154EMC).

Electrical resistivity techniques have mainly been developed to assess reinforcement corrosion. Steel corrosion in concrete is largely dependent on moisture and chloride ingress, and these two factors influence the electrical properties of concrete. The technique is cheap and relatively quick, and is therefore naturally interesting for the assessment of the probability of
Table 12.1 Some concrete resistivity values as a function of composition and storage conditions (Lataste, 2002; Woelf and Lauer, 1980)

<table>
<thead>
<tr>
<th>Influence of concrete</th>
<th>Electrical resistivity (ohm m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ordinary Portland cement (35 MPa)</td>
<td>500–1400</td>
</tr>
<tr>
<td>Self-compacting concrete (35 MPa)</td>
<td>300–1000</td>
</tr>
<tr>
<td>High-performance concrete(^1) (65 MPa)</td>
<td>850–1500</td>
</tr>
<tr>
<td>Fibre reinforced concrete</td>
<td>80–400</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Influence of environmental conditions</th>
<th>Ordinary Portland cement concrete ((\rho) in ohm m)</th>
<th>Other concrete ((\rho) in ohm m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very wet, sprayed atmosphere</td>
<td>50–200</td>
<td>300–1000</td>
</tr>
<tr>
<td>Natural atmosphere</td>
<td>100–400</td>
<td>500–2000</td>
</tr>
<tr>
<td>External atmosphere (sheltered concrete, 20 °C, 80% RH)</td>
<td>200–500</td>
<td>1000–4000</td>
</tr>
<tr>
<td>Carbonated concrete</td>
<td>&gt;1000</td>
<td>&gt;2000</td>
</tr>
<tr>
<td>Internal atmosphere (20 °C, 50% RH)</td>
<td>&gt;3000</td>
<td>&gt;4000</td>
</tr>
</tbody>
</table>

\(^1\) Containing slag (>65\%) or fly ash (>25\%) or silica fumes (>5\%).

corrosion in structures. Other applications are under development to exploit resistivity sensitivity to other concrete characteristics, independent of rebars. They aim to describe or compare concretes, or, in some cases, to describe changes in concrete.

The first part of this chapter presents electrical conduction through concrete and explains what concrete resistivity is; the influences of intrinsic and external parameters will then be described. In the second part, the various measurement techniques, their methodology, objectives, advantages and limits are proposed. The third part presents various new developments or original uses of the electrical resistivity method. Finally, in the last part, the impedance technique is considered as an alternative electrical technique.

### 12.2 Physical principles and theory

#### 12.2.1 Electrical conduction in concrete

Resistivity is the intrinsic expression of the material property called electrical resistance, which represents a material’s ability to impede the flow of electrical current. Resistivity is independent of the geometry or size of the tested element (if the material is homogeneous) or of the measuring device. Electrical resistivity, \(\rho\), is expressed in ohm-metres (ohm m), since resistance (\(R\)) is expressed in ohms. The inverse of resistivity is conductivity (\(\sigma\)), measured in Siemens per metre (S m\(^{-1}\)); the inverse of resistance is conductance (\(C\)), measured in Siemens (S).
From an electrical point of view, concrete is generally considered to be biphasic. Hunkeler (1996) described it as non-conducting aggregate particles embedded in an ionically conducting cement paste matrix. Conduction through concrete is, then, essentially electrolytic in wet concretes, by Ca\(^{++}\) ions (only in the first hours), and by Na\(^+\), K\(^+\), OH\(^-\) and SO\(_4\)^{2-} ions in free water. When humidity decreases, the electrical conduction then becomes an electronic phenomenon, through water and non-hydrated cement particles. This gel is rich in aluminium and calcium. The resistivity of concrete evolves between less than 100 ohm m (in the semiconductor range) for a moistened concrete, to 10\(^9\) ohm m for a very dry one (in the insulator range) (Monfore, 1968).

The electrical resistivity of porous materials can be described by Archie’s law (1942). The law, equation [12.1], derives from studies of porous rocks in oil-bearing reservoirs and can be applied to concrete:

$$
\rho = a\phi^m\rho_f S^n
$$

[12.1]

where \(\rho\) is the rock resistivity, \(\phi\) its porosity, \(\rho_f\) the resistivity of pore fluid, \(S\) the saturation rate, and \(a, m,\) and \(n\) are three constants linked to the material. The part of the equation linked to the solid phase can also be expressed as the formation factor: \(F = a\phi^m\). Furthermore, \(F\) can be viewed as the contrast, or ratio, between the resistivity of saturated rock and the resistivity of the saturating fluid. Equation [12.1] highlights the strong influence on electrical resistivity caused by the parameters of saturation rate, pore water composition, and porosity.

To enable forecasting, parametrical studies of concrete aim to evaluate \(a, m\) and \(n\). Research considering various concretes by their formulation (Cabrera and Ghodoussi, 1994; Naar, 2006) and their ages (Tumidajski \textit{et al.}, 1996) reveal the variation for these parameters over a large range (Table 12.2). Assessment of \(a\) is complex as long as \(\rho_f\) is not known exactly, thus explaining the dearth of data published. Parameter \(n\) is quite stable for rocks and even for concrete, and has a weak effect on resistivity. The

<table>
<thead>
<tr>
<th>Table 12.2 Archie’s law parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
<tr>
<td>(a)</td>
</tr>
<tr>
<td>Shape factor on rocks</td>
</tr>
<tr>
<td>On rocks (Telford \textit{et al.}, 1990)</td>
</tr>
<tr>
<td>On concrete (Naar, 2006)</td>
</tr>
</tbody>
</table>

© Woodhead Publishing Limited, 2010
variation range for $m$ is wide and it is difficult to consider an average value for a concrete without performing a specific study in the laboratory.

12.2.2 Influencing factors

_Intrinsic factors_

Many intrinsic factors influence electrical resistivity, notably those affecting porosity, which is that part of the electrolytic condition preponderant over all others. Porosity affects the electrical properties of concrete: the higher the porosity, the more the resistivity decreases. Some methodologies are based on using resistivity measurement to rank concretes by their porosity (Rengaswamy _et al._, 1994). As well as being influenced by pore volume, resistivity can also be an indicator of pore distribution (Lakshminarayanan _et al._, 1992). Concrete resistivity is dependent on pore volume and distribution, and also on pore radii. Andrade _et al._ (2000) consider that resistivity is an indicator of the interconnectivity and tortuosity of the pore network. However, resistivity is influenced only by open porosity, the effective porosity, which is different from total porosity. Only the electrically interconnected pores play a role; close porosity has no influence on electrical properties. Although both types of porosity influence mechanical strength, resistivity is generally correlated with mechanical properties (Lakshminarayanan _et al._, 1992). Furthermore, if only open porosity rigorously influences electrical behaviour, then it is that same porosity which has a big role in transfer properties (such as diffusion and permeability). This last point underlines the interest in resistivity measurements in the area of durability assessment. The link between resistivity and porosity, and its sensitivity to characteristics of porosity, in addition to the great influence of porosity on the deterioration process and on the mechanical behaviour of concrete, explains why resistivity is an interesting and prominent technique in describing concrete conditions and its ability to resist aggressive external conditions.

The water/cement ratio ($w/c$) is a parameter affecting many concrete properties. Electrical resistivity and $w/c$ are inversely linked: schematically, the higher the water content ($w/c$ high), the more the concrete is porous, and the more it is conductive. According to Polder _et al._ (2000), in favourable cases, when influencing factors are well mastered, resistivity allows the assessment of local variations in $w/c$; there have been a number of studies into this relationship (Hammond and Robson, 1955; Whittington _et al._, 1981; Woelfl and Lauer, 1980). Figure 12.1 shows the influence of $w/c$ ratio, in the range 0.45–0.90, on electrical resistivity for an ordinary Portland concrete (water saturated, with siliceous aggregates) (ANR SENSO, 2009).
Cement is one of the main parameters affecting concrete properties. There are various kinds of cement, differing by their chemical compositions, particularly by their aluminate or silicate content. Cement, depending on its make up, is the source of the chemical elements and ions supporting electrical flow through the concrete material. Thus, the chemical make up of the pore water is linked to the cement used. For instance, the resistivity of concrete made from calcium aluminate cement is about ten times greater than the resistivity of concrete made from Portland cement (Monfore, 1968). Studies show resistivity ratios ranging from one to one hundred for several concretes differing by formulation (notably by cement type), tested under the same conditions and by the same methods (Hammond and Robson, 1955; Neville, 1996).

Concrete is a composite of cement and water (which forms the porous matrix), and also of aggregates of various sizes. The aggregates influence the concrete’s electrical properties in several ways. The resistivities of the aggregates are several orders of magnitude higher than that of the water in the pores; therefore, concrete can be considered to be non-conducting aggregate particles embedded in an ionically conducting cement paste matrix (McCarter et al., 1981). A simplistic view of concrete for electrical resistivity models makes a distinction between that matrix and the volume fraction for aggregates (Tumidajski et al., 1996). According to Whittington et al. (1981), the electrical resistivity of concrete is three to four times higher than the corresponding mortar. A normal concrete contains about 60–75 vol% of aggregates. The effect of the aggregates on resistivity can be studied according to various points of view: total volume, mineralogy, and distribution.

Aggregate quantity can be expressed as $A/C$ (where $A$ represents the aggregate mass, and $C$ the cement mass). In this way, it can be seen that the increase in resistivity is related to the increase of $A/C$, as a direct
consequence of aggregate quantity (Woelfl and Lauer, 1980). Other researchers have reached the same conclusion: the more aggregate there is, the more the concrete is resistive (Hughes, 1985; Xie Ping and Tang Ming-Shu, 1988).

With regard to grain size distribution, research has shown that:

(a) an increase in larger diameters leads to an increase of local electrical measurement variability (Morris et al., 1996);
(b) the less continuous the aggregate distribution, the higher the variability (Lataste et al., 1999).

Resistivity is sensitive to mineralogy in several ways: the less the aggregate is resistive, the less the concrete is resistive, so concretes can be ranked (ANR SENSO, 2009); and the less contrast in resistivity there is between aggregate and mortar, the less variability there is in resistivity measurements (Morris et al., 1996).

Figure 12.2 illustrates the values and variabilities of concretes made with different aggregates, relative to a water saturated Ordinary Portland Cement (OPC) concrete with siliceous round aggregate ($D_{\text{max}} = 14 \text{ mm}$). Figure 12.2 shows resistivity at porosity of 14.2 to 18.1%. It can be seen that, for concrete comprising siliceous aggregates, resistivity values can be described by the same regression as taken from the reference set of concrete (SR14); the influence of $D_{\text{max}}$ is only viewable in terms of variability (higher when $D_{\text{max}}$ increases) and there is no significant influence from aggregate shape. For calcareous concrete, the variation range of resistivity with regard to the porosity of concrete is significantly different from the reference regression curve in this study, revealing the high influence of mineralogy of aggregates on the electrical behaviour of concrete (ANR SENSO, 2009).

![Figure 12.2](image_url)

12.2 Influence of aggregate on electrical resistivity: for siliceous or calcareous aggregate (S or C), round or crushed (R or C), and various $D_{\text{max}}$ (14 or 20 mm) (ANR SENSO, 2009).
Additions are used to modify (i.e. improve) a particular property of concrete, in terms of its strength and durability, but also to improve properties during the pouring phase (to improve workability, for instance). Electrical resistivity is sensitive to them, to their composition and their effects on the final concrete (Cabrera and Ghodoussi, 1994).

Because of its very fine particle size and Pozzolanic characteristics, silica fume creates a very fine pore structure and a low ionic concentration in the pore solution (Berke et al., 1991), resulting in an increase in resistivity. OPC is more porous than other concretes, and porosity within concrete with fly ash should be considered less continuous than others (Cao et al., 1994), even for silica fume, owing to the Pozzolanic reaction in fly ash cement (FAC). The influence of mineral admixtures is very disturbing in the sense that, when they are present, electrical resistivity can be greatly increased (several times higher with additions). Without being aware of the additions, an operator can overestimate concrete quality. Additions influence the variation range of electrical resistivity for an individual concrete but, in terms of measured variability, their effects have to be underlined, especially for fly ash. Figure 12.3 represents the comprehensive result of a study performed on samples of various mixes characterised by strength range (25 to 120 MPa; shown as M25 etc. in Fig. 12.3), and concrete formula (OPC, with fly ash or

![Resistivity (ohm m) vs. Mixes]

12.3 Range of variation of electrical resistivity for 10 concrete mixes of various strength: CV for fly ash, and FS for silica fume (Lataste et al., 2006).
silica fume). Owing to numerous measurements of each sample being taken, the results present variability ranges and, in two samples, show repeatability in measurement sessions (for M50, repeated M50-0), and between two specimens with the same composition (for M75FS and M75-E30). The resistivity range is larger for concrete with fly ash than for others.

Concrete is a material that evolves over time. During the first hours of setting, hardening results from microstructural evolution. The chemical reaction continues until reagent consumption is complete. Generally, it can be observed that concrete achieves most of its final properties after twenty-eight days. Mechanically, as well as for electrical resistivity, this asymptotic evolution owing to hydration should not be forgotten during the first months, Fig. 12.4. However, it is not a phenomenon which can be taken into account because each concrete, each structure (owing to exposition) has a distinct evolution in time, and it can even differ because of the curing process (Hammond and Robson, 1955). For concrete with a mineral admixture, significant evolution of resistivity can be measured over several years (Hansson and Hansson, 1983). The increase in resistivity with time can be measured using normal methods (Lakshminarayanan et al., 1992; Neville, 1996). This link between the evolution of resistivity and a concrete’s age begs the question as to what the ‘reference value’ for the material should be.

![Concrete resistivity evolution with time for various concretes and cures, comparison between neat cements and concretes with a mix ratio of 4:2:1 by mass between the coarse aggregate, fine aggregate and cementitious materials; storage in air at room temperature (Hammond and Robson, 1955).](image-url)
**Fluids**

Because electrical conduction through concrete is mainly electrolytic, the influence of fluids on concrete resistivity is very important. Moisture is, along with porosity, clearly one of the most important factors conditioning the resistivity range (Fig. 12.5) and, whilst porosity is a relatively steady-state parameter, moisture can evolve spatially and temporally in a concrete structure. Studies show that below 40% threshold of relative humidity (RH), defined also as the saturation rate ($Sr$), water in concrete does not conduct, i.e. it is not mobile (Hunkeler, 1996). At that level, water is totally adsorbed. Up to this critical RH value, the link between RH and resistivity has been widely studied (Fiore *et al.*, 1994; Lopez and Gonzalez, 1993). From 40 to 100% RH, resistivity can decrease from 1000 to 100 ohm m (Hunkeler, 1996). Other values can be found but, in this moisture range, resistivity change is normally of one order of magnitude. Figure 12.5 illustrates the influence of $Sr$ on the electrical resistivity of concrete for OPC, with siliceous aggregates, in the range of 12.5 to 18.1% porosity. Within the range of 40 to 100% $Sr$, regression models can be calculated, that do not fit measured resistivity variations when $Sr$ is below 40%.

The amount of water influences the electrical continuity of conductivity between pores filled by water, but also contributes to a change in the ionic composition of the pore water, i.e. to the conductivity of the interstitial fluids: the more the concentration increases, the more the resistivity is lowered. Figure 12.6 illustrates this point by showing resistivity variations for four concrete mixes with various $w/c$ values, and for one of them (M6) with calcareous crushed aggregates (whereas the three others have siliceous round gravels). Samples are saturated by water with different concentrations of sodium chloride (NaCl) (ANR SENSO, 2009).

![Graph showing influence of saturation rate on electrical resistivity of concrete for OPC, with siliceous aggregates, at porosity of 12.5 to 18.1%; regressions are drawn in the range of 40 to 100% saturation rate (ANR SENSO, 2009).]
Although the influence of moisture variation on resistivity is more important than the variation in ionic concentration, the two influences are linked. Saleem et al. (1996) explain that the higher the ionic concentration, the less the moisture variations influence resistivity. The relationship between pore fluid properties and concrete resistivity is used in practice. Under laboratory conditions, where moisture content can be controlled, resistivity measurements have been correlated to the ionic concentration gradient, as documented by Morris et al. (2002), or taken to assess its evolution in time (Polder and Peelen, 2002) (as part of a rebar corrosion study).

As described by Archie’s law, ionic concentration of pore water is an influencing parameter on the apparent resistivity of concrete. In a number of papers, the modification of $\rho_i$ was studied by saturating concrete in a known solution. However, the question is more complex than this because the composition of saturation water is not the same as pore water. This is why the resistivity of saturation solution is not equal to $\rho_i$ in Archie’s law, and why this parameter remains the most difficult to assess. A relatively theoretical approach based on molar concentration and the influence of the pH delivers good results (Snyder et al., 2003) and a second approach, totally deductive, agrees with the first (Tumidajski et al., 1996). The similar results predict $\rho_i$ to be normally about 0.10 to 0.25 ohm m, weakly variable, whatever the external concentration conditions are.

**External factors**

The influence of temperature on the electrical properties of concrete is well known (Fig. 12.7). Concrete resistivity decreases when temperature rises; up to 40°C the tendency is inverse owing to microstructural transformations as a result of the phenomenon of an increase of ionic mobility.
The effect of temperature should be viewed as a bias on resistivity measurement, and there are several ways to correct the disturbance: one, proposed by Spencer (1937), is empirical, proposing a temperature reduction curve adapted for all concretes (Woelfl and Lauer, 1980); a second is an analogous procedure by use of an equation, linking variation in temperature and resistivity variations (Gowers et al., 1993; McCarter et al., 1981; Whittington et al., 1981). If the first approach is not adopted today (because of the uncertainties surrounding the method), the second approach, though more rigorous, is also limited. Indeed, to correct measurements, an operator needs a factor representing the material behaviour; in other words, for accurate corrections, a calibration of the material under consideration has to be made. In practice, this work is rarely completed, so operators use a general factor that is not accurate enough to totally correct measurements (when it does not create supplementary errors) (Lataste, 2002). Castelotte et al. (2002) suggest a law to calculate the intrinsic thermal coefficient for a concrete, taking into account its composition (cement and aggregate ratio) but still based on the paste’s intrinsic thermal coefficient. An estimate of the thermal effect on concrete resistivity is a gradient of about $-3.3 \text{ ohm m °C}^{-1}$ (Gowers et al., 1993).

According to Millard (1991), temperature influence during investigations on site is more important than the moisture effect, because thermal variations change more quickly than humidity over the same time, leading to erroneous interpretation owing to the drift in resistivity with temperature.

Geometry and edge closeness can be factors that affect interpretation by their bias effect on measurements; for them, the quality of interpretation has, of course, to be taken into account. When measurements are taken close to an edge, it results in a limitation of the volume investigated. The concentration of the current density then leads to an overestimation of the concrete resistivity. The range of sensitivity depends on the device used and on

---

12.7 Electrical resistivity variations with temperature (Millard, 1991).
its size. According to the methodology selected, this bias may or may not be taken into account: on site, a correction has to be made to map resistivity variations close to edges; in the laboratory, working on test samples with the same geometry, any bias can be considered a constant and thus does not disturb any relative interpretation. Electrical conduction is well known and theoretical approaches can be made to correct geometrical biases.

Figure 12.8 presents the comparison of an experimental approach (by measuring the resistivity increase close to the edge, in a homogeneous concrete), numerical modelling (by using a finite element computation), and analogical calculation (by the use of conduction equations) (Lataste et al., 2003b).

The rate of disturbance is defined as the ratio between the increase of resistivity (because of an edge) and theoretical resistivity (without disturbance) reported, where 'd' is the distance to the edge and 'a' the size of the device measuring resistivity. The three approaches are in agreement so, for simple geometry, the analogical approach appears to be the most straightforward method, whilst finite element computation can be used for more complex cases.

Very conductive material embedded in an insulator has an important influence on electrical current flows. Therefore, the influence of steel reinforcements in concrete is an important factor to observe when interpreting electrical resistivity measurements. Electrical resistivity measurements can be used to characterise the probability of rebar corrosion and the influence of the rebar on the measurement is discussed in section 12.3.1 under the heading ‘Four-probe resistivity measurement technique’. However, when resistivity measurements are performed solely to study the concrete’s properties, the predominant effect of steel on electrical flows conduction has to
be known. Some on-site methods allow the influence of the rebar to be limited: performing the measurement far from a rebar, for example, or minimising the rebar influence by having current applied perpendicular to it (Polder et al., 2000). It is also possible to compute the influence of a rebar and then to process the data to remove the influence of the reinforcement, in the same way as for boundaries (Lataste et al., 2003b). This computational work can become very complex for a real structure so, generally, any investigation must be designed to consider the location of the reinforcement (found by the previous application of a detection method to locate it, see chapter 2). For characterisation in the laboratory, measurements on samples including a rebar have to be avoided; if that is not possible, resistivity measurements should be taken perpendicular to the rebar.

Electrical resistivity measurement, as shown by Archie’s law, is influenced mainly by porosity and moisture. Degradation of concrete influences these parameters: carbonation, for example, over time leads to a reduction of porosity from the surface, and resistivity is sensitive to the carbonated layer on the concrete (Dhir et al., 1989; Millard and Gowers, 1992); cracks or micro cracks can be seen as creating ‘connected pores’ and can be studied by resistivity (Lataste et al., 2003a); the alkali–silica reaction (ASR) generates moisture and the position of ASR can be localised by resistivity mapping (Klysz et al., 2006). These parameters have to be considered when interpreting resistivity measurements, but their influence can also be used to characterise by resistivity, the alteration itself.

12.3 Use of electrical resistivity

12.3.1 Equipment and measurement

Electrical resistivity is measured by application of an electrical current (I) into the material, and by the measurement of the potential difference (U). The calculation of the ratio \( \frac{U}{I} \) gives the electrical resistance (R), according to Ohm’s law, followed then by a calculation of electrical resistivity (\( \rho \)). The relationship between R and \( \rho \) (and hence the equation used to calculate resistivity) depends on the technique and the device used for measurement.

Resistivity measurement is performed with a low frequency signal. Because electrical conduction in concrete is essentially electrolytic, polarisation can occur close to the probes if the measurement is taken with direct current, leading to measurement errors (McCarter and Brousseau, 1990). The use of alternating current affects other phenomena, such as capacity and inductance. Electrical resistivity measurement is a function of signal frequency (Hughes et al., 1985). Studies show that for low frequency measurement, resistivity values are similar to resistivity assessed with a direct
current signal, though with variable accuracy: under 300 Hz, the error is less than 1.5% (Millard et al., 1989), and thus the 10–300 Hz range is considered relatively free of the affects of any disturbance. Measurements with higher frequencies are not representative enough to characterise the electrical resistivity of concrete. When very high frequencies are used, the technique is called impedance spectroscopy, as described in Section 12.5.

**Electrical resistivity measurement cell: for laboratory resistivity assessment**

The electrical resistivity measurement cell technique consists of the establishment of a regular potential gradient by applying an electrical field intensity. The measurement of the gradient, relative to the size of the tested sample, allows assessment of resistivity. In practice, the sample is positioned between two probes (on two opposite faces); wet sponges are used to improve the electrical contact between the concrete and the probes (Fig. 12.9).

The application of $I$ (A) and the measurement of $U$ (V) results in the calculation of the total resistance ($R_T$). A second measurement, with only the two sponges (i.e. without the concrete sample), allows $R_S$, the resistance of the sponges in series, to be assessed. The concrete resistance $R_C$ is then determined as $R_T - R_S$. By multiplying $R_C$ by the cell factor, the material resistivity ($\rho$) can be assessed. The cell factor is defined by the ratio between the surface area $S$ of the electrodes and the path length $l$ of the electrical current:

$$\rho = \left(\frac{S}{l}\right)(R_T - R_S) \quad [12.2]$$

This test can be performed on a sample of concrete cast in a mould or a cored sample taken from a structure by drilling. To avoid any bias owing to water content, measurements are taken from saturated concrete. Saturation can be achieved under vacuum, as suggested in recommendations made by Baroghel-Bouny and Belin (2007), inspired by Chlortest (2007), or simply by storing the concrete in water. Control of the saturation is always recom-
mended (as is temperature) but no recommendations are given as to the conductivity of the saturation water. Simple mains water can be used, or Ca(OH)$_2$ can be added; results in either case are similar (Baroghel-Bouny and Belin, 2007); the only recommendation here is that the water used should be indicated in any report. A more important factor affecting the results is the quality of contact between probes and concrete, and sponges improve this. Several parameters can influence the contact: the quality of the mechanical contact between the probes, sponges, and concrete; the influence of the contact solution; and the influence of the concrete contact surface itself.

As far as mechanical contact is concerned, contact under pressure is proposed, to avoid measurement variability; it is recommended that the same pressure is always used (Baroghel-Bouny and Belin, 2007). Furthermore, the higher the pressure, the less the variability is (Newland et al., 2008). As far as contact solution is concerned, Newland et al. (2008) suggest using a solution with at least 1M sodium chloride to reduce variability (working with 6 kPa pressure contact). When measurements are taken using only water (to avoid any ionic ingress into the sample, for instance), the variability of the measurement is still acceptable: repeatability measurements on 120 samples from three different concretes result in a scatter in reproducibility smaller than 1.5% (with 3.2 kPa pressure) (ANR APPLET, 2009). The influence of surface quality was also studied, comparing resistivity measurement on a rough surface, a sawn surface, and a smooth surface. The study also considered the length of samples (Fig. 12.10). Results show that, on a regular surface (sawn or smoothed by grinding), results are slightly better in terms of variability. However, the main parameter influenc-
ing the quality of the result is the length: the longer the sample is, the more representative the resistivity (i.e. there is less variability) (ANR APPLET, 2009).

This technique is used in a laboratory, and allows accurate assessment of electrical resistivity. Refinements aim to improve the reliability of results and limit measurement variability. However, the technique is quite robust in the sense that, even with light conditioning of the samples (e.g. no saturation under vacuum) and a simple measurement protocol (e.g. use of only water), values are reliable with an acceptable range of reproducibility. The main parameters to be considered are: temperature (to be kept constant), saturation (has to be complete), pressure on probes (to be kept constant), and dimensions (with regard to aggregates, the diameter of the samples should be at least three times that of the largest gravel, and the length should be at least 1 diameter).

**Single-probe resistivity measurement technique**

The single-probe resistivity measurement technique is adapted for the on-site assessment of cover concrete on reinforced concrete structures. The principle is to connect one probe (the counter electrode) with the rebar mesh (by opening a window to the reinforcement), then by putting a second probe (a metallic disc) on the surface near the rebars (Fig. 12.11). Thus the concrete between reinforcement and surface is totally crossed by the electrical current (Broomfield, 1997; Feliu *et al.*, 1996).

Electrical resistivity ($\rho$ in ohm m) is assessed by resistance measurement ($R$ in ohm) and the size of the metal probe (diameter $D$ in m):

$$\rho = 2RD$$  \[12.3\]

This technique is not disturbed by the rebars. It is complementary to the measurement of polarisation resistance; however, resistivity measurements

---

![Diagram of single-probe device](image-url)
can be used independently of electrochemical measurement. The main disadvantage is the high sensibility to variation in contact resistance, causing noise in the results.

*Two-probe resistance measurement technique*

The two-probe resistance measurement technique consists of the application of an electrical current between two probes a few centimetres apart, and the measurement of the potential difference between these same two probes. The study of the potential distribution of an investigated volume shows that such a device is essentially sensitive to the electrical properties very close to the probes. The resistance measured is representative of the volume, defined by 10 times the radius of the concrete–electrode contact area (Millard, 1991).

The method does not allow the computation of resistivity and the results are expressed in terms of resistance, limiting the comparison of results for different sized devices. The method can be used as an indicator of variability of surface properties, by relative observation of the variation. For instance, the technique is used in the wood industry to assess moisture variations before use.

*Four-probe resistivity measurement technique*

The four-probe device, directly inspired by geophysical applications, is particularly adapted to measure reliable resistivity variations on site. It is the electrical device most used on structures. Various probe configurations are possible, but two are generally used: an in-line arrangement (called Wenner probes), which is the most used, and a square arrangement (less common) (Table 12.3). Both devices lead to identical resistivity variation measurement (Lataste et al., 2009).

The measurement principle is to apply an electrical field intensity (current, I) between two probes (current electrodes, and denoted as C1 and C2), and to measure the difference of the potential (U) between the other two (called potential electrodes, and denoted P1 and P2). From U and I, the resistance R can be calculated. By multiplying R by the geometrical factor (k in m), dependent on the configuration and size of the device (Table 12.3), the apparent resistivity (ρa) is assessed. The apparent resistivity is an average value of the resistivity distribution in the investigated volume. In a very homogeneous material, resistivity and apparent resistivity are equal. Furthermore, the measurement is performed from the surface and the influence of the concrete skin and the gradient of properties in the concrete cover cannot be forgotten. Nevertheless, ρ and ρa are correlated. A laboratory study, comparing resistivity values from cylindrical samples (11 cm
in diameter and 22 cm high) for different 90-day-old water-saturated concretes, achieves very similar results (Fig. 12.12).

The investigated volume essentially depends on the distance between probes, but cannot be rigorously determined; it is a function of the resistivity distribution (e.g. depending on whether the surface is more or less conductive than the bulk material below). Five centimetres is a standard distance between two probes, allowing roughly 3 cm to be investigated, corresponding to the cover concrete depth. A distance of 5 cm also allows any disturbances resulting from aggregates to be limited. Gowers and Millard (1999) suggest $a > 1.5D$ (with $a$ the distance between probes, and $D$ the size of the largest aggregate), to decrease the error owing to aggregates to <5%. Larger

Table 12.3 Four-probe square devices

<table>
<thead>
<tr>
<th>Device</th>
<th>Geometrical factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wenner configuration</td>
<td>$k = 2\pi a$</td>
</tr>
<tr>
<td>Square configuration</td>
<td>$k = \frac{2\pi a}{2 - \sqrt{2}}$</td>
</tr>
</tbody>
</table>

12.12 Resistivity (from resistivity cell) versus apparent resistivity (four-probe square device, $a = 5$ cm) measured on water saturated concrete samples (ANR APPLET, 2010).
distances are also possible, the main interest in enlarging the distance being to obtain information from several depths of investigation (Lataste et al., 2008).

The main parameter affecting measurement quality, i.e. its reliability, is the contact resistance. This resistance is at the contact between probes and concrete, and is due to the contrast of electrical properties between the materials. To allow measurement, and to improve them, devices are generally equipped with moistened sponges or wooden plugs at the tips of the electrodes. More elaborate devices automatically discharge conductive gel when the electrodes contact the surface (Broomfield, 1997). A light moistening of the surface just before measurement can contribute to lowering the contact resistance (Lataste et al., 2001).

Edge proximity is another source of bias, which can be corrected (for instance by using a square section device; Fig. 12.8), or can be limited by avoiding measurement closer than four times the distance between probes for Wenner devices (Gowers and Millard, 1999), or two times for the square section device (Lataste et al., 2003b). The influence of rebars also has to be considered and, on reinforced structures, this influence can be very important. To limit it, the location of the measurement has to be chosen carefully (Fig. 12.13). Applying electrical current perpendicular to a rebar allows the effects of the rebar to be minimised. Rilem TC 154EMC makes some recommendations on this subject (Polder, 2001).

On-site electrical resistivity measurements can be taken for many reasons, and there are various illustrations of how these techniques can help in the assessment of structure conditions. The main use of the measurements is to assess the probability of reinforcement corrosion (see also chapter 14) (Andrade and Alonso, 2001). Interpretation of results is based on the hypothesis that electrical resistivity is closely representative of the variation in concrete moisture, with humidity being one of the main parameters affecting corrosion. An empirically determined relationship has been proposed (RILEM TC154EMC, 2004) between resistivity and the intensity of current of corrosion, i.e. the speed of corrosion (Fig. 12.14), written as:

12.13 Optimum locations of four-probe devices relative to reinforcement: (a) and (b) Wenner configuration, (c) and (d) square configuration.
This research followed previous published thresholds, suggested by several workers, discussing the risk of corrosion and the range of resistivity of cover concrete (Table 12.4) these values are generally given for normal concrete but other values can be found in the literature (Langford and Broomfield, 1987).

Independently of the risk of corrosion, resistivity measurements are used as a tool to map humidity variations in concrete facings (Klyscz et al., 2006; Woelfl and Lauer, 1980). Very few other applications can be found in the
literature because, on real structures, moisture is generally the most variable parameter for a facing. Sirieix et al. (2007) provide an example where resistivity variation is linked to only one parameter, based on the hypothesis that all others factors are constant.

12.3.2 Calibration and reliability

Measurement of electrical resistivity is a technique used in the laboratory but also adapted to on-site measurement. It is a cheap and relatively easy technique to use on real structures. Noise levels, related to the measurement process itself and to material variability, are acceptable. Several R&D programs, under the umbrella of the French National Research Agency (ANR), have assessed different levels of variability (ANR APPLET, 2010; ANR SENSO, 2009) and the results have enabled various levels of variability of electrical resistivity to be distinguished and quantified over a wide range of concretes. Each value is assessed by at least 80 measurements, performed either with a resistivity cell (in the laboratory), or with the four-probes square device (in the laboratory or on site). Early results seem to show that orders of magnitudes are: less than 2% for repeatability (independent of the type of the device and assessed by repeated, successively performed measurements); about 5% for reproducibility (independent of the type of the device and reproducing measurements on the same sample and the same zone but integrating the measurement process); about 7% for concrete variability assessed in the laboratory (independent of the type of the device and performing the measurement by changing the location of the device on the sample, or changing samples in the same batch) and less than 9% on site (with the four-probe device on an homogeneous wall, $2 \times 2$ m²); and about 12% between batches (independent of the type of device). These values are given to evaluate different levels of variability, but also taking into account noise from the apparatus, the measurement process, non-mastered weak variations relative to the measurement position (the distance from a large aggregate, for instance), or owing to non-mastered concrete changes between batches. These values can differ according to the concrete composition and conditions of measurement but allow the assessment of the variability range, which has to be considered for interpretation.

On site, with a four-probes square device, on a real structure (not mastered), a resistivity variability of less than 10% can be considered representative of good measurement conditions; if the value is more than 20% a reliable interpretation is impossible. These values are in accordance with TC RILEM 154EMC which gave recommendations for the use of resistivity on site. Furthermore, they suggest a calibration process allowing the control of the apparatus and detection of material failure before investigation (Polder, 2001).
On site, in instances where edges and reinforcement need to be considered to limit their influence, it is very difficult to take moisture, chloride and temperature variations into account. Generally speaking, the hypothesis that all (or some) of these parameters are constant is required to determine another single parameter. For instance, one can consider moisture and temperature (and even porosity) constant to interpret resistivity variation as a function of chloride ingress only. This approach can be performed on a structure that is already relatively well known, allowing mapping of electrical variation for study, but it does not allow a quantification of the parameter investigated.

Concrete make up is also an influencing parameter. This parameter asks the question of the representativeness of a single value of resistivity. The question explains the various thresholds for resistivity to indicate corrosion risk that are found in the literature, for instance (Table 12.4), defined according to different approaches, and on various concretes. To avoid any misinterpretation, the solution is to restrict calibration of resistivity to a core specimen. A resistivity measurement by itself does not allow the interpretation of results in terms of exact value for moisture, or porosity. Resistivity measurement is a technique particularly well-adapted to mapping variations in a structure. For quantification, the calibration of a well-defined and located core specimen (from a previous electrical investigation) is the best solution. New developments are being made in non-destructive testing (NDT) data fusion, by exploiting several NDT methods (including electrical resistivity) to identify water, chloride, or porosity variations (Breysse et al. 2009).

12.4 Other developments

12.4.1 Monitoring for durability

Assessment of the risk of corrosion

Electrical resistivity is a property of concrete that is influenced by a number of parameters related to the concrete’s durability. This is clearly correct if moisture or chloride ingress is considered, for instance. This sensitivity can be used to monitor concrete structures using an integrated resistivity device cast into the structure itself. A study based on measurement with probes cast into a concrete sample has highlighted the correlation between resistivity variation with time and the probability of rebar corrosion, considering various factors evolving over time (Polder et al., 2002). This study has also led to a discussion of the apparent diffusion coefficient (conditioning the concrete’s durability) being inverse to resistivity. The resistance electrodes used were stainless steel screws of 5 mm diameter, positioned at depths of 10 and 50 mm, and the concrete was exposed to drying–wetting cycles with
NaCl to roughly represent the influence of de-icing salts and intermittent drying periods.

Other laboratory studies have used resistivity measurements taken by embedded probes. Ten different concrete mixes were subjected to a cyclic ponding regime with 0.55 M sodium chloride solution and the changes in concrete were monitored by measuring the changes in resistance between pairs of stainless-steel electrodes embedded in the concrete at different depths from the exposed surface (four pairs of 2.5 cm electrodes spaced at 1, 2, 3, and 4 cm depth) (Basheer et al., 2002). Basheer et al. also propose a relationship between the electrical resistance ratio (calculated by dividing the measured resistances value for each type of concrete with that of the OPC concrete, for a given depth) and the apparent diffusion coefficient. Such an application is also presented, with a similar approach, by Chrisp et al. (2002).

Assessment of moisture/ionic ingress

Knowing the importance of moisture in the alteration process of reinforced concrete structures, and exploiting the sensitivity of electrical resistivity to saturation rate, a device is proposed to monitor humidity variation in concrete. The device, which is set in a core hole, allows resistivity to be measured at different depths. Resistivity variation is linked to moisture variation, making the realistic hypothesis of conservation of all others parameters. In some cases, resistivity variations can also reveal the influence of the ingress of chloride. But the most interesting point is the ability to describe the gradient from the surface of moisture or chloride (link to the ionic ingress, for instance) (Weydert and Gehlen, 1999). The ability to continuously monitor the concrete cover-zone (covercrete), in real time, could also allow a more informed assessment of the actual and future performance of the cover-zone. The development of sensor systems to assess covercrete performance and durability, form an important component of an overall integrated monitoring system (McCarter et al., 2005). Resistivity sensors are adaptable, both in terms of the information that can be obtained from them but also with regard to their adaptability to concrete structures and the environment. Their limitations are the need to take core samples, and the difficulty of generalising from information measured at a single point.

Various configurations are possible. One of them is the technique based on multi-ring electrodes (MRE). The probes are steel plates sealed in a single core hole, at various depths (Fig. 12.15). The measurements made between pairs of plates allow the determination of resistivity by the application of a geometrical factor to the electrical resistances measured (Bäßler et al., 2000). MRE has been successfully used on a real structure to detect variations in depth (Prückner and Gjørv, 2001). Tests performed by
BrittEuram (2002) confirm the ability of such probes to monitor moisture variations.

12.4.2 Characterisation of electrical anisotropy

Any electrical anisotropy measured in concrete could be an important indicator if the material, in general, is totally isotropic. New developments, using techniques performed with the four-probes square device, specially designed to assess anisotropy at the same time as resistivity, are presented here. The principle of measurement is always the same: resistivity is measured, with a few seconds time lag, whilst electrical current is applied in two different directions. The ratio between the resistivity assessed in the first and second directions represents a value characterising the anisotropy, expressed by a logarithm (LogAn). When LogAn equals zero, this is isotropy; when it does not equal zero, anisotropy of concrete is present.

Fibre orientation in steel-fibre-reinforced concrete

Steel fibres in concrete can be viewed as conducting elements in an insulator material. As such, they greatly influence electrical resistivity. As a function of the spatial distribution of the fibres, electrical anisotropy can be used to characterise their orientation and density. Blind tests have been performed on concrete samples poured in several ways to influence their fibre distribution. By measurements in several zones, the complete distribution
of the fibres (in terms of orientation) has been determined (Lataste et al. 2008). The method is to find the axes of high and low resistivity, affected locally by fibre orientation (Fig. 12.16). Results are expressed in terms of local anisotropy. A diagram is then drawn indicating the supposed orientation of fibres in the zone under inspection, and the intensity of anisotropy is an indicator of the intensity of orientation. In this study, fibres are mainly oriented perpendicular to the marked anisotropy axis.

**Study of cracks in concrete by non-destructive testing**

A crack can be visualised as long as its volume is conducting (i.e. filled with water), or an insulator (i.e. filled by air), as a preferential way for current to flow, or as an insulator barrier. In any of these instances, if the electrical contrast between crack and concrete is sufficient, the crack creates a particular electrical flow in its neighbourhood when resistivity is measured. The use of the square device, to assess local electrical anisotropy, has shown its potential for crack detection (Lataste et al. 2003a). The electrical resistivity method allows the description of the crack intensity, where this indicator is influenced by a combination of depth, opening and filling of crack. Numerical studies and measurement sessions on samples and on structures highlight the ability of resistivity to be sensitive to crack parameters (Fig. 12.17).
The next step is to generalise results on real cracks. In any case, resistivity appears to be an interesting tool for NDT characterising of concrete cracking.

### 12.4.3 Electrical resistivity tomography

Future developments will deal with the assessment of resistivity in a concrete volume by surface measurement. This technique is widely used in geology and is called electrical resistivity tomography (ERT). The principle is to apply multiple measurements by use of a set of electrodes to have the distribution of apparent resistivity for various volumes and depths of investigation (depending on the position of probes). From these apparent resistivity values (raw data), an inverse calculation allows a model for the true resistivity distribution to be proposed. The final result is presented as a 2D map. Interpretation is based on the analysis of the variations in resistivity in the investigated plane. Some technological problems have to be solved such as, for example, the quality of the electrode-concrete contact. The data obtained will be used to describe the gradient of properties in the concrete volume. Some interesting tests have already been completed (Fig. 12.18) but a full adaptation to concrete surfaces, with a device based on a centimetre scale, has yet to be developed.

### 12.5 Impedance spectroscopy

Alternating current impedance spectroscopy (ACIS), also called electrochemical impedance spectroscopy (EIS), consists of the measurement of
the electrical properties of concrete at varying frequencies, up to the very high frequency (VHF) range (from Hz to MHz). The aim is to describe the entire electrical concrete behaviour, including its resistive but also its capacitive response. Results are very generally represented by the Nyquist diagram: the imaginary part of impedance ($Z_i$) as a function of the real part of impedance ($Z_r$) (Fig. 12.19).

The various loops and their sizes indicate electrical behaviours on various scales: the values at low frequencies are representative of pure resistive behaviour, whereas those with a higher frequency reflect the measurement of influences with capacitive response (McCarter and Brousseau, 1990; Ping et al., 2002).
Gu et al., 1992). Each one describes a specific microstructure characteristic: a high-frequency loop allows characterisation of hydrating cement (Ping Xie et al., 1993; Scuderi et al., 1991), following the ionic ingress in concrete (McCarter et al., 1996) or moisture variations in depth during time (Schießl et al., 2000), but also the description of interfaces such as aggregate–cement or steel–concrete (in order to evaluate the speed of corrosion by assessment of reinforced concrete (Millard et al., 1992)).

Concrete is described by its electrical behaviour through various equivalent circuit models (Fig. 12.19), which are elaborated to a greater or lesser extent (Brantervick and Niklasson, 1991; Guangling Song, 2000; Ping Gu et al., 1992, 1993). They are composed of resistances and capacities explaining, respectively, the real and imaginary behaviour observed in the Nyquist diagram: schematically, interfaces produce capacitive responses and electrical flow is characterised by resistance. The models allow the large range of phenomena influencing ACIS in concrete to be taken into account and described.

From a practical point of view, ACIS is a technique adapted for laboratory study. The apparatus used and the measurement process are difficult to apply easily on site. The measurement session can also be long and is not really adapted to investigation of a complete, real, structure. Nevertheless, some practical developments are on the way to adapt this powerful approach on site. The best improvements are directed at monitoring structures (McCarter et al., 2003): this limits the number of measurement positions required and allows significant material characterisation. Tests (still at the laboratory stage) are directed at the characterisation of the cover concrete zone, relative to ionic or moisture ingress (Rajabipour et al., 2004). Other tests have been performed on a steel-fibre-reinforced concrete beam to monitor fibre orientation (Ozyurt et al., 2006). In both cases, measurement can be made using sealed electrodes.

12.6 References


Anr Applet, Durée de vie des ouvrages: Approche Prédicitive Performantielle et probabiliste, Mode opératoire de la mesure de résistivité électrique (2009).


ASTM C1202-97, Standard test method for electrical indication of concrete’s ability to resist chloride ion penetration, 6p.


Abstract: The capacitive technique is an electromagnetic non-destructive technique, in which an alternating current, 30–35 MHz, is applied on electrodes that are laid on the surface of the medium to be surveyed. The global capacitance of the system is related to the sensor, to the shape of the electrodes, and to the permittivity of the medium. As the shift of the oscillator’s frequency, the parameter that is measured, is proportional to this permittivity, a calibration done on known homogeneous materials allows the corresponding value of permittivity to be obtained. Moreover, various dimensions of the electrodes are used to survey different depths of the medium from a few millimetres to several centimetres, thus giving some information on any gradient in the medium, such as water content in concrete mixing.

Key words: electromagnetic, capacitance, non-destructive testing (NDT), electrodes, surface analysis, concrete.

13.1 Physical principle and theory

Capacitive techniques have been commonly applied mainly in order to quantitatively estimate the water content, or indirectly the porosity. From soils (Baron et al., 1977; Fares et al., 2002) including rocks (Rust et al., 1999) or snow (Louge et al., 1998), or agricultural products (Nelson, 1992), some studies have shifted towards the evaluation of reinforced concrete structures (Dupas 2001; Iaquinta, 2004; Dérobert et al., 2008). The principle is to place two (or more) electrodes on the surface of interest and to apply an alternating electric current between them. This system corresponds to a capacitor in which capacitance $C$ is related to the electromagnetic nature of the internal components (such as the nature of the materials, the moisture or the porosity).

The measurement is performed by using an oscillator connected to an inductance $L$ and our special capacitance $C$, with frequency $f_{osc}$ defined by:

$$f_{osc} = \frac{1}{2\pi\sqrt{LC}}$$

[13.1]
The capacitance value is a function of the various elementary capacitors related to the sensor and to the permittivity of the surveyed medium. Variations of the permittivity of moister or drier media modify the capacitance and then the oscillator’s frequency. The choice of this frequency is therefore a compromise between the loss factor, which should be minimized, and the depth of penetration, which has to be sufficient for the application, Fig. 13.1.

If there are no capacitive problems of the contact, the sensor with its electrodes placed on a material can be represented as two capacitors in parallel, the first one $C_{\text{rest}}$ corresponding to the support and the near environment, the second one $C_{\text{material}}$ to the electrodes and the surveyed material. Then, the global capacitance $C$, in equation [13.1] is defined by:

$$C = C_{\text{rest}} + C_{\text{material}}$$  \[13.2\]

These elementary capacitances can be described as:

$$C_i = \varepsilon_0 \varepsilon_i \chi$$ \[13.3\]

where $\varepsilon_0$ is the permittivity of vacuum ($= 8.854 \times 10^{-12} \text{ F m}^{-1}$), $\varepsilon_i$ is the dielectric constant of the material and $\chi$ is a geometrical factor, assuming that this factor is constant for all these elementary capacitors and then defining a virtual dielectric constant $\varepsilon_{\text{rest}}$ corresponding to the rest and the near environment.

Equation [13.1] can be modified thus:

$$f_{\text{osc}} = \frac{1}{2\pi} \sqrt{\frac{\varepsilon_{\text{rest}}}{L\varepsilon_0 \chi}} \left(1 + \frac{\varepsilon_{\text{material}}}{\varepsilon_{\text{rest}}} \right)^{-1/2}$$ \[13.4\]

In the field of civil engineering, the range of materials dielectric constants remains small compared to $\varepsilon_{\text{rest}}$, and that fact allows some simplification such as:

$$f_{\text{osc}} = \frac{1}{2\pi} \sqrt{\frac{\varepsilon_{\text{rest}}}{L\varepsilon_0 \chi}} \left(1 - \frac{\varepsilon_{\text{material}}}{2\varepsilon_{\text{rest}}} \right)$$ \[13.5\]

indicating that the oscillating frequency is linearly proportional to the dielectric constant of the surveyed medium.
The choice of the frequency band is a compromise between the electronic technology that can be used in the sensor and the volume of material that is coupled to the sensor. As the relative permittivities of concrete mixing present important variations at low radar frequencies, the capacitive technique appears to be very sensitive at such frequencies to the components or the state of the concrete. Indeed, Fig. 13.2 shows some examples of relative permittivity measurements of a common concrete dry and saturated, for different levels of chloride content (0, 30 and 120 g l\(^{-1}\) of NaCl, the two last corresponding, respectively, to sea water and deicing water) confirming these variations.

For saturated conditions, we can note that the relative permittivity is much more sensitive to water and chloride content at lower frequencies, with very high values for the imaginary part increasing with chloride content. In dry conditions, the relative permittivity seems almost non-dispersive, with a very slight effect of the chloride, which is crystallized in
the pores. This phenomenon is confirmed for the imaginary parts, which seem insensitive to the chloride content.

### 13.2 Equipment

As the choice of frequency is a compromise between the electronic technology that can be used in the sensor and the volume of material coupled to the sensor, the network of LPC has studied for years the system presented in Fig. 13.3(a). This system comprises a single capacitor, working at 35 MHz, associated with a set of limited types of electrodes (Baron and Tran, 1977; Dérobert et al., 2008), in order to be able to investigate plane concrete structures, of different thicknesses from 2–3 mm with the small electrodes $5 \times 70$ mm, to 7–8 cm with the largest electrodes $40 \times 70$ mm. Recently, some studies by the LPC network have led to the design of a capacitive sensor for the grouting evaluation of post-tensioned high-density polyethylene (HDPE) ducts (Dupas 2001; Iaquinta, 2004), as shown in Fig. 13.3(b) in data acquisition position around a 1-m duct sample. The oscillator’s frequency is about 65 MHz corresponding the best compromise between sensitivity of the measurements and the volume coupled with the probe, for this specific geometry. In both cases, the acquisition system, combining an oscillator and a frequency divisor, which decreases the signal towards lower frequencies in the range of 5000 Hz, is linked to a laptop using Hertzian communication and home-made software.

13.3 (a) Capacitance probe prototype for a flat concrete structure, with two complementary electrodes shown in the bottom. (b) Capacitance probe prototype for the grouting evaluation of external post-tensioned HDPE ducts.
13.3 Calibration

The calibration is carried out on homogeneous materials, presenting flat surfaces that ensure perfect contact with the electrodes, and whose dielectric constants are known, such as Teflon, PVC, granite, marble, limestone and a low-loss resin.

The measurements are done as follows: the measurement on the material is subtracted from another one done in air. This approach allows the minimizing, or removing, of any influence of the electronic shift, the sensor itself and the hand of the operator, and any environmental effect (temperature, moisture).

Calibration results are presented in Fig. 13.4 showing linear relations between the measurement and the electromagnetic property of the homogeneous material. The slope of each calibration curve expresses the sensitivity of the capacitance of each electrode to the dielectric constant of the surveyed medium.

It must be noted that the small-size electrodes provide visible dispersion in the results, suggesting a higher level of uncertainties in real concretes.

13.4 Data acquisition and interpretation

Some experiments have been performed on homogeneous mixtures of concrete, conditioned in the laboratory for various water contents, in the frame of a French National Project (RGC&U 2000–2003). Three types of concrete were designed, two 25-MPa concretes (CEMI 350 kg m⁻³) using different
Regional siliceous aggregates and one 40-MPa concrete (CEMII/A-LL 385 kg m$^{-3}$).

Results from Fig. 13.5 show a linear evolution of the variation of frequency function of the volumetric water content. The slight shift between the two 25-MPa concretes comes from the difference of dielectric constant of the aggregates themselves.

13.5 Applications

Even if the estimation of the water content of concrete surfaces of civil engineering structures remain the major application of the capacitive technique, the system operates in a frequency band where any chloride ingress, in concrete, induces important variations in the dielectric constant values (Fig. 13.2). This phenomenon is not so clear at much higher frequencies, in the radar frequency band (above 1 GHz). Some examples are presented in Fig. 13.6, from experiments done in the frame of a French National Project (SENSO 2006–2009) on homogeneous concrete slabs confirming these assumptions.

A standard concrete, defined by siliceous aggregates, 370 kg m$^{-3}$ of cement with a water–cement ratio w/c = 0.58, was filled homogeneously with salted water of different water content. Figure 13.6 presents the results obtained from radar and capacitive techniques. One can note that the radar velocity, which is directly correlated to the dielectric constant, is only modified for high water content, when the concrete is almost saturated.

For the capacitive measurements, the result varies significantly, until the concrete becomes moist, in proportion to the water content.

For concrete structures not exposed to significant chloride ingress, far from any coastal area, this technique can give accurate information on the

![Diagram](image-url)
moisture content of the concrete cover, but because the results integrate any gradient of water versus depth in the measurement, and lacking any calibration (the dielectric constant of the aggregates being unknown) no quantitative results are possible.

13.6 Limitations and reliability

The most important limitations of the technique are related to the flatness of the concrete surface, the absence of information about the location of the reinforcement and the need for calibration of the concrete material itself in order to obtain water and/or chloride content values.

Concerning the flatness of the surface, all the effects resulting from the building of the concrete structure, such as the border of successive formworks forming steps, or spherical voids owing to air bubbles in the surface laitance, can lead to important errors. These effects become vital for the small-size electrodes.

Secondly, for accurate measurements, we have to get information on the location of the reinforced bars in order to do spatially localized measurement in the centre of the reinforcement mesh. Indeed, the presence of rebars in the coupling volume under the electrodes induces a slight decrease in the measurement values, which could be interpreted as the concrete being dryer than in reality.

Then, the knowledge of an average of dielectric constant remains insufficient to get accurate water-content values. Knowing the strength of concrete (e.g. C25 or C40) can give a good indication of the relationship between the capacitive measurements and the water content, but we have to remember that the nature of the aggregates themselves strongly affect the relationship.
For the last aspect related to reliability, the relative measurements induce a strong limitation of the temperature influence and prevent any ‘hand effect’. Moreover, taking an average of elementary measurements (about five) a few centimetres around the measurement location decreases the noise in the value.

Finally, this technique is limited to the water, or chloride, content estimation of homogeneous concrete where the thickness is above 10 cm.

13.7 References


Techniques for measuring the corrosion rate (polarization resistance) and the corrosion potential of reinforced concrete structures

C. ANDRADE and I. MARTÍNEZ, Instituto de Ciencias de la Construcción Eduardo Torroja (CSIC), Spain

Abstract: The importance in real structures of an accurate identification of the zones suffering corrosion, and the determination of the actual loss in cross-section in these zones and the rate of its progress is recognized. Although there exist several techniques used on-site for characterizing corrosion of reinforcements, the most appropriate are those of an electrochemical nature because this is the basis of the corrosion process. The three parameters most frequently used for the corrosion evaluation are: the corrosion potential \( E_{\text{corr}} \), the concrete resistivity \( \rho \), and the corrosion rate \( I_{\text{corr}} \) calculated from the polarization resistance \( R_p \). In a real structure, the measurement of the three corrosion parameters allows the identification of the zones with high risk for damage. The principles of the use of the corrosion rate and the corrosion potential measurements for the evaluation of corrosion in reinforcement structures are described and current and future applications outlined.

Key words: polarization resistance, corrosion potential, reinforced concrete.

14.1 Introduction

Corrosion is one of the main durability problems that affect reinforced concrete structures. This degradation process involves the dissolution of the steel, which transforms into oxides or other salts. The importance in real structures of an accurate identification of the zones suffering corrosion, and the determination of the actual loss in cross-section in these zones and the rate of its progress is recognized. Although there exist several techniques for on-site characterization of corrosion of reinforcements, the most appropriate are those of an electrochemical nature because this is the basis of the corrosion process. The three parameters most frequently used for corrosion evaluation are: the corrosion potential \( E_{\text{corr}} \), the concrete resistivity \( \rho \) and the corrosion rate \( I_{\text{corr}} \) calculated from the polarization resistance \( R_p \).

In a real structure, the measurement of the three corrosion parameters enables the identification of the zones at high risk of damage. Through the
Techniques for measuring corrosion rate and potential

resistivity, it is possible to determine, owing to their higher water content, those zones corroding and those still not corroding, but only the calculation of the $I_{\text{corr}}$ enables the prediction of the evolution of the corrosion process. Corrosion measurement techniques can then be applied for:

- identifying corroded zones or zones at risk from corrosion,
- predicting the rate of corrosion of reinforcement,
- predicting the residual life of the structure, and
- monitoring the efficiency of repair or prevention systems.

The three parameters can be obtained in a single measurement of polarization resistance. Thus, when describing $R_p$, the whole set of parameters needed for the better interpretation are contained in it, whereas when $E_{\text{corr}}$ or $\rho$ are individually measured, these parameters are insufficient for the full interpretation.

For determination of the accumulated corrosion penetration $P_x$ or loss in cross-section, it can be calculated from the integration of the $I_{\text{corr}}$ values during the propagation time if the depassivation event is identified. This method is applicable for structures where the depassivation moment is known. Otherwise, in an existing corroding structure, the actual loss in diameter can be only obtained by direct visual observation at the time of inspection.

### 14.2 Principles

#### 14.2.1 Corrosion potential

Because of its simplicity, the measurement of the potential $E_{\text{corr}}$ is the most frequently used method in field determinations. The corrosion potential $E_{\text{corr}}$ (half-cell rebar/concrete) is measured as potential difference (or voltage) against a reference electrode (half-cell). As a corrosion detection technique, this was first used by Stratful.¹ Half-cell potentials of steel in concrete cannot be measured directly at the concrete–rebar interface owing to the presence of the concrete cover. Thus, the numerical value of the measured potential difference between the steel in concrete and the reference electrode will depend on the type of reference electrode used and on the corrosion condition of the steel in concrete.

The corrosion of metals in concrete is based on the formation of coupled anodes and cathodes leading to corrosion microcells (distances of micrometers) and macrocells (distances of cm). Corrosion of steel leads then to the co-existence of passive and corroding areas in the same bar which present different mixed potentials and, depending on the position where the measurement is performed, to an overall corrosion potential. The voltage differences between the corroding and passive zones of the bar induce current
flows which lead to potential-differentiated fields. These electric fields can be measured and represented as equipotential lines that allow the study of the state of reinforcement as given in Fig. 14.1.

From a practical point of view, potential maps\(^2\) are drawn from the \(E_{\text{corr}}\) measurements, and these reveal those zones that are most likely to undergo corrosion in the active state.\(^3\) However, such measurements have only a qualitative character.\(^4\) This is because the potential map only informs on the risk of corrosion and not on its actual corrosion activity. Potential mapping is still able to give a qualitative indication of the general performance of a structure and is a complement of the other on-site techniques.

The values of the potential found can be ranked in order to qualify a corrosion ‘risk’. In Table 14.1 a ranking is given following the criteria given by the ASTM C876.\(^5\)

A wide range of factors may influence the corrosion potentials and these are outlined below.

**Concrete moisture content**

Changes in moisture content mean that the isoelectrical lines differ from those of the dry case, leading to a difference of potentials up to 200 mV.
Potential values become more negative as concrete moisture increases. For a homogeneous moisture distribution, differences between corroding and passive zones are clearer, but it must also be considered that, for very dry concrete, active corrosion areas may not be well detected. The moisture content of concrete can be checked by resistivity measurements and therefore the joint consideration of potential and resistivity maps substantially help in the interpretation.

**pH influence**

Nernst behaviour indicates that one unit of pH corresponds to 60 mV or more of anodic potential value. In non-carbonated concrete, the pH differences are not significant, but in carbonated concrete the effect is relevant. In dry carbonated concrete the potentials may be very positive whereas in wet carbonated concrete, owing to the active corrosion, the potential may become very negative.

**Chloride content**

More chlorides result in more intense corrosion or extensive corrosion zones. Field experience on a large number of bridge decks has shown a certain correlation between concrete chloride content and the potential values.

**Cover thickness**

As concrete cover increases, the potential difference between corrosion and passive zones is more attenuated, resulting in a uniform potential value at infinite. Thus, the positioning of small corrosion spots becomes more difficult with increasing cover depth.

**Macrocell polarization effects**

The cell is composed of anodes and cathodes. In concrete there is anode control, i.e. the anode is polarizing the passive rebars in the vicinity of the

---

**Table 14.1 Ranges of potential and corrosion risk according to ASTM C876**

<table>
<thead>
<tr>
<th>$E_{\text{corr}}$ (SCE) (V)</th>
<th>Corrosion risk (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$&gt;-0.2$</td>
<td>10</td>
</tr>
<tr>
<td>$-0.2$ to $-0.35$</td>
<td>50</td>
</tr>
<tr>
<td>$&lt;-0.35$</td>
<td>90</td>
</tr>
</tbody>
</table>
corroding area to negative potentials. The shift of potentials to more negative values is larger in low-resistivity concrete than in high-resistivity concrete. Thus, the cathodic areas surrounding the anodic ones have a similar potential, leading to misinterpretations. Therefore, the interpretation needs always to take into account that the anodes are not alone but need a cathodic area to be active.

**Oxygen content**

Conditions of aeration, i.e. oxygen access, strongly determine rest potential values of passive steel in concrete. Low oxygen content leads to a pronounced decrease towards negative values of the rest potential and this is less pronounced in wet concrete. This leads to the risk that passive areas under low aeration conditions could be considered to be corroding areas. In corroding steel, the lack of oxygen may result in very negative corrosion potentials but also in low corrosion rates. Therefore, in the absence of oxygen, the interpretation of the corrosion state cannot be made in terms of the potential values.

### 14.2.2 Resistivity

Electrical resistance \((R_e)\) is the ratio between applied potential and the current circulating in an electrolyte. It is expressed through Ohm’s law.

\[
R_e = \frac{E}{I} \quad [14.1]
\]

The resistance depends on the geometry of the element, but the resistivity is independent of the geometry.

\[
\rho = R_e \times K_{cell} \quad [14.2]
\]

where \(K_{cell}\) is the geometrical cell constant, which, for a cube, is defined as the cross-sectional area \(A\) divided by the distance \(L\) between electrodes.

\[
K_{cell} = \frac{A}{L} \quad [14.3]
\]

The concrete \(\rho\) values indicate the degree of moisture content of the concrete, which is related to the corrosion rate when the steel is actively corroding, but which may mislead the interpretation under passive conditions. Regarding the corrosion state for the potential as well as for resistivity \(\rho\) the qualitative character can be stated. However, there is a relationship between \(\rho\) and \(I_{corr}\), discussed below, that enables the correct interpretation of the values of corrosion rate. The measurement of \(\rho\) has already been
Techniques for measuring corrosion rate and potential

Chapter 12; therefore, in this chapter only references to the correlation with the other electrochemical techniques for their interpretation in on-site measurements will be made.

The corrosion risk can also be classified as a function of the resistivity as shown in Table 14.2.

### 14.2.3 Corrosion rate

**Polarization resistance**

The corrosion rate $I_{\text{corr}}$ is the only electrochemical parameter that quantifies the rate of loss of metal. Traditionally, the loss of metal has been measured in the laboratory by gravimetric determinations. The gravimetric method gives average values of the corrosion rate because it considers the behaviour over a certain period of time. However, as metal corrosion in contact with aqueous electrolytes is an electrochemical phenomenon, the techniques used for its study are of the same electrochemical nature. These techniques are based on applying an external voltage or current to the metal and measuring the response generated. The change in potential is currently monitored by means of a reference electrode.

Instantaneous corrosion measurements were first electrochemically determined by extrapolation from the initial straight part of the polarization curve. A method later introduced was based on the work of Stern et al., who suggested the so-called polarization resistance or linear polarization $R_p$ technique based on the mixed-potential theory of Wagner and Traud. Applying this method, the corrosion current is determined by measuring the slope of the polarization curve at the corrosion or mixed potential in such a way that the polarization applied to the metal is small enough not to alter the corrosion process.

Owing to the small polarization applied, the $R_p$ technique is non-destructive in character, which has been the main reason for its generalized use, in spite of the initial criticisms of the technique. Currently, the $R_p$ determination is accepted as a fast and reliable method for calculating

<table>
<thead>
<tr>
<th>$\rho$ (kΩ cm)</th>
<th>Corrosion risk</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt;100–200</td>
<td>Negligible</td>
</tr>
<tr>
<td>50–100</td>
<td>Low</td>
</tr>
<tr>
<td>10–50</td>
<td>Moderate in carbonated or Cl$^-$ contaminated concrete</td>
</tr>
<tr>
<td>&lt;10</td>
<td>Resistivity is not an appropriate parameter. Other parameters must be taken into account</td>
</tr>
</tbody>
</table>
corrosion rate in numerous metal–electrolyte systems. The authors have successfully used it for many years for the determination of the corrosion rate $I_{\text{corr}}$ of reinforcements.13

The polarization resistance $R_p$ of a reinforcement embedded in concrete is defined as the ratio between applied voltage $\Delta E$ (shift in potential from $E_{\text{corr}}$) and the step of current $\Delta I$, when the metal is slightly polarized (about 20–50 mV) from its free corrosion potential $E_{\text{corr}}$.4 It can also be defined as the slope of the potential-current polarization curve at the corrosion potential $E_{\text{corr}}$.8,14

$$R_p = \left( \frac{\Delta E}{\Delta I} \right)_{\Delta E \to 0} \quad [14.4]$$

where the current density $i = \Delta I/S$, with $S$ being the steel area polarized. The units in which $R_p$ is usually measured are kΩ cm$^2$ or kΩ depending on whether the polarized area is taken into account or not.

The corrosion rate or current density $I_{\text{corr}}$ is related to $R_p$ by means of the expression:

$$I_{\text{corr}} = \frac{B}{R_p} \quad [14.5]$$

The units of $I_{\text{corr}}$ are $\mu$A cm$^{-2}$ (with $R_p$ in kΩ cm$^2$ and $B$ in mV).

The Tafel constant $B$ lies between 26 and 52 mV for reinforcement measurements. In general, the higher the corrosion is, the lower is the Tafel constant measured. For reinforcements, it is recommended to expect a $B$ value of 26 mV as an averaged value because it applies to active corrosion and the error of 2 when the steel is passive is relatively negligible.15

The calculation of the corrosion rate and corrosion penetration from the polarization resistance $R_p$ technique is well established.8,14,16–18 The application to the measurement of the corrosion of steel reinforcement started about 197313,19–33 and the agreement between gravimetrically determined weight loss and the electrochemical measurements has been largely demonstrated.13,19,20

It has to be also mentioned that an ‘analogue or electrical model’ is used to simulate the steel/electrolyte interface (double layer). The model used in corrosion studies is the Randles one34 (Fig. 14.2), which is composed of a capacitance to simulate the double layer and a resistance for the faradaic corrosion (weight loss or the polarization resistance) that is the resistance to corrosion. It has to be emphasized that this circuit is too simple for modelling the dispersion of the applied current in real-life full-scale structures.

The $R_p$ can be calculated by solving the circuit:

$$\frac{\Delta E}{\Delta I} = R_p\left(1 - e^{-\Delta I/R_pC}\right) + R_c \quad [14.6]$$
14.2 Randles circuit or electrical analogue model of metal–electrolyte interference.

Table 14.3 Ranges of the corrosion penetration rate and their relation to the velocity of the process

<table>
<thead>
<tr>
<th>Range of $V_{corr, rep}$ ($\mu$m year$^{-1}$)</th>
<th>Corrosion progression</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;1</td>
<td>Negligible</td>
</tr>
<tr>
<td>1–5</td>
<td>Low</td>
</tr>
<tr>
<td>5–10</td>
<td>Moderate</td>
</tr>
<tr>
<td>&gt;10</td>
<td>High</td>
</tr>
</tbody>
</table>

Ranges of corrosion rate values

The values of corrosion rate can be translated into corrosion penetration depth rate by means of Faraday’s law, which says that one equivalent of iron is lost for every coulomb passed. Thus:

$$\frac{It}{F} = \text{Equivalence number}$$  \[14.7\]

The conversion of $I_{corr}$ in $\mu$A cm$^{-2}$ into corrosion penetration rate $V_{corr}$ in $\mu$m year$^{-1}$ is made by using:

$$1 \mu\text{A cm}^{-2} = 11.6 \mu\text{m year}^{-1}$$  \[14.8\]

In Table 14.3, the ranges of $V_{corr}$ are given against how fast the corrosion process is progressing.

Accumulated corrosion penetration depth

The instantaneous values are plotted in graphs of $I_{corr}$ versus time in order to follow the corrosion process and distinguish among the variables under study as shown in Fig. 14.3.

Thus, by monitoring the $R_p$ values and integrating the instantaneous corrosion rates calculated from them, it is possible to determine the accumulated corrosion depth, or corrosion penetration $P_x$. The integration can be expressed as:
or, if $I_{corr}$ is constant or is averaged over a certain period $t$ (in years), by means of a more simple expression:

$$P_x = I_{corr} t$$ \hspace{1cm} [14.10]$$

As mentioned, $P_x$ can be expressed in two different units:

$$P_x = I_{corr} t \rightarrow P_x \, (\mu A \, cm^{-2} \, year)$$ \hspace{1cm} [14.11]$$

and

$$P_x = 0.0116I_{corr} t = V_{corr} t \rightarrow P_x \, (mm \times year \, year^{-1}) = P_{wi} \, (mm)$$ \hspace{1cm} [14.12]$$

Figure 14.4 enables another concept to be introduced that is related to the maximum pit depth, which, in turn, is related to $I_{corr}$ and $P_x$. In a metal corroding by localized attack, generally there is a relation between averaged corrosion and the maximum pit depth achieved.

$$P_{wi} = \alpha \, P_x$$ \hspace{1cm} [14.13]$$

This expression means that there is a factor $\alpha$ that can be multiplied by the averaged accumulated corrosion penetration to give the maximum depth of the maximum pit. The values of $\alpha$ are 3–15 with a value of 10 being the most frequent. This expression serves to identify the ‘worst’ scenario for prediction of the loss in cross-section.
14.3 Measurement methods

14.3.1 Measurement of corrosion potential $E_{corr}$ and polarization resistance $R_p$

It is essential to always quote the reference electrode being used for half-cell potential measurements. The unpolarizable reference electrodes saturated Calomel (SCE) or Cu/CuSO$_4$ (CSE) are more accurate than other types such as carbon. In addition, their maintenance regime (contact membrane) is crucial for their accuracy.

In practice, the following reference electrodes with a defined, constant and reproducible potential versus the standard hydrogen electrode (SHE) are used:

- copper/copper sulfate saturated CSE + 0.318 vs. SHE,
- calomel (Hg / Hg$_2$Cl$_2$) KCl saturated SCE + 0.241 vs. SHE,
- silver chloride [Ag/AgCl (SSCE)],
- KCl saturated SSCE + 0.199 vs. SHE.

The copper/copper sulfate electrode is the electrode most used for in situ potential measurement, whereas calomel and silver chloride electrodes are used more in laboratory investigations. Care has to be taken to wet the concrete surface correctly and sufficiently in order to minimize the first carbonated layer effect. The carbonated layer may introduce a high resistivity and partially shield the potential measurement. Abundant wetting for several minutes allows the water to reach a depth of a couple of millimetres and thus minimize the effect of the surface carbonation. It is more difficult to avoid the perturbation of potential lines if the concrete is dry, and if the carbonation is deeper than a millimetre or two, the carbonated front does not reach the bar.

The measurement of $E_{corr}$ is made first by measuring $R_p$ around the corrosion potential. Then, the electrical resistance is registered either during the $R_p$ measurement or after it in a separate measurement. The removal of the value of the electrical resistance from the $R_p$ measurement...
is essential in concrete, because the measurement gives both resistances together.\textsuperscript{13,19,20} Thus:

\[ R_p = R_{\text{total}} - R_e \] \hspace{1cm} [14.14]

The measurement of the $R_p$ is made by a three-electrode arrangement using the reinforcing steel as working electrode (WE), whose potential is monitored by a reference electrode (RE). A counter electrode (CE), at least of equal size to the rebar, injects the polarizing current inducing a shift in the potential of about 20 mV from the corrosion potential, either in the cathodic or in the anodic direction or going from the cathodic to the anodic direction.

Care has to be taken to ensure a correct ohmic drop, RI, or electrical resistance, compensation. Because of the relatively high electrical resistivities of the concrete, too low $I_{\text{corr}}$ values can be obtained if compensation of RI drop is neglected or not adequately performed.\textsuperscript{13,19,20} As mentioned in equation [14.14], this is because the $R_{\text{total}}$ calculated is the sum of the resistance associated with the actual corrosion process and the resistance associated with the electrolyte resistance (concrete). Currently, compensation can be electronically made by means of two methods: \textit{positive feedback} and \textit{current interruption}, both of which may be equally accurate if performed correctly.

14.3.2 Various methods of obtaining $R_p$

The sweep rate or the waiting time used for the calculation of the $R_p$ has to be selected in such a range that stationary conditions are achieved. Stationary conditions means that the effect of the capacitor indicated in the figure of the Randles circuit (Fig. 14.2) is negligible. In other words, during the first few seconds (called the ‘transitory period’) of applying a current to a metal–electrolyte system, the response shows a change owing to the charge of the capacitance of the double layer. Only when reaching a more or less stable value does the double layer influence disappear. This is the moment at which the $R_p$ value, corresponding to the resistance of the Randles circuit, can be measured. To measure during the transitory period, which is possible using certain other equipment, results in erroneous results. Therefore, correct waiting times or sweep rates are an essential part of the operational procedures.

The $R_p$ value can be obtained from either dc or ac techniques.\textsuperscript{35}

\textit{Potentiodynamic techniques}

The potentiodynamic method is based on the observation of the linearity of the polarization curves just around the value of $E_{\text{corr}}$ (Fig. 14.5), that is,
Techniques for measuring corrosion rate and potential

the slope $\Delta E/\Delta I$ of the polarization curves just around $E_{\text{corr}}$, as given in equation [14.4]. The summary of the main aspects to be considered already mentioned related to its application in concrete are:\textsuperscript{36}

- The need to compensate for the ohmic drop between working and reference electrodes: all modern laboratory potentiostats have the option of eliminating this drop, so, nowadays this is not a problem.
- The condition of linearity: it was also demonstrated for concrete that, for potential shifts of 20–30 mV around the $E_{\text{corr}}$, the polarization curve remains linear. For this reason, usually, the $R_p$ measurement is performed by a linear sweep of $\pm 10$ mV around the $E_{\text{corr}}$.
- The achieving of a stationary value: this requires a sufficiently long waiting time in static measurements, or appropriate sweep rates in dynamic ones. The recommended waiting times are around 15–60 s in potentiostatic operation and around 60–100 s in a galvanostatic one. With a sweep rate of 2.5–10 mV min$^{-1}$ similar results are obtained. In all instances the corrosion rates are in good agreements with the gravimetric losses.\textsuperscript{13,15,36}

\textbf{Pulse methods: galvanostatic and potentiostatic pulses}

Pulse methods are based on the application of a constant current or a potential step to the steel during a certain time, and the registration of the response obtained from the steel. Depending on the state of the steel, pulses in the range of 5–100 $\mu$A can be applied. The polarization measured in the steel during the application of the pulse must be preferably between 10–20 mV.

For optimum polarization time,\textsuperscript{13,15,36} corroding rebars need shorter polarization periods whereas passive steel needs longer waiting times to
reach a stable value. As already stated, in potentiostatic measurements, waiting times of 15–60 s are enough to achieve the quasi-stationary regime necessary to obtain the $\Delta I$ value to be used in the expression $\Delta E/\Delta I = R_p$. In galvanostatic measurements, longer waiting times are necessary to record a quasi-stationary $\Delta E$ value: periods of 60–100 s are the most appropriate. Figure 14.6a shows the response in potential to a galvanostatic pulse. The difference in response is shown for passive and active steels as well as the most suitable waiting time for each. In Fig. 14.6b, the different contribution of the corrosion rate and the ohmic drop to the pulse are documented.

Several procedures\textsuperscript{37–41} are reported which have made use of the recording of the transient response after the application of a galvanostatic pulse (very short waiting times).

**Electrochemical noise**

The electrochemical noise technique is based on the analysis of the very small voltage or current variations (in the range of microvolts) recorded during some time at the rest potential. In most of the cases, three identical electrodes are used. This method has been applied in experiments related to the pitting process. By recording the fluctuations with very sensitive and accurate equipment, the noise signal is transformed from the time domain into the frequency domain applying a fast Fourier transform spectrum analyser and plotted in the form of amplitude versus frequency.\textsuperscript{15} Other studies attempted to calculate the $R_p$ value from the recording of the potential noise divided by the current noise. The relationship of electrochemical noise to the $I_{corr}$ values for the steel–concrete system is still under discussion, and its interpretation in real concrete structures is far from easy.
Electrochemical impedance spectroscopy (EIS)

The EIS method is based on the application of an alternating signal instead of a direct signal to the rebar acting as a working electrode. The instrumentation needed is much more sophisticated, because, in addition to a potentiostat for recording and maintaining the $E_{\text{corr}}$, a spectrum frequency analyser able to apply and analyse the electrical ac signal is needed.

The technique applies a small amplitude (10–20 mV peak-to-peak) sinusoidal voltage in an extended frequency range. The response at every frequency is another sinusoidal signal with different amplitude (measured as $\Delta I$) and a phase shift relative to the input signal. The ratio $\Delta E/\Delta I$ is now an impedance $Z$, also sinusoidal, which can be composed into a resistive term in phase with the input signal, and the capacitive term with a phase shift of 90°.

This technique in principle is very interesting because, in addition to the fact to provide $R_p$ values related to the corrosion rate by applying Stern formula, it may give complementary information on the corrosion process, the dielectric properties of the concrete or the characteristics of the passivating film, depending on the range of frequencies to be evaluated. The main disadvantage, apart from the more expensive and sophisticated equipments needed, is that the measurement time to obtain the $R_p$ value is much longer than in the case of dc techniques. This is because of the necessity to work in the low frequency range of about $10^{-3}$ Hz.

14.3.3 Measurement of $I_{\text{corr}}$ on-site in concrete structures: actual procedures

Not all of the methods used in the laboratory can be used on-site. This is because the counter electrode on-site cannot be the same size as the reinforcement. Because the counter electrode is much smaller the current applied is not homogeneously distributed in the polarized area. For this reason, it is necessary to use well-defined methodologies not affected by the rebar size, which allow the calculation of the actual metallic surface being effectively polarized during the measurement. The metallic surface polarized is named the ‘critical length’ and it allows the calculation of the metal area affected by the polarization.

Critical length reached by the signal in large structures

As already mentioned, in small specimens the uniform distribution of the current applied between auxiliary and working electrodes is usually guaranteed. However, in large structures, the auxiliary electrode is much smaller than the working electrode (the rebar). This situation gives rise to an essen-
tially non-uniform distribution of the applied current along the reinforcement as shown in Fig. 14.7. The electrical signal tends to vanish with the distance from the counter electrode CE. The required uniform distribution is, therefore, not met and the $\Delta E/\Delta I$ slope cannot be referred to any specific rebar surface. Consequently, either the so-called critical length $L_{\text{crit}}$ (Fig. 14.7) reached by the electrical field has to be calculated or the current has to be confined within a well-defined delimitated area.

The critical length $L_{\text{crit}}^{46,47}$ is the distance reached by approximately 90% of the current applied by means of a small auxiliary electrode placed on the surface of the concrete as Fig. 14.7 depicts. It is the distance reached by the current relative to the border of the external auxiliary electrode. This critical length is a function of the square root of the ratio $R_p/R_e^{37}$ and independent of the size of the counter electrode; it is not a fixed length and it can be calculated by means of the transmission line model$^{46}$ (Fig. 14.8) and not from a Randles circuit. The calculation or measurement of this $L_{\text{crit}}$ enables the $R_{\text{p, true}}$ to be obtained, because it allows the calculation of the polarized area and therefore, the $R_{\text{p, true}}$ can be accurately expressed in $\Omega \text{ cm}^2$.

14.7 Non-uniform distribution of the current. The length of the rebar polarized to a significant level by the externally applied current is termed the ‘critical length’ $L_{\text{crit}}$.

14.8 Transmission line model (electrical analogue) representing the lateral distribution of the current along the reinforcement bar.
If the ratio $\Delta E/\Delta I$ is obtained without referring to $L_{\text{crit}}$ but, for instance, is calculated only from the CE area, an $R_{p,\text{app}}$ is obtained,\textsuperscript{46,47} which results in serious errors if used for quantitative calculations, because large differences between $R_{p,\text{true}}$ and this $R_{p,\text{app}}$ may exist. Thus, in Fig. 14.9, the error factor ($R_{p}/R_{p,\text{app}}$) is shown. The values have been confirmed in experiments made in large slabs. For non-corroding rebars, the error factor easily exceeds a value of 100, whereas for corroding rebars, it may be about 10 or smaller. The ratio $R_{p}/R_{p,\text{app}}$ depends on the value of the particular $R_{p}$ itself and of the concrete resistivity (level of concrete moisture content).

A general or fixed value of this error factor cannot be given because of the variability of moisture conditions and the degree of chloride contamination of the concrete which affect both $R_{s}$ and $R_{p}$. Therefore, it does not produce a single value for the error factor relating $R_{p,\text{true}}$ and $R_{p,\text{app}}$ values.

Another aspect to be taken into account is that the current is drained by the corroding spots as Figure 14.10 shows. For on-site measurements, the locally corroding spots change the current lines from the central CE resulting in a non-uniform distribution as Fig. 14.10 depicts. This effect invalidates not only the calculation of $I_{\text{corr}}$ but also the assessment of the location of the pits.

One of the solutions for the problem of the current dispersion along the bar is the confinement of it by a guard ring.\textsuperscript{48} The non-confining techniques are not able to correctly localize the isolated corroding zones, because they draw the applied current tens of centimetres away from the CE. On the other hand, the technique using a modulated guard ring allows correct identification of the localized corrosion spots as shown in the following subsection.

![Graph showing the relationship between $R_{p,\text{true}}/R_{p,\text{app}}$ and the area of the counter electrode](image-url)
Modulated or sensorized confinement of the polarizing current

The modulated or sensorized confinement method is based on the calculation of the $R_{p, \text{true}}$ using the effective confinement of the polarizing current within a specific area through a second circular counter electrode. This arrangement enables the current applied through the central CE to be confined to a defined area under the central CE and between it and the external guard ring (GR) (Fig. 14.11). If the counter-field from the GR is not correctly controlled and adjusted, the area being polarized may greatly...
differ (being smaller or larger) from the predetermined one. This control is achieved by means of the two additional reference electrodes, \( S_1 \) and \( S_2 \), located between CE and GR. These twin electrodes permanently control the external ring by means of detecting the current lines coming from the CE in order to adjust them within the predetermined area of diameter \( D \). The method then promotes an electrical delimitation of the area instead of determining it.

The electrode arrangement of the modulated or sensorized confinement method comprises:

- a small central counter CE,
- a ring-shaped counter electrode, GR, which surrounds the central one,
- a main reference electrode, RE, placed in the centre of the central CE, and
- two auxiliary reference electrodes, \( S_1 \) and \( S_2 \), placed between the central CE and the GR.

**Multiple electrode or potential attenuation method**

This method\(^{49}\) is based on the calculation of the critical length \( L_{\text{crit}} \) and allows calculation of the steel area actually polarized by the applied current by means of the transmission line model. The method makes use of a small counter electrode CE which is placed on the concrete surface, as Fig. 14.12 depicts. In the centre of this CE electrode, the main reference electrode RE is placed to measure the local corrosion potential \( E_{\text{corr}} \) of the reinforcing steel. Other auxiliary reference electrodes (R1, R2, and R3) are located aligned with the CE and the main RE as shown in Fig. 14.12.

When the current is applied through the CE, the rebar is polarized for a certain distance \( L_{\text{crit}} \). The auxiliary reference electrodes R1, R2 and

---

14.12 Attenuation of the current with the distance from the counter electrode (only one side of the electrical field is illustrated).
R3 serve to measure this $L_{\text{crit}}$ by recording the electrical potential at various distances from CE (potential attenuation) before and after the application of the current. The position where this potential difference is smaller than approximately 10% of the potential difference calculated from RE indicates $L_{\text{crit}}$ and, therefore, allows the geometrical rebar area actually polarized by the signal to be calculated. As soon as the polarized area is known and the geometrical characteristics of the concrete element are determined, the corrosion current $I_{\text{corr}}$ can be calculated. This method is the only one able to correctly determine $I_{\text{corr}}$ in submerged structures. The modulated guard ring method cannot be applied in these conditions.

14.3.4 Devices for on-site determinations

The equipment to be used with any of the methods previously described are based on a potentiostat or galvanostat as a means of controlling and measuring potential and current. The potentiostats to be used for on-site measurements have to be able to measure the ohmic drop (electrical resistance), $R_{\text{e}}$, or to compensate for its influence during the recording of the $R_p$ measurement. They need also to be portable and adapted to the measurement using a much smaller counter electrode size than the size of the reinforcement.

The potential measuring circuit of the instrument should be able to maintain an electrode potential within 1 mV over a wide range of currents and should have high input impedance, higher than 10 MΩ, in order to minimize current drawn from the system during measurements.

The current circuit should have sensitivity such that the $I_{\text{corr}}$ could be measured at least of the order of 0.05 μA cm$^{-2}$. Furthermore, the instrument should have a sensitivity to detect variations of 0.5 mV in a potential range from −1.0 to +1.0 V. The current circuit has to have a sensitivity of at least 0.05 μA in a range of 0.05 to 10$^4$ μA.

The apparatus used has to be equipped with an auxiliary sensor (electrode), containing all the counter and references electrodes. Although the counter electrodes can be of any metallic material able to produce the current, stainless steel is the most cost-effective material enabling an easy maintainability.

The reference electrodes can also be any of the traditional ones: calomel, silver/silver chloride or copper/copper sulfate. This last one has been demonstrated to be one of the most suitable because of its measurement range, accuracy and precision, although it has to be maintained and its junction membrane cleaned periodically.

As an example, Fig. 14.13 shows the commercial device Gecor 08 used for on-site corrosion evaluation.
14.4 How to interpret the measurements

14.4.1 $E_{\text{corr}}$ and electrical resistance

The interpretation of the potential readings has been controversial as an absolute and universal ranking seems impossible to achieve. In Table 14.1 a ranking is presented. According to ASTM C786–87 standard, a threshold potential value of $-350$ mV vs. CSE electrode was established. Lower values of potential suggested corrosion with 95% probability; if potentials are more positive than $-200$ mV CSE, there is a greater than 90% probability that no corrosion of the reinforcement steel occurs, and for potentials between $-200$ mV and $-350$ mV corrosion activity is uncertain. However, practical experience has shown that different potential values indicate corrosion for different conditions, so absolute values can not be taken into account to indicate corrosion risk, i.e., the relationship between concrete condition and potential values is not well-defined enough, with the exception of those potentials at the extreme ends.

Regarding electrical resistance, the ranking is less controversial although sometimes the values applied for soils are given, and these cannot be the same as for concrete. The ranking of concrete is shown in Table 14.2. This ranking is more coherent than that for the potential which is based on the general relationship between $I_{\text{corr}}$ and resistivity that will be discussed later.

14.4.2 $I_{\text{corr}}$

The experience on real structures has confirmed the ranges of values previously recorded in laboratory experiments. In general, as shown in Table 14.3, values of corrosion rates higher than 10 $\mu$A cm$^{-2}$ are seldom measured whereas values of 0.01–1 $\mu$A cm$^{-2}$ are the most frequent. When
the steel is passive, very low values (smaller than 0.05–0.1 \( \mu \text{A cm}^{-2} \)) are recorded. These current density values can be translated into ‘loss in diameter’ applying the Faraday law, resulting in a corrosion rate of 1 \( \mu \text{A cm}^{-2} \) produce a section reduction of 10 \( \mu \text{m} \) per year. When the corrosion rate is presented as \( \mu \text{m/year} \), it is named \( V_{\text{corr}} \) instead of \( I_{\text{corr}} \).

The accumulated corrosion is termed ‘corrosion penetration’ \( P_x \) that can be expressed as ‘loss in diameter’ \( \phi = (\phi_0 - 2P_x) \) with \( \phi_0 \) being the diameter after time \( t \) and \( \phi_0 \) = the initial one. This reduced diameter can be caused by localized (\( P_{\text{pit}} \)) or homogeneous corrosion, as Fig. 14.4 shows. It has been established that \( P_{\text{pit}} \) can be calculated from \( P_x^{30} \) as there is a relationship between the homogeneous corrosion penetration and the maximum depth of a pit. This relationship is named ‘pit factor \( \alpha \)’ and is usually between 3 and 15, whereas a factor of 10 is a suggested average value.

As the corrosion rate does not have a constant value over time, either at constant humidity and temperature, and much less when the structure is exposed to the open atmosphere. There is a need to calculate what has been termed a ‘representative corrosion rate, \( V_{\text{corr,rep}} \)’. There are several ways to obtain this averaged value:

1. The value of \( V_{\text{corr,rep}} \) can be the result of the integration of the instantaneous \( I_{\text{corr}} \)-time curves presented in Fig. 14.3 over time and dividing the thus obtained integral value by the years of corrosion propagation.
2. When no corrosion sensors have been installed and direct readings of the corrosion rate are not available for the particular structure, another approach to estimate an annual averaged value for \( V_{\text{corr,rep}} \) is to ascribe a value as a function of the aggressivity of the particular environment. In Table 14.4 averaged \( V_{\text{corr,rep}} \) values are given for the exposure classes of EN 206.
3. When isolated measurements are the only possibility, obtaining representative \( I_{\text{corr}} \) is more uncertain. In order to interpret the readings in the most accurate way, the procedure recommended, is based on the relation between resistivity and \( I_{\text{corr}} \). Figure 14.14 indicates that in the same structure the usual relation between \( I_{\text{corr}} \) and \( \rho \), when plotted in a log–log diagram, is linear with a slope of \(-1 \) \((I_{\text{corr}} - 3 \times 10^4/\rho)\). In consequence, the procedure proposed is the following:
   - After having measured the corrosion current, cores should be taken close to the measurement points. Cores are returned to the laboratory, where they are conditioned in an atmosphere of 85% RH (for structures sheltered from rain) or vacuum water saturated, (for non-sheltered or submerged ones) when the cores are equilibrated their minimum electrical resistivity \( \rho_{\text{min}} \) is measured.
   - Finally, a plot of \( I_{\text{corr}} - \rho \) (point A) is obtained (Fig. 14.14). The extrapolation to the maximum \( I_{\text{corr}} \) (point C) is made to reach the \( \rho_{\text{min}} \) (point B) at 85% RH or core saturation.
Table 14.4 Averaged corrosion rates $V_{\text{corr, rep}}$ for the exposure classes of EN 206

<table>
<thead>
<tr>
<th>Exposure class</th>
<th>$V_{\text{corr, rep}}$ (μm year$^{-1}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>No risk of corrosion, very dry</td>
</tr>
<tr>
<td>XC1</td>
<td>Corrosion due to carbonation. Dry or permanent wet</td>
</tr>
<tr>
<td>XC2</td>
<td>Corrosion due to carbonation. Wet, rarely dry</td>
</tr>
<tr>
<td>XC3</td>
<td>Corrosion due to carbonation. Moderate humidity</td>
</tr>
<tr>
<td>XC4</td>
<td>Corrosion due to carbonation. Cyclic, wet and dry</td>
</tr>
<tr>
<td>XD1</td>
<td>Corrosion due to chlorides contamination. Moderate humidity</td>
</tr>
<tr>
<td>XD2</td>
<td>Corrosion due to chlorides contamination. Wet, rarely dry</td>
</tr>
<tr>
<td>XD3</td>
<td>Corrosion due to chlorides contamination. Cyclic, wet and dry</td>
</tr>
<tr>
<td>XS1</td>
<td>Corrosion due to chlorides contamination. Airborne salt conditions</td>
</tr>
<tr>
<td>XS2</td>
<td>Corrosion due to chlorides contamination. Submerged</td>
</tr>
<tr>
<td>XS3</td>
<td>Corrosion due to chlorides contamination. Tidal, splash and spray zones</td>
</tr>
</tbody>
</table>

14.14 $I_{\text{corr}}$ versus $\rho$ (VH, very high; H, high; M, moderate; L, low; A, measurement points; B, extrapolation to minimum $\rho$; C, maximum expected $I_{\text{corr}}$).

The $I_{\text{corr, rep}}$ is calculated by averaging both values, the single value $I_{\text{corr, single}}$ and the maximum value achieved at laboratory $I_{\text{corr, max}}$:

$$I_{\text{corr, rep}} = \frac{I_{\text{corr, single}} + I_{\text{corr, max}}}{2}$$ [14.15]
14.5 Practical application

14.5.1 $E_{corr}$ measurements

To measure the half-cell potentials on a structure, a good electrical connection to the reinforcement has to be made. This can be achieved by means of a compression-type ground clamp or by brazing or welding a protruding rod, but a direct contact should not be made if reinforcement steel is connected to an exposed steel member. The other input of the high impedance voltmeter must be the external reference electrode placed on the concrete surface by a wet sponge in order to provide a good electrolytic contact between them. The sponge should be always wetted with a diluted solution of detergent.

It is essential to assure the electrical continuity of the reinforcement steel, and this can be done by measuring the resistance between separated areas. If resistance values are $\leq 0.3 \, \Omega$, electrical continuity is indicated.

As already mentioned, the objective of the measurement of potential is to determine areas of corroding reinforcement. To achieve this goal, first it is essential to define a work strategy that provides a fast and economical overview of the state of the structure. This strategy must involve the definition of a co-ordinate system to correlate readings and measuring points. A grid is usually applied with a cell size that varies from 15 cm$^2$ to 2 m$^2$, depending on the type of the structure, its characteristics and the scope of the work. The size of this co-ordinate system determines the accuracy of the measurements.

To determine the optimum grid spacing, it is necessary to determine the maximum distance from a corroding rebar at which there is no evidence of potential change. Measurements made with a big grid cell size could not detect corrosion activity whereas minimum spacing generally should provide high differences between readings. The spacing must be adequate to the type of structure surveyed and the expected use of measurements.

Potential measurements can be performed with a single electrode or with one or several wheel electrodes, which facilitate the potential survey of large bridge decks and parking decks (up to 300 m$^2$ per hour) when they are connected to microprocessor controlled data-loggers.

Once the data are obtained, the best method of representation depends on the amount of data and the type of structure. Therefore, tables may be used, or coloured grid maps of the potential field, where every individual potential reading can be identified as a small cell and a contour line map can be obtained interpolating between point measurements with different algorithms.

In a grid map, a coloured cell associated with a potential value represents each measurement point. The colour gradation step should not be greater
Techniques for measuring corrosion rate and potential

than 50 mV in order to provide the clearest interpretation of the result. A 3D surface can also be represented both by measured and interpolated values.

In addition, potential measurements can be represented by all the standard statistical methods such as cumulative frequency distribution or histograms. The type and depth of the study to be developed should impose the design requirements for these graphics.

14.5.2 $I_{corr}$ measurement

**Before starting the survey**

Corrosion rate equipment, similar to other electronic devices, does not work in extreme conditions of temperature or humidity. The devices should not be at operated temperatures below 2 °C or above 50 °C. If the environment exceeds these limits, then the system has to be operated in an air-conditioned enclosure or vehicle.

It should be noted that below freezing, the wetted sponge may freeze resulting in misleading or unstable readings. If it is essential to collect data under these conditions then an alcohol solution (10–30% of alcohol by weight) will reduce the freezing point. It should also be noted that below 5 °C the corrosion rate may reduce to negligible values which may mislead the interpretation ($Re$ may be very high or $I_{corr}$ very low).

Complete water saturation of concrete may also cause measurement difficulties owing to the high conductivity path that may be established through the concrete surface. This is particularly clear when deicing salts have been used. The extremely conductive concrete surface may facilitate the dispersion of the current lines to very long distances making it difficult to obtain a reliable or reproducible $R_p$ value. Therefore, measurements in submerged structures may give very unreliable results, unless the potential attenuation described previously is used.

Before starting the survey, it is necessary to select the number and location of points where corrosion rates will be measured. The number of points will depend upon: (a) the time available, access and size of structure, and (b) the aim of the inspection.

Measurements may be taken at strategic locations chosen because they represent one or more of the following:

- High or low readings from half cell potentials or resistivity.
- Locations of structural importance (different elements, construction joints, sources of water or chloride, the ground, water level).

With regard to the duration of each corrosion rate measurement, each reading may take from less than 1 min to about 5 min depending upon the actual corrosion conditions and the method of measurement. The physical
processing of placing the sensor may also take a time of 2–5 min. Therefore, the operator must allow 5–15 min per location to obtain a corrosion rate measurement.

Location of the bars

The actual geometry of the rebar arrangement is determined using a steel detector. If needed the bar pattern can be marked on the concrete surface, as well as the cover depth registered. The bar diameter and their distances are needed for the calculation of the steel area to be polarized during the measurements.

It is recommended that readings are taken over a rebar so the grid size is partly dependent on the rebar spacing. A grid spacing of 0.25 m is recommended, except on small structures or elements with severe changes in exposure conditions. Measurements may be also taken on a simple straight line, if the corrosion condition is likely to vary with distance along an element.

Preparation of the concrete surface

The surface has to be well wetted before applying the auxiliary sensor. Care has to be taken to avoid contamination of the reference electrodes with alkaline substances from the concrete. This is achieved by simply placing a clean wetted sponge between the sensor and the concrete surface. On coated or hydrophobically treated surfaces, trials have to be made in order to verify the feasibility of the electrolytic (ionic) connection. Hydrophobic treatments may not result in correct contact. There must be complete electrolytic contact between the sensor and the concrete surface. Any local deformation or insulating layer must be avoided or removed by grinding or choosing another location. Small deformations in the surface can be, if needed, ‘ironed out’ by using additional sponges.

When there is excessive superficial chloride contamination or very conductive concrete surface layer, a correct measurement may be difficult but it can be checked by measuring concrete resistivity. If the values obtained are below 1 kΩ cm an unacceptably high conductive concrete surface can be expected. It is then recommended to clean the surface to remove the salts, and to wash the contact sponge thoroughly with uncontaminated water. In very wet concrete, it may happen that measurements are not valid owing to very high corrosion rates or improper functioning. In these instances, the method for measuring submerged structures, potential attenuation method, may be the only feasible method.

Measurements can be performed in cracked concrete. However, locations with major voids, delaminations or large cracks (> 1 mm) within the
concrete must be avoided, particularly if the concrete is wet, because they may cause the signal to deviate from the required electrolytic path, resulting in erroneous readings.

**Placement of the auxiliary probe**

The auxiliary sensor must be located preferentially directly over a rebar of known diameter, either a single bar or at a crossover. Metallic (electronic) short circuits between this sensor and the bar caused by tie wire or nails must be avoided as these will invalidate the reading. The sponge beneath the auxiliary sensor has to be well wetted in order to enable an adequate correct electrolytic contact with the pre-wetted concrete surface. If the surface is vertical or horizontal overhead, the auxiliary sensor has to be secured with appropriate fixing tools (plastic straps, screws or some kind of pressing devices). The area below the counter sensor or that which will be polarized during the measurement has then to be calculated, and it must also be taken into account whether two or more bars intersect in that area.

**Connection between equipment and reinforcement**

In order to complete the circuit, an electrical connection has to be made between the equipment and the reinforcement. A window to the rebar must be opened by coring, excavation or a connection through an exposed steel connected to the reinforcement and through a cable attached to the rebar. The rebar must be cleaned (for instance by brushing with a metallic brush or by means of a screw) to ensure a good electrical contact.

In order to avoid opening several holes to have electrical contact with the reinforcement network, it is necessary to check the electrical continuity between rebars. If the reinforcement is electrically continuous then this connection can remain in place for testing at other points on the same span or concrete member. Otherwise, the connection must be moved from bar to bar for each additional test point.

It is essential that there is good electrical continuity between the rebar connection and the steel being measured. Reasons for discontinuity include construction joints with separated rebar cages, excessive corrosion and light reinforcement content. If discontinuities are found then it may be necessary to make regular rebar connections rather than just one or two.

**Accurate measurement**

A correct connection with the reinforcement may be verified by checking the stability of the corrosion potential $E_{corr}$, which is necessary to ensure an accurate measurement of the voltage shift after (applying) the
current. The stability is insufficient if the reading fluctuates more than 0.5 mV every 5 s.

After switching on the device, the measurement has to be carried out until completion of the testing time. If repetition of the measurement is necessary, the time interval between consecutive measurements has to be long enough to allow the corrosion potential to recover its stability.

14.6 Monitoring systems

The introduction of small sensors in the interior or on the surface of the concrete, usually when placing it on-site is one of the most promising developments for monitoring the long-term behaviour of concrete structures. Sensors to be embedded in concrete have to be robust enough to support concrete fabrication and to resist concrete alkalinity in the long term.

For monitoring corrosion by electrochemical techniques, the most usual method, for non-permanent on-site techniques, is to embed reference electrodes or resistivity electrodes, but the corrosion rate can also be monitored using the techniques described previously, in particular, the potential attenuation method.

Depending on the state of the structure to be monitored (existing structures or structures under construction), the installation of surface sensors is more suitable for existing structures. Embedded sensors are appropriate for structures under construction in which the implementation of the sensors is possible before casting. The type of embedded and surface sensors used can be observed in Fig. 14.15a and Fig. 14.15b, respectively.

It is necessary to study the long-term behaviour of the reference electrodes used in concrete. For this reason, Ti, MnO₂, Ag and Pb reference electrodes have already been tested. The MnO₂ electrodes show better durability and stability. A stainless steel disc is usually used as counter electrode.

For monitoring, the system used is a galvanostat–potentiostat able to measure the electrochemical parameters and also able to register deformation and temperature results. It is appropriate to have a high number of available channels (50–100) and to have a pre-programmed activated alarm system when the values exceed a predefined range. The equipment is also a data-logger able to store all the information and to process the results obtained.

Figure 14.16 shows an example of $E_{corr}$ and $I_{corr}$ monitoring in a real concrete structure. As can be observed, the annual cycles owing to the environment influence are clearly detected in all the parameters monitored.
14.7 Future trends: new techniques

Although methods based on the $R_p$ measurement for the $I_{corr}$ calculation in concrete structures have been widely studied in the last decades, there are some aspects that have not yet been resolved, particularly to the need to remove the concrete cover to make electrical contact with the rebar, or to develop a technique compatible with the wireless technique for the monitoring on site (reducing the energy consumption)
To polarize the metal acting as the working electrode and to measure its response, it is necessary to establish a physical contact between all the electrodes involved and the potentiostat through appropriate wires. A new measurement method in which direct physical contact between the potentiostat and the metal is not necessary to measure equivalence to the $R_p$ value has been developed recently. The method is based on the observation that an electrode can be polarized by placing it under the influence of an external electrical field generated by applying a current between two external electrodes.

The model used for calculation of the corrosion rate assumes that the applied current runs in parallel field lines through the electrolyte and the metal, and electrostatically polarizes the metal. The overall electrical resistance of the system can then be expressed as

$$\frac{1}{R_{e+M}} = \frac{1}{R_e} + \frac{1}{R_M}$$

where $R_{e+M}$ is the total resistance measured, $R_e$ is electrolyte resistance and $R_M$ is resistance owing the metal.

The feasibility of measuring the corrosion rate without physical contact to the working electrode opens up new application possibilities in concrete structures, as the access to the working electrode is not available and a real NDT is needed.

14.8 Conclusions

- The use of NDT for corrosion evaluation has been developed during the past two decades. Currently, the use of specific developments for
application of the on-site polarization technique are well defined and its use has become necessary for structure evaluation.

- Even when parameters such as corrosion potential or concrete resistivity are useful for the determination of the corrosion state of the structure, the corrosion process can only be quantified by the corrosion rate measurement $I_{corr}$.
- Owing to the variation that $I_{corr}$ presents in real structures that are exposed to the environment, it is necessary to establish a methodology to determine the representative value of the corrosion rate obtained in the structure.
- The introduction of small sensors in the interior or at the surface of the concrete can be considered as one of the most promising developments in order to monitor the long-term behaviour of concrete structures.
- New possibilities for the on-site determination of corrosion rate to allow faster measurement and avoid contact with the rebar are being studied. There is a new technique under development so that calculation of the polarization resistance $R_p$, through the polarization induced by an external field should be feasible without electrically touching the metal. This technique has been named the non-contacting (NC) corrosion method.

14.9 References


Techniques for measuring corrosion rate and potential


© Woodhead Publishing Limited, 2010
316 Non-destructive evaluation of reinforced concrete structures


© Woodhead Publishing Limited, 2010
Abstract: The main advantages of ground penetrating radar, an electromagnetic inspection method used in various fields such as glaciology and non-destructive testing, are that it is fast (hundreds of measurements per second) and that no contact with the inspected object is required. A wide range of radar equipment including software for data processing and interpretation is available on the market.

Key words: reinforced concrete, non-destructive testing (NDT), ground penetrating radar (GPR).

15.1 Introduction to ground penetrating radar (GPR)

Ground penetrating radar (GPR) is an electromagnetic investigation method. It is also known as surface penetrating radar, electromagnetic reflection method or radar. The main advantages of the method are that it is:

- non-destructive,
- fast (hundreds of measurements per second), and
- can be used in non-contact mode.

In principle, GPR can be used in reflection or transmission mode. As reflection methods are by far the most common, the following description will focus on this mode. The alternative approach of transmission/tomographic GPR is described in chapter 16. The basic principle of GPR in reflection mode is presented in Fig. 15.1, where an electromagnetic pulse is emitted via a transmitter antenna, reflected at the surface and interior layer boundaries of an object and recorded via the receiver antenna. The result of a single GPR measurement is a time series containing, amongst others, amplitude and two-way-traveltime (the time required to propagate to a reflector and back) of the reflected energy.

The radar method can be applied to a wide range of problems such as:

- layer thicknesses, including concrete cover of rebar, asphalt pavement, concrete tunnel walls, and sub-base and geological layers;
• locating of structures such as tendons or tendon ducts, anchors, dowels, and cavities;
• material properties, including humidity, chloride content, voids, and air content.

In general it can be stated that the investigation of material properties is more demanding than the investigation of structural elements and, in many instances, it is still under development.

A detailed description of the GPR method is given by Daniels (2004), the application of GPR for testing problems in civil engineering is described by DGZfP (2008) and the testing of concrete structures is outlined in the technical report No. 48 of the Concrete Society.

15.2 Physical principles and theory

Radar (radio detection and ranging) and GPR are electromagnetic methods. They are thus governed by the Maxwell equations (Crawford 1968, Maxwell 1865, Gerthsen et al., 1982) which can be written in their differential or integral forms:

\[
\begin{align*}
\text{Differential} & \quad \text{Integral} \\
\text{rot } H &= D^* + j \int_A H \, ds = \int_A D^* \, dA + I \\
\text{rot } E &= -B^* \quad \int_A E \, ds = -\int_A B^* \, dA \\
\text{div } D &= \rho \quad \int_V D \, dA = Q \\
\text{div } B &= 0 \quad \int_V B \, dA = 0
\end{align*}
\]
with volume $V$ and area $A$, displacement current $D$, electric current $I$ and electric current density $j$, electric charge $Q$ and charge density $p$, magnetic field $H$ and electric field $E$, temporal derivatives $\cdot$ as well as $B$ and $D$ which are proportional to $H$ and $E$, respectively.

The meaning of these equations can be summarized as shown below.

1. Electric currents and electric displacement currents cause magnetic fields.
2. Changing magnetic fields cause electric (eddy) fields.
3. Electric charges cause electric fields.
4. There are no magnetic sources (in the sense of magnetic charges).

Several conclusions can be drawn directly from Maxwell’s equations. Based on equations [15.3] and [15.4] it can be concluded that electromagnetic waves are transverse ($B$ and $D$ are orthogonal to the direction of propagation) because otherwise there would be temporal sources and sinks (Fig. 15.2) which is generally not consistent with equation [15.4] and in vacuum not with equation [15.3]. Another conclusion is that $E$ and $H$ are orthogonal because of equation [15.1]. The integral of $D\cdot$ over an area $A$ (Fig. 15.3) reaches its maximum value if $A$ is orthogonal to $D\cdot$. According to equation [15.1], the integral of $H$ around the edges of area $A$ will also reach its maximum value which implies that $H$ is parallel to $A$ and hence orthogonal to $D\cdot$. Another important conclusion is that electromagnetic waves are travelling in vacuum with the speed of light. From:

\[ D = \varepsilon \varepsilon_0 E \]  \hspace{1cm} [15.5]

equation [15.1] can be written as:

\[ \oint_A H \, ds = \int_A \varepsilon \varepsilon_0 E^* \, dA \]  \hspace{1cm} [15.6]

because $I = 0$ in a vacuum. Assuming a plane harmonic wave:

\[ E = E_0 \sin \omega (t - x/v) \]  \hspace{1cm} [15.7]

where $x =$ direction of propagation, $v =$ velocity of propagation and $\omega =$ angular frequency; equation [15.7] can be written as:

\[ \oint_A H \, ds = \varepsilon \varepsilon_0 E_0 \int_A \cos (\omega x/v) \, dA \]  \hspace{1cm} [15.8]

15.2 Sources and sinks in a longitudinal wave.
Using a rectangular area $A$, with $E$ being orthogonal to the surface of $A$ and with side lengths of $b$ orthogonal to $x$ and $\lambda/2$ parallel to $x$ with $\lambda$ = wavelength, then equation [15.8] can be written as:

$$\oint_A H \, ds = b\varepsilon\varepsilon_0 E_0 \int_{\lambda/4}^{3\lambda/4} \cos(\omega x/v) \, dx$$  \hspace{1cm} [15.9]$$

Integration over half a wavelength reaching from maximum to minimum leads to

$$\oint_A H \, ds = b\varepsilon\varepsilon_0 \omega (v/\omega) 2E_0$$  \hspace{1cm} [15.10]$$

Because $H$ and $E$ are in phase, the left side of equation [15.10] can be written as:

$$\oint_A H \, ds = 2bH_0$$  \hspace{1cm} [15.11]$$

A comparison of equations [15.10] and [15.11] leads to:

$$H_0 = \varepsilon\varepsilon_0 v E_0$$  \hspace{1cm} [15.12]$$

Starting with equation [15.2] and proceeding analogously to equations [15.5–15.12], it can be shown that:

$$E_0 = \mu\mu_0 v H_0$$  \hspace{1cm} [15.13]$$

and substituting [15.13] into [15.12] leads to:

$$v = 1/\sqrt{\varepsilon\varepsilon_0 \mu\mu_0}$$  \hspace{1cm} [15.14]$$
In a vacuum ($\mu = \varepsilon = 1$), equation [15.14] can be written as:

$$v = 1/\sqrt{\varepsilon_0 \mu_0} = 3 \times 10^8 \text{ m s}^{-1} = \text{speed of light} = c$$  \[15.15\]

For materials where the radar method is applicable, it can be assumed that $\mu \approx 1$ (in ferromagnetic materials this is not valid but ferromagnetic materials cannot be investigated with the radar method) and hence:

$$v = c/\sqrt{\varepsilon}$$  \[15.16\]

This means that the signal velocity within materials is mainly defined by their relative permittivity. For practical purposes, it can be assumed that dielectric constants of materials are in the range between 1 (air) and 84 (water) which leads to signal velocities between $3 \times 10^8 \text{ m s}^{-1}$ (air) and $0.33 \times 10^8 \text{ m s}^{-1}$. The high velocity of the radar signal is responsible for one of the main advantages of the radar method. As the signals travels with such a high velocity, a single measurement takes very little time and therefore the number of measurements per second is, from a physical point of view, almost unlimited.

If an electromagnetic wave hits an interface, part of the energy will be transmitted and part will be reflected. For a plain electromagnetic wave in a low loss material hitting at vertical incidence an interface between two materials with $\varepsilon_1$ and $\varepsilon_2$, the reflected wave can be described as:

$$\text{reflected wave} = R \times \text{incident wave}$$  \[15.17\]

where the reflection coefficient $R$ is:

$$R = (\sqrt{\varepsilon_1} - \sqrt{\varepsilon_2})/(\sqrt{\varepsilon_1} + \sqrt{\varepsilon_2})$$  \[15.18\]

which means that there is no reflection if $\varepsilon_1 = \varepsilon_2$ (materials with identical relative permittivities) and the reflection amplitude becomes negative (phase shift by 180 degrees) if $\varepsilon_2 > \varepsilon_1$.

The time a radar signal requires to travel through a material layer, be reflected, and travel back through the layer is described by the two-way-traveltime ($T_{\text{wt}}$)

$$T_{\text{wt}} = 2d/v$$  \[15.19\]

where $d$ is the thickness of the layer. If this thickness has to be computed from $T_{\text{wt}}$ this equation can be rearranged to:

$$d = T_{\text{wt}} \times v/2$$  \[15.20\]

This corresponds to the usual case where the thickness is computed with a known $T_{\text{wt}}$ derived from the radar data. The velocity $v$, necessary to compute depths or thicknesses from $T_{\text{wt}}$ is a priori unknown. It can be estimated using experience or velocity tables, calibrated by comparison of $T_{\text{wt}}$ with known thicknesses or by special setups during radar data
acquisition. Equation [15.20] is valid only if the distance between transmitter and receiver is zero or small enough to be neglected.

The ability of the method to separate single objects (lateral resolution) is limited by the size of the Fresnel region which characterizes the area where reflected waves add together constructively. Single points within this area can not be distinguished by the radar signal. The radius of the Fresnel region is described as:

\[ r = \sqrt{\frac{d\nu}{2f}} \]  

[15.21]

A larger radius of the Fresnel region corresponds to a decrease in resolution. Thus, lateral resolution increases with frequency and decreases with the depth (distance) \( D \) of the target.

### 15.3 Display formats for ground penetrating radar (GPR) data

GPR data can be acquired and presented in different modes. A measurement in a single position will produce a time series that can be presented as a curve or colour coded (Fig. 15.4). The vertical axis is a time axis and it is usually pointing downwards because larger times correspond to larger depths.

If the antenna is moved along a horizontal line, GPR measurements can be recorded at close intervals (Fig. 15.5). The recorded time series can be plotted side by side with the time axis pointing downwards. The horizontal axis is then corresponding to the line along which the antenna was moved. This display format is called a radargram.

If data acquisition is carried out over an area, for example by measuring along many parallel lines, a data cube can be built and then slices corre-
15.4 Data processing and interpretation

Data processing is an important step in GPR inspections and is carried out for several reasons such as an improvement of the signal/noise ratio or for the correction of unwanted physical and/or geometrical effects. The processing of GPR data has many similarities to the processing of seismic data (Yilmaz, 1987). A detailed description of data processing is given in chapter 7.

An example of a simple processing sequence applied to data (a radargram) acquired on an industrial railway track embedded in concrete is
presented in Fig. 15.7 (raw data) and Fig. 15.8 (processed data). The horizontal length of the section shown is 3.0 m, the vertical time scale is 12 ns. The processing sequence included a bandpass filter (improvement of signal/noise ratio), the correction of the reflection at the concrete surface (black arrow in Fig. 15.7) to time 0, migration (focusing of energy that has not been reflected vertically but sideways) and gain correction (amplification with respect to $T_{wt}$).

In Fig. 15.9, the original time scale has been replaced by a depth scale. In order to do this the signal velocity within the material has to be calibrated with a core, obtained from the radar data or estimated. In this example an estimated signal velocity within concrete of 0.08 m ns$^{-1}$ was used for time–depth conversion.

Interpretation is the final step of a radar inspection during which reflections are related to physical structures within the object. The reflection
marked with arrows pointing upwards in Fig. 15.9 have been interpreted as single bars of a layer of rebar, those marked with arrows pointing downwards as sleepers embedded in concrete.

### 15.5 Equipment

Today a wide range of GPR equipment is available from various manufacturers. A radar system consists of one or several antennae, a central unit usually including a monitor for real time data display and accessories such as cables and energy supply.

#### 15.5.1 Antennae

The frequency content of the emitted and recorded radar signal is mainly defined by the antenna. As a general rule of thumb, the higher the centre frequency of the antenna, the better the resolution, but the lower the depth of penetration of the radar signal and, thus, the possible depth of investigation. This means that the choice of the appropriate antenna(s) is crucial for the success of radar investigations. Currently, antennae with centre frequencies between some MHz (low resolution, high depth of penetration, for geological applications) and some GHz (high resolution, low depth of penetration, used for non-destructive testing) are available. Antennae can be monostatic (transmitter and receiver at fixed distance usually in the same box) or bistatic (transmitter and receiver as separate units). Depending on the antenna type and characteristics, antennae are coupled to the object or used in non-contact mode (e.g. horn antennae).
15.5.2 Central unit

Central units have one or several channels for using one or several antennae at the same time. The possible data acquisition rate (number of measurements per second) depends mainly the central unit and can reach up to several hundreds of measurements per second. Data storage and real time display are other tasks performed by the central unit. Often data can be processed in real time for an enhanced display of the data.

15.5.3 Accessories

As GPR is a fast and non-contact method, it can be used for the inspection of large structures such as roads or bridges. In this context, knowledge of the position of each measurement is essential. Modern surveying equipment such as GPS or automated theodolites can provide a means for an efficient position control.

Acquisition of radar data is only the first step of a radar survey, and it is followed by data processing and interpretation. The availability of appropriate software is essential for performing those steps satisfactorily.

15.5.4 Example

In Fig. 15.10, a mobile acquisition system is shown that is used for the inspection of roads and bridges. The two horn antennae (arrow 1, transmitter and receiver) are mounted at a height of about 0.25 m above the ground. This facilitates data acquisition while the vehicle is moving. The GPS antenna (arrow 2) sits on top of the radar antennae for position control. A radio antenna (arrow 3) allows communication of the on-board GPS system with the GPS base-station thus ensuring an accuracy of the position control of some millimetres.

15.6 Limitations and reliability of ground penetrating radar (GPR)

There are several limitations restricting the use of radar and the accuracy of results. Equation [15.18] showed that a sufficient contrast in material properties is required if the interface between two materials is to be investigated. Without such contrast the boundary will not appear in the radar data. If the thickness of a certain layer has to be investigated, the signal velocity within this layer has to be known, equation [15.20]. If the velocity is calibrated with a single core and used for the time to depth conversion, this implies the assumption of constant velocity within the layer. This assumption is used successfully in many cases. However, it should be
remembered that existing velocity variations will lead to errors in the result for the layer thickness.

The attenuation of the radar signal within materials is caused by many factors, the electrical conductivity being an important factor for practical purposes. Radar waves in conducting materials lead to stray currents reducing the depth of penetration of radar signals. Within certain limits this can be avoided using low frequency antennae (in most materials lower frequency waves experience less damping) but this will lead to a reduced resolution, equation [15.21] and may therefore not be feasible.

As described above, interpretation links reflections in radar data to physical structures within the object under inspection. In many instances an unambiguous relation between reflections and physical structures is not straightforward. In such instances, it is a necessity to have additional information such as cores or plans available to support the interpretation, in all other instances the availability of such additional information is also desirable.

15.7 Current and future trends

GPR is a method under continuing development. This is true for many aspects such as electronics and equipment, data processing and inversion methods, new applications and the investigation of material properties or
advanced field techniques. This progress and current trends are documented in the literature such as the proceedings of the biennial ‘international workshop on advanced GPR – IWAGPR’ or the ‘International conference on ground penetrating radar’. The following considerations will focus on three important trends, inversion, three-dimensional radar surveys and the investigation of material properties.

15.7.1 Three-dimensional radar surveys

As modern radar units are able to record hundreds of scans per second, a natural approach is data acquisition along lines. The antenna is moved horizontally while recording traces at fixed distances. The resulting dataset is two-dimensional (horizontal distance and time or depth) and is called a radargram.

An example recorded on a bridge girder (Fig. 15.11; Hugenschmidt and Mastrangelo, 2006) is presented in Fig. 15.12. Two groups of reflections at around 1 ns and 2–3 ns can be distinguished. The interpretation of these reflections is not straightforward without additional information or assumptions. If many (hundreds) of such datasets are acquired covering the whole front of the girder, they can be combined into a three-dimensional dataset. From this dataset, slices representing constant time (time-slices) can be cut and displayed. An example from $t = 0.8$ ns is presented in Fig. 15.13. Using this time slice, the distinction between the vertical rebars and two tendons running from top left to bottom right is straightforward.

Three-dimensional data acquisition and processing require considerably more effort than two-dimensional radar surveys. However, three-dimensional surveys do provide additional information that can make the difference between success and failure.
15.7.2 Inversion

Inversion (Ernst 2007, Soldovieri et al., 2007) can be described as a process where a numerical model representing a physical environment is modified until it would produce a certain dataset. In this sense inversion can be considered as the opposite of modelling where a numerical model is used to produce the corresponding dataset (Fig. 15.14). In other words, if a radar dataset has been acquired on a structure, a numerical starting model is built and synthetic radar data are generated using the starting model. The synthetic data are then compared with the real data and the model is adjusted. Then synthetic data are again generated using the updated model. This
Inversion and modelling.

The output of a successful inversion process is a model that represents the physical reality of the inspected object. An example is presented in Fig. 15.15 (Hugenschmidt et al. 2010). Data were acquired on a retaining wall made of reinforced concrete and the inversion was carried out using the approach described by Soldovieri et al. (2007). The result is given in terms of the modulus of the contrast function that accounts for the difference...
between the dielectric and conductive properties of the targets and those of the host medium (concrete hosting rebar).

15.7.3 Material properties

In the past, radar surveys on concrete were carried out almost exclusively to obtain structural information such as thickness of concrete, concrete cover of rebar or position of tendons. Recently, material properties such as moisture or chloride content have become an important subject for research (Barnes et al., 2008, Hugenschmidt and Loser, 2008, Klysz et al., 2008).

In a laboratory experiment (Hugenschmidt and Loser, 2008), nine concrete samples were built. Three different amounts of chlorides (0, 0.4 and 1%) were added to the concrete mixture and the specimens were stored at three different relative humidities (35, 70 and 90%). Radar data were acquired on each of the specimens and the quotient of the amplitudes of the reflections at the concrete surface and the bottom of the specimens was computed. The result presented in Fig. 15.16 shows a clear relationship between the quotient of the amplitudes and the chloride/moisture content of the specimens.

15.8 Symbols and constants

- $B = \mu_0 H$ (not valid for ferromagnetic materials)
- $c$ speed of light in vacuum $= 2.998 \times 10^8$ m s$^{-1}$
- $d$ thickness of a layer or depth of a target
\[ D \quad \varepsilon_0 E \]

\[ E \quad \text{electric field} \]

\[ \varepsilon \quad \text{relative permittivity of medium} \]

\[ \varepsilon_0 \quad \text{absolute permittivity of free space} = 8.85 \times 10^{-12} \text{ C V}^{-1}\text{m}^{-1} \]

\[ f \quad \text{frequency} \]

\[ H \quad \text{magnetic field} \]

\[ \lambda \quad \text{wavelength} \]

\[ \mu \quad \text{relative permeability of medium} \]

\[ \mu_0 \quad \text{absolute permeability of free space} = 1.26 \times 10^{-6} \text{ Vs A}^{-1}\text{m}^{-1} \]

\[ R \quad \text{coefficient of reflection} \]

\[ Twt \quad \text{two-way traveltime} \]

\[ \omega \quad \text{angular frequency} \]

### 15.9 References


DGZFP (GERMAN SOCIETY FOR NON-DESTRUCTIVE TESTING), 2008, Merkblatt über das Radarverfahren zur Zerstörungsfreien Prüfung im Bauwesen, Merkblatt B10, Berlin, Germany.


Proceedings of the 11th International Conference on Ground Penetrating Radar, GPR 2006, June 19–22, Columbus, Ohio, USA.
Proceedings of the 9th International Conference on Ground Penetrating Radar, GPR 2002, April 29–May 2, Santa Barbara, California, USA.
Proceedings of the 8th International Conference on Ground Penetrating Radar, GPR 2000, May 23–26, Gold Coast, Queensland, Australia.
16

Radar tomography for evaluation of reinforced concrete structures

L. ZANZI, Politecnico di Milano, Italy

Abstract: Radar tomography is a non-destructive technique that can be applied to reinforced concrete structures to map internal inhomogeneities, defects or property variations (moisture, chlorides concentration, density, steel fibre distribution). The method is illustrated by presenting the physical principles and the procedure for performing the experiments, i.e. the required equipment, the acquisition methods and the processing algorithms. Resolution, possible artefacts and criteria for interpreting the results are discussed to introduce the reader to a correct use of the technique. A few examples are also illustrated to show possible results. Advanced algorithms such as full-waveform inversion or diffraction tomography are briefly introduced and references are given to more in-depth studies.

Key words: radar tomography, data processing, reinforced concrete.

16.1 Introduction

Originally, the term tomography was introduced to indicate a medical imaging technique based on x-ray measurements. In applied geophysics, tomography generally refers to the reconstruction of the velocity distribution in a medium by inversion of the traveltimes of waves that travelled through the medium (Kak and Slaney, 1988; Nolet, 1987; Nolet, 2008; Stewart, 1991; Worthington, 1984). In non-destructive testing (NDT) of structures (walls, pillars, columns), tomography is generally applied by using sonic, ultrasonic or radar waves. To perform radar tomography on a concrete structure, a ground penetrating radar (GPR) system is needed (Daniels, 1996). GPR is a geophysical technique that has found a variety of applications in civil engineering including non-destructive investigations of concrete structures (Bungey, 2004).

A GPR system normally works in reflection mode producing echograms called radargrams and it consists of a radar unit and an antenna that is moved against the structure to be investigated. For tomographic surveying, two antennae (a transmitter and a receiver) are separately moved on different sides of a structure. Antennae with different frequencies are used depending on the required penetration: energy absorption
increases with frequency and lower frequencies are needed to investigate deeper targets. The distribution of the radar wave velocity inside the tomographic section is the standard result but another parameter (attenuation) is sometimes extracted from the radar records. Attenuation maps that describe the distribution of the attenuation coefficient of the material are extracted by processing the amplitudes of the radar wave. More complex approaches (full waveform inversion) are also possible in order to extract other information and/or to improve the resolution.

The parameters that are mapped by radar tomography (velocity, attenuation or other) are somehow related to the electromagnetic properties of the material and an understanding of these relationships is essential to interpret the results for diagnostic applications. These relationships are summarized in the following section. Then, the basic equation of ray tomography are introduced (16.3) followed by a discussion about resolution (16.4). In 16.5 the specific equipment needed for tomographic experiments is illustrated. The following sections (from 16.6 to 16.8) show how to perform a radar tomography from acquisitions to data processing. Artefacts that may appear in tomographic images are explained in 16.9. In 16.10 methods for interpreting the results are illustrated and some of the most interesting applications of radar tomography on concrete structures are described. A few laboratory and on-site examples are shown in 16.11. Some comments about advanced algorithms for tomographic inversion are given in 16.12. Finally, section 16.13 summarizes the potential of radar tomography for concrete investigations.

16.2 Physical principles

For an electromagnetic plane wave travelling along the $x$ axis, the expression of the electric field $E$ is:

$$ E = E_0 e^{j(\omega t - \gamma x)} \quad [16.1] $$

where $\omega$ is the angular frequency, $t$ is the time, and $\gamma$ is the propagation constant given by:

$$ \gamma = \sqrt{\omega^2 \mu \varepsilon - j \omega \mu \sigma} = \beta - j \alpha \quad [16.2] $$

where $\varepsilon$ is the permittivity, $\mu$ is the magnetic permeability and $\sigma$ is the conductivity.

Polarization phenomena are significant in most materials; these phenomena produce an out-of-phase component of the permittivity. A simplified description of the complex relative permittivity is given by the Cole–Cole model:
where $\varepsilon_s$ is the real relative permittivity at zero frequency, $\varepsilon_\infty$ is the relative permittivity at infinitely high frequency, $\tau$ is the time constant of the relaxation, and $\nu$ is a parameter that accounts for the presence of different polarization processes with different time constants (its value is usually in the range 0.5–1). The conductivity may also present an out-of-phase component:

$$\sigma = \sigma' - j\sigma''$$

although the imaginary part is usually negligible for most of the GPR applications.

Because $\varepsilon'$ and $\sigma''$ produce current out of phase with the electric field and $\varepsilon''$ and $\sigma'$ produce current in phase with the electric field, it is worth defining (Turner and Siggins, 1994) the effective permittivity $\varepsilon_e$ and the effective conductivity $\sigma_e$ as:

$$\varepsilon_e = \varepsilon_0 \varepsilon' - \frac{\sigma''}{\omega}$$

$$\sigma_e = \sigma' + \omega \varepsilon_0 \varepsilon''$$

With this approach, $\varepsilon_e$ and $\sigma_e$ are quantities that come from physical measurements of the electric properties of the materials.

Substituting equations [16.5] and [16.6] into equation [16.2], the components of the propagation constant $\gamma$ are obtained

$$\alpha = \omega \sqrt{\frac{\mu \varepsilon_e}{2} \left(1 + \frac{\tan^2 \delta}{2} - 1\right)}$$

$$\beta = \omega \sqrt{\frac{\mu \varepsilon_e}{2} \left(1 + \frac{\tan^2 \delta}{2} + 1\right)}$$

where $\tan \delta$ is the loss tangent, defined as:

$$\tan \delta = \frac{\sigma_e}{\omega \varepsilon_e}$$

Concrete can be considered a low-loss material, i.e. a material for which $\tan \delta \ll 1$. Therefore, $\alpha$ and $\beta$ can be approximated as:

$$\alpha \approx \omega \sqrt{\frac{\mu \varepsilon_e \tan \delta}{2}} \approx \sqrt{\frac{\mu \sigma_e}{2 \varepsilon_e}}$$

$$\beta \approx \omega \sqrt{\mu \varepsilon_e}$$
Both $\alpha$ and $\beta$ are frequency-dependent.

By using $\alpha$ and $\beta$, equation [16.1] becomes:

$$E = E_0 e^{-\alpha x} e^{j(\omega t - \beta x)}$$ \[16.12\]

where $\beta$ is responsible for the propagation velocity so that according to [16.11] we can write:

$$V = \frac{\omega}{\beta} = \frac{1}{\sqrt{\mu \varepsilon_e}}$$ \[16.13\]

Traveltime tomography (TT) measures velocity and, since for concrete we can assume that magnetic permeability is invariant and equal to the freespace value, equation [16.13] indicates that TT is really a measurement of effective permittivity $\varepsilon_e$.

Amplitude tomography (AT) measures attenuation and, from equation [16.12], we see that $\alpha$ is responsible for attenuation so that according to equation [16.10], we can conclude that AT is really a measurement of effective conductivity $\sigma_e$.

### 16.3 Fundamental equations

TT is based on the measurement of traveltimes for all the possible transmitter–receiver positions around the target section. The first arrival traveltime is the first energy burst emerging from the noise level in a radar record, i.e. it is the time that the pulse emitted by the TX antenna takes to arrive at the RX antenna along the fastest trajectory.

In TT, the inversion problem is typically expressed by a system of equations in which for each trajectory the equation of the measured traveltime $t_i$ is given by:

$$t_i = \sum_k l_{ik} w_k$$ \[16.14\]

where $l_{ik}$ is the path length of the $i$th ray in the $k$th cell and $w_k$ is the slowness (inverse of velocity) of the $k$th cell.

Thus, the assumptions in such an approach are that the radar wave propagates along rays, the propagation rays correspond to the fastest trajectories, and the medium inhomogeneity can be described by geometrical decomposing the tomographic section in a number of homogeneous cells characterized by constant velocities. We will return later to the approximations and implications related to these assumptions.

The TT assumptions are also used for the AT implementation. The objective of AT is to measure the attenuation coefficient $\alpha$ from the amplitude of the radar wave at the receiver. In principle, this is possible if any
Other phenomenon that might affect the amplitude at the receiver have been adjusted. We will see that we can adjust the wavefront divergence effect and the antenna pattern effect. However, we cannot separate the effects of scattering phenomena from absorption so that the attenuation coefficient that will result from AT is actually a combination of absorption and scattering. This observation is quite important for a correct interpretation of the AT results because scattering can be sometimes more noticeable than absorption.

For AT the inversion problem is a system of equations such as:

\[
\ln\left(\frac{A_i}{A_0}\right) = -\sum_k l_{ik} \alpha_k
\]

where \(A_i\) is the measured amplitude associated with the \(i\)th ray after divergence and antenna pattern adjustments, \(A_0\) is the amplitude measured at zero distance, i.e. at the TX position, \(l_{ik}\) is the path of the \(i\)th ray in the \(k\)th cell and \(\alpha_k\) is the attenuation factor of the \(k\)th cell.

### 16.4 Resolution

The assumption that radar waves propagate along rays is a very loose approximation of what really happens, corresponding to an assumption of an infinite bandwidth experiment with best possible resolution. A more realistic description of what really happens in a radar experiment is described by the concept of Fresnel zone. According to this concept, all anomalies included in the Fresnel zone constructively contribute to the electromagnetic field at the receiver.

The width of the Fresnel zone (FZW), i.e. the width of the raypath, can be computed according to the following expressions for monofrequency and for band-limited sources, respectively (Woodward, 1992):

\[
FZW_m = \sqrt{\frac{L}{wf}}
\]

and

\[
FZW_b = 2\sqrt{\frac{1}{2wB}\left(\frac{1}{2wB} + L\right)}
\]

where \(L\) is the source–receiver distance, \(w\) is the slowness, \(f\) is the frequency and \(B\) is the bandwidth. Figure 16.1 shows how wide the actual rays are for an example where the tomographic section is 1 m wide. The effects of the wavepath width are that anomalies smaller than FZW are poorly reconstructed and anomalies spaced less than FZW are hardly distinguishable.
from one another. In other words, FZW is a good prediction of resolution for a tomographic experiment.

In practice, this is true if the measurements are properly carried out, i.e. the angular coverage and the spatial sampling honour the Fourier Slice Theorem (Rector and Washbourne, 1994; Valle and Zanzi, 1996, 1998). When the measurements do not satisfy the above conditions because of limited accessibility to the structure or because of a particular structure geometry, the resolution limits could be even more unfavourable.

### 16.5 Equipment

The nominal frequency of the radar antenna that is more appropriate for ensuring the required penetration with the highest resolution is normally from 500 MHz to 2 GHz depending on the size of the structure. Both [16.16] and [16.17] indicate that resolution improves by selecting higher frequency antennae. Unfortunately, penetration follows the opposite behaviour. Lower frequency antennae ensure deeper penetration for two main reasons: a) scattering and absorption increase with frequency; b) lower frequency means larger antennae, and larger antennae can radiate more power. As a
result, the final selection of the proper antenna is always a trade-off between resolution and penetration requirements.

Table 16.1 presents a rough indication of the proper antenna frequency versus the length range of the tomographic trajectories. Actually, these indications can vary depending on the moisture condition of the concrete structure. Higher moisture means higher conductivity and hence higher absorption and lower penetration. Thus, preliminary tests of frequency penetration are always recommended for any new acquisition to select frequencies as high as possible to maximize the expected resolution.

High frequency antennae usually consist of TX and RX elements hosted in a single shielded box (Fig. 16.2) to prevent interference with radio equip-

<table>
<thead>
<tr>
<th>Nominal frequency (MHz)</th>
<th>500</th>
<th>1000</th>
<th>2000</th>
</tr>
</thead>
<tbody>
<tr>
<td>Distance range (cm)</td>
<td>200–400</td>
<td>100–200</td>
<td>50–100</td>
</tr>
</tbody>
</table>

16.2 Shielded antenna operating at very high frequencies (2 GHz).
ment, mobile phones and other telecommunication systems. As a result, to perform a tomography where we need TX and RX elements to be independently moved on different sides of the wall we have to use two antennae connected to the radar unit with only the TX and the RX, respectively. As a result, compared with what is really needed to perform a tomographic acquisition, antenna elements are oversize and overweight and the whole equipment is unnecessarily expensive. Unfortunately, the use of GPR systems in tomographic configuration is not sufficiently widespread to motivate the GPR supplier to design dedicated systems with separate TX and RX elements.

An exception exists for borehole tomographic applications. Borehole GPR antennae are commercially produced with separate unshielded TX and RX elements that are expected to be lifted inside two parallel boreholes. They are typically applied for geotechnical or mine studies (Becht et al., 2004; Gloaguen et al., 2007; Gourry et al., 1996; Liu et al., 1998a; Tronicke et al., 2001, 2002 and 2004) but, in principle, they can be also used to perform NDT testing of concrete foundations or huge concrete structures provided that at least two boreholes are available or can be drilled (Giroux et al., 2007; Jha et al., 2004).

16.6 Acquisition procedures

Accuracy in traveltime measurements is essential in tomographic investigations. GPR systems are designed to work in reflection mode and they collect data on a relative time scale activating the receiver slightly in advance of the expected arrival of the coupling wave, i.e. the direct wave from TX to RX. The fact that the time axis is not calibrated is not a problem in reflection mode because the GPR users can rely on the direct wave to perform an a posteriori calibration of the timescale. This is not the case for tomographic experiments. TX and RX elements are independently moved on different sides of the tomographic section and the timescale must be calibrated with a specific procedure.

In principle, once the equipment have been assembled and warmed-up, a single measurement of a zero distance radar record, i.e. a measurement with physical contact of TX and RX boxes, would be enough to calibrate the time scale, i.e. to estimate the zero time of the experiment. However, since a zero distance measurement is expected to introduce some wavelet distortion because of near-field effects, a more reliable procedure can consist of recording a number of radar traces with the TX facing the RX in air at known distances that are progressively increased. As a result, the radar record will look like Fig. 16.3 and the zero time of the experiment can be accurately estimated by extrapolating the first arrival line to read the intercept time at zero distance. Since GPR systems might have a time
drift during acquisitions, it is advisable to repeat this calibration procedure at the end of the experiments or sometimes during the experiments when they take long. This calibration must definitely be repeated when a battery is replaced, when an antenna cable is changed and any time the equipment or the acquisition software is restarted.

Amplitude tomography also needs a specific calibration. The objective is to measure the term $A_o$ in equation [16.15], i.e. the amplitude that would be measured at the TX position. Again, in principle, the problem could be solved with a single measurement of a zero distance radar record, but in practice this is not possible because the result would be strongly distorted by the near-field effects and by a radiation pattern totally different from the radiation pattern resulting when the antenna box is facing the concrete structure. Instead, the calibration procedure might consist of performing two transillumination experiments through a homogeneous concrete specimen as shown in Fig. 16.4. Once the amplitudes of the two measurements are extracted and adjusted for divergence effects, the attenuation coefficient of the specimen can be obtained and used to extrapolate the zero distance amplitude $A_o$. This is intended to be the amplitude measured in the direction perpendicular to the antenna element. Any tomographic trajectory that will leave the TX or reach the RX element at different angles will need to be adjusted for the antenna pattern effect. Because the calibration experiment requires a proper specimen that cannot be moved on-site, it is essential to perform the laboratory calibration with exactly the same

---

**16.3 Time calibration.** The intercept time $t_o$ will be subtracted from the tomographic traveltimes.

---

© Woodhead Publishing Limited, 2010
Once the traveltime calibration has been performed, the tomographic acquisition can start. Different strategies can be adopted to move the TX and RX antennae. When the section geometry is complex, a stop-and-go procedure might be required for both TX and RX elements. A proper spacing between measurement points along the structure perimeter must be selected. This has to be calibrated according to the expected resolution of the experiment. A spacing larger than the resolution is excessively large and will undersample the tomographic section. On the contrary, a spacing ten times smaller than resolution is unnecessarily small and time-consuming. When the section geometry is simple, one of the elements (e.g. the RX) follows the stop-and-go procedure whereas the other element (e.g. the TX) runs profiles during the RX stop intervals. For this, the TX is equipped with an odometric wheel so that GPR traces are collected with the expected spacing along the sides of the tomographic section. The second procedure is therefore preferable whenever possible in order to reduce the acquisition times that in tomographic experiments are significant. Of course, whatever strategy is adopted, proper care must be taken to report accurately TX and RX positions of each measurement. Care must be also taken to properly design the whole sequence of measurements. The objective is to ensure a good coverage of the investigated section. This depends on the number and orientation of the TX–RX point-to-point measurements. An insufficient coverage will result in artefacts and loss of resolution (see Section 16.9).

16.4 Amplitude calibration set-up.
16.7 Data pre-processing

Table 16.2 shows a possible pre-processing sequence needed to prepare the tomographic inversion. The tomographic software needs the description of the section geometry and the coordinates of TX and RX positions for each radar measurement. Bandpass filtering and data cleaning prepare the data for the most important step that consists of traveltime picking. Data cleaning means that radar traces presenting a poor SNR (signal-to-noise ratio) or contaminated by some interference can be removed during a preliminary check of data quality. Traveltime picking is performed by reading the time at which the first energy burst appears in a trace. A consistent criterion should be used in associating a time of arrival to each event, e.g. by choosing to read the time of the first positive or negative peak or the first zero-crossing (Fig. 16.5). In principle, this criterion should be the same that was used to read the traveltimes of the calibration test described above. Traveltime calibration (i.e. subtraction of the calibration time) concludes the pre-processing of traveltime tomography.

In addition, amplitude tomography needs some more processing (Giroux et al., 2007; Holliger et al., 2001; Peterson, 2001). Amplitude picking can be performed automatically, for example by analysing the first break wavelet to extract the peak-to-peak excursion or to measure the square root of the signal energy (Fig. 16.5). These amplitudes must be adjusted for wavefront divergence by applying a gain proportional to the TX–RX distance and must be normalized to the reference amplitude \( A_0 \). Finally, amplitudes must be adjusted for the effects of the radiation pattern of the antennae. This is a critical issue because the prediction of the directivity function of a GPR antenna is a complex problem. The knowledge of the directivity function of the antenna in the freespace is useless because it is well known that the directivity function is strongly modified as soon as the antenna is put against the structural element under investigation. To make the problem more

<table>
<thead>
<tr>
<th>Table 16.2 Pre-processing sequence for preparation of tomographic inversion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Geometry input</td>
</tr>
<tr>
<td>Band-pass filtering</td>
</tr>
<tr>
<td>Data cleaning</td>
</tr>
<tr>
<td>Traveltime picking</td>
</tr>
<tr>
<td>Time calibration</td>
</tr>
<tr>
<td>Amplitude picking</td>
</tr>
<tr>
<td>Amplitude normalization</td>
</tr>
<tr>
<td>Adjustment for wavefront divergence</td>
</tr>
<tr>
<td>Adjustment for radiation pattern</td>
</tr>
</tbody>
</table>

© Woodhead Publishing Limited, 2010
complex, the resulting radiation pattern depends on the dielectric properties of the material. Analytical expressions of the radiation pattern are available in literature (Smith, 1984) for simplified assumptions, e.g. assuming that the GPR antenna behaves as a Hertzian dipole in far-field conditions. Unfortunately, these assumptions are very restrictive and far from real situations. First, high frequency GPR antennae are not simple dipoles: they generally consist of shielded bow-tie dipoles. The shield prevents interference from spurious signals or echoes coming from the environment and improves the directivity of the antenna. Secondly, the dipoles are resistively loaded. Finally, TX–RX distances in tomographic experiments are not long enough to satisfy the far-field assumption. A numerical approach can be more successful in approximating the real antenna behaviour. As an example, Fig. 16.6 shows the real radiation patterns of a GPR antenna resulting from laboratory tests on two specimens with different dielectric properties. The experimentally measured pattern is compared with the far-field analytical expression and with a numerical simulation of the near-field behaviour obtained by solving the Sommerfeld integral (Valle et al., 2001). The results confirm that the numerical near-field solutions, though related to an unshielded infinitesimal dipole rather than to a shielded bow-tie element, are more reliable than the far-field analytical solutions and should be preferred for adjusting tomographic amplitudes.

16.5 (a) Traveltime picking and (b) amplitude picking.
The apparently linear problem expressed in [16.14] is actually a non-linear problem since both velocities and wave trajectories are unknown and intrinsically related. The trajectory that is expected to generate the first arrival at the receiver entirely depends on the velocity distribution within the tomographic section. As a result, these trajectories can be traced only once the velocity field is known. Vice versa, the velocity field can be determined by inverting the system in [16.14] only once the trajectories are known. The common strategy to approach the problem is to apply an iterative inversion procedure (Fig. 16.7). A preliminary velocity field is assumed to calculate the initial trajectories, equation [16.14] is solved and the new velocity field is used to update the trajectories and prepare equation [16.14] for a new inversion. Iterations will continue until convergence is reached, i.e. until the velocity field variations between two consecutive iterations become negligible. As a result, we come to the final solution by solving a sequence of linear problems that are obtained by linearizing the problem around the solution given by the current velocity field. It is understood that the final result will depend on the initial assumption accepted for the velocity field. Thus, a good initial velocity model, based on any available a priori information or preliminary measurements, is very important to guide the solution towards the correct result.

The design of the grid is also very important to stabilize the solution. A common error by inexperienced users consists of designing a very dense grid, which can lead to numerical instability and inaccurate results.
grid of small pixels. This would appear to be a smart approach because it produces a high-resolution image and it reduces the residual level. In reality, as the number of velocity pixels is increased the inversion problem becomes more and more underconstrained and the final result is totally unreliable unless strong constraints are simultaneously added to preserve inversion stability. A good criterion for designing the grid consists of selecting a pixel dimension consistent with the resolution of the experiment according to the Fresnel theory (see Section 16.4; Valle and Zanzi, 1998). Some tomographic software also offer the option of designing non-regular grids; this is a powerful method for reducing the null space and for increasing the resolution in the target areas (Bohm and Vesnave, 1999; Bohm et al., 2000, Valle and Zanzi, 1998; Vesnave, 1996). The design of the irregular grid can be fully automatic or interactive; in any case, it should be guided by any available a priori information, by the coverage map that indicates the areas with a low rate of observations, and by the previous reconstruction that helps the user to localize the areas of main interest where higher resolution is desirable. Staggered grids have been also proposed as a valid approach to achieve a good tradeoff between stability and resolution (Vesnave and Bohm, 2000).

There have been many studies about regularization, i.e. to discuss mathematical methods for stabilizing the tomographic solution and for improving the image resolution. A very common method consists of introducing an appropriate stabilizing functional (stabilizer) such as the traditional

---

16.7 Flow chart of the typical iterative approach to tomographic inversion.
minimum norm stabilizer (Vignoli and Zanzi, 2005). The effect of these stabilizers is normally to drive the final solution towards a smoothed reconstruction of the velocity field. Nevertheless, alternative methods exist. Gloaguen et al. (2005, 2007) recently tested a self-regularization technique based on a geostatistical analysis of the traveltime data. Various stabilizers, such as the minimum support stabilizer, can be also used to incorporate a priori knowledge in the inversion process (Bertete-Aguirre et al., 2002; Geman and Reynolds, 1992; Geman and Yang, 1995; Portniaguine and Zhdanov, 1999; Vogel, 1997). These new stabilizers are particularly indicated to reconstruct velocity fields with abrupt variations. It was demonstrated that they generate clearer and more focused images of the anomalies than the conventional maximum smoothness functional (Vignoli and Zanzi, 2005; Vignoli and Zhdanov, 2005; Zhdanov, 2002). A similar objective can be also achieved by using the $N$-colour tomography approach (Valle and Zanzi, 1998; Valle et al., 1999) where a constrained inversion is performed assuming that only a limited number ($N$) of homogenous different materials (e.g. concrete and void) may exist inside the investigated structure. Another valid alternative is the variable damping factor approach suggested by Bernabini and Cardarelli (1997). However, we cannot say that a stabilizing approach is always preferable to the others. In reality, the optimal selection of the proper stabilizer is again related to a priori information and with the target of the investigation. As an example, if a cavity is expected inside a concrete structure, the minimum support stabilizer can be the proper choice whereas if the aim of the investigation is to map gradual variations of moisture level inside the tomographic section, the minimum norm stabilizer can be more appropriate.

Although a stabilizing strategy has also the positive effect of attenuating the noise impact on the final result, specific noise reduction actions can be adopted during inversion. Noise in traveltime tomography is the result of inaccurate traveltime picking and mispicking. Inaccurate picking, i.e. poor precision in reading the traveltime associated with the selected wavelet feature (e.g. the first positive peak or the zero crossing or any other feature), is expected to introduce a widespread noise equally distributed on the whole tomographic section. Instead, mispicks, i.e. large picking errors that occur when ambiguities exist in recognizing the first arrival among multiple events, are expected to produce local artefacts in the final velocity map. However, large errors such as those generated by mispicks should appear as outliers of the inversion problem. In other words, they can be gradually detected during the iterative inversion as the data that are too far from the synthetic data associated with the current estimate of the velocity field. Thus, it is possible to prevent the mispick artefacts by including into the inversion algorithm a strategy for automatic detection and correction of outliers (Valle and Zanzi, 1998).
Most of the above observations can be also extended to the inversion of equation [16.15], i.e. to amplitude tomography. However, amplitude tomography always follows traveltime tomography because equations [16.15] still need the wave trajectories and trajectories need the velocity field. It follows that amplitude tomography does not require an iterative procedure. Trajectories will be calculated with the final velocity field resulting from traveltime tomography and equation [16.15] will be instantly inverted.

16.9 Artefacts

The interpretation of a tomographic result can be misleading if the interpreter is not aware of possible artefacts resulting from a limited coverage experiment. Limited coverage is often forced by the geometry of the structure or by physical constraints that do not allow access to every side of the tomographic section. This issue can be discussed by invoking the Fourier Slice Theorem under the hypothesis that the electromagnetic wave travels along an infinitely thin straight ray.

For crosshole geometry (Fig. 16.8a), the coverage, i.e. the ray density in each cell of the tomographic section, is quite inhomogeneous (Fig. 16.8b) and the detectable wavenumbers are included in a fan filter (Fig. 16.8c) defined by the equation

$$f_y = \pm \frac{f_x}{\tan(\theta)} \quad [16.18]$$

where $\theta$ is the maximum angle of the projections, i.e. the widest angle between TX and RX positions. The associated impulse response shows a X-shaped artefact with the larger extension in the direction perpendicular to the sides where antennae move along (Fig. 16.8d). The consequence is that any local velocity anomaly in the investigated section will be similarly distorted in the tomographic reconstruction.

If the section is accessible from all directions (assuming a rectangular section) a ‘double crosshole’ geometry is obtained (Fig. 16.9a), where the detectable wavenumbers are included in the combination of two perpendicular fan filters. If the maximum angle of the projections is smaller than 45°, the section coverage is still somehow inhomogeneous (Fig. 16.9b) and the fan filters do not overlap so that a non-observed region of wavenumbers exists (Fig. 16.9c). Figure 16.9d shows the associated impulse response where the X-shaped artefact is still present, although less pronounced. If the maximum angle of the projections is wider than 45° (Fig. 16.10), the section coverage tends to be homogeneous and a complete wavenumber coverage is assured so that artefacts disappear from the impulse response. As a result, a double crosshole geometry with maximum projection angle
of at least 45° is preferable, whenever possible, to prevent the artefacts and to preserve the resolution.

### 16.10 Interpretation of results

Velocity and/or attenuation maps produced by traveltime and amplitude tomography contain information about morphology and physical properties of the materials and inhomogeneities inside the investigated area. Morphological information must be carefully interpreted. As mentioned above, the geometry of small anomalies might be distorted by artefacts produced by limited coverage so that the knowledge of the acquisition geometry is a necessary information to be transmitted to the interpreter. Resolution is
another parameter that must be known for a correct morphological interpretation. In principle, anomalies much smaller than the expected resolution tend to be ignored whereas anomalies of a size close to the expected resolution tend to be poorly reconstructed. A distinction has to be made here depending on the sign of the velocity contrast created by the small anomaly. The first arrival energy travels along the fastest path so that a fast anomaly attracts energy whereas a slow anomaly rejects energy. The effect on the inversion is that fast anomalies are better detected although they tend to be over dimensioned; on the contrary, slow anomalies tend to be smoothed or even undetected.

Careful interpretation is also necessary when small anomalies are observed on attenuation maps. Anomalies of a size close to the experiment
resolution are expected to produce scattering. Amplitude inversion cannot separate scattering effects from absorption effects. This generates ambiguities in trying to identify the material of the anomaly: very different materials (e.g. void and metal) can produce a similar attenuation anomaly regardless of their absorption coefficients.

Sometimes, this type of ambiguity can be solved with a joint interpretation of velocity and attenuation maps. In general, this joint approach is the best strategy for a reliable interpretation of tomographic results in terms of materials and physical properties. Let us assume that the possible materials that we expect in a tomographic investigation on a concrete structure are: concrete (with different properties as a result of different mixtures,
different compaction, honeycombing, moisture level, salt concentration, carbonation), cavities, metal reinforcements, and steel fibres.

We can immediately exclude metal reinforcements and carbonation because tomography is not an appropriate approach to detect these targets. The size of metal reinforcements is normally much smaller than resolution. A weak indication of their presence might be observed in attenuation maps as a result of scattering but cheaper and more effective methods exist to detect metal reinforcements, e.g. metal detectors or GPR in standard reflection mode (Annan et al., 2006). Carbonation is a near-surface phenomenon that is expected to affect a thickness that is normally much lower than tomographic resolution. In addition, tomography is intrinsically a weak method for studying the external perimeter of the structure where coverage is normally lower than inside the structure.

Variations in compaction, porosity, and concrete mixtures (water, cement, sand, aggregates) are expected to produce different radar responses (e.g. honeycombing is expected to reduce both permittivity and conductivity), but water influence is strongly dominating over the other material variations (Balayssac et al., 2009; Bungey et al., 1996; Bungey, 2004; Klysz and Balayssac, 2007; Klysz et al., 2007; Laurens et al., 2005; Maierhofer et al., 2008; Sbartai et al., 2006a; Sbartai et al., 2006b; Sbartai et al., 2009; Soutsos et al., 2001; Villain et al., 2009). Higher water concentration is expected to increase both permittivity and conductivity of concrete. As a result, according to equations [16.10], [16.11] and [16.13], areas of higher saturation are mapped by traveltime and amplitude tomography as slow velocity and high attenuation areas, respectively. An increase in the concentration of chlorides at constant saturation is expected to further increase the conductivity and the imaginary part of permittivity and thus to produce higher attenuation (Derobert et al., 2009; Hugenschmidt and Loser, 2008; Mizobuchi et al., 2008; Sbartai et al., 2006a, 2008; Soutsos et al., 2001). Effects of chlorides on the velocity map should be negligible at frequencies of interest for GPR tomography, i.e. higher than 500 MHz (Al-Qadi et al., 1997a, 1997b; Haddad and Al-Qadi, 1998; Robert, 1998). However, some studies have reported minor effects on the real part of permittivity at frequencies of about 1 GHz (Derobert et al., 2009; Kalogeropoulos et al., 2009; Krause et al., 2006; Villain et al., 2009).

The presence of cavities should be readily detected by traveltime tomography as high velocity anomalies, provided that the void size is not too small compared with resolution (Valle and Zanzi, 1996). Amplitude tomography can also detect cavities, but the response might differ according to the void size. Small cavities are expected as high attenuation anomalies because of scattering whereas large voids might appear as low attenuation anomalies because of lower absorption occurring travelling inside the cavity. The prediction of the amplitude tomography response to an elongated cavity
might be very tricky and very much dependent on the acquisition geometry compared with the orientation of the elongated target.

Finally, steel fibres are expected to increase attenuation because of scattering and increased conductivity (Maierhofer et al., 2008). According to Soutsos et al. (2001), they also increase permittivity and thus reduce velocity. As a result, provided that the proper frequency is selected for the experiment to ensure penetration preserving the sensitivity to fibre scattering, the presence/absence or variations in fibre distribution can be mapped. However, steel fibres are generally used in horizontal thin structures that can be inspected with more convenient and effective methods, e.g. with GPR in reflection mode.

16.11 Examples

A laboratory test and a real application are presented to illustrate some examples where moisture and chloride variations are detected by travel-time and amplitude tomography.

16.11.1 Laboratory test

A concrete sample was prepared consisting of two $30 \times 30 \times 60$ cm concrete blocks. The two blocks were positioned over a bed consisting of three stone layers and surrounded by a concrete structure consisting of small cubic concrete specimens (Fig. 16.11). The tomographic experiment was per-
formed horizontally on the two concrete blocks (Fig. 16.12). The surrounding stone and concrete structure was created to increase the size of the sample so that no radar energy could travel in the air around the concrete blocks instead of in the concrete. The elements of the sample were not cemented to allow the replacement of the concrete blocks. A preliminary situation with a homogeneous section was created by using two dry concrete blocks. In practice, the blocks were not really dry because they were not dried in an oven but they were left indoor for a few weeks so that they were presumed to be relatively dry compared with the following treatments. GPR tests were performed to check that all the elements of the specimen were sufficiently in contact to prevent any possible direct or refracted air trajectory for the radar signal. A first tomographic experiment was also performed on this structure by using two 1 GHz antennae that were moved step-by-step along two opposite horizontal lines with a station spacing of 5 cm (Fig. 16.12). A total number of $12 \times 12 = 144$ radar traces were measured with a crosshole geometry. The velocity map obtained from traveltine inversion confirmed a homogeneous condition of the two specimens with a radar velocity of about 9.5 cm ns$^{-1}$.

After the validation of the specimen, one concrete block was removed and saturated with fresh water. The weight of the block changed from 121 kg to 122.5 kg corresponding to a 3% increase of water volume. The tomographic experiment was repeated immediately after the extraction of

16.12 Tomographic acquisitions. Two 1 GHz antennae were moved on the exposed sides of the concrete blocks along a horizontal line with a step of 5 cm.
the concrete block from the water pool and the recomposition of the specimen. Fig. 16.13 shows the resulting 144 radar records organized in 12 common- TX fans. The six fans on the left were collected with the TX on the wet block so the radar signal mainly travels inside wet concrete. As a consequence, traveltimes are slightly higher with an increase of about 0.3–0.4 ns compared with the traveltimes measured on the following fans. In addition, amplitudes on the left fans are lower than on the right fans as expected for the absorption increase associated with the conductivity increase generated by water. We can also note that an additional decrease of amplitudes is observed on the most lateral fans and in the middle fans. This is related to scattering effects produced by metal reinforcements (Fig. 16.14). Traveltime inversion produced the result shown in Fig. 16.15 where the dry and wet concrete blocks are clearly imaged with velocities of about 9.5 and 9.1 cm ns\(^{-1}\), respectively. This corresponds to an increase of relative permittivity from 10 to 11. The result is consistent with the observations reported by Soutsos \textit{et al.} (2001) and by Villain \textit{et al.} (2009). According to figure 4 in Soutsos \textit{et al.} (2001) and to figure 2 in Villain \textit{et al.} (2009), an increase of one in relative permittivity approximately indicates a 3–4% increase of water volume in concrete that is about the water volume increase observed after the block treatment in the water pool.

Amplitude inversion was also performed producing the result shown in Fig. 16.16. The attenuation map is strongly influenced by the scattering effects produced by metal reinforcements (Fig. 16.14). Nevertheless, if we

![Image of radar records and traveltime inversion](image-url)
16.14 Schematic representation of the reinforcement structure of the concrete blocks.

16.15 Velocity map from traveltome tomography on the concrete sample: wet block above (fresh water), dry block below.
look at the attenuation measured in the middle of each concrete block where the results are less affected by scattering effects we observe higher absorption on the wet block (upper part of Fig. 16.16) indicating the expected increase of conductivity. An average increase of about 6 dB m\(^{-1}\) (from 75 to 81 dB m\(^{-1}\)) is measured from dry to wet block.

To check the effect of salt water, the wet concrete block was returned to the previous relatively dry condition and then submerged in a salt-water container. The NaCl concentration was about 40 g l\(^{-1}\) to simulate a seawater condition. After treatment, the sample was rebuilt and the tomographic experiment repeated with the same scheme. As expected, the velocity map (Fig. 16.17) from traveltime tomography reproduced a result very similar to the previous one (Fig. 16.15) whereas amplitude tomography (Fig. 16.18) was able to detect a relative variation of salt concentration. Compared with Fig. 16.16, the new attenuation map presents the same behaviour caused by metal reinforcements. However, if we compare the attenuation values measured in the middle of each block, we observe a higher increase of attenuation from about 75 to 85 dB m\(^{-1}\), i.e. an average increase of about 10 dB m\(^{-1}\). The higher values observed after salt water treatment compared with fresh water treatment (85 versus 81 dB m\(^{-1}\)) are interpreted as the effect of the higher conductivity increase determined by salt water.

![Attenuation map from amplitude tomography on the concrete sample: wet block above (fresh water), dry block below.](image_url)
16.17 Velocity map from travelttime tomography on the concrete sample: wet block above (salt water), dry block below.

16.18 Attenuation map from amplitude tomography on the concrete sample: wet block above (salt water), dry block below.
16.11.2 Bridge column test

The real test was performed on one column of the bridge illustrated in Fig. 16.19. The bridge was built in 1994 after the collapse of the existing bridge during a river flood. Eight concrete columns with a diameter of 142 cm bear the metallic structure of the new bridge. At the time of the experiment, the sediments at the base of the selected column (Fig. 16.20) were emerging from water and the column was apparently dry. Nevertheless, a minor increase of moisture at the base of the column could be envisaged and the goal of the experiment was to check moisture variations with radar tomography.

Again, a pair of 1 GHz antennae was used to perform a vertical tomography at the base of the selected column by moving the antennae along 13 stations with a spacing of 15 cm. The 169 radar records are shown in Fig. 16.21. From left to right the TX position is moving up so that left fans are mostly caused by trajectories that travel at the base of the column. A very slight increase of traveltimes and a decrease of amplitudes are observed in the left fans as a possible result of higher moisture at the base of the column. Tomographic results confirm this impression. Figure 16.22 and Fig. 16.23 illustrate the velocity and the attenuation maps, respectively. A very gentle vertical gradient of velocity is observed. A more pronounced effect is shown by the attenuation map. Attenuation decreases moving up towards the top of the column.

![Bridge built in 1994 on eight concrete columns with diameter of 142 cm.](image)
16.20 Column selected for the tomographic test.

16.21 Full tomographic dataset recorded on the selected column.

with only one exception between 140 and 160 cm where a return to higher attenuation values is observed, probably caused by scattering effects produced by metal reinforcements. On the whole, the tomographic test validated the assumption of a vertical negative gradient of moisture from the base of the column.
16.22 Velocity map from traveltime tomography on the selected column.

16.23 Attenuation map from amplitude tomography on the selected column.
16.12 Hints on advanced algorithms

16.12.1 Frequency downshift method

The inability of amplitude tomography to separate absorption from scattering can create interpretation ambiguities. Attenuation maps can be also produced with the frequency downshift method (Giroux et al., 2007; Liu et al., 1998a, 1998b; Quan and Harris, 1997; Zanzi et al., 2001; Zanzi and Lualdi, 2002). This approach ignores amplitudes and is based on the frequency analysis of the data, more specifically on the observation of the frequency downshift effect. The assumption is that attenuation is proportional to frequency.

The parameter that is generally used to measure the frequency downshift effect is the spectrum centroid. The expression of the spectrum centroid $f_S$ is:

$$f_S = \frac{\int_B fS(f) df}{\int_B S(f) df}$$ [16.19]

where $f$ is frequency, $S(f)$ is the amplitude spectrum of the signal and the integrations are performed over the signal band $B$. If we assume a linear relation between attenuation and frequency:

$$\alpha = \alpha_0 f$$ [16.20]

we can demonstrate that the projection of $\alpha_0$ along a source–receiver raypath can be evaluated as:

$$\int_{ray} \alpha_0 dl = \frac{(f_S - f_R)}{\sigma_S^2}$$ [16.21]

where $f_S$ is the centroid of the spectrum of the signal generated by the source, $f_R$ is the spectrum centroid of the received signal, and $\sigma_S^2$ is the variance of the source spectrum defined as:

$$\sigma_S^2 = \frac{\int_B (f - f_S)^2 S(f) df}{\int_B S(f) df}$$ [16.22]

Equation [16.21] indicates that attenuation can be revealed by the frequency downshift effect measured as the difference between the spectrum centroids at the source and at the receiver. By backprojecting the downshift effect estimated on all the available data, we can obtain a distribution map of the attenuation parameter within the section. Equation [16.21] is strictly valid under the assumption that the source generates a gaussian spectrum.
However, different assumptions on the shape of the source spectrum still lead to similar relations such as:

$$\int \alpha_r dl \propto \frac{(f_S - f_R)}{B^2}$$  \[16.23\]

Because the different assumption only affects the proportionality factor between the downshift effect and the attenuation, we see that the gaussian assumption is not a requirement for the validity of the method.

On the whole, this procedure looks less rigorous than amplitude tomography because of the required assumptions. Thus, the reliability of quantitative results is probably lower. However, a qualitative interpretation of higher and lower attenuation areas is expected to be meaningful and benefits can be envisaged from a joint interpretation of attenuation maps produced by the two approaches. As an example, void is a low attenuation material for the electromagnetic wave. As a result, a void is not expected to generate a frequency downshift effect and the void will be mapped as a low attenuation area by the frequency method. On the other hand, we know that a void is sometimes observed as a high attenuation area by amplitude tomography because of scattering. Thus, the frequency downshift method looks promising to help the interpreter in separating scattering and absorption phenomena. However, the frequency downshift method is relatively young and more laboratory investigations are needed to consolidate these expectations.

16.12.2 Diffraction tomography

When the scale of the variations is similar to the wavelength, diffraction phenomena become predominant. In this instance, ray tomography techniques like TT and AT provide only a partial extraction of the information contained in the data. A full-wave approach is needed to overcome this limit. Diffraction tomography (DT) is one of the most rigorous full-wave imaging method (Valle et al., 1999, 2000). Both amplitude and phase information are simultaneously treated by DT and all the energy collected by the receiver can be properly processed, including the diffraction. Nevertheless, DT also requires some approximations to get a linear inversion problem. Born and Rytov approximations are the most common approaches to linearize the relationship between the measurements and the velocity perturbations. They are based on different weak scattering hypotheses. Under these hypotheses, DT performs the tomographic inversion one frequency at a time.

Because the full wavefield is inverted, in principle, this method is the most appropriate to pursue the maximum resolution allowed by the data wavelength. The main drawback of DT and the main reason that tends to
limit its popularity is that a delicate and accurate data pre-processing (windowing, Fourier transform, phase unwrapping) is needed to extract amplitude and phase of each component. As a result, the final image might be quite sensitive to the pre-processing quality. A complete mathematical discussion of this method is beyond the scope of this contribution. The reader can refer to Valle \textit{et al.} (2000) and to Zhou and Liu (2000) for further details, algorithms and examples.

Full-waveform inversion of tomographic data is also possible in time-domain through an iterative procedure based on a Finite Difference Time Domain (FDTD) algorithm. This approach is described by Ernst \textit{et al.} (2007a, 2007b) and Meles \textit{et al.} (2009).

16.13 Conclusions

Most concrete defects are strongly associated with water because water can be a vector of chlorides and CO$_2$, and because water plays a major role in activating chemical reactions, in reinforcement corrosion and in freezing/thawing cycles. Radar tomography is sensitive to permittivity and conductivity of the material and these parameters in concrete are strongly influenced by moisture and water salinity. Thus, traveltime tomography can produce velocity maps that can be interpreted in terms of moisture distribution. Traveltime tomography combined with amplitude tomography can also give indications of different chloride concentrations. Cavities are targets that can be detected by both traveltime and amplitude tomography because they affect both velocity and attenuation maps. Honeycombing is also expected to be indicated by an increase in velocity and a reduction in attenuation. The presence and inhomogeneous distribution of steel fibres can be potentially mapped provided that the proper antenna frequency is selected. As a result, radar tomography can find several interesting applications as a NDT for concrete investigations.

16.14 References


\textsc{balaysac j p, laurens s, arliguie g, ploix m a, breyse d, derobert x and piwakowski b} (2009), ‘Evaluation of concrete structures by combining non-
destructive testing methods (SENSO project), *Proceedings of NDTCE*’09, June 30–July 3, Nantes.


Active thermography for evaluation of reinforced concrete structures

C. MAIERHOFER, M. RÖLLIG and J. SCHLICHTING, BAM Federal Institute for Materials Research and Testing, Germany

Abstract: Infrared (IR) thermography, which encompasses the determination of the surface temperature of an object using an IR camera, is an imaging technology that is contactless and completely non-destructive. Its applications are classified into passive and active methods. By using passive thermography, differences in emissivity and, if a temperature gradient is present, differences in temperatures can be related to subsurface structures. If additional energy is induced into the structure by heating or cooling, the procedure is called active thermography. Active thermography methods enable structural investigations of building elements taking into account many different testing problems. In this chapter, the physical background, the equipment used, and the influences from environment and material properties are discussed. Several results of applications concerning the detection of subsurface defects are presented.

Key words: active thermography, pulse-phase thermography, emissivity, reinforced concrete, subsurface defects.

17.1 Introduction

With infrared (IR) thermography, the temperature distribution at the surface of any structural element can be recorded as an image, called a thermogram. For the recording of thermograms, no direct contact to the surface is required. The application of IR thermography in civil engineering is well known for the identification of heat losses in building envelopes. This includes the location of thermal bridges caused by structural elements or missing insulation, of air leakage, or of modified thermal properties owing to an increased presence of moisture. However, thermography is not only limited to passive usage. Active thermography methods allow structural investigations of building elements to consider many different testing problems. As advanced non-destructive testing (NDT) methods like radar, ultrasonic and sonic methods are mainly suited for the detection
and characterization of inhomogeneities deeper than 5 to 10 cm, active thermography closes the gap for testing the near surface region between the surface and a depth of 10 cm. Many safety-relevant cases of damage originate from defects that are close to the surface, e.g. voids and honeycombing in the top layer of reinforcement in concrete structures, delaminations of carbon-fibre-reinforced plastic (CFRP) laminates used for strengthening of concrete structures, delaminations of protective coating systems as well as surface and subsurface cracks. Therefore, active thermography is very well suited for condition assessment, damage evaluation and quality assurance of the built infrastructure.

With active thermography, the structure to be investigated is actively heated (or cooled) with an additional energy source, which is either applied at the same side as the IR camera or at the opposite side in transmission configuration. Owing to the resulting temperature differences, a non-stationary heat transfer is induced. Inhomogeneities of material properties, structural elements, voids and delaminations can be detected in the thermograms, if the thermal properties are different to the surrounding material. The difference between temperature distribution and its change as a function of time above non-defect regions and inhomogeneities includes information about the defect parameters like depth, lateral size and type of material. Sophisticated data analysis of temporal temperature data in time and frequency domains (e.g. pulse-phase thermography) affords the detection of flaws and inhomogeneities with high reliability. The combination of experimental data and numerical simulation enables the selection of optimum measurement parameters as well as of obtaining quantitative information from experimental results.

Concrete structures are usually inhomogeneous, containing a variety of materials (including cement, aggregates, pores, plaster, rebars, and steel fibres) with various thermal properties. Thus, active thermography is, in general, very well suited to assess the following testing problems in the near-surface region up to a depth of 10 cm concerning the investigation of thermal material properties, the location of structural inhomogeneities, and the detection of enhanced moisture directly at or close to the surface.

In this chapter, first the physical principle and the theoretical background of the method are described. An overview of the state-of-the-art of current developments and applications concerning active thermography in civil engineering including existing guidelines and recommendations is given subsequently. Requirements for typical experimental set-ups are described with further introduction to the different heating sources and IR cameras. Finally, an outlook provides information about future trends in technology, data processing and integration.

© Woodhead Publishing Limited, 2010
17.2 Physical principle and theoretical background

17.2.1 Theoretical background of heat transfer

The active approach of the thermographic investigation of structures implies a direct heating or cooling. Common heating techniques are either direct such as transient impulse, step impulse (long impulse) and periodic heating (lock-in technique for data analysis) or indirect such as mechanical vibration and electromagnetic induction. There are several further methods for thermal stimulation, but these are not relevant for current applications in civil engineering.

The heating process induces an intended non-stationary heat flux. Derived from the first law of thermodynamics by Fourier, this process of heat conduction can be described by the general equation of three-dimensional heat transfer in anisotropic media:

$$\frac{\partial}{\partial x} \left( \lambda_x \frac{\partial T}{\partial x} \right) + \frac{\partial}{\partial y} \left( \lambda_y \frac{\partial T}{\partial y} \right) + \frac{\partial}{\partial z} \left( \lambda_z \frac{\partial T}{\partial z} \right) + w(x, y, z) = \rho c \frac{\partial T}{\partial t} \quad [17.1]$$

where $T(x,y,z,t)$ is the temperature as a function of position and time; $\lambda_x$, $\lambda_y$, $\lambda_z$ is the thermal conductivity in different directions; $\rho$ is the density; $c$ is the specific heat capacity; and $w(x,y,z)$ is the internal heat source (here $w = 0$).

Thermal conductivity $\lambda$, density $\rho$ and specific heat capacity $c$ can be combined to give two different terms being specific for each material. The thermal diffusivity $\alpha$ describes how fast temperature differences are compensated in a material:

$$\alpha = \frac{\lambda}{\rho c} \quad [17.2]$$

The units of diffusivity are m$^2$ s$^{-1}$.

The thermal effusivity $e$ is a measure of how much heat is exchanged at the interface between two different materials:

$$e = \sqrt{\lambda \rho c} \quad [17.3]$$

The units of effusivity are Ws$^{0.5}$ K$^{-1}$ m$^{-2}$.

For estimating the detectability of defects and inhomogeneities in structures, the heat transfer can be described by the propagation of thermal waves (a harmonic heating process with a wavelike temperature field). In this model, the reflectivity of thermal waves at interfaces is determined by the differences of the effusivities of the contiguous materials. For the one-dimensional case, the reflection coefficient $R$ of a plane thermal wave for transmission from medium 1 to medium 2 is equal to:
\[ R_{12} = \frac{e_1 - e_2}{e_1 + e_2} = \frac{\sqrt{\rho_1 c_1 \lambda_1} - \sqrt{\rho_2 c_2 \lambda_2}}{\sqrt{\rho_1 c_1 \lambda_1} + \sqrt{\rho_2 c_2 \lambda_2}} \]  

[17.4]

Thus, for a successful detection of inhomogeneities, there must be a sufficient difference between the thermal properties of medium 1 and 2. The larger the difference of the effusivities of the two media, the higher the thermal signature of the inhomogeneities, e.g. the reflection coefficient at the concrete/air interface is about 100\%, whereas the reflection coefficient of concrete/steel is about –24\%.

Furthermore, thermal waves are strongly damped. The thermal diffusion length \( l \) represents the depth at which the amplitude of the thermal wave (temperature) is reduced to \( e^{-1} \) of its value at the surface:

\[ l = \frac{2\alpha}{\omega} \]  

[17.5]

where \( \omega \) is the angular frequency of the thermal wave. Typical values for brick and concrete are between 1 and 2 cm for a heating period of 15 min. However, in several cases, information from deeper layers (up to 10 cm) was also obtained.

The strong damping of thermal waves implies a very high dispersion of the phase velocity \( v \):

\[ v = \omega l = \sqrt{2\alpha \omega} \]  

[17.6]

When impulse heating is used instead of harmonic heating, equations [17.4] to [17.6] can only be used as a rough estimation.

### 17.2.2 Physical principle of temperature measurement

For recording the space-resolved temperature distribution at the surface of structures, the focal plane array (FPA) inside the IR camera detects the thermal radiation emitted by the surface. The total radiated power of a structure over the whole spectrum is described by the Stefan–Boltzmann law:

\[ M = \varepsilon \sigma T^4 \]  

[17.7]

where \( \varepsilon \) is the emissivity, \( \sigma \) is the Stefan–Boltzmann constant, and \( T \) is the temperature.

Usually, IR cameras are only sensitive for a distinct temperature interval, e.g. 3 to 5 \( \mu m \) (MWIR) or 8 to 12 \( \mu m \) (LWIR). In this instance, the detected radiated power has to be calculated by Planck’s law, which gives the spectral distribution of the flux at various temperatures.
The emissivity $\varepsilon$ is the ratio between the radiated power emitted by a general structure in relation to the power emitted by an ideal radiator (blackbody). For a blackbody, the radiated power is independent of material properties and depends only on its temperature. Therefore, the emissivity of a blackbody is always 1. The emissivity of real structures varies between 0 and 1 ($0 \leq \varepsilon \leq 1$) and depends more or less on the temperature, the wavelength, the polarization and the angle to the normal. This dependency is influenced by surface properties such as roughness and contamination. For each body, the directed spectral emissivity is equal to the directed spectral absorptivity (Kirchhoff’s law).

Therefore, the wavelength dependency of the emissivity has to be considered. Typical data for various building materials were presented by Walther and Gerber. Spectra of concrete, red brick and granite recorded by Baldridge et al. are depicted in Fig. 17.1. For concrete, in particular, the emissivity is higher than 0.9 in the relevant wavelength intervals of 3 to 5 $\mu$m (MWIR) and 8 to 12 $\mu$m (LWIR) and varies only slightly. Hence, concrete is well suited for thermographic investigations.

In addition to the influence of emissivity, active thermography measurements also suffer from interference from various other effects, namely direct reflections from other thermal radiators (owing to the emissivity of the structure being less than 1), indirect reflections, other non-stationary thermal influences of the environment such as airflow around the object and various material characteristics such as density, thermal conductivity and heat capacity. Techniques will be introduced to give a better separation of these effects by combining complementary measurements based on passive and active thermography.

![Emissivity as a function of wavelength for concrete, red brick and granite recorded at room temperature.](image)
17.3 State of the art

Until the 1980s, the limited sensitivity of the available instrumentation restrained the full development of thermography as a NDT technique. Then, rapid developments in electro-optical and signal-processing techniques led to more advanced IR imaging systems. Scanner cameras using a single detector with a thermal resolution of less than 100 mK already enabled the recording of temperature distribution with a high resolution of several micrometres. In the 1990s, focal plane arrays (FPA) were developed and the increase of array size, thermal resolution and image repetition frequency during the last ten years has accelerated the increase in applications in the areas of material testing, mechanical and civil engineering, power generation, aerospace, preventive maintenance, and protection of the environment. Adjusted to the area of application and to the environmental conditions, various IR cameras, mainly distinguished by the detector types, can be used. These detectors are characterized by features such as spectral range, spatial resolution, noise-equivalent temperature difference (NETD, thermal resolution), integration time, and long-term stability. The FPA detectors can be classified into two different types: quantum detectors and thermal detectors. For quantum detectors, the incident thermal radiation is converted directly into an electrical signal. Usually, these detectors have to be cooled by a Stirling cooler. Quantum detectors consist of semiconductors or semiconductor heterostructures and are limited by an upper wavelength, which is given by the size of the band-gap of the semiconductor used. The quantum efficiency is high for semiconductor materials with a direct band-gap, e.g. InSb or HgCdTe. With these detectors, commercial IR cameras with a full frame rate of up to 1 kHz for a FPA size of $256 \times 256$ were achieved. For lower frame rates, larger arrays ($1024 \times 1024$) can be used. The maximum thermal resolution (NETD) is less than 15 mK.

Thermal detectors can be operated without cooling at stabilized temperatures. The signal is generated indirectly by the incident thermal radiation heating up the surface of the detector. The higher temperature changes the material properties, e.g. the thermal conductivity of the material of a microbolometer array. Currently, microbolometers are made of vanadium oxide (VO$_x$) or amorphous silicon (a-Si). Microbolometers have a much lower thermal resolution, less thermal contrast and higher thermal crosstalk than quantum detectors. For new microbolometers, an NETD of about 30 mK can be obtained. The main advantages of IR cameras with microbolometer arrays are lower costs, lower weight and that they are not sensitive to vibration and shock. Therefore, these cameras are well suited for long-term use, e.g. for monitoring in
industrial production, or for use on building sites, e.g. for building inspection and diagnosis.

These new technologies afford the development and application of active thermography as a fast and reliable tool in many areas of NDT. It is well known for material testing in several branches of industry, where it is mainly used for the detection of defects and delaminations.

Applications of direct heating in civil engineering were started in the 1990s. Initially, qualitative detection of concrete deterioration and plaster delaminations as well as the investigation of external bonded CFRP sheets were successful. The method is also very useful for the determination of the built-in position of anchoring elements at curtain facades. On-site applications first focused on the location of delaminations in concrete and masonry bridges. Further applications used the sun as a natural heat source for inspections of bridge decks and of paving in general. For locating delaminations in bridge decks using sunlight, an ASTM standard, published in 1988 and reapproved in 1997, with the title Standard test method for detecting delaminations in bridge decks using infrared thermography exists.

As shown in recent publications, active thermography by inducing a non-stationary temperature distribution (e.g. impulse thermography) in combination with the analysis of temporal data in frequency domain (e.g. pulse-phase thermography) is very well suited for the visualization of inhomogeneities and defects close to the surface (up to a depth of 10 cm) of concrete as well as of historic masonry structures. Quantitative information about the lateral geometry of defects is gained from lateral temperature distribution. Much more difficult is the analysis of the defect depth, which might be performed semi-empirically (e.g. by determining the blind frequency in the phase images) or by a comparison of experimental data with the results of numerical simulations, as mentioned above. However, in this instance the main problems are manifold: the inaccessibility of several structures, the changing environmental conditions, the inhomogeneity of the investigated surfaces, and the relatively thick building structures in relation to the low thermal diffusivity of the building materials. Although many investigations of the various measurement problems occurring in civil engineering have been carried out, most of the hitherto presented are results of laboratory measurements. In the following sections, some on-site applications will be presented.

17.4 Experimental equipment and calibration

The experimental set-up for the application of active thermography in civil engineering usually consists of a thermal heating unit, an infrared camera, and a computer system, which enables digital data recording in real time.
The schematic principle of the method is shown in Fig. 17.2(a) and a typical example for such a system is shown in Fig. 17.2(b).

17.4.1 Heating sources

The optimal heating source will generate a homogeneous heat flow without any delay along the surface area of the structure under investigation. In reality, these conditions can only be approximated. Heating sources can be classified in relation to the three different heat transport processes: radiation, convection and conduction. An overview of different sources is given in Table 17.1.
In several instances, infrared radiators have proven to be the most suitable sources by being fast and efficient, and generating a homogeneous temperature increase. For this, the heating procedure is usually done dynamically by moving the radiators (either computer controlled or manually) an appropriate distance from the surface to obtain the best possible homogeneous heating. The heating time varies from several seconds up to 60 min. It must be considered that the surface temperature should not rise above 50 °C to avoid any damage. This temperature has to be even lower for sensitive surfaces as paintings and other coverings. For detecting structures that are very close to the surface (less than 1 cm), halogen lamps and flash lights can be used as radiation sources. Local space-resolved heating is possible, e.g. with a laser system, as demonstrated for the detection of cracks by Legrandjacques et al. In selected cases and under optimum conditions, sunlight can also be exploited.

For convective heating, it is easy to use a conventional electric fan heater with up to 2000 W. This is a very suitable tool for sensitive surfaces avoiding radiation in other wavelength intervals such as the infrared spectrum, and for curved surfaces. As contact units, electrical heating mats are an option, but are usually not applied in practice owing to the low energy insertion. Additionally, a direct contact to the surface is required. For generating a transient temperature decay at the surface, in principle also cooling of the surface can be applied. For on-site measurements, the heating source has to be placed close to the surface under investigation (10 to 20 cm) and should be moveable to obtain homogeneous heating. An appropriate power supply must be available.

### 17.4.2 Infrared camera

After heat intrusion, the cooling down process of the surface is observed with an IR camera. This camera and the related software for data acquisition and analysis should at least fulfil the following conditions:

<table>
<thead>
<tr>
<th>Radiation units</th>
<th>Convection units</th>
<th>Contact units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Infrared radiator</td>
<td>Fan heater</td>
<td>Heating mat</td>
</tr>
<tr>
<td>Flash light</td>
<td></td>
<td>Cold pack</td>
</tr>
<tr>
<td>Halogen lamp</td>
<td></td>
<td>Cooling and heating</td>
</tr>
<tr>
<td>Laser</td>
<td></td>
<td>with Peltier units</td>
</tr>
<tr>
<td>Use of daily sunlight</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Propane gas with electrolyte oven</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
• Spectral sensitivity in the area of atmospheric transmission (3 to 5 μm or 8 to 12 μm). Between 8 and 12 μm, the radiation intensity at the relevant temperatures is much higher, but between 3 and 5 μm, the sensitivity for the detection of temperature changes is higher.

• Thermal images should be recorded with a minimum frame rate of 5 to 10 Hz for monitoring fast cooling down processes (a line array or focal plane array consisting of several detectors will be preferred to a scanning system with only one detector enabling also higher frame rates).

• Minimum temperature resolution: 0.07 K or better.

• Spatial resolution ≤2 mrad.

• Although the most important information in active thermography is the change of infrared radiation of the surface, the camera should be temperature calibrated at least between −10 and 100 °C. This might be helpful for a better determination of the initial thermal condition of the structure and for the recording of environmental influences.

The camera should be positioned on a fixed tripod on a stable base at a distance of 2 m or more, depending on the size of the investigated area and on the spatial resolution of the camera/objective system. The required objective lens has to be selected (wide-angle, normal, telephoto). Examples for the field of view (FOV, the angular extent of a given scene that is imaged by a camera), instantaneous field of view (IFOV, the angle subtended by the geometrical projection of a single detector element), minimum focus distance, and maximum spatial resolution for various objectives of the SC 1000 Inframetrics IR camera (256 × 256 pixels) are given in Table 17.2.

17.4.3 Calibration

The camera should be switched on at least 30 min before starting data acquisition to ensure a stable temperature of the camera itself. An operational test should be performed by measuring the surface temperature of a homogeneous object with known emissivity (reference object, black body).

<table>
<thead>
<tr>
<th>Objective lens</th>
<th>Field of view (FOV)</th>
<th>Instantaneous field of view (IFOV)</th>
<th>Minimum focal distance (cm)</th>
<th>Spatial resolution at minimum distance (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal</td>
<td>17° × 16°</td>
<td>0.067°</td>
<td>25</td>
<td>0.3</td>
</tr>
<tr>
<td>Telephoto</td>
<td>8° × 8.5°</td>
<td>0.031°</td>
<td>70</td>
<td>0.4</td>
</tr>
<tr>
<td>Wide angle</td>
<td>32° × 34°</td>
<td>0.130°</td>
<td>50</td>
<td>1.1</td>
</tr>
</tbody>
</table>
The emissivity of the structure under investigation should be checked with contact thermometers on site or, in more detail and if sample extraction is possible, in the laboratory. A first thermogram of the surface to be investigated should be recorded to have a reference image in a more or less thermal equilibrium. Also a photograph from the area should be taken for analyzing surface inhomogeneities.

Additional direct or indirect radiation should be avoided. During the whole time range of data recording, it has to be ensured that nothing or nobody is passing the area between camera and observed surface. For image reconstruction, planned data fusion, and/or a required high accuracy of geometric data, an optical camera calibration as described by Rzeszotarski\textsuperscript{28} has to be performed.

### 17.5 Data processing

#### 17.5.1 Thermal contrast

For qualitative data analysis, thermograms of experimental data recorded at various time intervals after heating were selected. The grey or false colour values of the images were scaled to the minimum and maximum temperature in each image. Shallow defects have maximum thermal contrast after a short cooling-down time whereas deeper defects appear later. For quantitative analysis, transient curves [surface temperature as a function of time for each pixel \((i,j)\)] from areas above defects and above homogeneous bulk material were compared and difference curves were calculated as shown in Fig. 17.3. These difference curves usually have a maximum

![Graph showing temperature as a function of time](image_url)

17.3 Temperature as a function of time at the surface above a reference point and above a defect and the corresponding difference curve (experimental data).
\(\Delta T_{\text{max}}\) at a distinct time \(t_{\text{max}}\) that depends on the depth and size of the defects, on the heating time and on the thermal properties of bulk and defect material. For further data interpretation and for solving the inverse problem, a numerical simulation program was adapted and developed based on finite differences and the differential equation of Fourier. Further simulations have been performed by using a finite element program.

17.5.2 Pulse-phase thermography

A further type of impulse thermography is the so-called pulse-phase thermography (PPT), which combines the method of data acquisition of impulse thermography with the approach of frequency analysis also used in lock-in thermography. The stored data received during impulse thermography are analyzed in the frequency domain via fast Fourier transformations of the transient curves of each pixel in a series of thermal contrast images. Defects lead to changes in amplitude or phase of the corresponding images. The main advantage of PPT is the information included in the phase images, which are reported to be less influenced by surface IR and optical characteristics. That also means less sensitivity to non-uniform heating compared with the thermal contrast images of impulse thermography.

In this experiment, the heating pulse is represented as a square pulse, which can be described as superposition of different frequencies with varying amplitudes. The available energy is concentrated in the low frequencies. The pulse duration determines the frequency spectrum in such a way, that for longer pulses lower frequencies contain more energy and therefore more information. This is especially true for the long pulse durations necessary for the investigation of concrete of up to 60 min. The maximum frequency is determined by the acquisition rate, the minimum frequency is limited by the recording time. In practice, only the first images at low frequencies are of interest, since most of the energy is concentrated here. Higher frequencies exhibit a higher noise level.

As an example for PPT, in Fig. 17.4 thermograms and the respective amplitude and phase images of a concrete test specimen are depicted for various cooling times and frequencies, respectively. The concrete test specimen has a size of \(1.5 \text{ m} \times 1.5 \text{ m} \times 0.5 \text{ m}\) (width \(\times\) height \(\times\) depth) and includes eight polystyrene cubes simulating voids with sizes of \(20 \text{ cm} \times 20 \text{ cm} \times 10 \text{ cm}\) and \(10 \text{ cm} \times 10 \text{ cm} \times 10 \text{ cm}\) at depths between 1 and 8 cm. The specimen was heated up for 30 min and the cooling down was observed for 120 min with an image rate of 2 Hz. In the thermograms, it can be seen that the shallow voids appear directly after heating whereas the deeper voids can only be recognized later. In each image, the temperature is scaled to the respective minimum and maximum to obtain the best possible con-
In the phase images of this thermal sequence at the lowest frequency of 0.05 mHz, all voids can be clearly seen with a higher signal-to-noise ratio as in the thermograms. For higher frequencies, the deeper voids disappear. The voids with a tilted orientation in respect to the surface appear half black and half white. It can be deduced that one corner is much deeper than the other. This effect is in accordance with a better geometrical resolution of phase images. The comparison of depth slices of data recorded

17.4 Thermograms, amplitude images, phase images, and radar depth slices recorded at a concrete test specimen with polystyrene cubes simulating voids.
with radar (GPR, ground penetrating radar) at the same specimen clearly shows that the frequency of the phase images can be related to a depth scale.

17.5.3 Segmentation

For enhancing the thermal contrast, i.e. for amplification of inhomogeneities against the background, distinct threshold values in the grey scale images of the thermograms can be related to these inhomogeneities (segmentation). Therefore, a histogram is generated displaying the frequency of a distinct temperature value (grey value). For a bimodal distribution with two clear maxima (optimum condition), a temperature interval exists that is not included in the thermogram. In this instance, the threshold can be selected inside this interval and a perfect separation of inhomogeneity and background is possible. If these maxima cannot be separated clearly, several pixels will be allocated to the wrong segment. Further processing, e.g. with seeded region growing methods, can be applied. Nevertheless, the threshold still can be used to enhance the contrast.

Figure 17.5 shows the histogram of a thermogram recorded at a concrete test specimen (1.5 m × 1.5 m × 0.5 m) with voids simulated by polystyrene at depths between 2 and 8 cm. The test specimen was heated for various durations (5, 15, 30 and 60 min) and thermograms with maximum contrast were selected. As expected, the histograms become broader and lower with
increased heating time. In all histograms, two areas can be separated, although there is a large overlap. An example for the selection of different threshold values is shown in Fig. 17.6 for the thermogram recorded after a heating time of 30 min. Figure 17.6(a) has no threshold. Fig. 17.6(b) has a lower threshold value of 29.32 °C and Fig. 17.6(c) 30 °C. In the latter, the small voids nearly disappear.

Further image processing especially for enhancing the contrast of shallow defects and cracks is based on the application of the Sobel operator for enhanced edge detection. This operator is based on the first derivative of the spatial temperature distribution, which is high at the edges. Orthogonal to the direction of large gradients, the image is smoothed. An example for the application of the Sobel operator to crack detection is presented by Bzymek,31 and a further study is presented below.

17.5.4 Reconstruction

As mentioned above, quantitative characterization of hidden imperfections in materials is desired. In particular, defect depth, shape and material
properties are of interest. To obtain these parameters, the reconstruction of experimental data by solving the inverse problem is required.

In most studies, the full-width-at-half-maximum (FWHM) of a defect-related temperature distribution is used to determine the defect diameter.\textsuperscript{32} Lugin and Netzelmann produced an algorithm for the 2D/3D defect shape reconstruction for pulsed thermography by combining a simulation unit with a defect shape correction.\textsuperscript{33} The method was tested for experimental data obtained on plate-shaped steel samples. Another possible method of defect reconstruction is related to a Green’s function approach.\textsuperscript{34} In this method, some initial information has to be defined, e.g. thermal properties of sound material, propagation speed, and frequency spectrum of thermal waves.

Currently, for quantitative information concerning defect depth, the application of reference methods (as described above) or use of a reference test specimen is recommended.

17.5.5 Combination with other methods

In most instances, only one method is applied to solve a certain problem. To increase the reliability, for enhancing the accuracy of quantitative results, for unambiguous data interpretation and for taking advantage of different and, in some instances, complementary physical effects, it is useful to combine complementary methods. It has been shown that, amongst others, the following combinations are very promising:

- active thermography and radar for depth calibration as described above;
- active thermography and elastic waves, also for depth calibration;\textsuperscript{35}
- active thermography and shearography of thermal-loaded specimen enabling flaw detection by temperature distribution and surface displacement;\textsuperscript{36,37}
- various methods for thermal lock-in excitation by light and ultrasound;\textsuperscript{38} and
- active thermography and digital imaging as described below.

Using the latter combination for the detection of voids below a stone pavement consisting of granite tiles 1.5-cm thick, laid on mortar on a concrete floor, an area of 40 m\textsuperscript{2} was investigated. Single areas of about 1 m\textsuperscript{2} were heated up with an infrared radiator for about 3 min. The cooling down behaviour was recorded with the IR camera. The voids could be easily detected with a temperature difference of about 2.5 K. In Fig. 17.7, the superpositioning of the photogrammetric equalized digital photo as well as of the equalized thermogram recorded 1.33 min after switching off the heating source is displayed. From Fig. 17.7, the position of the joints can be clearly derived and the main voids, represented as light areas in the
thermogram, are seen to appear at the edges and sometimes also along the joints of the tiles.

17.6 Areas of applications

17.6.1 Investigation of material properties

For the investigation of the influence of bulk material properties, three concrete test specimens (no. 1 to 3) with a size of 1 m × 1 m × 0.3 m were constructed. A grain size distribution curve of A/B 16 and a water/cement ratio of 0.6 were realized for all mixtures. Further properties such as air content in fresh concrete, compressive strength, and density determined for 28-day-old cubes are given in Table 17.3.

The three test specimens contained four voids with a size of 10 cm × 10 cm × 5 cm at depths of 6 and 10 cm. Two voids were simulated by inclusion of polystyrene cuboids and two by integration of pieces of gas concrete parts. Test specimen no. 1 (NC) was made of normal (reference) concrete together with ten cubes and was investigated during hydration (3, 7, 28, 56, 180 and 360 days after concreting) with active thermography. Parallel to each measurement, one of the concrete cubes was tested by conventional destructive investigation methods. In test specimen no. 2, the larger aggregates were replaced by porous aggregates in normal cement (PAC). This leads to lower density and compressive strength. Air-entraining agents were put into the cement mixture of no. 3 (AAC) resulting in low density and compressive strength as shown in Table 17.3. Except for test specimen no. 1, the measurements were performed more than one year after concreting.
The heating up was performed for about 30 min, and the cooling down was observed for 120 min.

For quantitative data analysis, transient curves (surface temperature as a function of time for each pixel) from areas above voids and above homogeneous material were compared and the difference curves were analyzed.

For test specimen no. 1, it is expected that the thermal properties of the bulk material change during hydration (which can be quantified by time or compressive strength) whereas the free water is transformed into chemically bonded water. Therefore, in Fig. 17.8(a), $\Delta T_{\text{max}}$ and $t_{\text{max}}$ of void 2 (polystyrene at a depth of 6 cm) are displayed as a function of compressive strength (determined for sample cubes) for a heating duration of 30 min. With increasing strength, the contrast decreases whereas the time of the maximum contrast increases. This is consistent with a decrease of thermal conductivity as outlined by the results of a numerical simulation.\(^{39}\)

The porous aggregates in specimen no. 2 (PAC) as well as the air-entraining agents in specimen no. 3 (AAC) reduce the density, and it is expected that the pores reduce the thermal conductivity of the material. As shown previously,\(^{39}\) the expected reduction of thermal conductivity should lead to a small decrease of $\Delta T_{\text{max}}$ and to a larger increase of $t_{\text{max}}$, whereas the reduction of the density has a larger influence on $\Delta T_{\text{max}}$ and a smaller influence on $t_{\text{max}}$. Temperature transient difference curves of test specimen no. 1 (one year after concreting) and test specimen no. 2 (PAC) are presented in Fig. 17.8(b).

\[\begin{array}{cccccc}
\text{Type of concrete} & \text{Pore content in fresh concrete (vol\%)} & \text{Density (kg dm}^{-3}\text{)} & \text{Compressive strength of cubes (N mm}^{-2}\text{)} & t_{\text{max}} (\text{s}) & \Delta T_{\text{max}} (\text{K}) \\
\text{Normal concrete (NC), (1)} & 0.9 \pm 0.2 & 2.33 \pm 0.03 & 48.5 \pm 2.0 & 900 \pm 50 & 1.72 \pm 0.08 \\
\text{Concrete with porous aggregates (PAC), (2)} & 6.0 \pm 0.2 & 1.85 \pm 0.03 & 27.4 \pm 2.0 & 1290 \pm 70 & 2.18 \pm 0.08 \\
\text{Concrete with air-entraining agent (AAC), (3)} & 3.9 \pm 0.2 & 2.28 \pm 0.03 & 37.6 \pm 2.0 & 880 \pm 50 & 2.57 \pm 0.08 \\
\end{array}\]
In Table 17.3, the properties of the various types of concrete, that cause the characteristics of these transient difference curves (specified by $t_{\text{max}}$ and $\Delta T_{\text{max}}$) of void 2, are listed. For specimen no. 2 (PAC), $\Delta T_{\text{max}}$ and $t_{\text{max}}$ increase clearly. Here, the effects of increased density and increased thermal conductivity are superimposed on each other. For specimen no. 3 (AAC),

17.8 (a) Maximum temperature contrast $\Delta T_{\text{max}}$ and time $t_{\text{max}}$ at which this maximum contrast appears for void 2 as a function of compression strength of cubes determined during hydration of test specimen no. 1 (heating duration: 30 min). (b) Temperature transient curves for void 2 for test specimen no. 1 (NC) and no. 2 (PAC) recorded 1 year after concreting and after a heating time of 30 min.
ΔT_{\text{max}} increases clearly, whereas t_{\text{max}} slightly decreases (within measurement accuracy). Therefore, the reduction of density and not the reduction of thermal conductivity have a main influence on the data here.

17.6.2 Location of voids in industrial pre-cast concrete beams

In the following, results are presented concerning the applications of active thermography for the location of voids and honeycombing in industrial pre-cast concrete beams of an underground car park. After the assembly of the beams, visual inspection, impact tests, and destructive openings have shown a large amount of voids and honeycombing owing to bad compaction of concrete around the reinforcement bars (Fig. 17.9(a)). These areas were partly refilled, but there were several voids left. For a comprehensive registration of all voids, active thermography was applied after a detailed pilot survey and optimization of the method as shown in Fig. 17.9(b). Two infra-red radiators were applied and an area of 1 m² was heated for 15 min. The cooling-down behaviour was observed for 1 h. Afterwards, a PPT data analysis was carried out. In Fig. 17.10(b), amplitude and phase images are displayed. In particular, in the amplitude image calculated at a frequency of $2.77 \times 10^{-4}$ Hz, the voids are clearly visible as light areas. Additionally, some rebars (dark lines) and six reinforcing bar spacers (light dots) can be recognized. For the area-wide investigation of all beams, the measurement time was optimized by recording of single thermograms at maximum temperature contrast ($t_{\text{max}}$).

17.6.3 Plaster on concrete

For the investigation of plaster delaminations, a concrete test specimen (1.5 m × 1.5 m × 0.5 m) was covered with three different thicknesses of plaster: 10 mm (bottom left quarter), 15 mm (top half) and 20 mm (bottom right). Delaminations were simulated by 10 cm × 10 cm large pieces of paper at the interface between concrete and plaster. The test specimen was heated up by infrared radiators for about 6 min and the cooling down was observed for 30 min.

In Fig. 17.11, three thermograms are displayed after different cooling-down times. Each thermogram was scaled to minimum and maximum temperature values. First, the voids below 10 and 15 mm plaster appear; later on also the void below 20 mm plaster is visible. Additionally, it can be observed that the surface temperatures are increasing for increasing plaster layer thickness. This can be explained by the different thermal properties of the plaster and the underlying concrete.
17.6.4 Carbon-fibre-reinforced plastic (CFRP) on concrete

Adhesive bond defects between carbon-fibre-reinforced plastic (CRFP) laminates and concrete can be easily detected using a short active heating time of 15 s with IR radiators as shown in Fig. 17.12. In addition, flashlights can be applied.41

17.6.5 Characterization of cracks

Crack detection in concrete can be easily performed by using water for contrast enhancement. The surfaces to be investigated can either be watered...
from the surface or from the back. In the latter instance, water appears at the surface by capillary moisture transport through the crack. As water has a higher emissivity than concrete, the crack can be detected by an infrared camera directly or, with better contrast, through the reflection of additional infrared radiation, which is projected onto the surface.

In the following example, a concrete test specimen with a natural crack through the whole cross section is watered from the back side for several hours and thermograms of the surface are recorded at distinct time intervals.

In Fig. 17.13, the respective thermograms within the first hour are displayed. The watering of the already existing crack can be clearly observed as a ‘growing crack’. The moist crack and moisture appears with a lower temperature as the water is cooler than the concrete surface.

For automatic crack detection in the thermograms, the following steps based on sliding-neighbourhood operations were applied. First, a Sobel filter is applied and, by selecting a threshold value, all information is reduced to a binary mask, Fig. 17.14(a). The noise from the thermogram becomes
17.11 Thermograms of an image sequence during cooling down (a) for 100 s, 26.7 to 40.0 °C; (b) for 400 s, 24.5 to 33.1 °C; (c) for 800 s, 23.7 to 29.2 °C, after 6 min of heating. Different thicknesses of plaster, 10 mm (bottom left), 15 mm (top half) and 20 mm (bottom right), were investigated. Low temperature values are black, high values are white.

17.12 (a) Test specimen with carbon-fibre-reinforced plastic laminates that have no contact at the hatched areas (nominal glue thickness is given to the right). (b) Thermogram directly recorded after a heating time of 15 s using an infrared radiator.
more visible. As the crack has a connected signature, the surroundings of each pixel can be analyzed for the selection of relevant indications. In a second step Fig. 17.14(b), all white pixels (value of 1) with less than two white neighbours are set to black (value of 0). Afterwards in a third step, all pixels were set to white, which fulfils the condition that inside a $5 \times 5$ matrix, a minimum of 5 white pixels is available Fig. 17.14(c). As larger connected structures have to be detected, at least in a fourth step, all pixels were set to black if they did not have at least 50 neighbours inside a $15 \times 15$ matrix. After this last operation, the remaining noise has been suppressed Fig. 17.14(d).

After applying these single steps by batch processing to several thermograms at various regular time intervals (in this instance, 6), the temporal capillary moisture distribution through the crack appearing at the surface can be reconstructed as shown in Fig. 17.15.

These algorithms can also be applied to other problems, although they have to be adapted to the size of the individual inhomogeneities to be detected. Batch processing enables automatic and fast data analysis.

17.6.6 Limits and requirements

The limits of the application of active thermography depend on the testing problem, the object under investigation, and the used equipment. For a
successful measurement campaign it should be ensured that the surface of the object under investigation is accessible and that the surface structure is more or less homogeneous having a defined high emissivity and thus a low reflectivity. Direct or indirect (reflected) radiation from additional sources (e.g. the sun) should be avoided. Wind strength and dust also have an influence on the data.

The detectability of inhomogeneities in the near-surface region depends on the thermal properties, on the depth, and on the size of the inclusions or defects. Under optimum conditions, e.g. a large void in homogeneous concrete, the void can be detected up to a depth of 10 cm. Delaminations behind plaster can be detected for plaster thicknesses larger than 2 cm. For these cases, the minimum lateral size (planar extension) of the void should be equal to or larger than the coverage. It must be remembered that moisture leads to an increase of thermal contrast, but also to a reduction of the

\[ 17.14 \ (a) \text{ First step: Sobel filtering and binary mask; (b) rejecting of white pixels with less than two white neighbours; (c) Pixels set to white if they fulfil the condition that inside a } 5 \times 5 \text{ matrix, a minimum of five white pixels are available; (d) pixels set to black if they do not have at least 50 white neighbours inside a } 15 \times 15 \text{ matrix.} \]
penetration depth of active thermography. Although enhanced moisture content close to the surface can be detected for delaminations or voids filled with water as shown previously, systematic studies for quantification have not yet been carried out.

The maximum geometric resolution in the plane parallel to the surface is given by the IR camera and the used objective and can be taken from the manual of the camera. The lateral accuracy in detecting the boundaries of inhomogeneities or defects depends strongly, however, on the thermal properties and the defect depth (the shape of the boundaries blurs with increasing depth).

Only qualitative information about the depth of inhomogeneities or defects is accessible. The depth of the inhomogeneities has an influence on the maximum contrast in the difference curves and on the frequency where the inhomogeneities appear and disappear in the phase images of PPT. Quantitative results can only be obtained from experimental data by comparison with calibration curves, numerical simulation, reference specimens, or reference methods. This is only possible if the thermal properties of the sound concrete and of the defect are known, or if the geometry of the structure is known.

The temperature resolution and the absolute accuracy of measurement of the temperature depend on the IR camera used (as given in the camera
manual) and on the accuracy and uniqueness of the assumed emissivity of the investigated object.

### 17.7 Future trends

Compared with other NDT methods which are already well established in industry, active thermography is a relatively new method. Owing to recent developments of advanced IR cameras as well as in computer technology, the possibilities of manifold applications of active thermography have increased considerably. Although currently there have been only a few applications in civil engineering, there is a high potential for a number of testing challenges to be successfully solved. Further developments will thus focus on feasibility studies as well as on automation for enhancing the efficiency of testing performance. Main areas of applications are likely to be preventive maintenance such as periodic monitoring for damage prevention, quality assurance during construction and after reconstruction and repair, building diagnosis and damage evaluation. In the course of the implementation of new energy-saving regulations, passive and active thermography will play an increasing role.

Further developments will include interfaces to optical methods. For the investigation of building surfaces, the combination with photogrammetry and, in the frame of 3D assessment of building geometry, with 3D laser scanners is applicable.

As previously mentioned, active thermography can be combined and fused with several other NDT and minor destructive methods as well as with integrated sensors. Here, it is required to know the advantages and limits, e.g. probability of detection (POD) and receiver operating characteristics (ROC), of the individual methods for selecting adequate complementary methods, which enhance the reliability as well as measurement accuracy.

Further developments of equipment are expected in the following topics:

- uncooled FPAs with enhanced temperature resolution,
- multispectral FPAs with several narrow bands for the recording of separated thermograms at various wavelengths at the same time and position,
- larger FPAs with higher frame rate, and
- stereo methods for 3D-recording of the distribution of surface temperature.

The development of software packages for data analysis contain e.g. automatic defect detection, data fusion, numerical simulation of 3D heat transport and reconstruction of thermal material properties.
17.8 Guidelines and sources of further information and advice

A good introduction to and general overview of active thermography, including equipment, data analysis and applications, can be found in the work of Maldague. More information about new technological developments and special applications are published in the proceedings of the following conference series: International conference quantitative infrared thermography (QIRT, http://qirt.gel.ulaval.ca/), International workshop on advanced infrared technology and application (AITA), ThermoSense – the annual infrared thermography applications conference (SPIE conference, www.thermosense.org). For applications in civil engineering, the following conference proceedings are recommended: Non-destructive testing in civil engineering (NDT-CE). Networks are e.g. the NDT database & journal (www.ndt.net, very good for literature research), the Fraunhofer Alliance (www.vision.fraunhofer.de), the North-American network for thermography of buildings (www.buildscanir.com) and some national networks such as www.vath.de in Germany and www.thermografie.co.at in Austria. Several interesting links can be found on the website of the British Institute of Non-Destructive Testing (www.bindt.org) and the European Federation for Non-Destructive Testing (www.efndt.org).

An overview of national and international standards, guidelines and recommendations is given in Table 17.4. In the light of the current state-of-the-art for the application of the above-mentioned methods in industry as well as in research and development, standards and guidelines are urgently required. At the moment, only a few standards for applications of active and passive thermography are available. In addition, there is a high demand for reference materials and test samples, and for round-robin tests enabling quantitative evaluation of different procedures for performing active thermography.

The European standard EN 473 Qualification and certification of NDT personnel – general principles provides a uniform level of qualifications of the personnel, which is acknowledged and accepted. Often it is demanded by authorities and users of NDT in industrial areas in European countries such as Germany. DIN 54190-1 to 3 Non-destructive testing – thermographic testing describes the general principles, equipment and terms and definition of thermographic testing applied to NDT. Qualification and certification of personnel for thermographic testing is included in DIN 54162 as well as in DIN EN 13187. The latter is related to thermography in civil engineering.

Standards for special applications have been developed by the American Society for Testing and Materials (ASTM): for the practice for thermographic inspection of insulation installations in envelope cavities of frame
Table 17.4 Selection of national (German) and international standards and guidelines concerning active thermography

<table>
<thead>
<tr>
<th>Organization</th>
<th>Standards, guidelines and recommendations</th>
</tr>
</thead>
<tbody>
<tr>
<td>DIN 54190-1, Edition: 2004-08, Non-destructive testing – Thermographic testing – Part 1: General principles</td>
<td></td>
</tr>
<tr>
<td>DIN 54190-3, Edition: 2006-02, Non-destructive testing – Thermographic testing – Part 3: Terms and definitions</td>
<td></td>
</tr>
<tr>
<td>DIN 54162, Edition: 2006-09, Non-destructive testing – Qualification and certification of personnel for thermographic testing – General and special principles for level 1, 2 and 3</td>
<td></td>
</tr>
<tr>
<td>DGZfP</td>
<td><strong>DGZfP-B 5</strong>, Edition: 1993-10, Guideline for thermographic investigations at building elements and building structures (in German)</td>
</tr>
<tr>
<td>DGZfP-TH 1, Edition: 1999-03, Characterisation of thermography systems (in German)</td>
<td></td>
</tr>
<tr>
<td>VDI/VDE</td>
<td><strong>VDI/VDE 3511 Blatt 4</strong>, Edition: 1995-01, Temperature measurement in industry – Radiation thermometers (German, English)</td>
</tr>
<tr>
<td>VDI/VDE 3511 Blatt 4.1, Edition: 2001-06 Temperature measurement in industry – Specification for radiation thermometers (German, English)</td>
<td></td>
</tr>
</tbody>
</table>
buildings, for in situ measurement of heat flux and temperature on building envelope components, for location of wet insulation in roofing systems, and a standard test method for detecting delaminations in bridge decks. Furthermore, there are standards for the characterization of infrared thermography systems and the environmental influences on IR radiometers.

The German Society for Non-destructive Testing (www.dgzfp.de) has published guidelines on the characterization of infrared thermography systems and on thermal investigation of buildings.

The Association of German Engineers (VDI, www.vdi.de) is today the largest engineering association in Western Europe and has published guidelines for supporting the industry with basic information, suggestions and support for the efficient use and application of infrared thermography technology.

The RILEM (Réunion Internationale des Laboratoires et Experts des Matériaux, systèmes de construction et ouvrages) Technical Committees TC SAM (Strategies for the assessment of historic masonry structures with NDT) and TC 207 INR (Interpretation of NDT results and assessment of RC structures) are currently developing guidelines for the application of active thermography in civil engineering (www.rilem.org).

17.9 References

Non-destructive evaluation of reinforced concrete structures

10 WALThER, L. and GERBER, D., Infrarotmesstechnik (Berlin: Technik Verlag, 1983).
25 ARNDT, R., ‘Rechteckimpuls-Thermografie für die qualitative und quantitative zerstörungsfreie


43 Maldague, Xavier, P. V., Nondestructive evaluation of materials by infrared thermography (Springer Verlag, 1993).
Abstract: The historical development of one-sided access nuclear magnetic resonance (OSA–NMR), a unique tool for non-destructive measurement of moisture-related properties in concrete is reviewed. The physical background is described. Applications of OSA–NMR, such as the determination of moisture content profiles, liquid transport coefficients, and early-age hardening in concrete are discussed and the limitations of the method are outlined.

Key words: nuclear magnetic resonance (NMR), one-sided access (OSA), moisture content, hardening, concrete, non-destructive testing.

18.1 Introduction

Since its discovery in 1946, nuclear magnetic resonance (NMR) has evolved from a scientific curiosity into one of the most powerful spectroscopic techniques in scientific research, used in chemical and biochemical research by default, and also in medical diagnostics, physics and material science.¹

Very soon after its discovery, researchers started to use NMR for studying the hydration of cement-based materials, such as mortar and concrete. In 1955, researchers observed that it was possible to distinguish between bound (crystal) water and free water in a hardening cement paste, based on the line width of the NMR signal.² A few years later, this approach was refined in order to characterize the development of water mobility in hardening cement.³

Differences in water mobility also allow tumors and normal tissue to be distinguished by NMR, as reported in 1971 by Damadian.⁴ He suggested that these differences could be used to diagnose cancer. Even though it later turned out that these differences were too variable for diagnostic purposes, the in vivo diagnosis of human diseases like cancer has become the most common application of magnetic resonance imaging (MRI). The MRI method of generating 2D- and 3D-resolved NMR images, was developed in the early 1970s by Paul Lauterbur.⁵
Only a few years later, this new technique also began to be used for investigating porous building materials. In 1979, Gummerson and his co-workers reported the first use of an MRI technique to monitor the dynamics of the internal water content distribution in several porous inorganic materials during capillary inflow.\(^6\)

NMR and MRI equipment is usually associated with the idea of large, expensive and sophisticated instrumentation, demanding a detailed expert knowledge for its operation. Therefore, they are often regarded as typical scientific laboratory techniques. Conventional NMR equipment requires the insertion of a small sample into the measuring device. For a large sample, e.g. a bridge deck or a building wall, it is necessary to cut out a sample. Therefore, traditional NMR instrumentation, even though it is non-destructive in principle, cannot be used without damaging the sample.

However, that is only half the story. Even in the early 1980s, the idea of ‘inside-out’ NMR devices had been developed, which allowed the investigation of objects outside the NMR apparatus. In 1980, Jackson patented his idea of NMR apparatus that could be lowered into an exploration well in order to characterize subsoil earth formations with regard to porosity, oil/water saturation, pore-size distribution and permeability.\(^7\) Today, these so-called NMR logging tools are commonly used during oil exploration. In 1981, employees of SwRI (Southwest Research Institute, San Antonio, USA) adapted this ‘inside-out NMR’ method for other applications. This work resulted in the first prototype of NMR apparatus for a one-sided access (OSA) technique. The main application of this device was for on-site moisture determination in concrete bridge decks.\(^8\) Because it was equipped with a large electromagnet, the serviceability of this device was limited owing to its great weight and the need for an auxiliary power supply for the electromagnet.

Later, OSA–NMR prototypes based on permanent magnets were developed. Since then, several other systems have been reported, e.g. the NMR-INSPECT from Fraunhofer IZFP, the NMR-MOUSE (mobile universal surface explorer) from RWTH and the NMR-MOLE (mobile lateral explorer) from Fraunhofer IBMT.\(^9\)–\(^11\)

To sum up the current situation: the enormous potential of NMR methods for the characterization of concrete and mortar, as described in many papers and books, is unquestionable.\(^12\) Devices for applying NMR directly to the component have, in principle, been available for several years.\(^13\) But NMR is still far from being a standard method for non-destructive concrete inspection.

### 18.2 Physical background

A full treatment of the physical background is beyond the scope of this text, but has been covered exhaustively elsewhere.\(^14\)\(^15\) The basic physical concept
underlying NMR is the simple fact that a moving electrical charge produces a magnetic field and vice versa. The angular momentum or spin is a property of all atomic nuclei having an odd number of protons or neutrons. As atomic nuclei are charged, the spinning motion causes a magnetic dipole in the direction of the spin axis, and the intrinsic magnitude of this dipole is a fundamental nuclear property called the nuclear magnetic moment, $\mu$. Hydrogen nuclei ($^1\text{H}$), possessing the strongest magnetic moment, are fortunately present in great abundance in moist material. Consequently, this isotope is used for NMR moisture measurements. Furthermore, a variety of other nuclei are routinely observed such as $^2\text{H}$, $^7\text{Li}$, $^{11}\text{B}$, $^{13}\text{C}$, $^{14}\text{N}$, $^{15}\text{N}$, $^{17}\text{O}$, $^{19}\text{F}$, $^{23}\text{Na}$, $^{27}\text{Al}$, $^{29}\text{Si}$, $^{31}\text{P}$, some of which are also used for studying cement-based materials.

For a large number, $N_0$, of $^1\text{H}$ nuclei, as found in an aqueous material (see Fig. 18.1a), in the absence of an external magnetic field, the magnetic moments have random orientations. However, if an external static magnetic field, usually labelled $B_0$, is applied, the magnetic moments tend to align with the field direction like small compass needles (see Fig. 18.1b). However, unlike compass needles, the magnetic moments may align either with the field or against it, according to the laws of quantum mechanics.

The energy state of the parallel orientation is slightly overbalanced compared with the anti-parallel orientation, leading to an excess population $\Delta N$, which provides a detectable macroscopic magnetization. Lowering the sample temperature and increasing the magnetic field strength usually leads to higher values of $\Delta N$, which is why NMR spectrometers are often equipped with extremely strong magnets (superconducting magnets).

Each magnetic dipole precesses around the $B_0$ direction with a characteristic frequency, called the Larmor frequency $\omega_0$, which is directly proportional to the strength of the magnetic field.

$$\omega_0 = \gamma B_0.$$ [18.1]

where the proportional constant $\gamma$, called the gyromagnetic ratio, is dependent on the type of nuclei ($2 \pi \, 42.58 \text{ MHz T}^{-1}$ for $^1\text{H}$).

18.1 Schematic description of the physical principle of NMR: (a) $^1\text{H}$ nuclei, (b) alignment of magnetic moments, (c) applied radiofrequency energy and (d) magnetic dipoles absorbing and emitting energy.
In order to detect an NMR signal, radiofrequency (RF) energy must be applied at exactly the Larmor frequency. This RF field is indicated by the amplitude of its magnetic field vector $B_1$, as shown in Fig. 18.1c. The magnetic dipoles will be induced to resonantly absorb and then emit energy (Fig. 18.1d).

The simplest NMR experiment is to apply a single pulse of the RF field. During the RF pulse, the net magnetization formed by the superposition of the magnetic dipoles gradually rotates about the direction of the field vector $B_1$. A so-called 90° pulse rotates the magnetization by an angle of 90°. Then, an NMR signal can be detected, which is called free induction decay, abbreviated to FID (see Fig. 18.2a). The FID signal progressively decays with time and, in some instances, it is only detectable for a few microseconds after terminating the pulse. During and shortly after the intense RF pulse, the NMR receiver electronics are saturated and signal detection is not possible. Therefore, a fast-decaying FID is not always detectable. Fortunately, a second NMR signal can be generated by applying a second RF pulse, in this instance a 180° pulse, $t_e$ seconds after the first one. This results in a so-called spin echo signal, which can be detected $t_e$ seconds after the 180° pulse (see Fig. 18.2b).

The amplitude $A$ of the emitted $^1$H-NMR signal is a direct measure of the hydrogen density in the material (see Fig. 18.2). Therefore, measuring this amplitude is a simple way of directly determining the moisture content in a wide range of materials. The functional dependence between the NMR signal and moisture content is linear. The same calibration curve can often be used for different materials.

$$S = S_0[1 - \exp(-t/T_1)]\exp(-t/T_2)$$

$18.2$ Schematic presentation of two typical NMR signals: (a) free induction decay (FID) and (b) spin echo; described by the formulae $S = S_0[1 - \exp(-t/T_1)]\exp(-t/T_2)$ and $S = S_0[1 - \exp(-t/T_1)]\exp(-t/T_2)$, respectively.
Different physical states, such as solid, liquid, or gaseous, adsorbed onto a solid surface and chemically combined, can be characterized qualitatively and quantitatively by means of specific time characteristics of the NMR signal, known as relaxation times $T_1$ and $T_2$ (see Fig. 18.2). In a mixture of components that have different values of $T_1$ ($T_2$), the relaxation signal curve is composed of overlapping exponential curves representing the individual components. By separating the measuring curve into single exponential curves, the relative hydrogen content of each component can be determined. For example, this relaxation analysis is routinely used in the food industry to determine the fat/water content of margarine. Pure liquid water at room temperature shows values for $T_1$ and $T_2$ in the range of a few seconds. If a water molecule is chemically combined or is in close proximity to a surface, the relaxation times change drastically.

For example, the hydrogen content of water in concrete has a range of $T_2$ values from $10^{-2}$ s for capillary pores, $10^{-4}$ s for gel pores, and up to $10^{-5}$ s for water of hydration. This relationship is used to obtain detailed analysis of the hardening (hydration) process of early-age concrete. In general, water within a confining geometry, such as a pore, shows a characteristic value for $T_1$ ($T_2$), that is proportional to the relation between the inner surface and the volume (surface-to-volume ratio, SV), which, in turn, is a property of the pore geometry. Water-filled porous materials are often characterized by a broad distribution of $T_1$ ($T_2$) values, corresponding to the pore-size distribution. Hence, analysis of NMR relaxation times is a precise method for quantitative determination of pore-size distribution.

The proportionality between the magnetic field strength and Larmor frequency, described in equation [18.1], can be used to spatially encode NMR signals, which is the principle of MRI. In a spatially varying magnetic field, each volume element is characterized by a specific Larmor frequency, according to the local magnetic field. Therefore, the frequency spectrum of the received signal can be directly translated into a corresponding spatial signal distribution.

### 18.3 Nuclear magnetic resonance (NMR) hardware

The main parts of an NMR spectrometer are the magnet, the probe coil, and the electronic control unit. The NMR magnet is one of the most expensive components, for which electromagnets made of superconducting wire, resistive electromagnets, and permanent magnets are commonly used. In conventional NMR, the sample is placed in a probe coil inside the magnet geometry (within the magnet coil or between the poles of the magnet) in order to ensure the sample sits in the homogeneous focal point of the magnetic field (see Fig. 18.3a). For MRI, additional gradient coils are used for applying the magnetic field gradient.
Because the sample has to be inserted into the enclosing apparatus, the maximum volume of the sample is restricted and the apparatus is often huge in dimensions and weight.

In OSA–NMR, the stray field of a U-shaped magnet assembly is used to obtain a $B_0$ distribution outside the NMR (see Fig. 18.3b). The required RF field with amplitude $B_1$ is generated by a flat surface coil. Owing to the inhomogeneous distribution of $B_0$ and $B_1$, only the nuclei in a small sensitive volume (SV) will be excited. This typically disk-shaped SV is located at a definite distance $x$ from the NMR probe-head surface, where $B_0(x)$ corresponds to the applied carrier frequency $\omega$ of the RF field. For a given depth-dependent $B_0$ distribution, the measuring depth $x$ can be varied by simply changing the frequency $\omega$. Alternatively $x$ can be varied at constant frequency $\omega$ by changing the $B_0$ distribution within the inspected material, either by using a variable (electro) magnet or by changing the lift-off between the material and NMR probe-head. The maximum measuring depth $x_{\text{max}}$ is always restricted. Small hand-held probe heads can reach a few millimeters at most. With larger, mechanically supported probe-heads, it is possible to measure several centimeters.

At Fraunhofer IZFP, NMR-INSPECT, the world’s first completely portable, battery-powered measuring unit based on the OSA–NMR principle was developed. This equipment was intended especially for on-site application in terms of diagnosing the moisture situation in porous building materi-
als. Its compact, robust design, its easy handling, and its optional battery power supply allows it to be used on building sites.\textsuperscript{13}

\section*{18.4 Application possibilities}

\subsection*{18.4.1 Determination of moisture depth profiles}

Usually, OSA–NMR equipment, such as NMR-INSPECT, does not achieve the sensitivity and spatial resolution of conventional MRI equipment. Whereas MRI provides 3D-resolved images with a resolvable linear increment in the sub-100\,μm range, OSA–NMR is used to determine 1D-profiles with an increment in the 1 mm range. However, OSA–NMR offers the possibility of on-site and online inspection of moisture profiles. With OSA–NMR, the current moisture situation and variations in moisture distribution during wetting and drying can be observed directly on the building component. Entire building constructions can be inspected in a completely non-destructive manner, without the need to impair their integrity or their appearance by taking a sample.

The diagram in Fig. 18.4 shows a typical calibration curve, representing the functional dependence between the amplitude $A$ of the $^1$H-NMR signal measured with NMR-INSPECT and the water content in lightweight concrete, expressed in mass-percent. The correlation is almost proportional. Such a calibration curve can be applied for almost all mineral construction materials. Depending on the measuring time, the inaccuracy of determined water content is typically between 0.3 and 1 mass-%. Moisture values as low as 0.5\% can still be detected.

\begin{equation}
y = 0.1981x + 0.1462 \quad R^2 = 0.9964
\end{equation}

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{calibration_curve.png}
\caption{18.4 Moisture content measurement in lightweight concrete with OSA–NMR; calibration to gravimetric (destructively) determined moisture content.}
\end{figure}
In Fig. 18.5, the application and results of a moisture profile measurement on a lightweight concrete pillar are presented. The dotted line in the diagram shows the moisture profile as it was determined on-site. The profile could be determined up to $x_{\text{max}} = 26$ mm. The entire profile measurement, with a resolved depth-increment of 1 mm and an inaccuracy in moisture determination of about 0.5%, took approximately 30 min.

In order to verify the measurement result, a drilling core of 150 mm in length was extracted and investigated in the laboratory later. The sample was cut into three pieces of about 50 mm in length. By measuring from the top side as well as from the bottom side of each piece, it was possible to determine the moisture content at each point of the drilling core, despite the limited maximum measuring depth of 26 mm.

This ‘laboratory’ moisture profile is shown as a black line in the diagram of Fig. 18.5. Apart from small differences at the beginning, probably as a result of the one-day delay between on-site measurement and sampling, the good correspondence between the ‘on-site’ profile and the ‘laboratory’ profile is obvious.

It should be noted that the strongly decreased moisture contents at about 50 and 100 mm are a consequence of the cutting process. Finally, the integral moisture contents of the cut pieces were determined by gravimetry (weighing, drying and reweighing). The moisture contents determined by this method are shown as grey horizontal lines in the diagram.

18.4.2 Determination of liquid transport coefficients

Existing methods for non-destructive characterization of the durability properties of porous building materials, such as concrete, are regarded as
being unsatisfactory. A key attribute for the evaluation of concrete’s durability is its resistivity to the ingress of chloride and sulfate ions. It has been shown that these substances invade the interior of the concrete structure by piggyback transport with water. Therefore, the main important long-term damage mechanisms in these materials are affected by the distribution and transport of water through the pore system. Hence, the determination of liquid storage and transport parameters is of fundamental interest for predicting the durability and service lifetime of these materials.

Nowadays, laboratory NMR equipment is routinely used to investigate the absorption and redistribution of water in different building materials. In this way, it is possible to determine the liquid transport coefficients and storage parameters for an exhaustive variety of materials. These coefficients can be used as a basic data set for computer programs, allowing precise one- and two-dimensional calculations of simultaneous heat and moisture transport in building components, even under complex conditions. Alternatively, OSA–NMR offers the possibility of determining these coefficients for concrete directly from the component.

Samples of concrete with various water transport properties were produced for an experimental study. The different numbers of capillary pores in the cement stone resulted from different water-to-cement ratios of the mixture, \( w/c = 0.45, 0.50 \) and \( 0.55 \). Additionally, the number of open pores was varied by changing the post-curing conditions, i.e. the samples were stored for 24 h at different temperatures (20, 45, 80 and 105 °C). Finally, the samples were exposed to water, which was applied without (capillary sorption) and with pressure (permeation). This moisture loading was interrupted at several points in order to monitor the moisture profile and the progress of the moisture diffusion front, respectively, with OSA–NMR.

Monitoring the time-dependent water uptake during capillary sorption allows the evaluation of the concrete’s water uptake coefficient \( U \), which is a measure for the absorption velocity. Firstly, \( U \) could be determined by the usual destructive method, i.e. by gravimetric determination (weighing, drying, reweighing) of the absorbed water amount in the sample at different times. Secondly, it was possible to determine the value of \( U \) by OSA–NMR. In Fig. 18.6a, both methods for water uptake coefficient determination are compared. An excellent correlation can be observed.

For the determination of the water permeability coefficient, the concrete is exposed on one side to water, which is applied with a specific pressure \( p \) and for a specific time \( t \) (Fig. 18.6b). The depth of penetrated water is determined as a function of \( p \) and \( t \). It is necessary to do this using several different water pressures and times in order to follow the procedure of the standardized method. In principle, such an investigation can only be done non-destructively. It should be noted that one sample of concrete had an
embedded steel bar, representing the reinforcement in concrete. The influence of ferromagnetic reinforcement is described in 18.5.

18.4.3 Early-age concrete hardening

Even though a variety of methods to measure the properties of fresh concrete are already available, monitoring strength development in early-age concrete is still an unsolved testing problem. During cement hydration, part of the mixing water is chemically combined and the residual water is confined in pores, which gradually decrease in size. These processes strongly affect the NMR relaxation times $T_1$, $T_2$. The same microstructural processes are also responsible for the development of the mechanical properties in cement stone. Therefore, measures of changes in $T_1$ and $T_2$ should be cor-
related to the development of strength and tightness in fresh concrete. The hardening of an individual concrete component can be monitored continuously by using OSA–NMR.

The hardening behavior of five different concrete mixtures as well as pure cement pastes was investigated. Different water-to-cement ratios \( \frac{w}{c} \) as well as the enrichment of some samples with a retarder should provide a wide range of hardening behaviors. Starting with their preparation, the development of the \( T_2 \) relaxation curve of each specimen was observed over three days.

Figure 18.7a shows the evolution of the \( T_2 \) relaxation curve for a single sample. It is very evident that the curve’s decay is accelerating as hardening, i.e. cement hydration, proceeds. Fitting the experimental data to a one-exponential approximation function provides the time constant of the decay curves, which is the relaxation time \( T_2 \). This relaxation time typically decreases greatly between the beginning and the end of hardening, (Fig. 18.7b). For concrete without a retarder (S1, S3 and S4), this decrease is moderate in the first 2–3 h after preparation. This induction period is followed by an accelerated decrease (acceleration period) up to 20–30 h and finally the \( T_2 \) decrease slows down again (decay period). Hence the \( T_2 \) evolution follows the qualitative behavior of hydration progress and strength development. As expected, the acceleration period is greatly delayed for specimens with a retarder (S2 and S5).

### 18.5 Reliability and limitations

Ferromagnetic inclusions, such as the steel reinforcement in concrete, generally cause a problem when applying NMR in analysis of building materi-
als. In a laboratory study, it was shown that just a single cylindrical steel bar with 10 mm diameter noticeably influenced the OSA–NMR measurement.\textsuperscript{25} The steel bar changes the $B_0$ magnetic field distribution and with it, the position, size and shape of the sensitive volume. This results in changes in the measuring depth $x$ which can not be predicted precisely. In addition, the resolved depth increment and NMR signal amplitude, i.e. the determined moisture content, are also changed. These effects are most pronounced when the bar is exactly parallel to the magnetic field direction of the NMR probe head.

When there is a very dense distribution of reinforcement bars in close proximity to the concrete surface, obtaining a reasonable OSA–NMR measurement can be completely prevented. Obtaining OSA–NMR measurements at depths below the embedded steel reinforcement is generally very difficult owing to the shielding of the RF field.

On the other hand, it has been shown that there are no problems obtaining OSA–NMR measurements above a wide-meshed (more than 10 cm spacing between two adjacent bars) reinforcement, if the probe head is fitted exactly in the ‘steel-free’ clearance of the mesh. Moreover, the $B_0$ distribution could be optimized, accounting for the distortion effect of the steel reinforcement. Nevertheless, both approaches require knowledge about the exact position and quantity of reinforcement, which is often not available in the case of an old concrete construction.

Materials with very small $T_2$ relaxation times (e.g., fired clay brick and blast furnace cement concrete) can cause another problem. If $T_2$ is very short, the hydrogen nuclei have already completely relaxed within the dead time of the NMR measuring device (Fig. 18.2), thus these nuclei cannot be detected. Additionally, the resolvable increment of MRI is inversely proportional to $T_2$. Therefore small $T_2$ values prevent high spatial resolution.

The field strengths $B_0$ and $B_1$, and with them the measuring sensitivity of an OSA–NMR probe head, rapidly decrease with increasing measuring depth $x$. Hence there is a threshold level, the maximum measuring depth $x_{\text{max}}$, that characterizes the measuring range in which the observed nuclei can be detected with an acceptable sensitivity. With currently available equipment, $x_{\text{max}}$ is typically in the region of 30 mm. Higher maximum measuring depths of 60 or even 100 mm have been achieved, but these were associated with an unacceptably coarse resolvable depth-increment or with a probe head of monstrous size and weight.\textsuperscript{8,26}

\section*{18.6 Conclusions and future trends}

NMR is not only an useful tool in laboratories of institutional and industrial research facilities. Exceptional NMR instrumentation is available, which gives OSA to the specimen in order to inspect a building component.
For more than 10 years, researchers at the IZFP have been concerned with the development and application of such NMR devices. NMR-INSPECT, a completely portable and rugged OSA–NMR device, was developed for on-site inspections.

Despite its broad potential for building material characterization, ranging from moisture profiling to the determination of storage and transport properties in porous media, OSA–NMR has not become a routinely used inspection tool in the building industry. The main obstacle is probably the equipment costs. NMR is still not a ‘mass production’ technique and requires highly sophisticated and hence expensive components. Compared with conventional NMR and MRI instrumentation used in biochemical research and medical diagnostics, the costs for OSA–NMR equipment are quite low. However, for standard application on building sites, such high equipment costs are still unusual.

If OSA–NMR was to be used for on-site inspection of old concrete structures with potential deterioration, it would be difficult to prove its cost-effectiveness without a distinct increase of the inspection depth (100 mm or more) and without increasing the size and weight of the probe head. This is still an unsolved but not unsolvable technical problem.

18.7 References


© Woodhead Publishing Limited, 2010
Stress wave propagation for evaluation of reinforced concrete structures

S. TESFAMARIAM, The University of British Columbia, Canada; B. MARTÍN-PÉREZ, University of Ottawa, Canada

Abstract: Failure of service-critical systems owing to ageing and/or faulty design practices highlights the vulnerability of these systems and the need for proactive management. Structural deterioration may be attributed to ageing, extreme environments and/or inadequate investment in regular maintenance and repairs. The proactive management can be integrated during initial construction quality control, monitoring of \textit{in situ} response and environmental changes (such as variation of load change and traffic volume), and during quality control of repaired structures. This chapter explores the application of stress wave propagation techniques during the life cycle of the structure to instigate proactive decision action.

Key words: stress wave propagation, ultrasonic through-transmission, impact–echo, setting time, compressive strength, reinforced concrete.

19.1 Introduction

Services rendered by civil infrastructure systems (e.g. water and transportation) are critical for a nation’s economy and the quality of life of its citizens (ASCE, 2009). However, failure of service-critical systems owing to ageing and/or faulty design practices, e.g. the failures of the I-35W bridge in Minneapolis (National Transportation Safety Board, 2008) and de la Concorde overpass in Montreal (Johnson \textit{et al.}, 2007), highlights the vulnerability of these systems and the need for proactive management. Structural deterioration may be attributed to ageing, extreme environments and/or inadequate investment in periodic maintenance and repairs (Rix \textit{et al.}, 1995), which can contribute to the reduction in the system structural capacity. The problem is further aggravated by the growth of cities and the corresponding increase in system demand. The decrease in structural capacity coupled with an increase in demand can instigate safety issues. Consequently, a reactive decision action is often invoked to repair, rehabilitate or demolish the system. This passive management is inefficient and a rather expensive practice. On the other hand, central to proactive management is the implementation of routine condition assessment and monitoring of \textit{in situ} response and environmental changes (such as variation of load change and traffic volume), and during quality control of repaired structures. This chapter explores the application of stress wave propagation techniques during the life cycle of the structure to instigate proactive decision action.
situ environmental changes (such as the variation of load change and traffic volume) to instigate proactive decision action.

The life cycle of a civil infrastructure system is schematically depicted in Fig. 19.1, which shows that a system generally goes through three main stages during its lifetime: new construction, in-service use, and repaired state. During these stages, the systems can be monitored at a micro level (e.g. material quality) and/or at a macro level (i.e. monitoring overall structural response). At each stage of the lifetime of the infrastructure, various engineering performance indicators need to be quantified for proper diagnosis and appropriate decision action. The performance indicators, for example, can be crack depth, crack width, crack distribution, crack development, honeycombing voids, delamination, concrete strength, elastic modulus, thickness, bar location, bar size, and bar corrosion.

The engineering performance indicators are assessed using different non-destructive testing (NDT) techniques. Relations between engineering performance indicators and their corresponding relevant NDT techniques are summarized in Table 19.1. For example, in situ concrete strength can be estimated through rebound hammer and ultrasonic measurements. Similarly, reinforcement corrosion-induced delamination can be estimated through impact–echo, ground penetrating radar (GPR), and infrared ther-
Table 19.1 NDT methods for concrete structures (adapted from IAEA, 2005)

<table>
<thead>
<tr>
<th>Performance indicator</th>
<th>Strength</th>
<th>Elastic modulus</th>
<th>Thickness</th>
<th>Crack depth</th>
<th>Crack width</th>
<th>Crack distribution</th>
<th>Crack development</th>
<th>Honeycomb voids</th>
<th>Delamination</th>
<th>Bar location</th>
<th>Bar size</th>
<th>Bar corrosion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rebound hammer</td>
<td>•</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Penetration resistance</td>
<td>•</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pull-out</td>
<td>•</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ultrasonic</td>
<td>• • • •</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Radar</td>
<td>• • •</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Thermography</td>
<td>• •</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Radiography</td>
<td>• • • •</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Acoustic emission</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Magnetic or eddy current</td>
<td>• •</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Half-cell potential</td>
<td>•</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Photography</td>
<td>• •</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Stress waves are generated when pressure or deformation are induced using external stimuli, such as a transducer or an impact, to the surface of a solid (ACI Committee 228, 1998). Various techniques are used to generate the stress waves: standard ultrasonic pulsers, tone burst systems, laser generation, and lithotripsy (Popovics et al., 1993). The choice of an appropriate stress wave generator takes into consideration the input control, penetrating strength, and simplicity (Popovics et al., 1993). The disturbance generated through the impact propagates as three different waves, P, S, and R waves. The P wave wave is associated with the propagation of normal stress and particle motion and is associated with the time of first arrival. The S wave is associated with shear stress, and particle motion is perpendicular to the propagation direction. The R wave travels away from the disturbance along the surface. The R wave is useful for quantifying surface flaws.
The P wave velocity $C_P$ is the fastest, and it is often used to quantify damage and strength. For an infinite, homogeneous, isotropic and uniform material, the P wave velocity is related to the dynamic modulus of elasticity $E_d$ (kg m$^{-2}$ s$^{-1}$), mass density $\rho$ (kg m$^{-3}$) and Poisson’s ratio $\nu$ of the material and is computed by the following equation (Krautkramer and Krautkramer, 1990):

$$C_P = \sqrt{\frac{E_d(1-\nu)}{\rho(1+\nu)(1-2\nu)}} \quad [19.1]$$

Although concrete is a heterogeneous and anisotropic medium and the rigorous application of equation [19.1] may not be valid, the P wave velocity is still considered to be strongly influenced by the material stiffness. The S wave velocity $C_S$ is related to the shear modulus of elasticity $G$ (kg m$^{-2}$ s$^{-1}$) and $\rho$:

$$C_S = \sqrt{\frac{G}{\rho}} \quad [19.2]$$

When the stress wave is generated through ultrasonic pulse, the method is referred to as ultrasonic through-transmission (UTT), also known as ultrasonic pulse velocity (ACI Committee 228, 1998). The UTT works with two transducers: a generator and a receiver (Fig. 19.2). The use of UTT in concrete structures dates back to the 1940s at the Road Research Laboratory in the UK for thickness measurement of pavement layers (Jones, 1953) and

![Ultrasonic through-transmission set-up](https://via.placeholder.com/150)

19.2 Ultrasonic through-transmission set-up (Tesfamariam, 2000).
at Ontario Hydro Canada for finding crack depths (Leslie and Cheesman, 1949). This method is standardized in ASTM (ASTM Standard C597, 2002).

When the stress wave is generated through an impact, the method is referred to as impact echo or IE (ACI Committee 228, 1998). In the IE technique, the stress is generated through an impact, and the signal is collected using an adjacent transducer. The development of IE in concrete applications dates back to the mid-80s (Sansalone and Streett, 1997). Various applications of UTT and IE in civil infrastructure systems have been reported, and a review is provided in section 19.3.

Once the stress-wave signal generated through UTT or IE is collected, the interpretation of the signal can be undertaken in the time domain using velocity and attenuation coefficient, or in the frequency domain using quality factor (Q-factor). Since concrete is a heterogeneous material, its strength is affected by the elastic (aggregate, sand, and cement) and inelastic (air voids and flaws) constituents. The velocity is used to measure the elastic component. The inelastic component can be quantified through attenuation measures.

### 19.2.1 Velocity

For the UTT, the P velocity is measured from the time of first arrival to the receiver. Calculation of the P wave velocity $C_p$ from the time of first arrival is given by:

$$C_p = \frac{X}{t}$$  \[19.3\]

where $X$ is the distance that the P wave is propagated between source and receiver, and $t$ is the travel time. For the impact–echo method, the P wave velocity is measured using the P wave resonance frequency $f_p$ as:

$$C_p = 2Xf_p$$  \[19.4\]

where $X$ is the distance between the top and bottom of a two-layer solid system. The velocity measurement can readily be related to the condition of the concrete (e.g. Table 19.2).

### 19.2.2 Attenuation coefficient

As the stress wave propagates through concrete, the energy of the propagating wave decreases with increasing path length. The decrease in wave energy is the result of attenuation and divergence, caused by the absorption and scattering of the wave energy. Absorption and scattering losses can give information about the structural and physical properties of the medium (Blitz and Simpson, 1996). Absorption is the result of converting the stress
wave energy into heat, caused by friction as a result of particle movement. Scattering in concrete structures is a result of wave reflection, refraction, diffraction, and mode conversion owing to dissimilar materials, microcracks and air voids. The stress wave intensity $I_d$ over a distance $d$ can be described by an exponential function (Krautkramer and Krautkramer, 1990):

$$I_d = I_o e^{-\alpha d} \tag{19.5}$$

where $I_o$ is the intensity at a distance 0 and $\alpha_d$ is known as the attenuation coefficient (m$^{-1}$). For a given distance, an exponential best-fit curve can be fitted through the time domain signal as depicted in Fig. 19.3, and the corresponding attenuation coefficient $\alpha$ (s$^{-1}$) can be calculated as:

$$I_t = I_o e^{-\alpha t} \tag{19.6}$$

where $I_t$ is the intensity at time $t$. 

### Table 19.2 Correlation of pulse velocity and condition of the structure (Whitehurst, 1951a)

<table>
<thead>
<tr>
<th>Pulse velocity</th>
<th>Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt;4570</td>
<td>Excellent</td>
</tr>
<tr>
<td>3600–4570</td>
<td>Generally good</td>
</tr>
<tr>
<td>3050–3660</td>
<td>Questionable</td>
</tr>
<tr>
<td>2130–3050</td>
<td>Generally poor</td>
</tr>
<tr>
<td>&lt;2130</td>
<td>Very poor</td>
</tr>
</tbody>
</table>

19.3 Attenuation coefficient $\alpha$ in the time domain.
The attenuation coefficients (decay constants) have been correlated with damage level as shown in Table 19.3. The results of Table 19.3 were generated by Kesner et al. (2004) using the IE technique for detecting distributed cracks in concrete. After each IE test, the crack densities were obtained from neutron radiographs. Furthermore, the crack densities were used to define the damage level that is further related to the attenuation coefficient (Table 19.3).

Research of these methods was first published in the mid sixties. Galan (1967; 1990) used ultrasonic pulse velocity (UPV) and the attenuation coefficient $\alpha$ to estimate elastic and inelastic components of concrete. This method has been utilized by other researchers (Ismail et al., 1996; Tesfamariam et al., 2006; Tharmaratnam and Tan, 1990).

### 19.2.3 Quality factor

It has been shown that attenuation in concrete can be estimated by the quality factor (Q-factor). The Q-factor is the measure of the spectral width of the peak frequency of the stress wave. The particular frequency can be identified after converting the time domain signal (Fig. 19.3) into the frequency domain signal (Fig. 19.4), using the fast Fourier transform (FFT). The equation for the calculation of the Q-factor is shown as (Krautkramer and Krautkramer, 1990):

$$Q = \frac{f_1}{f_2 - f_1}$$  \[19.7\]

where $f_1$ is the peak frequency, and $f_2$ and $f_1$ are the frequencies at 70.7% of the spectral peak amplitude. It should be highlighted that dispersion is a frequency-dependent phenomenon that also affects the Q-factor measurement. The propagations of stress waves having higher frequency (lower wave length) are more affected by the anomalies in the concrete that low frequency waves.

The Q-factor, as a measure of the attenuation index, has been used to monitor deterioration of concrete owing to the alkali–silica reaction.
(Tesfamariam et al., 1999; Thomas et al., 1999), freeze–thaw durability (El-Korchi et al., 1989), the quality of Berea sandstone (Shankland et al., 1993), and concrete strength (Tesfamariam et al., 2006).

19.3 Applications

A review of applications of the various stress wave propagation and analysis techniques to the condition assessment and monitoring of concrete structures during their lifetime is summarized below according to the three stages depicted in Fig. 19.1.

19.3.1 Initial time of set and strength development for newly cast concrete

The initial design and construction quality of a concrete structure considerably influences the life cycle of the system. Factors detrimental to the initial construction may be design error, error in site selection or development, and/or material deficiencies (Mailvaganam et al., 2000). Clearly, once there are slip-ups, and flaws of any kind are detected in a built structure, the rectification process can be very costly. Hence, prudence from the design engineer and construction manager is of great importance. Quality control of material specifications and construction can be monitored by implementing various NDT techniques. For instance, in a freshly placed concrete, the
quality of concrete, strength development, honeycombing, location of reinforcement and any other flaws can be identified using various types of NDT techniques (Chang and Liu, 2003; Chong et al., 2003). Furthermore, in situ sensors can be embedded into the core of the structure before concrete placement to monitor its performance (Culshaw et al., 1996). In this case, sensors should not be intrusive and impinge on structural performance. The results from different sensors and NDT measurements have then to be integrated into a generalized architecture to infer a decision action about the initial quality and development of benchmarking for future assessment (Hannah et al., 2000).

The use of stress wave propagation techniques has been successfully applied in the monitoring of the quality of fresh concrete, in particular in the determination of the initial and final setting time. Early determination of setting time can be used, for example, to accelerate formwork removal and, consequently, to save construction time. The velocity or attenuation coefficients can thus be used to determine in situ setting time. However, in order to develop a correlation between these coefficients and setting time, the stress wave propagation measurements need to be carried out concurrently with ASTM penetration test measurements (ASTM Standard C403/C403M, 2008). Whitehurst (1951a) reported an early attempt to correlate setting time and UTT velocity measurements, and he identified the challenge of removing formwork to measure the early age of concrete. Tesfamariam (2000) has developed an experimental set-up (Fig. 19.5) to measure velocity at an early age. Figure 19.5 shows the freshly cast concrete placed in a 6" cylinder, where a circle is cut from the bottom end in order to insert a cone shape transducer. However, the top transducer has open access and it is supported using adjustable transducer support.

Factors that influence the compressive strength of concrete are the properties of the individual components, mix proportions, curing history, moisture condition and environment as reflected by deterioration (Sturrup et al., 1984), all of which also influence velocity, quality factor and attenuation coefficients. Various workers have reported the effect of these factors on the setting time of freshly cast concrete and concrete strength, e.g.:

1. Properties of the individual components:
   - cement type (Ravindrarajah, 1992);
   - aggregates: shape, type and composition (Jones, 1953; Sturrup et al., 1984);
   - admixtures: accelerators, retarders (Ravindrarajah, 1992).
2. Proportions:
   - water-to-cement (w/c) ratio (Pessiki and Carino, 1998; Popovics and Popovics, 1998; Sturrup et al., 1984; Tesfamariam et al., 2006);
   - aggregate ratio (Pessiki and Carino, 1998; Sturrup et al., 1984).

Reported theoretical and empirical relations between early concrete strength and stress wave propagation parameters are shown in Table 19.4 and Fig. 19.6 (UTT) and Fig. 19.7 (IE). Figure 19.6 shows that, for example, increasing the \( w/c \) ratio decreases the velocity measurements, and also that, for a given \( w/c \) ratio, increasing the aggregate to cement \( (a/c) \) ratio, increases the velocity measurements. Figure 19.7 shows the impact of adding accelerator and retarder admixture in the velocity measurements.

Once the concrete has reached the final setting, the rate of increase in strength decreases and reaches the design level at 28 days. Theoretical and empirical relations between concrete strength and stress wave propagation parameters are reported in the literature as shown in Table 19.5 and Figs 19.8 and 19.9. Figure 19.8 shows the impact of \( w/c \) and \( a/c \) ratios on the strength and velocity measurements. Figure 19.9 shows the effects of curing
Table 19.4 Application of stress wave propagation methods for freshly cast concrete

<table>
<thead>
<tr>
<th>Reference</th>
<th>Application</th>
<th>Analysis technique</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Whitehurst (1951b)</td>
<td>Setting time</td>
<td>UTT–velocity</td>
<td>Fig. 19.6</td>
</tr>
<tr>
<td>Tesfamariam (2000)</td>
<td>Setting time</td>
<td>UTT and IE–velocity, attenuation coefficient</td>
<td>Fig. 19.6</td>
</tr>
<tr>
<td>Pessiki and Carino (1998)</td>
<td>Setting time</td>
<td>IE–velocity</td>
<td>Fig. 19.7</td>
</tr>
<tr>
<td>Krauß and Hariri (2006)</td>
<td>Dynamic Poisson’s ratio $\mu_d$</td>
<td>UTT: P and S wave velocity</td>
<td>$\mu_d = \frac{1 - 2(V_s/V_c)^2}{2 - 2(V_s/V_c)^2}$</td>
</tr>
<tr>
<td></td>
<td>Dynamic Young’s modulus $E_d$</td>
<td>UTT: P and S wave velocity</td>
<td>$E_d = \frac{(1 + \mu_d)(1 - 2\mu_d)}{(1 - \mu_d)}V_c^2$</td>
</tr>
</tbody>
</table>

\[
\mu_d = \frac{1 - 2(V_s/V_c)^2}{2 - 2(V_s/V_c)^2}
\]

\[
E_d = \frac{(1 + \mu_d)(1 - 2\mu_d)}{(1 - \mu_d)}V_c^2
\]

![Graph](image)

19.6 Ultrasonic through-transmission setting time measurement.
temperature, \(w/c\) ratio and \(a/c\) ratio. Figures 19.8 and 19.9 show that, for a given velocity measurement, increasing the aggregate content decreases the concrete strength. Figure 19.8 shows that varying the curing temperature has no marked effect on the velocity and compressive strength relation.

Results reported in Figs 19.6–19.9 shows the effect of various material mix properties on the strength development and setting time. However, the velocity and compressive strength correlation provided, in Table 19.5, for example, does not take into consideration the variation in the mix proportion. This will not cause a serious error if a baseline data and calibration is carried out on a control mixture. For general application, without having a baseline calibration, prediction equations provided in Table 19.5 can be enhanced by introducing the mix design proportions.

19.3.2 Ageing and deterioration of concrete infrastructure

The presence of deficiencies in concrete infrastructure reduces the performance of these structures over their service life. These deficiencies are the result of defects introduced at the design/construction stage, material deterioration resulting from interaction with the exposure environment, and/or damage caused by overloading or extreme events such as earthquakes (Paik and Melchers, 2008). From the two previous categories, there are several
phenomena that can lead to the degradation of in-service concrete infrastructure over the long term (second stage in Fig. 19.1):

(i) repeated loading;
(ii) overloading;
(iii) non-uniform dimensional changes, such as shrinkage of constrained concrete;
(iv) severe loading;
(v) loss of foundation support;
(vi) surface wear by abrasion or erosion; and/or
(vii) chemical attack resulting from the interaction between mix ingredients or between the structure and the exposure environment (Li et al., 2009).

<table>
<thead>
<tr>
<th>Reference</th>
<th>Application</th>
<th>Analysis technique</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Galan (1967; 1990)</td>
<td>Strength development</td>
<td>UTT–velocity, attenuation coefficient</td>
<td>$f_c(C_P, \alpha) = aC_P^b\alpha$, where $a$, $b$, $c$ are model parameters obtained through regression analysis</td>
</tr>
<tr>
<td>Pesski and Carino (1998)</td>
<td>Strength development</td>
<td>IE–velocity</td>
<td>$f_c'(C_P) = a\left(\frac{C_P}{n} + 1\right)^b$, where $a$, $b$ are model parameters obtained through regression analysis, $n$ is number of samples used</td>
</tr>
<tr>
<td>Tharmaratnam and Tan (1990)</td>
<td>Strength development</td>
<td>UTT–velocity, attenuation coefficient</td>
<td>$f_c(C_P) = ae^{C_P}$ $f_c'(\alpha) = \alpha^{e^{C_P}}$ $f_c'(C_P, \alpha) = \alpha^{e^{C_P}e^{C_P}}$, where $a$, $b$, $c$ are model parameters obtained through regression analysis</td>
</tr>
<tr>
<td>Tesfamariam et al. (2006)</td>
<td>Strength development</td>
<td>UTT and IE–velocity, attenuation coefficient</td>
<td>$f_c(C_P) = aC_P^b$ $f_c'(C_P, \alpha) = aC_P^b\alpha^c$ $f_c'(C_P, Q) = aC_P^b\alpha Q$, where $a$, $b$, $c$ are model parameters obtained through regression analysis</td>
</tr>
</tbody>
</table>
19.8 Ultrasonic through-transmission strength development measurement, impact of w/c and a/c ratios.

19.9 Impact–echo strength development measurement, impact of w/c ratio and curing condition.
These phenomena acting on concrete often result in excessive deformations and/or the loss of mass, strength and rigidity, thus affecting the in-service performance of the structure. The majority of the deterioration mechanisms affecting concrete structures as a result of interaction with the environment have cracking as the most common manifestation (Table 19.6). Although concrete cracking does not necessarily lead to structural failure, cracks in concrete adversely affect its in-service durability properties by providing easy access to water and other aggressive agents, thus increasing the rate of deterioration and impairing the remaining lifetime of the structure. It is the amount and intensity of concrete cracking that indicates damage accumulation in concrete infrastructure and triggers intervention for repair/rehabilitation. Therefore, in order to design a proper repair/rehabilitation system, there is a need to assess the actual physical condition of the structure as well as to determine the location and severity of cracking within. NDT offers an excellent resource to this end, since it can provide data on in situ and undisturbed conditions, and its equipment has the advantage of being portable and covering large areas. Furthermore, NDT also offers the capability of monitoring the in-service condition of a structure in order to detect any change in its performance.

Among several NDT techniques, stress wave propagation methods, such as ultrasonic testing and impact–echo, have successfully been used to assess the condition of concrete structures that are deteriorated as a result of the exposure environment. Although these methods do not provide direct measurement of performance indicators, the data obtained from them can be related to the physical condition of the medium through which the stress waves propagate (e.g. Fig. 19.10). Moreover, results from NDT are often corroborated with destructive testing on concrete cores. A summary of some of the applications of stress wave propagation methods in the assessment of deteriorated concrete infrastructure is presented in Table 19.7.

<table>
<thead>
<tr>
<th>Cause</th>
<th>Manifestation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shrinkage</td>
<td>Tension cracks; map cracking</td>
</tr>
<tr>
<td>Reinforcement corrosion</td>
<td>Longitudinal cracking parallel to reinforcement;</td>
</tr>
<tr>
<td></td>
<td>spalling; delamination of concrete cover</td>
</tr>
<tr>
<td>Alkali–aggregate reactions</td>
<td>Map cracking; popouts; differential movement</td>
</tr>
<tr>
<td></td>
<td>owing to expansion</td>
</tr>
<tr>
<td>Freeze/thaw effects</td>
<td>Scaling; popouts; cracking; D-cracking; disintegration</td>
</tr>
<tr>
<td>Sulfate attack</td>
<td>Cracking; expansion; softening and disintegration</td>
</tr>
<tr>
<td>Acid and seawater attack</td>
<td>Surface erosion; disintegration</td>
</tr>
</tbody>
</table>
19.10 Expansion over time owing to alkali–aggregate reaction and UTT measurements: (a) velocity and (b) attenuation coefficient (Tefsamariam, 2000).
Table 19.7 Application of stress wave propagation methods for ageing/deteriorating concrete infrastructure

<table>
<thead>
<tr>
<th>Reference</th>
<th>Application</th>
<th>Analysis technique</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chung and Law (1985)</td>
<td>Fire damage</td>
<td>UTT–velocity</td>
<td>Measurement of depth of fire-damaged surface layer</td>
</tr>
<tr>
<td>Hobbs (1986)</td>
<td>Alkali–silica reaction</td>
<td>UTT–velocity</td>
<td>Damage was defined as $D = 1 - (V/V_o)$, $V$ and $V_o$ are velocities (amplitudes) for loaded and unloaded specimen</td>
</tr>
<tr>
<td>Daponte et al. (1990)</td>
<td>Detection and monitoring of crack growth</td>
<td>UTT–velocity, attenuation</td>
<td>Results corroborated by destructive testing</td>
</tr>
<tr>
<td>Olson and Wright (1990)</td>
<td>Corrosion-induced spalling and delamination of parking garage deck with asphalt overlay</td>
<td>IE–frequency domain</td>
<td>Results corroborated by destructive testing</td>
</tr>
<tr>
<td>Olson (1992)</td>
<td>Assessment of cracking in thin-arch concrete dam</td>
<td>IE–frequency domain</td>
<td></td>
</tr>
<tr>
<td>Olson (1992)</td>
<td>Condition evaluation of severely cracked post-tensioned bridge segment</td>
<td>UTT–velocity and peak signal amplitude</td>
<td></td>
</tr>
<tr>
<td>Cheng and Sansalone (1993)</td>
<td>Corrosion-induced delamination of bridge deck</td>
<td>IE–frequency domain</td>
<td>Verified by destructive (coring) and non-destructive (sounding) methods; delaminated areas exhibited waveform with very large-amplitude flexural frequency</td>
</tr>
<tr>
<td>Teodoru and Herf (1996)</td>
<td>Freeze–thaw damage</td>
<td>UTT</td>
<td>UTT velocity and attenuation with increasing expansion (Fig. 19.10)</td>
</tr>
<tr>
<td>Tesfamariam (2000)</td>
<td>Alkali–silica reaction</td>
<td>UTT and IE–velocity and attenuation</td>
<td></td>
</tr>
<tr>
<td>Epasto et al. (2009)</td>
<td>Fire damage</td>
<td>IE–wavelet time-frequency analysis</td>
<td></td>
</tr>
</tbody>
</table>
The ultrasonic pulse velocity method is mostly used to verify the homogeneity of a member, detect large voids or internal cracks (Olson, 1992), to determine the depth of visible surface cracks (Leslie and Cheesman, 1949), or to study the presence of horizontal layers that are formed when concrete is exposed to an aggressive environment, such as surface damage from frost (Teodoru and Herf, 1996) or fire (Chung and Law, 1985). In general, as the pore volume of concrete increases owing to micro- or macro-cracking, the ultrasonic pulse velocity goes down. Furthermore, stress waves are partially or totally reflected at discontinuities created by voids or cracks, making it possible to locate them by analyzing the recorded spectra of the pulse. Once the pulse velocity in concrete has been determined by applying equation [19.3], the increased travel time required by the pulse in going around a crack can be used to calculate the path length and, thus, the depth of the crack.

The impact echo method has been used to measure thicknesses as well as to detect flaws such as honeycombs, voids, cracks, and delaminations. Its advantage over the pulse velocity method resides in the fact that it only requires access to one surface, making it very useful for condition assessment of concrete slabs, walls, bridge and parking garage decks (Cheng and Sansalone, 1993), dams (Olson, 1992) and tunnel linings.

19.3.3 Repaired concrete infrastructure

Routine maintenance and repair is critical for extending the service life of infrastructure systems (third stage in Fig. 19.1). Although NDT is very useful in the condition assessment of deteriorated infrastructure as previously discussed, it can also be very valuable in the quality assurance monitoring of repair materials/systems and their compatibility with the existing structure. Improper installation of repair systems affects the integrity and durability of the repaired structure. Similarly, the in-service performance of repaired structures also has to be evaluated and monitored during their remaining lifetime, from both serviceability and ultimate states viewpoints. The use of stress wave propagation methods has been limited to a few applications related to the repair of concrete infrastructure, such as the evaluation of bond quality between repair and host systems (Lin and Sansalone, 1996; Lin et al., 1996) and the quality assurance of crack repairs (Olson, 1992). A summary of these applications is presented in Table 19.8. The methodologies employed are similar to those used in the condition assessment of the structure, namely the location of flaws (such as debonding) or the comparison of results obtained before and after repair.
19.4 Discussion and future trends

As depicted in Fig. 19.1, the life cycle of concrete infrastructure comprises three different stages: design/construction, in-service, and repair/rehabilitation. As part of a robust infrastructure management program, maintenance strategies in each of these stages should be identified. The integration of NDT in the management of concrete infrastructure during these three stages ensures the availability of knowledge on the actual condition of the structure to allow better decision-making for resource allocation. During the design/construction stage, optimization can be carried out between future maintenance, life-cycle costing and proper protection to aggressive exposure conditions (Bijen, 2003). However, deficiencies can be identified at this stage, as discussed in 19.3.1, and a new maintenance program then needs to be designed for the remaining lifetime of the structure. NDT can thus provide the necessary information to ensure the structure is performing above a minimum level (performance monitoring), to design the best-suited repair system, and to assure that the implemented repair system restores the minimum performance level required.

Stress wave propagation methods are efficient in controlling the quality of construction as specified in design during the first stage of the life cycle of the structure, in particular for setting time and strength development. The assessment of the in-service stage is related to the initial quality of construction as well as the different operational and environmental loadings to which the infrastructure is subjected. Monitoring of in-service infrastructure through NDT can be done both at the micro and macro levels. At the micro level, monitoring entails observation of any prevalent deterioration mechanism and consequent changes in the material behavior that results from different physical/chemical attacks and interaction

<table>
<thead>
<tr>
<th>Reference</th>
<th>Application</th>
<th>Analysis technique</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Olson (1992)</td>
<td>Epoxy injection repair</td>
<td>UTT</td>
<td>Filling of cracks was verified by comparing spectra records before and after repair</td>
</tr>
<tr>
<td>Lin et al. (1996)</td>
<td>Repair of parking garage deck</td>
<td>IE</td>
<td>Debonding was identified by analyzing response spectra</td>
</tr>
</tbody>
</table>

Table 19.8 Application of stress wave propagation methods for repaired concrete infrastructure
with the exposure environment. Both the pulse velocity and impact–echo methods are valuable techniques to locate and characterize damage resulting from these phenomena. At the macro level, the response of the overall system to daily operation is monitored, e.g. vibration response (Xu et al., 2004). The ‘performance limit’ here is different for each system type, but, in general, it may include: drift, span deflection, crack width, and stress level. For example, in measuring structural response, some of the main responses of the system that are of importance are temperature, strain (Watkins, 2003), displacement (Fu and Moosa, 2002) and vibration (Xu et al., 2004).

The ‘performance limit’ has to be established before real-time monitoring, in order to intervene and implement corrective/mitigative actions if necessary. For this purpose, the prevalent condition state (micro level), the demands from operational use, and the overall response of the structure (macro level) have to be monitored and integrated within the same monitoring framework, in which routine NDT plays an important role. Each non-destructive sensor furnishes unique and critical information, which has to be collated to better assess the condition state of the system and, therefore, support decision-making. However, for a full integration of NDT in such a system, there is an urgent need to establish baseline-data for interpretation of results early on.

### 19.5 Conclusions

Maintenance of concrete infrastructure systems requires reliable decision-making tools to direct limited resources more prudently into the most deserving system. These decisions should be based on all available information, including the actual condition of the infrastructure. In this chapter, the importance of stress wave propagation methods as a NDT tool has been presented within a holistic framework for condition assessment of such systems, from inception and construction to in-service performance and repair/rehabilitation. It has been emphasized that the operational condition of the system throughout its life cycle has to be monitored in order to make robust decisions for intervention. Successful applications of these methods in the monitoring and assessment of the concrete infrastructure have been presented within this framework. However, further research is required in the improvement and refinement of these techniques in quality assurance and performance monitoring of repaired/rehabilitated systems.

### 19.6 References


Non-destructive evaluation of reinforced concrete structures


Proceeding of sessions sponsored by the Engineering Mechanics Division of the ASCE in conjunction with the Structures Congress, San Antonio, TX, pp. 115–126.

IEEE proceedings of frontiers in education conference, pp. Session 4c1, 4c11–4c14.


In In situ non-destructive testing of concrete, ACI SP-82 (Ed, Malhotra, V. M.) American Concrete Institute, Detroit, MI, pp. 201–227.

In In situ non-destructive testing of concrete, ACI SP-82 (Ed, Malhotra, V. M.) American Concrete Institute, Detroit, MI, pp. 247–276.


Abstract: This chapter describes non-destructive test methods based on surface-guided mechanical waves for application to concrete. After a summary of the history of development of the methods, surface wave propagation in homogeneous and layered media is reviewed, and analytical and numerical modeling efforts are described. Specific time domain and frequency domain surface wave methods are introduced, including the spectral analysis of surface waves (SASW) and multichannel analysis of surface waves (MASW) methods. The needed equipment are described and finally successful applications to concrete structures are reviewed.

Key words: attenuation, concrete, cracks, multichannel analysis of surface waves (MASW), non-destructive evaluation (NDE), non-destructive testing (NDT), spectral analysis of surface waves (SASW), surface wave, ultrasound.

20.1 Introduction

In the 1940s and 1950s, the development of one-sided wave velocity measurement techniques for concrete was motivated by the interest in estimating Young’s modulus and other elastic moduli of the concrete in situ (Andersen and Nerenst 1952, Long et al. 1945, Whitehurst 1954). For these tests, a hammer blow was used to generate transient mechanical wave pulses that propagated along the surface of the concrete. Two receivers mounted on the same surface a known distance apart detected the arrival of the disturbance of the passing wave pulse. The arrival time of the first rise in signal above a given threshold value was determined in the time domain signals from each receiver, then the P-wave wave propagation time was determined by an attached electronic timer. After a lengthy dormant period, work on time-domain surface-guided wave propagation measurements re-emerged with interest in measuring Rayleigh surface wave characteristics in addition to those of P-waves (Abraham et al. 1998, Nazarian et al. 1997, Popovics et al. 1998, Qixian and Bungey 1997, Sansalone and Streett 1997, Wu and Fang 1997, Wu et al. 1995). In this more recent work, additional
signal parameters beyond the very first arrival are utilized. The velocity values obtained are used either to characterize the concrete or to estimate pavement thickness in conjunction with the impact–echo method.

In the 1960s, frequency domain-based approaches to characterize layered structures, where the layer stiffness decreases with depth as in a pavement structure, were studied (Jones 1962, Vidale 1964). In that work, the dispersion of surface waves as a function of frequency was evaluated in order to make inferences about the layered structure itself. The surface waves were generated using a variable frequency continuous surface wave source attached to a surface of the pavement, and the resulting dynamic response of the pavement structure was monitored with a set of surface sensors as the frequency of the wave source was increased incrementally. However, the limited computer power available at that time restricted complete and effective inversion of the collected data. Decades later in the 1980s, the spectral analysis of surface waves technique (SASW) was developed at the University of Texas at Austin, extending the early work of Jones and Vidale. SASW also determines the dispersive property of surface waves in the frequency domain, but a transient, broad frequency wave source is used instead of a swept single frequency source. The obtained frequency-dependent phase velocity of the surface wave is a function of the material properties at different depths, and estimates of the layered structure are obtained from inversion of the collected data. The multichannel analysis of surface waves (MASW) method, which was developed at the Kansas Geological Survey (KGS) (Park 1999) in late 1990s, is an extension of the SASW concept. MASW is based on signal processing and inversion of multi-mode seismic data sets, an approach that was pioneered by the geophysics community (Gabriels et al. 1987, Nolet et al. 1976, McMehan et al. 1981, Mokhtar et al. 1988). In MASW, multiple receiver data are manipulated to produce the phase velocity curves of the layered structure from which the data are collected. A good review of the development of such surface wave methods is provided by Park and Ryden (2007).

Over the past decade, the development of non-destructive techniques (NDT) to characterize surface-opening crack depth in concrete using surface waves have been reported (Hevin et al. 1998, Kee and Zhu 2010, Lin and Su 1996, Popovics et al. 2000, Sansalone et al. 1998, Wu et al. 1995, Sellick et al. 1998, Song et al. 1999, 2003, Yang et al. 2009). In this approach, the reflection/scattering interaction of a surface wave with a crack is monitored in order to characterize the nature of the crack. The method has also been applied to characterize distributed micro-cracks within a material (Gallo and Popovics 2005).

With recent improvements in instrumentation, air-coupled transducers have been used for detection of leaky surface (Zhu et al. 2001) and leaky
guided waves (Castaings et al. 2001). All NDT methods that utilize surface waves can be carried out using air-coupled sensors. With the advantage of non-contact sensing, air-coupled transducers provide an opportunity for quick scanning and imaging of large civil engineering structures by detecting the leaky surface wave.

Readers should refer to the list of cited references at the end of this chapter for guideline reference material on this topic.

20.2 Basic principles of surface wave propagation

20.2.1 Surface waves in a homogeneous elastic halfspace

The NDT methods that use ultrasound, acoustics, seismic waves and vibration are based on mechanical wave (also known as ‘stress wave’) phenomena. Two types of mechanical waves can propagate within the body of a solid material: P waves (also known as compressional waves) and S waves (also known as shear waves). In addition, surface waves propagate along the free surface of a solid. Figure 20.1 shows the transient wave field in a solid half-space owing to the action of a dynamic load applied on the free surface, where hemispherical P-wave and S-wave fronts are seen; the surface wave travels along a cylindrical wavefront near the free surface and contains most of the dynamic displacement energy. The P waves always travel with the highest velocity and surface waves the lowest. The characteristics of mechanical waves, such as propagating wave velocity, and amount of wave energy reflected from an interface between two distinct media, are a function of the elastic properties and mass density of the solid materials. For example, the velocity (speed) that these waves propagate through a material is controlled by the small strain elastic moduli (Young’s modulus, Shear modulus and Poisson’s ratio) and mass density of the material. Wave propagation characteristics are also influenced by the severity and location of internal air-filled defects such as cracks, voids, honeycombs and delaminations. Thus, mechanical wave measurements can be used to provide direct information about the condition of the material or structure under investigation.

The behavior of surface guided waves is now discussed. For a homogeneous, isotropic, elastic half space with a free surface, the wave that propagates along the free surface of the half-space is referred to as the Rayleigh surface wave (R wave). The Rayleigh wave phase velocity is defined by the equation (Achenbach 1973):

$$\left(2 - \frac{c^2}{C_S^2}\right)^2 - 4 \left(1 - \frac{c^2}{C_F^2}\right)^2 \left(1 - \frac{c^2}{C_S^2}\right)^2 = 0$$  [20.1]
where $c$ is the multiple surface wave phase velocity solution, $C_P$ the P-wave velocity, and $C_S$ the S-wave velocity of the material. The real-valued, non-negative value of $c$ is taken as the Rayleigh wave phase velocity $C_R$. We note that the Rayleigh wave as rigorously defined is non-dispersive, that is to say the phase velocity ($C_R$) is independent of frequency. This situation is attained practically when a single solid layer has a thickness that is much greater than the wavelength of the propagating wave, and any inhomogeneities within the solid material are much smaller than the propagating wavelength. If these conditions are not met, the surface wave behavior may become dispersive, and, as a result, this type of wave cannot be properly called a Rayleigh surface wave. Rather, non-Rayleigh surface waves will be identified generally as surface-guided waves, dispersive surface waves, and Lamb waves. Various different approximations to the solution of equation [20.1] for $C_R$ are provided in the literature (Achenbach 1973, Nazarian et al. 1999). The formulation provided by Achenbach is

$$C_R = \frac{0.862 + 1.14\nu}{1 + \nu} C_S$$ [20.2]

where $\nu$ is the Poisson’s ratio of the material. For concrete, $\nu$ typically varies from 0.15 to 0.25, and $C_R$ ranges from 90 to 92% of $C_S$ and from 58 to 53% of $C_P$ and $C_P$ typically ranges from 3800 m s$^{-1}$ to 4500 m s$^{-1}$ for sound concrete, and $C_R$ from 2100 to 2600 m s$^{-1}$.

Unlike P waves and S waves, most of the energy of Rayleigh surface waves is relegated to the near-surface region away from the surface (ACI 2005, Aki and Richards 1980). The amplitude of motion of the R wave decreases exponentially with depth. The affected depth is generally con-
Surface wave techniques for evaluation of concrete structures

Defined within a single wavelength, although a smaller level of motion can exist at deeper levels. So assuming \( C_R = 2200 \text{ m s}^{-1} \), a wave frequency of 30 kHz results in R waves that effectively penetrate 75 mm, whereas a wave frequency of 3 kHz results in 750 mm effective penetration. Since the penetration depth is inversely related to the frequency of the wave, lower frequencies appear to penetrate deeper into the material. For shallow (in the order of the propagating wavelength) layered media, the Rayleigh wave becomes dispersive because of the interaction of the surface wave motion with the underlying layer surface; this dispersion serves as a phenomenological basis of several NDT methods. Because longer wavelengths (lower frequencies) will be affected by the material properties of the deeper layers, these waves are more properly called surface-guided waves rather than Rayleigh waves. Unlike P waves, which are dominated by in-plane motions, R waves result in large amplitude out-of-plane motions, which are usually the dynamic characteristic being measured at the free surface.

Rayleigh waves and other types of surface-guided waves are most efficiently generated by a dynamic point load (small surface excitation area with respect to propagating wavelength), such as an impact event or a point harmonic vibrator acting on the free surface of a solid. Through the process of geometric attenuation, the energy of the wavefront decreases with radial distance \( (r) \) from a point source: as the surface of the wavefront expands the fixed amount of wave energy is spread over a wider area (Richart et al. 1970). For body waves, the amplitude decreases as a function of \( 1/r \) along the direction of propagation at the free surface, whereas for the Rayleigh wave, which propagates along a cylindrical front, the amplitude decreases as a factor of \( 1/r^{0.5} \) (Achenbach 1973). Because the attenuation is geometrically lower, Rayleigh waves propagate over large distances, and they often dominate the transient response generated by a point source of waves. However, because the Rayleigh wave velocity is lower than that of the body waves, it arrives later.

20.2.2 Surface waves in layered elastic media

Surface guided waves in layered media are affected by the interaction of the waves that are multiply reflected at the interface between layers. This behavior is much more complicated than the simple Rayleigh wave case, and cannot be expressed by simple analytical formulations. These surface-guided waves are expressed in terms of multiple (infinite) possible solution modes, both propagating and evanescent, and are usually dispersive, meaning that the phase velocity is frequency dependent and phase velocity does not necessarily equal group velocity. These modes are actually set up by the multiple reflections of body waves within the layers. Interpretation of the phenomena in terms of guided waves becomes appropriate beyond...
a certain distance away from the wave source – in the ‘far-field’ zone of the source. Within the near-field zone of the source, application of a guided wave mode model may not be appropriate.

The possible guided wave mode solutions are commonly expressed through dispersion curves that show wave propagation characteristics (e.g. phase velocity or intrinsic attenuation) as a function of wave frequency. Dispersion curves reflect properties of the layered structures (elastic constants and boundary conditions), and thus can be used for structure characterization. Real-valued phase and group velocity dispersion curves for a single solid layer overlaying a solid half-space are shown in Fig. 20.2. In one example, the 0.03-m-thick top layer is acoustically stiffer \( (C_P = 4340 \text{ m s}^{-1}, C_S = 2600 \text{ m s}^{-1}, \rho = 2400 \text{ kg m}^{-3}) \) than the underlying half-space \( (C_P = 4000 \text{ m s}^{-1}, C_S = 2400 \text{ m s}^{-1}, \rho = 2400 \text{ kg m}^{-3}) \), whereas in the other example the top layer is acoustically more compliant \( (C_P = 3619 \text{ m s}^{-1}, C_S = 2200 \text{ m s}^{-1}, \rho = 2400 \text{ kg m}^{-3}) \). The dispersion curve computation is carried out with GEOPSY (Wathelet 2008). In both examples, the phase velocity of the fundamental mode converges towards the Rayleigh wave velocity of the top layer as frequency increases. At low frequency the fundamental mode converges toward the Rayleigh wave phase velocity of a homogeneous medium having the properties of the underlying half-space. The excitation amplitude of the fundamental and the higher modes depends on the receiver and source location, excitation source and the excitability of each particular mode (Ryden 2004). The nature of these curves reflect properties of the layered structures (elastic constants, thicknesses and boundary conditions), and thus can be used for structure characterization. Much work has been

![Dispersion curves](image-url)

20.2 Real-valued phase velocity (solid lines) and group velocity (broken lines) dispersion curves up to 400 kHz frequency for a single solid layer overlying a solid half-space: (a) the top layer is acoustically more compliant than the underlying half space and (b) the top layer is acoustically stiffer than the underlying half-space.
Surface wave techniques for evaluation of concrete structures

carried out toward understanding the dynamic wave propagation response of general multi-layered structures by researchers in seismology (Ewing et al. 1957, Rosenbaum 1960, Gilbert 1964). Similar work has been carried out to consider guided ultrasonic waves in single isotropic and anisotropic plates for non-destructive evaluation (NDE) purposes (Chimenti 1997). Matrix techniques, which provide the far-field and long time characteristics of plane wave propagation in arbitrary multilayered structures, have been used effectively to understand the wave propagation phenomena in these types of structures (Lowe 1995). The analytical eigenvalue problem can be solved by numerical methods, such as Runge–Kutta direct integration (Aki and Richards 1980, Saito 1988, Takeuchi et al. 1972). Layered structures where the velocity of the top-most layer is higher than that of the underlying layers, called inverse media, often exhibit a dispersion curve where the modes cannot be easily separated or identified (Lu et al. 2007, Tokimatsu et al. 1992). In that instance models that take into account the source function, source and receiver location are needed to interpret the dispersion curve (Lai 1998). Analytical models have been used to obtain an exact formulation based on ray-superposition theory to directly compute the transient near-field response (within a radius of ten top layer thicknesses) at a point, owing to point-source excitation in a structure consisting of two layers on a half-space. These ‘exact’ models account for all possible modes (true and leaky) and are applicable to all layer thicknesses and wave frequencies (Ma and Lee 2006, Zhou and Popovics 2001, Zhou et al. 2000). Because of computational limits, however, these models cannot simulate the response for large time durations or at large distances (in the far field) away from the wave source.

Lamb waves are a specific case of surface-guided waves that travel within a thin (relative to propagating wavelength) single elastic and homogeneous plate of large areal extent. Two families of wave motion exist. The propagating wave modes of the symmetric family ($S_i$) are computed by

\[
\frac{\tan\left(\frac{\beta d}{2}\right)}{\tan\left(\frac{\alpha d}{2}\right)} = -\frac{4\alpha\beta\kappa^2}{(\kappa^2 - \beta^2)^2} \tag{20.3}
\]

and the anti-symmetric family ($A_i$) of wave motion by

\[
\frac{\tan\left(\frac{\beta d}{2}\right)}{\tan\left(\frac{\alpha d}{2}\right)} = -\frac{(\kappa^2 - \beta^2)^2}{4\alpha\beta\kappa^2} \tag{20.4}
\]
where $\alpha^2 = \left(\frac{\omega^2}{C_p^2}\right) - \kappa^2$ and $\beta^2 = \left(\frac{\omega^2}{C_s^2}\right) - \kappa^2$, $\omega$ is the angular frequency $= 2\pi f$, $\kappa$ is the wave number, and $C_p$ and $C_s$ are the P-wave and the S-wave velocities. The solutions of propagating wave modes can be solved in terms of $\omega$ and $\kappa$ by solving equations [20.3] and [20.4] and results are presented in a dispersion curve format. The phase velocity of a given wave mode is defined as $\omega/\kappa$, so a phase velocity dispersion curve set can be computed from the general dispersion curve data. Lamb wave phase velocity dispersion curves depend on the frequency and the thickness and elastic constants of the plate material.

In a fluid/solid half space system, for example a solid elastic half-space bounded by air, the wave field in the solid is similar to that in the free surface solid half space: all body waves and Rayleigh waves exist in the solid. In the near interface region, there also exists an interface wave, called the Scholte wave, propagating along the interface in both the fluid and solid. The propagating P-, S- and Rayleigh waves in the solid cause small disturbances at the interface resulting in out-of-plane surface motion. The resulting surface motion at each point causes an acoustic wave to ‘leak’ into the surrounding fluid, assuming the fluid has lower acoustic impedance than the solid. The superposed leaky waves that emanate from each point in motion form leaky P-, S- and Rayleigh wave wavefronts. The response of these leaky waves is of significance to NDE researchers where air coupled sensors are used to detect the surface-guided wave propagation in contactless fashion.

Leaky Rayleigh waves propagate with a velocity slightly higher than the ordinary Rayleigh wave, and attenuate more intensively with distance owing to continuous energy radiation into the fluid (Viktorov 1967). Leaky Rayleigh waves exist when the fluid wave velocity is slightly smaller than the leaky Rayleigh wave velocity of the solid (Mozhaev and Weihnacht 2002). For common fluids such as water and air at normal driving frequencies (<10 MHz), the viscous effect of the fluid can be neglected because the Reynolds numbers are far above the critical value of 2500 (Qi 1994). The properties (velocity and amplitude dispersion) of leaky waves are affected by both the fluid and solid (Gusev et al. 1996). The fluid is usually a homogeneous material that has uniform distribution in space, and the density and velocity are easy to determine. Therefore, material properties of the solid can be indirectly obtained by measuring leaky waves. Material variation of the solid influences leaky waves in the same way that it affects ordinary waves in the solid–vacuum cases. This provides the basis of air-coupled sensing techniques.
20.3 Signal processing and data presentation

20.3.1 Time domain analysis for homogeneous media

Surface-guided waves can be analyzed and interpreted either in the time domain or the frequency domain. Basic information about surface-guided wave arrival time and amplitude can be obtained from time domain signals, but this approach is effective only when the surface wave energy is principally relegated to the top surface, for example for high frequency waves or a thick (with respect to propagating wavelength) top layer, where the propagation medium must be homogeneous to a depth of approximately one wavelength from the surface. In these cases, the Rayleigh wave velocity of the top layer is obtained from determination of the group velocity arrival between multiple sensed locations on the surface. Impact-based one-sided techniques have been applied to measure velocity of surface-guided waves (Popovics et al. 1998, Sansalone and Streett 1997, Wu et al. 1995). Time domain signals obtained from sensed points that are separated a known distance ($\Delta x$) are analyzed. The arrival time of the surface wave in each signal is associated with a specific feature in the time signal: e.g. the occurrence of a peak (maximum) that precedes a sharp drop in signal value. The difference in arrival times is computed as $\Delta t_R$ and the surface wave velocity is then computed as:

$$V_R = \Delta x/\Delta t_R$$  \[20.5\]

Few improvements for one-sided velocity measurement for Rayleigh waves (R waves) in concrete have been reported. Most previous work assume that concrete behaves as a linearly elastic and homogeneous material. To satisfy this condition, considering the heterogeneous nature of concrete, relatively large wavelengths and distances between source and receiver, or between the receivers, is required in order to maintain stable R-wave velocity measurements (Sansalone and Street 1997, Wu et al. 1995). This requirement may restrict application of measurement from smaller sized samples. A large testing zone also works against the merit of one-sided impact-based methods, which can measure both the P-wave and R-wave velocities simultaneously when the spacing is smaller.

The maximum energy arrival concept has been adopted to determine $\Delta t_R$, for instance through phase match filtering (Dziewonski 1972). Another possibility is to employ a continuous wavelet transform (CWT). Experimental studies show the effectiveness of the proposed method, and a finite element analysis validates the findings (Shin et al. 2007).

Time domain data from the multiple sensors are usually presented in an x-t (‘waterfall’) plot, as shown in Fig. 20.3. Complementary information beyond Rayleigh wave velocity can be obtained by time domain analysis of
P-wave velocity using such multiple sensor data. This method, often called ‘seismic refraction’ is pertinent when the P-wave velocity of the material increases with depth. The arrival of the first propagating wave front (P wave) is plotted as a function of distance from the wave source. Discontinuities of the wave front arrival time with respect to distance indicate the presence of a distinct subsurface layer beneath the surface (Abraham and Derobert 2003).

20.3.2 Spectral analysis of surface waves (SASW) method

When the wavelength of propagating waves is of the order of the layer thicknesses, then frequency domain analysis methods are more appropriate. Two commonly applied frequency domain methods are the spectral analysis of surface waves (SASW) method and the multi-channel analysis of

![Image of an x-t 'waterfall' plot presentation of multiple time domain signals obtained from equally spaced receivers configured in a line away from the wave source.](image-url)
surface waves (MASW) method. The dispersion curves for a layered medium provide information about wave velocity as a function of depth that can be used to estimate stiffness profile of a material.

The SASW method is based on in situ near-surface seismic profiling, which can provide an estimate of shear modulus of the individual layers of a medium as well as estimate the thickness (Roesset et al. 1990). In SASW analysis, a surface wave is generated on the structure using an impact source or a frequency controlled point vibrator (Matthews 1996). The surface wave motion is then measured by two transducers a certain distance away from the source. The signals measured at each transducer are then analyzed, and phase velocity is calculated according to the phase difference between signals from the two receiving transducers. The elastic profile for the structure can be estimated from this obtained dispersion curve, which plots wave phase velocity vs. frequency. In layered media, phase velocity is expected to vary with frequency, corresponding to different depths through which the wave travels (Nazarian and Stokoe 1984). Figure 20.4 shows a schematic of the SASW testing set-up on a concrete pavement structure.

The phase velocities are calculated as a function of frequency by determining the time it takes for each frequency component to travel between the two wave sensors,

\[ V_{\text{phase}}(f) = X \frac{360}{\phi_i} f \]  

[20.6]

where \( V_{\text{phase}}(f) \) is the phase component for each frequency \( f \), \( X \) is the distance between receivers and \( \phi_i \) the phase angle of component at

![20.4 SASW test method testing configuration.](image)
frequency \( f \) in degrees. The wavelength \( \lambda \) at a given frequency component is given by

\[
\lambda = X \frac{360}{\varphi_i}
\]  

[20.7]

Phase velocity dispersion curves \( (V_{\text{phase}} \text{ versus } f) \) have been used to characterize layered structures, such as pavements (Jones 1962, Nazarian et al. 1988, Park et al. 1999 and Ryden and Lowe 2004).

An iterative process is used to estimate the stiffness as a function of the wavelengths and the phase velocities. A dispersion curve from acquired data is compared with modeled results and iteratively adjusted to minimize the error between the two. Using least squares error method, the best fit with the shear wave dispersion curve is found (Park et al. 1999).

Commercially available equipment for the implementation of this method is available, notably the ‘portable seismic pavement analyzer’ (PSPA) (Nazarian et al. 1998), which is effective for determining a modulus profile in a multi-layered system with known layer thicknesses. However, the use of SASW method to determine thickness is at best an estimate, as it relies on empirical relationships rather than precise measurement (ACI 2005).

The location and configuration of the testing set-up are important. For finite slabs, or structures where cracks and edges are present nearby, careful sensor orientation will help avoid undesired wave reflections from the edges. For a vertical reflecting boundary, the adverse effect of reflected waves can be reduced by placing the source between this reflecting boundary and the first sensor (Park et al. 1999).

According to some researchers, SASW is very effective for top layer characterization, when careful data collection procedures are followed and when the shear wave velocity of the top layer is greater than that of the underlying layer (Nazarian et al. 1988, Popovics et al. 2006, Ryden and Lowe 2004). In this case, SASW can be applied to predict the mean surface wave velocity of that top layer if the top layer is homogeneous and thick enough: the surface wave velocity is represented by the straight asymptotic portion of the fundamental mode dispersion curve (shown in Fig. 20.2), which is relatively easy to determine. However, computations to characterize intermediate underlying layers are more complicated, and several trials must be made to find the most accurate impactor-receivers set-up and frequency-wavelength relation (Nazarian et al. 1988, Park et al. 1999, Ryden and Lowe 2004). Since only two receivers are used, the SASW method has difficulty separating different propagating modes over a multi-layered system; thus SASW measures a superposition of all propagating waves at the specific receiver locations (Foti 2000). This problem is caused by jumps between different solution branches in the dispersion curve of the layered system;
the phase unwrapping process is disrupted by these jumps, leading to errors in the predicted dispersion curve (Ryden and Lowe 2004). Low signal to noise ratio (SNR) also hampers SASW analysis because constraints related to the desired signal consistency range are not always possible to achieve. Park and Miller proposed a set up to increase SNR and to use the dispersive characteristics of the evaluated media to match the fundamental modes more accurately (Park et al. 1999).

20.3.3 Multi-channel analysis of surface waves (MASW) method

The MASW method uses multiple sensors to record the complete wave field and thus is able to resolve the various wave mode branches. Because multichannel recording and processing schemes are employed, the MASW test gives reliable results even in the presence of higher wave modes and various types of ambient noise. One principal difference between SASW and MASW is the number of data collection points (collected wave signals) needed to carry out the analysis. SASW requires only two data points, whereas MASW requires more, of the order of tens of points depending on the required depth of analysis and image resolution. Figure 20.5 shows one possible MASW testing configuration with a phase velocity dispersion curve obtained from a pavement section using this configuration.
Another important difference is the number of wave modes that are assumed in the analysis. For SASW, only one dominant wave mode is assumed to exist, where the computed curve is iteratively adjusted until a best match is obtained between experimental results and that of the model. In the MASW method, the surface/plate wave dispersion curves of all the excited wave modes are computed across a range of frequencies. The dispersion curves that represent the system are obtained by processing the multichannel signals collected from sensors placed along a line with consistent spacing between each. The arrivals of multiple wave propagation modes are discerned, providing plots of phase velocity dispersion curves for the excited modes. With this technique, the fundamental and higher mode dispersion curves can be extracted from any type of layered structure (Park et al. 1999, Ryden 2004, Ryden and Lowe 2004, Ryden et al. 2004). Because multichannel recording and processing schemes are employed, the MASW test gives more reliable results even in the presence of higher modes of surface waves and various types of ambient noise.

In the MASW method, the time–space wave data collected from multiple channels are mapped to the phase velocity–frequency domain with the application of an appropriate mathematical transform, such as the p-omega transform (Mokhtar et al. 1988), also known as the ‘slant-stack’ transform (McMechan et al. 1981). Data collection is based on $N$ wave signals collected from the surface along an equally spaced (with spacing $X_1$) linear array of sensed points from the wave source (such as an impact event). The total length of sensed region is given by $X = N X_1$. The value of $X$ indicates the depth of analysis with MASW, and generally $X$ should be more than four times the desired depth of inspection (Bodet 2005, 2009). A set of $N$ time signals is obtained, which can be represented in a spatial offset-time domain plot. The signal set can be obtained by using several receivers and a single wave source, or a single receiver and multiple wave sources (Ryden et al. 2002). The most important testing parameter in the latter MASW setup is the trigger of the wave source signal when the single sensor–multiple wave source testing option is applied. An instrumented hammer is the recommended wave source. If an impact event is used to generate the waves, the impact must be provided by a high frequency impactor, having a small contact area.

The assembly of the plots across a frequency range provides a representation of the phase velocity dispersion curve for that pavement system, which can be constructed from multiple signal data. The data are not useable below a certain phase velocity–frequency ratio, because of signal spatial aliasing effects. This threshold phase velocity value is determined by the spacing between sensed points $X_1$, where closer spacing gives a lower threshold phase velocity for a given frequency. The resolution of phase velocity is controlled by the total length of the sensor array.
Lamb wave analysis has been applied to determine the top layer thickness and material properties of pavements. In this approach, only the top layer of a layered system is considered in the analysis, and it is appropriate in cases where a single layer lies atop a relatively thick underlying layer with a notably lower acoustic impedance. The obtained experimental dispersion phase velocity curves are matched to Lamb wave dispersion curves computed for a single layer of known thickness and mechanical properties. Typically analytically computed fundamental A0 and S0 Lamb mode dispersion curves are fitted to the MASW data by an iterative procedure. The computed Lamb wave curves are adjusted until a best match is obtained between experimental results and that of the model (Ryden 2004). Shear wave velocity, Poisson’s ratio and layer thickness estimates can then extracted (Popovics et al. 2007). The thickness is estimated by relating the maxima associated with the fundamental thickness resonance in the summed frequency domain amplitude spectra with the zero group velocity frequency of the S1 Lamb mode (Ryden and Lowe 2004). The inversion of deeper embedded layers is based on the full phase velocity spectrum of surface waves (Ryden 2004).

20.3.4 Surface wave attenuation analysis

Several studies on NDT to determine surface-breaking crack depth in concrete with waves have been reported. Many efforts make use of time-of-flight methods: the time required for waves to travel around the tip of a surface-breaking discontinuity is measured, from which depth is estimated. Several studies report success in determining the depth of simulated cracks (notches with well-defined tips) in concrete when the velocity of wave propagation in concrete is known and a particular wave pulse-crack interaction is realized (Lin and Su 1996, Sansalone et al. 1998, Wu et al. 1995). The time-of-flight method, however, is not effective when realistic concrete cracks are tested, that is when the crack tip is ill defined and the crack is tightly closed (Song et al. 1999).

More recently, NDT for characterizing the surface-opening crack depth in concrete using surface waves have been reported (Hevin et al. 1998, Kee and Zhu 2010, Popovics et al. 2000, Song et al. 2003, Yang et al. 2010). Among them, the self-calibrating surface wave transmission method seems to be a more promising approach for crack depth assessment of in situ surface opening cracks in concrete structures. Numerical studies and surface wave transmission measurements performed on concrete demonstrate superior sensitivity to the presence of cracking along the wave path in concrete as compared with time-of-flight based methods (Hevin et al. 1998, Popovics et al. 2000, Sellick et al. 1998, Song et al. 2003). However, accurate crack depth estimation from surface transmission remains a challenge. Studies on
crack depth estimation based on the cut-off frequency of the transmission function (TRF) have been reported (Hevin et al. 1998, Li et al. 1997, Scala and Bowles 2000). Even though the self-calibrating surface transmission method greatly reduces the effects of experimental variability in impact source and receiver characteristics, the variations in the measured TRF owing to variability in impact position and type of couplant may still be significant. Thus, the experimental determination of the cut-off frequency is not necessarily straightforward and the estimated crack depth may contain error (Hevin et al. 1998).

Wave transmission or attenuation measurements do show high sensitivity to realistic cracking in concrete under laboratory conditions (Suaris and Fernando 1987). However, the practical measurement of the wave transmission coefficient across cracks in concrete structures is made more difficult by disrupting effects of wave path dependence in heterogeneous materials, incoherent signal noise and source, and receiver and coupling variability. Practical one-sided surface wave transmission coefficient measurements in relatively homogeneous materials such as metals have been achieved through the use of a self-compensating testing scheme: after the disrupting experimental variability is eliminated, the obtained surface wave transmission coefficient is used to calculate the depth of surface-breaking cracks (Achenbach et al. 1992). The self-compensating approach has several advantages: the wave velocity of the material need not be known in advance and the technique is independent of the type of wave transmitter and receiver and coupling conditions. Numerical studies (Hevin et al. 1998, Yang et al. 2009), and self-compensating surface wave measurements on concrete (Popovics et al. 1998, 2000, Song et al. 1999) demonstrate great sensitivity of self-compensating surface wave transmission to the presence of cracking along the wave path, although the precise relationship between the crack depth in concrete and the surface wave transmission coefficient from the crack has yet to be established. The testing scheme for concrete is a modified version of that reported by Achenbach et al. (1992); the modification is needed to allow testing in inhomogeneous materials. Signal transmission obtained with the modified self-compensating scheme reduces sharply as crack depth relative to the wavelength of the surface wave increases, but is largely unaffected by concrete composition, the nature of a crack (crack opening condition) and the testing characteristics such as the type of wave transmitter and receiver (Popovics et al. 2000). In order to apply the results effectively, however, the accuracy of the measurement must be established and a unique relationship between the surface wave transmission coefficient across the crack and the depth of the crack in concrete obtained. In a recent study, numerical analyses were used to establish the relationship and provide guidelines for proper sensor arrangement in the surface wave transmission measurement (Kee and Zhu 2010). Refined experimental
results were obtained by using air-coupled sensors in the self-compensating scheme. The key findings of this study include the following:

1. Surface wave transmission measurements are affected by near field scattering by the crack tip. To obtain consistent and reliable surface wave transmission across a crack, the sensors must be installed in the far field. The far field distance depends on the crack depth and wavelength.

2. The relationship between surface wave transmission coefficient and crack depth obtained from numerical analysis and experimental measurements in the far field shows good agreement with the analytical solution (Achenbach 1980), as shown in Fig. 20.6 (Kee and Zhu 2010).

3. Air-coupled sensing improves signal consistency and test speed. Temporary mounting of accelerometers will induce a resonant response in low frequency range, which reduces the signal coherence.

20.4 Equipment

The following equipment is needed to perform the surface wave tests: a wave source, wave detection sensors, and a data acquisition and analysis system. The wave source is typically a local dynamic event, for example the impact of a steel sphere on the surface of concrete or an attached ultrasonic transducer. The size of the impactor controls the frequency content: smaller size provides higher frequency contents. The wave field produced by a dynamic impact event has broad spatial directivity, meaning that both body
wave components are produced and propagate in all directions along hemispherical wave fronts within the solid. Surface waves are also produced, and as discussed earlier they propagate in all surface directions along cylindrical wave fronts near the surface. Usually the wave detectors are surfacemounted sensors, such as geophones or accelerometers. These sensors provide a signal that is proportional to the dynamic surface motion caused by the wave propagation events; geophone signals are proportional to the velocity of the surface whereas accelerometer signals are proportional to the acceleration. Generally geophones are sensitive and effective sensors for low frequencies (below 1 kHz) and accelerometers for higher frequencies, although exceptions may occur. The sensors provide an analog voltage output, which is collected over a certain time period to give a time domain signal.

A typical instrumented hammer can generate signals up to 10 kHz on a hard concrete surface, whereas a specially designed impactor can generate signals up to 20 kHz. Thus, the penetration depth of R waves can be controlled by choosing an appropriate impact source.

One disadvantage of MASW is that it takes many steps to collect a complete set of data, and furthermore satisfactory coupling between receivers and the test surface may be difficult to achieve. It is time-consuming to couple the sensors required for the MASW test. Recent developments aim to increase the speed of surface wave measurements by removing the contact required between the propagation medium and the source and receivers. An additional advantage of a non-contact set-up is more accurate measurement of signal attenuation values. Laser interferometers have been used to record surface wave pulses propagating on concretes (Chekroun et al. 2009). A laser testing set up is shown in Fig. 20.7, where the surface waves are usually generated by a piezoelectric transducer in contact with the concrete surface. The advantage of the laser is the direct measurement of surface displacement, as opposed to surface acceleration, and the broad band (with respect to frequency) nature of the sensed signal. One disadvantage is the need in most instances to improve the surface reflectiveness, for instance with metal tape. Air-coupled sensors provide another solution to this surface-coupling problem, but it is expensive to use multiple sensors and multichannel data acquisition equipment. One alternative is to use the MSOR procedure (multichannel simulation with one receiver) (Ryden et al. 2004). The lone receiver is fixed at one position, whereas the source moves consecutively at equal spacing.

The data acquisition system digitizes the analog signal data, performs the analysis in either the time and frequency domain, and, where needed, performs the inversion and matching processes that should be carried out. These processes can be executed on PC-based computer systems, using commercially available software platforms. Commercially available testing
systems that can carry out many of the tests described here are also available.

20.5 Field application of surface wave methods

Surface wave methods have been applied to obtain estimates of layer thickness and stiffness in multilayered systems such as pavements and tunnel liners. For example the velocity of measured Rayleigh wave and refracted P-wave modes have been used to evaluate subsurface conditions: low velocity values can indicate weak surface conditions owing to scaling, spalling and minor cracking. The method has also been applied to detect the extent of a damaged zone near the surface of the tunnel liner, for example that caused by internal fire to a concrete (Abraham and Derobert 2003).

The SASW method has been successfully applied to determine the stiffness profile of asphalt and concrete pavement systems, although the accuracy of the thickness estimate is not as reliable as provided by other NDT methods such as impact–echo (ACI 2005). The application of SASW has been extended to concrete structures, although the relatively complicated signal processing and inversion procedure limit effective application to concrete defect detection.

MASW has been recently used to characterize cover-concrete with the objective of quantifying parameters required for the assessment of

The analysis and inversion methods can be numerically intensive, and they may not always reach the correct solution; this is especially true for the SASW method. As a result, a considerable amount of user expertise is needed to apply the SASW method. Because the sensors require physical contact with the surface of the tested specimen, the methods (using current, standard technology) are not readily applied for rapid scanning. Also, the surface conditions of the concrete may adversely affect the results, and where there is an extremely rough surface or limited access it cannot be applied. However, recent developments in technology such as wheeled sensor systems and contactless air-coupled sensors provide some solutions to the contact problem (Zhu and Popovics 2005).

20.6 References


Surface wave techniques for evaluation of concrete structures


Popovics, J.S., Song, W., and Achenbach, J.D. (1998) ‘A study of surface wave attenuation measurement for application to pavement characterization’, In: Medlock,


© Woodhead Publishing Limited, 2010


Impact–echo techniques for evaluation of concrete structures

O. ABRAHAM, LCPC, France; J. S. POPOVICS, University of Illinois, USA

Abstract: A summary of the history of the development of the impact–echo non-destructive test method is presented and the basic physical phenomena underlying the method are described in relation to non-destructive evaluation of concrete. Approaches to the data analysis and signal processing techniques, including time and frequency domain processing, are described. The equipment required and classical measurement configurations are outlined. Finally, classical applications of the impact–echo method are summarized.

Key words: concrete, impact–echo, non-destructive testing (NDT), non-destructive evaluation (NDE), ultrasound, vibration, data processing.

21.1 History of the development of the method

The impact–echo method was first developed at the US National Bureau of Standards (NBS), which is now called the National Institute of Standards and Technology (NIST) and Cornell University during the mid-1980s for non-destructive testing (NDT) of concrete (Carino et al., 1986; Sansalone et al., 1988, 1997; Carino, 2001). According to this method, an impact source is applied at the surface of the structure, and a transducer near the impact point measures the dynamic surface response. The success behind the impact–echo approach lies in processing the recorded signal within the frequency domain rather than attempting in the time domain to separate wave arrivals. In concrete structures, material heterogeneity induces high attenuation and scattering that complicate time-picking techniques to a greater extent, for instance, than in steel (Carino, 1984). The frequency domain classically encountered in concrete applications extends between 2 and 40 kHz, which corresponds to an investigation depth ranging from 0.05 to 1 m. Interestingly, this type of frequency content may be generated merely by steel ball impacts, which are very inexpensive and easy to use. Moreover, measurements in this frequency band were made possible at the time thanks to a transducer developed at NBS (Proctor, 1982).
The impact–echo field of application has been broadening over time, beginning with thickness measurements and extending to various flaw detections (Sansalone et al., 1997). The impact–echo method can be used to detect voids, debonding (Sansalone et al., 1989), honeycombing (Sansalone et al., 1988) and cracks (Cheng et al., 1995a, 1995b). Recent applications have focused on detecting delamination (Cheng et al., 1993), voids in grouted tendon ducts (Abraham and Côte, 2002; Carino et al., 1992; Dérobert et al., 2002; Jaeger et al., 1996), as well as on the non-destructive testing of concrete pipes, columns and bars (Kim et al., 2002; Lin et al., 1992, 1993, 1994a, 1994b; Popovics, 1997) and masonry structures (Williams et al., 1997; Sadri, 2003).

21.1.1 Reference manuals, guidelines and standards

A reference book was published by Sansalone and Street in 1997; it summarizes all of the results obtained at both NBS (now NIST) and Cornell University (USA) over the previous two decades. The ASTM C1383-4 Standard entitled ‘test method for measuring the P-wave speed and the thickness of concrete plates using the impact–echo method’ is available. Similar guidelines are currently being developed in Germany (Merkblatt B11, to appear in 2010).

21.2 Basic principles of the impact–echo method

21.2.1 Wave propagation

The impact–echo method is a testing method that remains fully non-destructive to the structures under study (and is typically applied to concrete slabs with two parallel surfaces). It is based on a frequency analysis of the structure’s vibrational response when subjected to a shock (Sansalone et al., 1997). Following an impact, the waves propagating inside the structure will be reflected from the boundaries of the underlying layers. After multiple reflection between the top and bottom surfaces of the slab, a resonance phenomenon can be observed. This results in a maximum peak in the frequency spectrum of the signal, which is then used to recover information on the slab thickness (see Fig. 21.1).

The amplitude reflected at the boundary of two media, labeled 1 and 2, is equal to the incident amplitude multiplied by the reflection coefficient \( R \) (0 > \( R > 1 \)), which is expressed as:

\[
R = \left( \frac{Z_2 - Z_1}{Z_2 + Z_1} \right)
\]  \[21.1\]

where the mechanical impedance, denoted \( Z_i \) for material \( i \), corresponds to the density \( \rho \) times the wave velocity \( V \), i.e.:
Thus, for a concrete slab, almost all of the energy will be reflected owing to the significant difference between the mechanical impedances of concrete and air \( (R \approx 1) \).

The primary applications concern the non-destructive testing of plate-like structures, for which traditional wave propagation equations in solids are used to interpret the data. In this configuration, the structural response is often recorded or modeled in the time domain. We will now see that the traditional set of main governing parameters consist of the compression wave velocity \( V_p \) and plate thickness \( e \), which appear in the equation as \( V_p/(2e) \), where:

\[
V_p = \sqrt{\frac{\lambda + 2\mu}{\rho}}
\]  \[21.3\]

\( \lambda \) and \( \mu \) are the Lame’s constants and \( \rho \) the density. \( \mu \) is also called the shear modulus and is directly related to the shear wave velocity \( V_s \):

\[
V_s = \sqrt{\frac{\mu}{\rho}}
\]  \[21.4\]

The compression wave velocity is notably higher than shear wave velocity for all known materials.
21.2.2 Vibrational resonances

A vibration resonance is a repetitive (usually harmonic in nature) motion of a structure with respect to a stationary frame of reference. Each resonance is associated with a natural frequency of repetition and modal shape that is bounded by the limits of the structure’s displacement field. A single structure will have an infinite number of different possible resonances, divided into families of distinct mode shapes with different modes orders within each. For example, a plate-like structure will exhibit flexural and thickness stretch families of vibration modes. Within each family, the fundamental mode has the lowest frequency and simplest mode shape. In all cases, the resonant frequencies are a function of the mechanical properties (elastic moduli and mass density), the shape and boundary conditions of the structure under study. They will also carry information about material damage, the presence of internal inclusion (such as steel, void, delamination, crack, etc.), local modifications of shape. Resonance frequency measurement can thus provide information about those parameters. In the following discussion about the impact–echo method, we will concentrate on plate-like structures.

21.3 Data interpretation

21.3.1 Conventional interpretation of impact–echo spectral response in plate-like structures

The principle behind the impact–echo method has traditionally been explained by the fact that the compression wave (or P wave) periodically reflects on the free surface and bottom surface, or on the top of cavities or, more generally, on the interface of two layers with different mechanical impedances. Figure 21.1 shows an example with a slab of thickness $e$ containing a void with depth $d$. The sensor is placed near the point of impact. In a conventional approach, the resonance frequencies of the slab thickness and above the void are computed based on the amount of time necessary for a back-and-forth oscillation of the P wave, $\tau_e$ and $\tau_d$, respectively, as given by the following equations:

$$\tau_e = \frac{2e}{V_P} \quad \text{and} \quad \tau_d = \frac{2d}{V_P} \quad [21.5]$$

where $V_P$ is the velocity of the compression wave in the slab.

The periodicity of the multiple echoes produces, in theory, resonance frequencies

$$f_e = \frac{V_P}{2e} \quad \text{and} \quad f_d = \frac{V_P}{2d} \quad [21.6]$$
It became apparent that for plate-like structures, the frequency $f_e$ should be multiplied by a shape factor $\beta$. Sansalone proposed that $\beta$ be set equal to 0.96 for concrete (Sansalone et al., 1997), to yield the impact–echo thickness frequency $f_{ZGVS1}$

$$f_{ZGVS1} = \beta \frac{V_P}{2e}$$  \[21.7\]

(usage of the subscript $\text{ZGVS1}$ is justified below). In the following section, the exact formulation of the thickness resonant frequency, which explains the origin of the shape factor $\beta$, is derived from Lamb wave dispersion theory.

### 21.3.2 Guided-wave interpretation of impact–echo spectral response in plate-like structures

In 2005, Gibson and Popovics explained the origin of the shape factor $\beta$ in the impact–echo method. Their explanation also included a proper analytical formulation of the impact–echo phenomenon. They demonstrated that $\beta$ depends solely on the Poisson’s ratio value of the material and, moreover, that the actual underlying physical phenomenon in the impact–echo method is stationary Lamb waves, which exist in plate-like structures and correspond to vibrational resonances set up by multiple reflections of both compression waves and shear waves within the thickness. This phenomenon ultimately yields the modes of propagation, which can be classified into two groups depending on the mode shape symmetry with respect to the middle plane of the plate: they are either symmetric (S) or anti-symmetric (A). For a linear, elastic homogeneous material, the phase velocity $c$ of the symmetric and anti-symmetric Lamb wave modes of a plate of thickness $e$ can be computed as a function of frequency $f$ with the two following sets of equations (Viktorov, 1967), for symmetric modes:

$$\tan\left(\sqrt{1-\zeta^2} \bar{e}\right) \quad \frac{\tan\left(\sqrt{\bar{e}} - \xi^2 \bar{e}\right)}{\tan\left(\sqrt{\bar{e}} - \xi^2 \bar{e}\right)} = -\frac{4\xi^2 \sqrt{1-\zeta^2} \sqrt{\xi^2 - \zeta^2}}{(2\xi^2 - 1)^2}$$  \[21.8\]

and for anti-symmetric modes:

$$\tan\left(\sqrt{1-\zeta^2} \bar{e}\right) \quad \frac{\tan\left(\sqrt{\bar{e}} - \xi^2 \bar{e}\right)}{\tan\left(\sqrt{\bar{e}} - \xi^2 \bar{e}\right)} = -\frac{(2\xi^2 - 1)^2}{4\xi^2 \sqrt{1-\xi^2} \sqrt{\zeta^2 - \xi^2}}$$  \[21.9\]

where:

$$\bar{e} = \frac{\pi ef}{V_P}, \quad \xi^2 = \frac{V_S^2}{V_P^2} \quad \text{and} \quad \zeta^2 = \frac{V_S^2}{c^2}$$  \[21.10\]
Figure 21.2 shows the phase and group velocity dispersion curves for a plate of thickness $e = 0.25$ m with $V_p = 4000$ m $s^{-1}$ and a Poisson’s ratio equal to 0.23 for both symmetric (S) and anti-symmetric (A) modes. The critical resonance frequency of each mode corresponds to the frequency at which the phase velocity tends to infinity. The impact–echo resonance frequency $f_{ZGVS1}$ corresponds to a frequency very close to the critical resonance frequency of the first symmetric mode (S1), which equals:

![Diagram of dispersion curves](image)

**21.2** Normalized dispersion curves of a slab of thickness 0.25 m with $V_p = 4000$ m $s^{-1}$, $\rho = 2400$ kg m$^{-3}$, with a Poisson’s ratio equal to 0.23: (a) phase velocity dispersion curves, (b) group velocity dispersion curves (after Benmeddour, 2006; Benmeddour et al., 2008a, 2008b). The impact–echo resonance frequency $f_{ZGVS1}$ corresponds to the frequency at which the Lamb mode S1 has a group velocity equal to zero and a finite phase velocity.
Non-destructive evaluation of reinforced concrete structures

\[ f_e = \frac{V_p}{2e} \]  \hspace{2cm} \text{[21.11]}

\( f_{ZGVS1} \) is indeed the frequency (slightly less than \( f_e \)) at which the group velocity of mode S1 is zero and the finite-valued phase velocity exhibits minimum frequency (Clorennec et al., 2007; Gibson et al., 2005; Holland et al., 2003). At frequency \( f_{ZGVS1} \), the energy is trapped inside the slab below the source; it does not propagate. The classical name in the literature for this frequency is ZGV, which stands for zero group velocity.

Gibson et al. (2005) showed that \( \beta \) in equation [21.7] only varies with the Poisson’s ratio. Figure 21.3 presents the values of the shape factor as a function of Poisson’s ratio. When the Poisson’s ratio equals 0.22, i.e. a classical value for concrete, the shape factor \( \beta \) is equal to 0.950. One issue yet to be resolved is the pertinence of holding the shape factor \( \beta \) above a circular void, e.g. in the case of a tendon duct inspection (Sansalone et al., 1997). Moreover, it should be noted that the same phenomenon exists for the second antisymmetric Lamb mode (A2) with a Poisson’s ratio of below 0.45 (Fig. 21.2). For instance, this frequency appears in Fig. 21.4a (as the second peak with maximum amplitude). Consequently, by using both the S1 and A2 ZGV resonance frequencies, it is possible to determine the Poisson’s ratio of the material under study (Clorennec et al., 2007). This property is certainly promising relative to impact–echo signals in the field of material characterization in civil engineering.

21.3.3 Interpretation of impact–echo spectral response in other structures

The impact–echo method has been applied to many other shapes (other than plates) for non-destructive testing purposes. In such instances, the
interpretation of the resonance frequencies, and eventually the mode shapes, is much more complicated than for plate structures. As a result, the application of impact–echo to these structures has been limited. The conventional interpretation of the resonant frequencies in these structures is still based on the function \( V_P/(2\pi) \) (Kim et al., 2002; Lin et al., 1992, 1993, 1994a, 1994b; Sansalone et al., 1997), although a guided wave-based interpretation has also been proposed (Popovics, 1997).

21.3.4 Time domain determination of the compression wave velocity

The estimation of thicknesses or defect depths requires measurement of the compression wave velocity, a step that involves two sensors separated by a known distance on the free surface. An impact is generated in line with both sensors, and their signals are recorded simultaneously. The arrival times of the compression wave are then selected on both signals. The compression wave velocity is computed as the distance between the two sensors divided by the difference in the two arrival times (the time elapsed for the compression wave to travel between the two sensors). The compression wave arrival time is indicated as the time of appearance of the very first disturbance in the signal. The shape factor value is then set equal to 0.95; as for the classical Poisson’s ratio of concrete, which varies from 0.18 to 0.22, \( \beta \) varies from 0.954 to 0.95.

The time-of-flight technique can also be applied to the fundamental Lamb modes that dominate the time signal and whose velocity is related to the shear wave velocity (Popovics et al., 1998). At high frequencies, the \( A_0 \) group velocities, as measured by means of time-picking, converge towards the Rayleigh wave velocity \( c_R \), given by (Viktorov, 1967):

\[
c_R = \frac{0.87 + 1.2\nu}{1 + \nu} V_S \tag{21.12}
\]

where \( \nu \) is the Poisson’s ratio, and

\[
V_P = V_S \sqrt{\frac{2(1-\nu)}{1-2\nu}} \tag{21.13}
\]

The velocity \( c_R \) can also be estimated by a spectral analysis of surface waves (Kim et al., 2006; Nazarian, 1993), although equation [21.13] is only valid when \( A_0 \) dispersion (i.e. velocity varying with respect to frequency) can be neglected, in which case both the phase and group velocity are identical. The information on shear wave velocity can also be obtained via a classical multichannel spectral analysis of surface waves (Ryden et al., 2006). The corresponding experimental set-up, which uses several measurement points,
allows for mode separation and thus avoids misinterpreting the dispersion curve.

Recent improvements include the use of impact–echo measurements together with multi-channel analysis of surface waves (MASW) in order to simultaneously invert the dispersion curve of the A0 Lamb mode phase velocity dispersion curve and the impact–echo thickness resonance frequency (Ryden et al., 2006).

### 21.4 Numerical simulations

According to Sansalone (1997), an important factor for ensuring successful impact–echo development is the effort devoted to numerical simulations. Even though dynamic finite element modeling has not been at the origin of recent explanations of the impact–echo phenomenon by means of the Lamb wave ZGV frequency, it has nonetheless offered the possibility to do the following: explain measured signals in the case of shallow delaminations (Cheng et al., 1993) and cracks (Cheng et al., 1995b); forecast the sensitivity of impact–echo measurement to various fault sizes (Abraham and Côte 2002); validate the hypotheses used in semi-analytical expressions for structures other than plates (Kim et al., 2002, Lin et al., 1993, 1994a); and lastly, visualize mode shapes (Hill et al., 2000), including those related to the Lamb wave (Gibson et al., 2005). Figure 21.4 displays the dynamic finite element computation of vertical displacement after an impact centered at 10 kHz above a 0.25-m thick slab. Near the impact source (at x = 0 m), the stationary S1 mode at \( f_{ZGVs1} \) is clearly visible. Use of the dominant A0 Lamb mode, which corresponds to the highest amplitude in the mea-

---

**21.4 Measurement carried out on a 0.25 m thick slab (the steel ball diameter is 0.011 m): (a) signal recorded near the impact point, (b) amplitude of the fast Fourier transform.**
sured signal, has been proposed to recover the shear wave velocity of the concrete (Ryden et al., 2006).

21.5 Signal processing, data presentation and imaging

21.5.1 Basic Fourier transform

To obtain information on the frequency content of a digitized impact–echo time signal, only a simple fast Fourier transform (FFT) is needed. Figure 21.5 shows a typical signal recorded above a wall 0.25 m thick with the impact of a ball 0.011 m in diameter. The resonance phenomenon in the time signal can be seen after the arrival of the A0 Lamb mode. Yet no time-picking can be used to recover information about the slab because several wave modes are mixed and interfere with one another. In the frequency domain, the impact–echo resonance frequency $f_{ZGVSI}$ is clearly observed.

![Finite element modeling of the surface response of a 0.25 m thick slab to an impact. The source is located at $x = 0$ m. The slab is 1.5 m long. The stationary vibration corresponds to the impact–echo resonance frequency at $f_{ZGVSI}$. The A0 Lamb mode is also clearly visible.](image-url)

© Woodhead Publishing Limited, 2010
When recording the signal, the signal sampling frequency $f_s$, which corresponds to a time interval between each point equal to $\Delta t = 1/f_s$ in the digitized time signal, should be chosen judiciously so as to avoid temporal aliasing. In practice, when no filtering exists before digitization, it is recommended that the highest frequency present in the signal be ten times smaller than the sampling frequency $f_s$.

The frequency resolution of the FFT $df$ is related to both $\Delta t$ and $N_t$ (the total number of signal points), i.e.:

$$df = \frac{1}{N_t \Delta t}$$  \hspace{1cm} [21.14]

### 21.5.2 Time–frequency analysis

A principal limitation of the impact–echo method, when the frequency domain of the source has been well chosen (Colla, 2003), is interpreting the Fourier domain maxima: this interpretation step becomes increasingly difficult in the presence of noise, attenuation and multiple waves (Abraham et al., 2000). The FFT modulus (also called amplitude or magnitude) sometimes proves difficult to interpret because of the many peaks it contains or when the expected resonance frequency is high and/or the source is lacking appropriate frequency content. It is indeed recommended to use a source with a frequency range close to the frequency to be determined. Nevertheless, when the targeted frequency is high, the use of an appropriate source always leads to dissipation phenomena owing to the diffraction and rapid attenuation caused by wave scattering from concrete aggregates and inhomogeneities. By reducing the observation time window, attenuation will broaden the characteristic spectral peak thickness and increase the amplitude of secondary lobes. The amplitude of these secondary lobes adds to the measurement noise content and is detrimental to method performance. Another difficulty stems from the A0 Lamb mode, which provides information on the frequency content of the source: its amplitude is often paramount in the Fourier domain and can mask useful peaks.

For improved frequency extraction, many signal-processing techniques have been tested. Sansalone and Streett (1997) proposed the technique of time signal ‘clipping’ in order to reduce influence of the A0 Lamb mode when clipping the signal if it exceeds a predetermined value. However, the non-repeatability of the source and experimental conditions make this technique arbitrary and introduce an artifact into the Fourier transform. In addition, the actual impact–echo test signal often features considerable noise and contains multiple frequencies. Among today’s signal processing techniques for improving the impact–echo method, the idea of performing a time–frequency analysis on the impact–echo signal seems
most reasonable because it allows for the consideration of *a priori* knowledge regarding the impact–echo test. Abraham *et al.* (2000) proposed a sliding window and later (Abraham *et al.*, 2002) a wavelet transform to better detect voids in a concrete slab. Yeh *et al.* (2008) adopted a wavelet transform to detect both shallow and deep cracks. Bédaoui *et al.* (2009) proposed a continuous wavelet transform (CWT) to extract the S1 Lamb mode in noisy signals based on the continuity of its ridge (Fig. 21.6). Figure 21.6a shows the signal energy (local spectral density) in the time–frequency domain, where the horizontal ridge (where the modulus of the CWT is maximum) corresponds to the zero group velocity frequency of the S1 Lamb mode. The energy of the ridge is extracted and the frequency content of the reconstructed signal is shown in Fig. 21.6b, which is superimposed on the FFT modulus of the original signal.

### 21.5.3 Other 2-D and 3-D image construction

For a rather long period, impact–echo data were measured manually using a sparse grid point system. An A-scan is one signal (as in Fig. 21.4), which here has been represented by the modulus of its FFT. If the measurement device is moved along a line, then the plot of the corresponding A-scan data collected and stacked into a single image is called a B-scan, which represents a slice image into the depth of the concrete. A horizontal slice image, at some depth below the surface, is called a C-scan. It is now well accepted that B-scans and C-scans (Abraham *et al.*, 2002; Colla *et al.*, 2000) are to be preferred since they improve the level of confidence on results and furnish a map of the required parameter. In addition, when searching...
for a localized fault, these scans provide information on slight thickness changes that may otherwise induce a false diagnosis.

Classically, a B-scan consists of a measurement line where the impact–echo signal is recorded every 2 to 5 centimeters, depending on the application. Figure 21.7 presents a B-scan crossing an empty tendon duct in a test wall 0.25 m thick (from LIER test walls at LRPC of Lyon France, Roenelle et al., 2006). Each measurement point is separated by 0.02 m. The downward frequency shift above the empty duct is clearly visible.

Figure 21.8 shows a C-scan measured with a laser interferometer above a wall containing several tendon ducts. The wall has an average thickness of 0.25 m. The plot depicts the FFT amplitude of each point at two frequencies: one lying below the thickness frequency (6.1 kHz) and the other around the thickness frequency (7.8 kHz). With the C-scan at 6.1 kHz, it is possible to follow the empty pipes. The C-scan around the thickness frequency

---

21.7 B-scan above an empty tendon duct in the 0.25 m thick slab. Each measurement point is separated by 0.02 m. The thickness frequency shift above the empty duct is clearly visible. The grey scale represents normalized spectral magnitude.
21.8 (a) A 0.25 m thick wall with several tendon ducts; (b) C-scan at the frequency 6.1 kHz. This frequency is lower than the impact–echo thickness resonance frequency $f_{ZGVS1}$. The lighter shaded zones correspond to the detection of an anomaly (partial filling). (c) C-scan at a frequency of 7.8 kHz. Measurements conducted in the ANR French project ACTENA. The grey scale represents normalized spectral magnitude.
resonance frequency indicates that the thickness at the top of the wall is slightly reduced.

### 21.6 Equipment

One advantage of the impact–echo method is that the cost of equipment required to carry out measurements remains quite low. This is partly because of the following characteristics:

- the source system, which has remained quite simple, being the impact of a steel ball on a concrete surface;
- signal acquisition, which can be performed using conventional ultrasonic systems. An acquisition frequency of 1 MHz is sufficiently high, in conjunction with a 5-ms time for data acquisition.

The most important part of the equipment is the sensor. First of all, it must be wide band, i.e. with a frequency response as flat as possible within the range 2 to 50 kHz. These frequencies correspond to average thicknesses of between 5 cm and 1 m. Second, the required number of measurements should make it possible to use the equipment without any surface preparation and without a coupling agent. The receiver is thus often composed of a sensor with a small tip head (Proctor, 1982). Therefore, to conduct an impact–echo measurement, the following items are needed:

- an established source: steel balls with diameters varying from 4 mm to 30 mm for a concrete thickness of up to 1 m;
- a receiver: a piezoelectric transducer with a large bandwidth, typically between 1 and 50 kHz;
- an amplifier for the received signal;
- a digitizing acquisition card with a sampling frequency greater than 500 kHz and with more than 2048 points;
- a computer to visualize and analyze the data.

The source size selection depends on the target thickness or depth. Its frequency content should include the expected resonance frequency (Abraham et al., 2000). As an initial approximation, the maximum frequency content of the impact is estimated as 3/2 of the inverse of impact contact time. The smaller the steel ball diameter, the shorter the contact time.

Recent developments have been aimed at increasing impact–echo measurement speed by the use of non-contact receivers. Mori et al. (2002) used an air gun as the source and a laser interferometer as the receiver. Zhu et al. (2007) successfully employed an air-coupling receiver both to detect shallow delaminations and voids with a steel ball impact and to perform a C-scan above a reference slab with numerous flaws. Abraham et al. (2009) used a laser interferometer coupled to a steel ball as a source for detecting
the high-frequency range of the void effect in a tendon duct on the impact–echo signal response.

21.6.1 Conventional testing systems

The first equipment was rather cumbersome to operate and required at least two technicians for the measurement phase, one to handle the source and sensor while the other recorded the signal on a computer. Handheld units, which allow one person alone to conduct the measurement, are now available on the market.

21.6.2 Automation equipment

Although a B-scan can be performed manually, an automated system (Streicher et al., 2006) is almost always necessary to handle C-scans, which correspond to a dense 2-D measurement grid with a grid size of between 2 and 5 cm. Such robots are currently being tested at BAM in Germany (Algernon et al., 2008; Kurz et al., 2009; Streicher et al., 2006; Wiggenhauser et al., 2005). In studying the performance of the impact–echo method, Beutel et al. (2008) showed that thickness measurements, using reference slabs ranging from 0.111 to 0.256 m thick (Taffe et al., 2003), could be improved by using a scanner (should the compression wave velocity be estimated from a measurement above a known thickness).

21.7 Impact–echo method applications

21.7.1 Slab/wall thickness

The impact–echo method was first applied to measuring the thickness of plate-like structures. Improvements made in the determination of compression wave velocity enables thickness measurements with an accuracy of better than 4% (Popovics et al., 2006) for concrete applications. One extension to the slab/wall thickness measurement consists of the detection of voids or debonding behind a concrete wall or below concrete pavements (Henriksen, 1995; Wouters et al., 1999). In this instance, the survey procedure calls for precise knowledge of the slab response in the absence of disorder.

21.7.2 Internal defect detection

Delamination

The impact–echo method has also been applied to detect delaminations within concrete slabs (Lin et al., 1990) or between different concrete layers,
e.g. the case of non-ballasted railway track (Colla et al., 2002), or between concrete and hard rocks (Song et al., 2009).

For a shallow delamination, with length greater than depth, the structural response to a classical impact–echo source excitation is dominated by a low resonance frequency $f_{\text{delam}}$, corresponding to fundamental flexural vibration of the delaminated layer. $f_{\text{delam}}$ is usually clearly visible in both the spectrum and the time signal: the periodic oscillations in the time signal attenuate slowly once energy is trapped between the delamination surface and the top. This resonance frequency is difficult to correlate with the depth and extent of the delamination because boundary conditions remain unknown. Nonetheless, the greater the delamination area, the smaller the value of $f_{\text{delam}}$. The extent of delamination can be estimated by means of impact–echo scanning by following $f_{\text{delam}}$; moreover, its depth may be estimated by a complementary impact–echo measurement using a short-duration (high-frequency) impact that will locally excite the stationary S1 Lamb mode resonance at the corresponding thickness resonance frequency. Unfortunately, if delamination is very shallow, the latter measurement may prove difficult to perform as a result of energy dissipation in the concrete owing to intrinsic damping and scattering at high frequencies, along with the difficulty of sending sufficient energy into the required frequency range with a small steel ball.

The detection of debonding between materials relies on a study of the modifications in thickness frequency of the multilayered structure. For a two-layer structure, the thickness frequency of the perfectly-bound structure, i.e. $f_{\text{twolayers}}$, as characterized by thicknesses $e_1$ and $e_2$ and compression wave velocities $V_{P1}$ and $V_{P2}$, is given by (Sansalone and Streett, 1997):

$$f_{\text{twolayers}} = \frac{1}{\frac{2e_1}{V_{P1}} + \frac{2e_2}{V_{P2}}} \quad [21.15]$$

This relation is obtained by considering the propagation time of the compression wave in both layers. $f_{\text{twolayers}}$ is measured like for a perfectly bound material. The reflection with the first layer will be visible if the contrast between layers 1 and 2 is sufficient (a minimum value commonly accepted for the reflection coefficient, equation [21.1], is: $R = 0.24$. If the material below layer 1 has a lower mechanical impedance $Z_2 < Z_1$ equation [21.7] can be used to recover the thickness of layer 1. If its mechanical impedance is higher $Z_2 > Z_1$, the resonance associated with the top layer is not set up (Sansalone et al., 1997).

For materials with similar mechanical impedances (equation [21.2]) that correspond to reflection coefficients of below $R = 0.24$, the thickness resonance frequency of the top layer will become visible only when
delamination exists. This new phenomenon can thus be regarded as a valuable debonding indicator.

**Internal voids, including post-tensioning (PT) duct voids**

Void detection makes use of the mechanical impedance contrast between air and concrete (the reflection coefficient $R$ is nearly 1) and corresponds to a traditional field of application for the impact–echo method.

If the lateral void dimension is greater than 1.5 times its depth (see Fig. 21.1), then the impact–echo response of a structure to an impact will be similar to that of a slab with a thickness equal to the void depth (Sansalone et al., 1997). The measured resonance frequency will thus directly yield information regarding void depth, through use of equation [21.6].

The detection of a void whose lateral dimension is less than 1.5 times its depth is also possible. Within a slab, voids will first be noticed by a downward shift in the thickness resonance frequency (Fig. 21.9). This shift is currently observed above voided tendon ducts, for which void localization is of primary importance (Abraham et al., 2002; Jaeger et al., 1996). Figure 21.9 provides an estimation of this frequency shift, in considering various empty duct sizes. In the field, however, a frequency shift can sometimes also be observed above fully filled ducts (Jaeger et al., 1997), and this is why current practice recommends caution when applying this technique to locate voids in tendon ducts. In this type of study, it is also important to know the slab thickness value: a B-scan crossing the tendon duct may serve as a reference to check whether a downward shift is not caused by an increase in slab thickness.

**Other applications**

Because the response of a structure to an impact is tied to the mechanical properties of the particular structure, the impact–echo method has also been used to characterize structural modifications during laboratory testing. In this instance, the geometry of the structure often remains unchanged, and variations in resonance frequencies are directly correlated with the evolution in material properties. As an example, laboratory experiments have been reported in studying concrete setting time and strength (Pessiki et al., 1988), whereby the resonance frequencies of a concrete cylinder are monitored during early-age hardening and provide information on evolution in the compression wave velocity directly linked to the material modulus. The impact–echo method can also be used to follow concrete material damage, e.g. when exposed to fire (Kumar et al., 2008), or microcracking in relation to rebar corrosion damage (Liang et al., 2001).
One final field application calls for use of the impact–echo method to locate honeycombing (Carino, 2001). In this instance, no reflection from the fault is usually visible, but the thickness resonance frequency is still modified to a lower value.

### 21.8 Future trends

As with contact seismic methods, the principal drawback of impact–echo is its limited measurement speed and that it does not offer wide spatial

---

21.9 (a) Schematic diagram of the impact–echo method for the detection of a small void; (b) finite element evaluation of the thickness resonance frequency shift (%) above a void versus both depth \( (p) \) and diameter \( (D) \).
coverage about each test point. Efforts to develop non-contact sensors may prove decisive for the rapid inspection of very large areas, as typically encountered in civil engineering, with impact echo (Zhu et al., 2007). The future of non-contact sensors will depend on their capacity to make measurements while in motion. Another important future application field is the mechanical characterization of material, for example the measurement of Poisson’s ratio with impact–echo. Indeed if the zero group velocity of S1 and A2 Lamb modes are measured on a slab, it is possible from a single measurement to recover the Poisson ratio (Clorennec et al., 2007).

21.9 References


486 Non-destructive evaluation of reinforced concrete structures


CARINO, N.J., Laboratory study of flaw detection in concrete by the pulse echo method. *ACI SP-82*, American Concrete Institute, Detroit, Michigan, USA, 557–579, 1984.


© Woodhead Publishing Limited, 2010


Ultrasonic techniques for evaluation of reinforced concrete structures

M. SCHICKERT, Institute for Materials Research and Testing (MFPA Weimar), Germany; M. KRAUSE, BAM Federal Institute for Materials Research and Testing, Germany

Abstract: An overview of ultrasonic pulse–echo and transmission techniques that can be used for flaw detection, localization of internal objects, and materials characterization is presented to provide information about the state of concrete structures. The underlying physics of elastic wave propagation and imaging methods are discussed. Although several transmission techniques are mentioned, the main focus is on pulse–echo imaging including synthetic aperture focusing technique (SAFT) reconstruction, which is described from the authors’ perspective. Descriptions of measurement systems and application examples are intended to show current and future measurement possibilities.

Key words: ultrasonics, elastic wave propagation, reinforced concrete, flaw detection, imaging, synthetic aperture focusing technique (SAFT) reconstruction.

22.1 Introduction

An ultrasonic wave travelling through concrete will interact with any obstacle in its path. Measured changes of the propagation characteristics can be attributed to material properties or structural characteristics of the concrete. These fundamental relations led to the use of ultrasound for non-destructive testing (NDT) in civil engineering more than 70 years ago (Meyer and Bock, 1939, Jones, 1948). Although the wavelength of ultrasonic waves is small compared with the size of constructional elements, it is large enough for sensitive interaction with features of interest. Since ultrasonic waves propagate best in solids and propagation is blocked by even thin layers of air, ultrasonic testing is mainly applied to concrete and cement and much less to masonry.

The application of ultrasonic testing is divided into two areas: the characterization of concrete properties and the provision of information about the inner structure of concrete elements. The characterization of concrete properties is typically done by transmitting an ultrasonic pulse through the concrete, measuring pulse velocity and/or attenuation, and relating the
results to concrete properties such as elastic moduli, compression strength, or the setting condition. Relatively inexpensive instruments are available that are easy to operate.

Investigations of the inner structure of concrete elements can be based on transmission and reflection measurements. In a transmission measurement, changes of the propagation time indicate the existence of inner objects. One-sided reflection measurements make direct use of signals reflected by inner objects in the concrete. Objects can be localized by the lateral measurement position on the surface and the propagation time of the ultrasonic waves. Examples of applications are the localization of inner objects such as tendon ducts and the detection of flaws. Recording a number of measurements enables imaging of interior areas of concrete, with much better detection and localization possibilities. For this more involved equipment, ranging from handheld instruments to large automated testing equipment is necessary.

In this chapter, the focus is on research, development, and application work done at the reporting institutes, with a limited overview of the field of ultrasonic testing of concrete. First, the fundamentals of wave propagation in concrete are explored including peculiarities caused by the inhomogeneous composition of concrete that affect the possibility and reliability of ultrasonic testing of concrete. A survey of the applications and requirements of ultrasonic NDT follows. A brief section on the characterization of concrete properties and on flaw detection in transmission is followed by a section on imaging of concrete elements, which starts with an explanation of concepts and notions. Actual measurement equipment of different complexity is discussed, and typical application examples performed by the reporting institutes are given. Actual developments of measurement techniques are summarized as future trends, and finally resources for further information are listed.

### 22.2 Ultrasonic wave propagation in concrete

Elastic wave propagation in homogeneous solids is a classical subject (Krautkrämer and Krautkrämer, 1990) and fundamental for considering wave propagation in concrete. The basics necessary for understanding the methods used in later sections are summarized in the following. The inhomogeneous nature of concrete causes a number of peculiarities which are explained afterwards.

#### 22.2.1 Elastic waves

The propagation of elastic waves in solids is most easily viewed in a fundamental configuration that is also a good approximation to many practical
situations. In this arrangement, two half spaces comprised of gas and solid, respectively, are divided by a boundary plane. The two spaces are each assumed to be homogeneous. In both spaces, pressure waves can propagate; these are also known as longitudinal waves because the mass particles are oscillating along the propagation direction. With $\rho$ the density, $E$ the (dynamic) modulus of elasticity and $\mu$ the Poisson ratio of the solid, the propagation velocity of the pressure wave in the solid is:

$$c_p = \sqrt{\frac{E(1-\mu)}{\rho(1+\mu)(1-2\mu)}}$$  \[22.1\]

The subscript P denotes pressure waves.

The second wave type is the shear wave. It propagates with the shear velocity $c_S$ (subscript S denotes shear)

$$c_S = \sqrt{\frac{E}{\rho(2(1+\mu)}}$$  \[22.2\]

and it is also called the transversal wave because it lets mass particles oscillate in a direction perpendicular to the propagation direction. Various oscillation directions are possible, and the actual plane the particles are oscillating in is called the polarization plane. Shear waves are confined to the solid.

A third wave type, the Rayleigh wave, propagates in the solid along the boundary between gas and solid, and is sometimes called the ‘surface wave’, although this use is unfortunate because other waves exist at surfaces that possess different properties. The amplitude of the Rayleigh wave decreases rapidly with the distance from the boundary, most of the energy being contained in a layer of one wavelength width. Its propagation velocity is roughly:

$$c_R = \frac{0.87 + 1.12\mu}{1+\mu} c_S$$  \[22.3\]

and this is always a bit smaller than the shear wave velocity. The Rayleigh wave causes particles to oscillate elliptically in a plane perpendicular to the boundary.

This elastic wave model is valid for all frequencies, leading to wavelengths that depend on the respective propagation velocity $c$ of the wave type and the frequency $f$ by:

$$\lambda = \frac{c}{f}$$  \[22.4\]

In the volume of a solid or close to its surface, these three wave types all propagate independently of each other with their respective propagation
velocity. This means that they can cross without interacting. It is only at boundaries, on the outer surface to the gas or at inner surfaces to objects or voids contained in the solid, that they are converted into each other owing to physical laws depending on materials and shapes.

When a wave reaches the boundary of an obstacle on or in the solid, part of the wave’s energy will be transmitted into the material behind the boundary, and the remaining part will be reflected. Depending on the relation of the wavelength to the size of the obstacle, the resulting effects are summarized under the terms reflection for small wavelengths and scattering for small obstacles. Diffraction describes the deviation of a wave from its propagation direction caused by obstacles. To estimate the amounts of the transmitted and reflected parts, it is convenient to evaluate a combined material property, the acoustic impedance. The acoustic impedance of a material depends on the density \( \rho \) of the material and the propagation velocity of the considered wave \( c \) by

\[
Z = \rho c
\]

If a homogeneous plane wave in material 1 is perpendicularly incident on a boundary to a material 2, and the boundary dimensions are large compared with the wavelength such as at a plane surface, reflection and transmission will occur. The reflection factor \( r \) and the transmission factor \( t \) computed from the connected acoustic impedances by:

\[
r = \frac{Z_2 - Z_1}{Z_2 + Z_1} \quad \text{and} \quad t = \frac{2Z_2}{Z_2 + Z_1}
\]

yield the amplitudes of the reflected and the transmitted wave, respectively. For smaller obstacles and concentrated wave beams, these factors still give a first approximation. It should be kept in mind, however, that these factors only describe boundary conditions for a certain wave type and do not account for mode conversion.

The model used so far was that of two half-spaces of gas and solid, separated by a single boundary. One can still apply the above relations when considering geometries involving additional boundaries if the distance between the boundaries is much larger than the wavelength in the solid. If this condition is not satisfied, other wave types occur that match the additional boundaries. One important example is wave propagation in a solid plate with two parallel boundaries to gas as occurring in ceilings and walls. Here Lamb waves propagate along the plate, with symmetrical or antisymmetrical displacement of the boundaries. Depending on the relationship between wavelength and plate thickness, different modes of this wave type
propagate, each with its own transverse amplitude distribution and propagation velocity, which can depend on frequency. Since Lamb waves are not used throughout the rest of this chapter no further details are given here.

22.2.2 Elastic wave propagation in concrete

In a step towards practical application, the two half-spaces comprised of solid and gas are replaced by concrete and surrounding air. Hardened concrete is a material composed of the three phases cement matrix, aggregate, and air pores. It is a rather heterogeneous material compared with the homogeneous solid assumed in the former section. In order to estimate the consequences for the propagation of ultrasonic waves, Table 22.1 contains typical acoustic impedance values of the three components, and (homogeneous plane wave) reflection factors relative to the cement matrix.

These values indicate that concrete is a strongly heterogeneous medium. As a consequence, a propagating wave is continuously scattered by aggregate and pores, which is the most prominent cause for attenuation of an incident ultrasonic pulse. Assuming an exponential attenuation model, the attenuation coefficient \( \alpha \) has been empirically determined as \( \alpha = 5.6 \text{ dB MHz}^{-1} \text{ cm}^{-1} \) (Schickert, 1999), which is about 5 to 10 times larger than the one characterizing biological soft tissue (Atkinson and Woodcock, 1982). All waves scattered at aggregate and pores superimpose to an incoherent, fluctuating wave that is called *structural noise* when received. On its way through the concrete, the energy of the coherent wave is more and more shifted to the incoherent part. Table 22.1 also shows that steel objects such as reinforcement bars or pre-stressing wires are clearly reflecting. Whether they can be detected depends on their actual size and on the amount of structural noise.

Figure 22.1 shows the main consequences of the scattering for the ultrasonic testing of concrete (Schickert, 2002); note that the plots have different time scales. If two ultrasonic transducers are placed on opposite sides of a concrete element in transmission mode, the originally excited, coherent

<table>
<thead>
<tr>
<th>Concrete component</th>
<th>Acoustic impedance, ( Z_L )</th>
<th>Reflection factor, ( r )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement matrix</td>
<td>( 7 \times 10^6 \text{ Ns m}^{-3} )</td>
<td>–</td>
</tr>
<tr>
<td>Aggregate</td>
<td>( 17 \times 10^6 \text{ Ns m}^{-3} )</td>
<td>0.4</td>
</tr>
<tr>
<td>Air (pores)</td>
<td>( 0.4 \times 10^3 \text{ Ns m}^{-3} )</td>
<td>-1.0</td>
</tr>
<tr>
<td>Steel</td>
<td>( 46 \times 10^6 \text{ Ns m}^{-3} )</td>
<td>0.7</td>
</tr>
</tbody>
</table>
part of the wave arrives first, followed by the incoherent part making up for the structural noise (Fig. 22.1a). Since all components of hardened concrete sit firmly in their places, neither part of the signal changes with time and time-averaging only reduces additional electrical noise. If, in contrast, a single transducer is used as sender and receiver in pulse–echo mode or two different sending and receiving transducers are placed side-by-side on the concrete, a signal similar to the example in Fig. 22.1b results. Scattering at aggregate and air pores starts immediately after the excited wave impinges on the concrete. The backscattered parts that reach the receiving transducer add up to form structural noise. To be detectable, any reflections from buried objects need to have higher reflection amplitudes than structural noise received at the same time. In the example in Fig. 22.1b, the later reflection (from a back wall) is quite easy to detect, but the earlier reflection (from a hole) is hard to distinguish from structural noise.
The strength of scattering at an obstacle depends on the ratio of cross-section and the wavelength of the incoming wave; it diminishes if this ratio becomes small. The easiest way to reduce structural noise is therefore to lower the frequency of the excited wave. This will increase the wavelength according to equation [22.4] and thus decrease the scattering strength, allowing deeper penetration of the concrete. Consequently, for ultrasonic testing of concrete much lower frequencies are used than for testing steel or for medical ultrasonics. In combination with typical sound velocities, wavelengths of a few centimetres follow. This is larger than the size of most aggregate and much larger than the size of air pores. Unfortunately, the sensitivity to detect searched objects is limited at the same time. As a consequence, broadband transducers are employed to balance between resolution and penetration depth. Another consequence of using long wavelengths which are in the same range of the transducer diameters or larger, is a broad divergence angle under which waves are sent into the concrete.

In Table 22.2 some test parameters are summarized in their typical ranges for pressure wave transducers and an often used shear wave transducer. The ranges may be exceeded in special cases.

Attenuation is not the only consequence of scattering. An additional effect is caused by the frequency-dependent behaviour of scattering and results in acoustic low-pass filtering of the propagating wave (Schickert, 1999). If the wave is excited by a short pulse which contains a wide frequency range then this range will be narrowed and shifted to lower frequencies during propagation. The effect will increase with longer propagation paths, larger aggregate, and higher porosity. In the time signal, the original pulse shape will be smaller and broader.

In Fig. 22.2 the low-pass filtering effect is illustrated by sample signals after travelling different path lengths through concrete (Schickert, 2007). The concrete had 16 mm maximum aggregate size and average porosity. For easy comparison, all signals were scaled to the same amplitude, time-shifted to a common onset, and the coherent signal parts were extracted by time-windowing. The broadening effect owing to frequency-dependent

### Table 22.2 Typical ranges of testing parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Pressure wave</th>
<th>Shear wave</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sound velocity</td>
<td>3800–4500 m s(^{-1})</td>
<td>2400–2750 m s(^{-1})</td>
</tr>
<tr>
<td>Frequency range</td>
<td>50–200 kHz</td>
<td>55 kHz</td>
</tr>
<tr>
<td>Wavelength</td>
<td>90–20 mm</td>
<td>45–50 mm</td>
</tr>
<tr>
<td>Range</td>
<td>40–600 mm</td>
<td>80–1000 mm</td>
</tr>
<tr>
<td>Divergence angle</td>
<td>40–100°</td>
<td>100°/180°</td>
</tr>
</tbody>
</table>

*In polarization–propagation plane/perpendicularly to it.*
sound attenuation is obvious. It also occurs in pulse–echo measurements, with the result that signal shape depends on concrete depth. This effect can be seen in the structural noise in Fig. 22.1b.

The effects of aggregate size and porosity on pulse-broadening are shown in Fig. 22.3a and 22.3b, respectively (Schickert, 2007). The signals travelled a propagation path of 50 cm through concrete test specimens made from different recipes, and all signals were time-shifted to a common onset, and the coherent signal parts were extracted by time-windowing. The relative signal amplitudes were maintained. Signals become wider and much smaller with increasing maximum aggregate size. Porosity has a similar effect, but to a lesser extent.

The changes of signal shape in Fig. 22.2 correspond to a frequency-dependent behaviour. This makes it difficult to meaningfully define the propagation velocity of a wave. Originally, the sound velocity is defined as being the phase velocity. But the phase velocity, despite the additional effect that the phase velocity is frequency-dependent in concrete owing to dispersion, is not easy to measure in a medium with frequency-dependent sound attenuation. Instead, the term ‘pulse velocity’ is used in this chapter to mean the propagation velocity of a designated point of a pulse such as the pulse onset, the first negative pulse maximum, or the maximum of the mathematical envelope. The pulse velocity $c$ can be calculated from a measured propagation time $t$ from one transducer to another and the propagation path $s$ between the two transducers by

$$c = \frac{s}{t}$$

[22.8]
Note that different definitions of designated points of a pulse may result in different pulse velocities. Here $t$ stands for the propagation time in the test object alone; signal delays through the measurement system need to be subtracted.

The most prominent effects of elastic wave propagation in concrete can be summarized as follows:

- structural noise caused by scattering at aggregate and pores,
- pulse attenuation, primarily at higher frequencies,
- small scattering amplitudes at obstacles, primarily at lower frequencies,
- broad divergence angle,

22.3 Examples of transmitted signals showing the influence of concrete properties on signal shape: (a) varying maximum aggregate size at average porosity, (b) varying porosity at 16 mm maximum aggregate size, all signals at 50 cm propagation path.
• change of pulse shape, and
• mode conversion between wave types.

All these effects are frequency dependent, and all effects caused by scattering are connected to the concrete recipe, most notably the sizes and volume fractions of aggregate and porosity.

In order to better understand the often quite complicated propagation of elastic waves in concrete, numerical simulation of elastic wave propagation can be helpful. The foundation and examples of simulations using the elastodynamic finite integrate technique (EFIT) can be found in chapter 8. A simulation result including synthetic aperture focusing technique (SAFT) image reconstruction will be discussed later in Section 22.5.4.

22.3 Applications and requirements of ultrasonic non-destructive testing

NDT with ultrasonic waves can be used to solve a number of tasks that arise in association with manufacture and use of concrete elements. The main fields of application are quality control, condition analysis, and flaw detection. A generic concrete element shown in Fig. 22.4 is used to illustrate the inspection possibilities. It contains reinforcement close to the upper and lower surfaces and embedded tendon ducts. In this example, the following common inspection tasks can be performed employing ultrasonic testing.

Characterization of concrete material properties:

• Elastic moduli: frequently used for the development of concrete recipes and quality control.
• Compressive strength: most important concrete property; difficult to determine.
• Other properties: such as porosity, moisture content, and setting of concrete.

22.4 Common inspection tasks at a concrete element.
Flaw detection in transmission:

- **Defects**: detection of voids, delaminations, and cracks.
- **Micro-cracking**: detection of micro-cracking regions owing to chemical processes or freezing.
- **Crack depth**: depth determination of surface-opening cracks.

Pulse–echo measurements and imaging:

- **Thickness**: at single points or on grids to image back walls of varying thickness.
- **Position**: lateral position and depth of internal objects such as tendon ducts, reinforcement bars, and built-in parts.
- **Honeycombing**: position and size of un-compacted concrete areas.
- **Voids in tendons**: detection and localization of voids inside and outside of tendon ducts.
- **Delamination**: position, size, and thickness of delaminations.

Any technique considered for testing concrete structures has to fulfil requirements specific to applications in civil engineering. Most importantly, results need to be clear and unambiguous. Evaluation results can cause far-reaching decisions, thus customers require statements such as ‘defect’/‘no defect’ or dimensions in centimetres including uncertainty. As such, conclusions can often not be drawn solely on grounds of ultrasonic measurements. Instead, it can be necessary to apply more than one NDT technique, possibly backed up by destructive probing. Two-dimensional maps or images of the results ease interpretation. Result interpretation should be made not only from a non-destructive, but also from a civil engineering point of view. Application of the techniques should be fast, require little effort, be reasonably priced, and should not interfere with construction work or traffic. In general, an application-oriented approach, not a method-oriented approach is demanded in NDT of concrete.

### 22.4 Transmission methods

Transmission methods have the longest tradition in ultrasonic testing of concrete. The first ultrasonic instrument for concrete testing was built by Leslie and Cheesman (1949) to determine concrete materials properties. Developments of transmission methods have mainly been driven by the civil engineering community.

The basic measurement principle is the determination of ultrasonic propagation velocity and/or attenuation. Results obtained from different samples or from different positions of the same sample are correlated with the designated quantity. Many applications are performed on laboratory samples to determine certain concrete properties. A survey of stress wave
applications for condition assessment of infrastructure systems is given in chapter 19. In this section, some special ultrasonic measurement possibilities are added. Extensive surveys on ultrasonic methods can be found in Galan (1990) and Malhotra and Carino (2004); a review of American literature is contained in Popovics and Rose (1994).

22.4.1 Measurement devices

Transmission measurement devices are used for the determination of ultrasonic pulse velocity and attenuation. During the past few decades, a variety of such systems have been developed. Two transducers, sender and receiver, are connected to the main unit. Usually low-frequency (20 to 100 kHz) transducers with low damping are employed to offer high-energy pulses. Pulse velocity results of different systems should be compared with caution since measured values depend on the measurement principle (Schickert, 1995a).

New developments include air-coupled and laser-generated measurement equipment. Piwakowski and Safinowski (2009) describe an automated device utilizing air coupled transducers. It generates Rayleigh waves on a single concrete surface and measures the dispersive propagation velocity and attenuation, which can then be correlated to concrete properties. Ultrasonic transmission experiments using laser-generated and laser-detected elastic waves were conducted by Erfurt et al. (2006). Applications are directed towards homogeneity control of concrete structures.

22.4.2 Characterization of concrete material properties

The dependence of ultrasonic propagation velocity or attenuation on concrete material properties is always indirect. Since propagation quantities depend on many properties of concrete, successful measurement of a single property requires that the influence of all other properties remains small. Additionally, disturbances by other factors such as material variations and temperature need to be ruled out. In reinforced concrete, the possible parallel wave propagation through reinforcement bars needs to be observed (RILEM, 1972).

Elastic moduli

The dynamic modulus of elasticity of concrete can be calculated from the propagation velocity of pressure waves using equation [22.1]. Lower precision requirements are met by estimation of the Poisson ratio; additional measurements of the shear wave velocity and inclusion of equation [22.2] allow for determination of both moduli at once. The values measured using
ultrasonic elastic waves can differ from those obtained with the resonant-frequency method, and both are generally higher than static moduli (Hohberg, 2001).

**Compressive strength**

The correlation of pulse velocity to compressive strength is among the earliest applications of ultrasound to concrete testing. This method relies on the empirical correlation curves found in transmission experiments, which are subsequently used to estimate the compressive strength of concrete samples and structures (Malhotra, 1984). The results are particularly sensitive to the influence of other parameters, so the method should be applied only under controlled conditions and with verified statistical significance (EN 13791:2007).

**Other properties**

Many other concrete properties have been correlated with ultrasonic transmission experiments, including porosity (Hernández et al., 2006), moisture content (Rollet et al., 2008), and the setting of concrete (Grosse and Reinhardt, 2003).

22.4.3 Flaw detection in transmission

Obstacles in the propagation path between ultrasonic sender and receiver cause a loss of signal amplitude and an additional pulse delay. These effects can be used for flaw detection if successive measurements are carried out on line or planar grids. Transducers can be placed in transmission on both sides or over an edge of a concrete element, or in pitch–catch configuration on the same side. The detection probability is affected by the grid interval and the size of an obstacle relative to the propagation path and the wave length. A survey of earlier applications is contained in Schickert and Schnitger (1985).

Although current pulse–echo methods usually have higher sensitivity and better localization ability and are therefore more often used, transmission methods are still advantageous in cases where clear reflectors are lacking or too much structural noise is encountered. Multiple transmission measurements can be combined for tomographic imaging (22.6.3).

The range of detectable defects comprises voids and honeycombing, delaminations, and cracks perpendicular to the propagation direction. On surfaces, Rayleigh waves can be used for detection of surface-breaking cracks (Zhu and Popovics, 2008). A method of crack depth measurement is described in JCMS (2003b). Freeze–thaw cycles can induce micro-cracking
in exposed concrete regions, which can also be revealed by use of transmission experiments (Auberg et al., 1999).

### 22.5 Imaging of concrete elements

#### 22.5.1 Concepts and notions

Imaging of concrete elements can be done in reflection and transmission mode. The emphasis here is on reflection mode, where echoes reflected at inner objects are measured at one-sided access. The techniques used have mainly been developed since the 1990s and are often adopted from NDT of steel or have a radar background. Two-sided transmission tomography will be discussed later in 22.6.3.

The general measurement setting is shown in Fig. 22.5. A number of single measurements are conducted on a linear grid (x direction). An image calculated from these measurements will be a cross-section in the xz plane. Alternatively, many single measurements are taken on a planar grid (xy plane). From this three-dimensional data set, either two-dimensional cross-section images or depth section images can be calculated, or the complete xyz volume beneath the measurement plane can be displayed as a three-dimensional image.

**Single A-scan measurements**

In a basic ultrasonic reflection experiment, a wave is sent from one point of the surface into the concrete, and reflected waves are received at the
same point. For the pulse–echo technique, a single transducer is used for sending and reception, and two parallel transducers are used in pitch–catch technique. Fig. 22.6 shows the setting for a pulse–echo experiment. The concrete test specimen in this illustration was made from concrete with 8 mm maximum aggregate size, has a pulse velocity of 4360 m s\(^{-1}\), and contains two side-drilled holes as artificial targets. Its width in the \(y\) direction is about 30 cm.

The time record of the received signal is called an A-scan (Schickert, 1996). An example acquired at this sample is shown in Fig. 22.1b. Signals can be displayed as bipolar, as in this example, or as rectified.

In a homogeneous medium, the A-scan would be zero except where echoes from obstacles are received. As was pointed out in 22.2.2, this is not the case for concrete where the useful signal is superimposed by structural noise. If an echo indication exceeds the structural noise it can be detected, and its propagation path \(s\) can be obtained from pulse arrival time \(t\) and pulse velocity \(c\) by:

\[
s = \frac{c}{2} (t - t_o).
\]  

according to equation [22.8]. The time offset \(t_o\) is the total time delay of the pulse through the measurement system. Thus, the distance of an object can be determined quite accurately provided the pulse velocity is known. The A-scan of the example in Fig. 22.1b was collected from the sample in Fig. 22.6 at \(x = 12\) cm. The most prominent reflection originates from the back wall, the depth of which can be determined using equation [22.9]. The left echo reflected from the left hole is almost lost in noise. To determine the

![Diagram](image-url)

**22.6 Ultrasonic transducer measuring a sample containing two side-drilled holes in the pulse–echo technique.**
lateral position of the hole, the transducer is moved along the surface until the amplitude of the reflection is maximized.

However, as pointed out earlier, a large divergence angle impedes accurate positioning. Structural noise and echoes stemming from Rayleigh waves or from other mode-converted waves superimpose and obscure the useful signal. At different measurement positions, the noise, clutter, and coupling conditions may vary in an unpredictable way, and it is difficult to forecast whether an object will be detectable. Thus single A-scans are often difficult to interpret and are used only for unambiguous tasks such as thickness measurements or the detection of large objects.

**B-scan line measurements**

When object reflections cannot be easily detected in single A-scans, more information from additional measurements may solve the problem. The idea behind B-scans is to convert many time signal measurements into an image helping the eye to discriminate between object indication patterns and structural noise (Schickert, 1996).

To this end, a number of A-scans are recorded on a linear grid (Fig. 22.5). The transducer is led by hand or by a mechanical scanner. From the received signals, a B-scan image is formed by converting the amplitudes of each A-scan into shades of grey or colours using a colour table, and plotting them side by side, yielding an image similar to a cross-section. The result is a two-dimensional image in co-ordinates of the scanning position and of the depth into the material, the latter being calculated as for the A-scans.

For the following example, the sample in Fig. 22.6 was measured at 51 surface positions at line intervals of 1 cm. The resulting B-scan is displayed in Fig. 22.7, which contains the original high frequency data converted to the shown grey scale.

The later reflection of Fig. 22.1b is contained in all measurements and now extends to a line. This is the indication of the back wall. The line is disturbed at two places owing to shadowing from the two side-drilled holes. Reflections from the holes are received at several transducer positions owing to the wide divergence angle of the transducers. They describe hyperbolae that are typical for reflections from localized objects. The lateral positions of the side-drilled holes can be determined as being the measurements with the shortest propagation times.

Although the signal-to-noise ratio remains the same as in the A-scans, the B-scan is easier to read. Obviously, the addition of the second dimension supports the human eye in interpreting the measurements. For these reasons, using B-scans may often be desirable when testing concrete provided the effort of scanning the surface on a linear grid is tolerable.
C-scan planar measurements

Producing C-scan images requires a planar scan of the concrete element. For each measurement position a single amplitude value of the received signal at a chosen, constant time instant is saved. These values are converted to grey scale or colour images similar to B-scans and plotted in a scan grid, imaging a two-dimensional depth cut through the concrete element at a certain depth in co-ordinates of the scanned plane.

The large number of measurements requires the use of a mechanical scanner. Compared with the relatively time-consuming planar scan, the extra effort to capture complete time signals and subsequent processing by the synthetic aperture focusing technique (SAFT) algorithm described in the next section requires little additional effort and results in a much higher information content. This makes C-scans much less used than in NDT of steel and fibre-reinforced composites. Instead, depth cuts of three-dimensional SAFT images are used.

SAFT imaging

The ultrasonic SAFT reconstruction is an imaging method utilizing the information content of many echo measurements (Schickert et al., 2003, Schickert, 1995b). The measurements are recorded on the surface of the concrete at a linear or planar grid, sometimes called an aperture. Since the measurement data is the same in single or multiple B-scans, SAFT reconstruction can be seen as a post-processing method. In the reconstruction calculation, the SAFT algorithm coherently superimposes the measured signals to the pixels or voxels of the reconstructed region, thus synthesizing a transducer of the size of the aperture with variable focus to each image element. FT-SAFT is a faster calculation variant which executes the reconstruction in the Fourier domain. More details are
given in chapter 8. Line scans lead to two-dimensional images and planar scans to three-dimensional images. Indications in SAFT images are reconstructions of internal object boundaries. Reconstruction results can be displayed as cross or depth sections through the reconstructed concrete region (Fig. 22.5) or, in the three-dimensional case, as iso-surface volume views (Fig. 22.18).

For illustration, the data from the B-scan in Fig. 22.7 was SAFT-reconstructed yielding the image in Fig. 22.8. The magnitude of the complex reconstruction data is displayed in the given grey scale, which is used for all SAFT images throughout this chapter. The image is depth-corrected based on statistical signal processing (Schickert, 2003). Compared with the B-scan in Fig. 22.7, the reflections of the two side-drilled holes are now concentrated to form separate indications representing the top of the drillings. The back wall is visible as a horizontal line, partially shadowed by the side-drilled holes. Using a pressure wave transducer with a relatively high frequency range of 330 to 680 kHz results in sharp indications but also in increased structural noise.

Within the limits set by wavelength, attenuation, and structural noise, SAFT reconstructions provide detailed information about the imaged concrete region and can therefore be used for detection and localization tasks. The superposition process reduces structural noise, which can be a severe problem in single pulse–echo measurements, by configurational averaging. On the other hand, the reconstruction process can introduce artefacts induced by physical limitations and wave propagation effects such as mode conversion. In order to avoid coming to the wrong conclusions, the resulting images need to be interpreted, utilizing additional information if possible.

22.8 Two-dimensional SAFT reconstruction of the sample containing two side-drilled holes.
Measurement equipment

Measurement equipment specifically designed for ultrasonic pulse–echo testing of concrete is still rare. Some instruments are exemplified in this section, complemented by imaging systems developed at the reporting institutes that were used for the application examples given in 22.5.3.

Measurement instruments and transducers

The development of ultrasonic pulse–echo instruments and transducers for concrete testing is more complicated than for transmission measurements. Ultrasonic instruments for concrete are often specialized versions of steel testing instruments. They are equipped with high-energy pulsers, low frequency amplifiers, and sometimes pulsers with adjustable frequency range and analog reception filters. Requirements for ultrasonic transducers suited for concrete testing are even harder to achieve, requiring a broad frequency response, clean signals without strong reverberations from within the transducer, and easy surface coupling. In particular, coupling to the concrete surface is decisive for easy handling. This determines how fast measurements can be carried out, an important point for imaging purposes that require many single measurements. Transducers that do not need a coupling agent are advantageous.

Ultrasonic instruments and transducers that comply with these demands are manufactured by companies such as General Instruments (Krautkrämer), Olympus (Panametrics), Deutsch, Acoustic Control Systems, and Hillger. Two systems offering complete measurement solutions for ultrasonic concrete testing are described in the following paragraphs.

Handheld instrument

A handheld instrument specifically designed for pulse–echo and transmission measurements in concrete is shown in Fig. 22.9. The results of A-scan and B-scan measurements can be evaluated directly on the display. The included software allows also B-scan and C-scan representations of planar scans on a separate personal computer. The instrument, an A1220 made by Acoustic Control Systems (ACSYS), Moscow, can be combined with a range of pressure wave and shear wave transducers, but is most often used with the ultrasonic pitch–catch transducer M2502 (Fig. 22.9). This transducer assembly combines 24 single, individually spring-loaded transducers, all 12 of them connected in parallel in a 4 × 3 configuration as sending and receiving areas, respectively. The transducer transmits and receives shear waves with a centre frequency of 55 kHz corresponding to about 45 mm wavelength in concrete. The so-called dry point contact (DPC) transducers
end in small ceramic tips and are coupled directly onto the concrete without a coupling agent. Number and frequency range of the transducer make it suited for larger penetration depths. Instrument and transducer are highly portable, and are best used for fast and simple single point and line scan measurements.

Line scanner

The only portable line scanner currently available contains an ultrasonic instrument and a transducer array in a single casing, and can be operated by one person. It was developed by ACSYS, Moscow, in cooperation with BAM (Kozlov et al., 2006), and is marketed under the name A1040 (MIRA). The linear array consists of 10 groups of four parallel transducers each. Because the instrument provides electronic switching of the transducer groups, all sender–receiver combinations can be measured quickly. The measurements are transferred to a notebook computer and are processed by a special SAFT algorithm that yields a cross-sectional image through the concrete under the linear array. The special SAFT algorithm makes use of all sender–receiver combinations and is called combinational SAFT or full matrix reconstruction (Samokrutov et al., 2006, Holmes et al., 2008). Measurement and reconstruction times total to just 3 s. By manual or automated acquisition of a number of parallel line scans, it is possible to build a three-dimensional volume that be viewed in two-dimensional cross-sections and depth sections (Krause et al., 2009a).
Automated scanning measurement systems

To date, automated scanning measurement systems need to be custom-built. The systems described here were developed at the reporting institutes, and were used for the application examples in 22.5.3.

BAM measurement equipment

The ultrasonic pulse–echo measurements of BAM were carried out using a PC-based measurement system with laboratory instruments, ultrasonic transducers, and an automated scanner for transducer movement. The transducer type used is a 24-element dry point contact transducer with 55 kHz centre frequency as described above. The exciting pulse is a rectangular signal with $\pm 300$ V amplitude and $50 \mu s$ pulse duration.

For transducer movement, various automated scanner systems are used. Each system consists of two linear axes for positioning and a pneumatic assembly to press transducers to the surface. The systems are operated from a combined movement and measurement control, and the step size is usually 2 cm. The light scanner version shown in Fig. 22.10a is equipped with suction bases for easy fixation even in suspended operation. Two ultrasonic transducers work simultaneously in order to decrease the measurement time. One axis of a different scanner consists of a rail system for ‘endless’ linear movement and measurement times around the clock (Fig. 22.10b). A watchdog system raises an alarm in case of failure via mobile phone (SMS) communication. The main technical specifications for the scanning systems are derived from their outdoor use and the requirement for accurate and fast positioning of the measurement devices.

MFPA measurement equipment

The automated measurement system of MFPA Weimar comprises a commercial ultrasonic instrument and proprietary units such as an automated mechanical scanner and an integrated software environment (Fig. 22.11). For one-sided access, two-dimensional and three-dimensional images can be generated in manual or automated scanning mode (Schickert and Tümmler, 2009). Whereas the primary use of the system is precision research work in the laboratory, it can also be used for tough in-the-field applications. Two persons can transport the system and make it operational within 30 min.

The ultrasonic instrument made by Ing.-Büro Dr. Hillger, Braunschweig, Germany, is contained in an aluminium-cased field computer. It covers a frequency range from 0 to 10 MHz and digitizes signals with 12 Bits at 50 MS s$^{-1}$. Features like a variable-width rectangular burst sender and a set
22.10 (a) and (b) Two types of BAM scanners for ultrasonic testing of a post-tensioned concrete bridge.
of reception filters make it specifically suited for low frequency applications. The automated scanner is driven by stepping motors to offer efficient line and area scanning. Water-coupled transducers that can glide on the concrete surface and also air-coupled transducers are directly moved in the $x$ and $y$ scanning directions. Direct (dry) coupled transducers are additionally moved perpendicularly to and from the concrete surface using the $z$ axis. The scan area is $1.00 \, \text{m} \times 0.80 \, \text{m}$; for larger scans the system is shifted. The scanner can be operated horizontally, upright, or suspended.

### 22.5.3 Application examples

The measurements described in this section were conducted with the automated equipment specified in Section 22.5.2 except where otherwise stated.

#### Thickness measurement

The first application example shows that ultrasonic imaging based on a large number of measurements can display the back wall of thick foundations even for heavily reinforced concrete. For systematic studies, a foundation test slab was constructed at BAM that contains two sections $0.75 \, \text{m}$ and $1.25 \, \text{m}$ thick, a strip foundation, and pile heads with $30 \, \text{cm}$ diameter. Two areas of reinforcement were realized from $12 \, \text{mm}$ reinforcement bars in a $15 \, \text{cm}$ mesh, and from $28 \, \text{mm}$ reinforcement in a $10 \, \text{cm}$ mesh, respectively. Measurements were made with an automated scanner at a step size of $2 \, \text{cm}$. Ultrasonic shear waves were excited at a centre frequency of $30 \, \text{kHz}$. 
Fig. 22.12 depicts a cross-section through the foundation slab as a result of a three-dimensional SAFT reconstruction (Taffe et al., 2006). Indications in the image correctly reveal the back wall at 0.75 m and the bottom of the strip foundation at 1.25 m. The detected width of the strip foundation of 0.5 m and its location also agree well with the construction. The elevated background noise is caused by the maximum aggregate size of 32 mm and the large thickness.

Reinforcement imaging

Imaging of reinforcement bars is generally best done using magnetic or electromagnetic methods. The following example shows that detailed ultrasonic pulse–echo imaging is also possible in regions close to the surface where reinforcement layers are located. If water coupling is used as in this case the surface of the concrete needs to be sufficiently smooth.

In this experiment, a pressure wave transducer with water coupling was used to scan the MFPA hall floor for localization of the upper reinforcement layer consisting of a mesh of 12 mm reinforcement bars (Fig. 22.17). The transducer has a relatively high centre frequency at about 200 kHz in concrete which qualifies it for high-resolution images in near-surface regions of the concrete. Fig. 22.13 shows bipolar depth sections of the three-dimensional SAFT reconstruction at two different depths (Schickert and Tümmeler, 2009).

In Fig. 22.13a, reinforcement bars of the upper layer extending in the $x$ direction are imaged at a depth of $z = 6.8$ cm. Their $y$-directed counterparts are shown in Fig. 22.13b at a depth of $z = 8.0$ cm; also shown are the second half wave of the $x$-directed reinforcement bars and also the starting indication of a pipe target (see also Fig. 22.18). Indications of reconstructed
reinforcement bars that have higher acoustical impedance than concrete start with a positive (light) half wave, whereas the indication of the empty part of the pipe that has lower impedance than concrete starts with a negative (dark) half wave.

Localization of tendon ducts

At a pre-stressed concrete bridge, vertical tendons in the box girder had to be localized as it was not clear which of two versions of the construction plan was realized. Ultrasonic measurements were conducted by BAM (Krause et al., 2008a) as part of a comprehensive NDT effort also including magnetic reinforcement bar localization and radar examinations.
The measurements were conducted on a planar grid using an automated scanning system. After completion, a three-dimensional SAFT reconstruction of the measurement data was calculated and evaluated in two-dimensional sections through the volume data. Figure 22.14a shows a depth section. It is calculated as the mean of depths from 16.3 cm to 35 cm, being the depth region of the first tendon duct layer. The image shows a single vertical duct and some indications of inclined reinforcement bars that partly shade the duct. A second depth cut in the range from 44 to 70 cm (not shown) imaged two vertical ducts in a second layer. Figure 22.14b shows a cross-section with all three ducts as point reflectors (mean of $y = 74$ cm to $y = 103$ cm) and their assumed positions from the two construction plans. From this image, the position of the tendon ducts in lateral and depth coordinates could be determined. It became clear that version two of the plans was realized.

Detection and localization of voids in tendon ducts

Often it is important to know not only position and shape of an object within the concrete, but also to detect whether it is bonded to the concrete.
or contains air. Examples are voids in tendon ducts which can cause corrosion of pre-stressing wires. Such detection is generally possible using elastic waves. Examining the reflection factors $r$ in Table 22.1 reveals that for a reflection at steel, $r$ is positive and the reflected wave starts with a positive half wave, and for reflection at air, $r$ is negative and the reflected wave starts with a negative half wave.

Based on these grounds a method has been developed that makes the phase of the reflected wave visible (Mayer et al., 2008, Millmann et al., 2006). Based on a SAFT reconstruction, the phase of an object indication is assigned a colour to it that corresponds to the phase of the corresponding reflected wave. This colour information is additionally modulated by the reflection intensity and overlaid to the SAFT image thus indicating ungrouted regions in tendon ducts.

The next example describes void detection results obtained at tendon D3 in the Large Concrete Slab sample at BAM. The tendon duct has a diameter of 35 mm and contains a single steel wire of 25 mm diameter. Parts of the duct were grouted using high-pressure field technology; other parts around $x = 1.5$ m and $x = 3$ m were left ungrouted (Fig. 22.15a). The concrete cover is 14 cm.

The measurement signals were acquired by an automated scanner on a planar grid. From the three-dimensional FT-SAFT reconstruction a cross-section along the duct displaying the reconstructed magnitude is shown in Fig. 22.15b. Although the tendon duct and the back wall are clearly visible, different grouting conditions within the duct cause only small intensity

22.15 Examination of a partly grouted tendon duct in a test specimen: (a) plan of grouted (hatched) and ungrouted (white) sections, (b) FT-SAFT cross-section showing reconstruction magnitude, and (c) colour-coded phase information overlaid (images originally coloured).
changes, which can not be securely assigned to grouted or ungrouted sections (Krause et al., 2006), because of the small difference between wire and tendon diameters. In contrast, the phase-sensitive evaluation displayed in Fig. 22.15c shows clear indications of the ungrouted sections around $x = 1.5$ m and $x = 3$ m (a coloured image representation would yield easier differentiation) (Krause et al., 2008c). The phase difference between indications of grouted and ungrouted sections is almost $180^\circ$ corresponding to the difference between acoustically hard steel wire and acoustically soft air voids.

In an effort to assess and to improve NDT methods, automated radar and ultrasonic measurements were performed by BAM at post-tensioned bridges. Investigations were carried out at the inner and outer sides of box girders and at the bottom and top sides of decks with transverse prestressing (Niederleithinger et al., 2007, Streicher et al., 2006, Wiggenhauser et al., 2008). The following example was selected as a typical measurement result.

As described before, air inclusions in tendon ducts cause large indication amplitudes in reconstructed ultrasonic images. Figure 22.16 depicts part of the result of a 15 m$^2$ area measurement at a box girder. In the resulting depth section of a three-dimensional SAFT reconstruction, three tendons are visible. A comparison with the construction plan reveals that the strong indications around $x = 3$ m correspond to the coupling areas of each two tendons.

**Three-dimensional imaging of concrete regions**

In the concrete floor of a new experimental hall for MFPA, a test site was integrated before concreting. The test site contains 18 point and line targets in an area of overall dimensions of 11.7 m $\times$ 0.9 m, embedded into the reinforced concrete floor with a compressive strength of C30/37 and a maximum aggregate size of 16 mm. On both surfaces the concrete is rein-
forced with a layer of reinforcement bars, each layer consisting of crossed bars with a diameter of 12 mm and a mesh size of 11 cm. The site is divided into six fields each containing three elongated targets or six circular targets in three different depths (Schickert and Tümmler, 2009).

In this experiment, a direct coupled shear wave transducer was used to scan the R1 field containing three pipes of 87 mm diameter, half of the length being filled with mortar, the other half left empty (Fig. 22.17). The transducer has a centre frequency of 55 kHz and contains 12 sending and 12 receiving transducers each wired in parallel (Fig. 22.9).

The scanned measurements were processed by three-dimensional SAFT reconstruction. A three-dimensional overview of the test field was created by calculating contours of the reconstruction data at a certain threshold (Fig. 22.18). The colour table (the image is originally coloured) varies with depth for easier object identification.

The resulting iso-surface image shows the empty parts of the pipes (Schickert and Hillger, 2009). All three targets can be identified and located. Uneven indications are the result of shadowing by reinforcement and concrete inhomogeneities. The filled pipe parts are reconstructed with low contrast and can not be seen in this image. Additionally, some parts of the lower reinforcement layer appear in the image. Image noise would be visible as ‘clots’. Iso-surface images like this give a good overview of the reconstructed region but are not as suited for detailed evaluation because finer image details are not visible.

22.5.4 Simulation

The interpretation of received signals or imaged indications can be difficult if objects contain different material boundaries, if different wave types
Simulation can help in such cases to understand the details of wave propagation and mode conversion of elastic waves. For example, elastic wave scattering at tendon ducts can lead to complicated indication patterns. Depending on their condition and construction, tendon ducts can have fillings of various materials. In the following simulation, a tendon duct of 40 mm diameter and 1 mm steel casing was modelled in three configurations (Fig. 22.19a): completely filled with air, completely grouted with mortar, and containing a 16 mm steel wire in the centre as well as grouting mortar (Krause et al., 2008c). For better representation of the results without structural noise, the concrete was modelled as a homogeneous material with a density of 2430 kg m$^{-3}$ and a shear wave velocity of 2570 m s$^{-1}$.

22.18 (a), (b) and (c) Various iso-surface views of the three-dimensional SAFT reconstruction of the three pipe targets in Fig. 22.17.
In a first step, wave scattering at these ducts was modelled using two-dimensional EFIT (see Chapter 8). The incident wave was a 110 kHz shear wave, polarized perpendicularly to the ducts. This is equivalent to simulating a tendon duct of 80 mm diameter imaged with 55 kHz shear waves. The waves were sent and received at a linear grid on top of the simulated test specimen. In the second step, the simulated signals were reconstructed by two-dimensional SAFT and imaged as a cross-section (Fig. 22.19b). A logarithmic scale was used for better visibility of all indications. As expected, the results show indications at the top of the ducts in all three cases. But even in the simple cases (Fig. 22.19b, left and centre duct), additional major indications appear stemming from waves propagating around the circumference (left duct; Krause et al., 2009b) and from the bottom reflection (centre duct). In the right duct, reflections from the wire and multiple reflections within the duct superimpose. The results clarify that ultrasonic SAFT images of tendon ducts contain useful information which needs to be interpreted carefully.

**Fig. 22.19** Elastic wave scattering at 40 mm tendon ducts with different fillings: (a) simulated arrangement and (b) resulting SAFT reconstruction based on EFIT simulation of wave scattering.
22.6 Future trends

Despite the relatively long history of the use of ultrasonic techniques in concrete testing, much progress has been achieved during the last few years, and more is to be expected until ultrasonic instruments are regularly seen on construction sites. As the general testing methods and algorithms are already developed to a usable state, continuing efforts are needed mainly in the following areas:

(a) Conducting measurements needs to be faster to be more effective and economical.
(b) Robust professional equipment needs to be developed for easy on-site usage.
(c) New and enhanced techniques are needed to initiate novel applications.

Some examples are included to show research on future trends such as using electronic scanning and air-coupled transducers to make measurements faster (see also chapter 3), and using ultrasonic tomography to test columns for voids. It is not too far-fetched to expect that in the future robots will perform automated measurements with a combination of various NDT techniques for increased significance and confidence.

22.6.1 Electronic scanning

Ultrasonic imaging of the volume of concrete elements requires a large number of single measurements on a planar grid. Whereas automated mechanical scanners are a necessity to master this task, the scanning process currently takes 2–6 h m⁻² and is thus too slow to be economical for many purposes.

One possible solution is to employ electronic scanning. Instead of dedicated sending and receiving transducers, a larger number of transducers is used that can be electronically switched between the sending, receiving, and off states, respectively. Thus, electronic scanning between a larger number of ultrasonic transducers replaces the equivalent number of mechanical scanning positions.

Although the substitution of the whole planar scanning grid by transducers is technically and economically not feasible currently, the use of linear arrays is possible. Figure 22.20 shows a linear array consisting of 48 transducers. The array is divided into 16 groups, each containing three transducers connected in parallel (columns in the figure). The transducer groups are electronically switched by a multiplexer during the measurement. Set-up files are used to define the switching order, also allowing transducer groups
to be combined to apertures of different size for optimized range-resolution ratios.

The ultrasonic array and the multiplexer are used with the scanning and measurement system shown in Fig. 22.11. Together they form an integrated scanning multi-channel measurement and imaging system for the application at concrete elements. With the aid of the electronically switched linear array, the acquisition time is reduced by a factor of 10 to about 12 min m$^{-2}$ (for roughly 2600 measurements). During measurements, a two-dimensional SAFT image of each electronic array scan is displayed, and the resulting three-dimensional SAFT image is shown directly after completion of the acquisition process (Schickert and Hillger, 2010).

Because the system is new, its merits and drawbacks are still under investigation. Figure 22.21 shows an example measurement at a sample containing a back wall displacement from 20 to 15 cm and a hard foam ball of
50 mm diameter with a cover of 11 cm. The original image is coloured and can be rotated for better position determination.

22.6.2 Air-coupled imaging

In order to accelerate ultrasonic scanning measurements, air-coupled ultrasonic transducers can be used. The absence of a mechanical contact to the concrete surface allows for faster scanning, thus reducing the measuring time considerably. Whereas air-coupled transducers already exist, two problems remain to be solved for one-sided experiments: firstly, the enormous acoustic impedance mismatch between concrete and air poses a barrier to waves passing this boundary in either direction. On its path into the concrete and then back into the air, most of the emitted energy will be reflected and only a very small fraction of it remains to be measured. Secondly, the same impedance mismatch leads to a high sensitivity of the incidence angle on the propagation angle in the concrete, requiring high accuracy in setting up the incidence angle.

The following example was conducted at a 20 cm thick concrete sample with a slightly inclined side-drilled hole of 58 mm diameter (Krause et al., 2008b). The sample was investigated using a pair of air-coupled transducers with a centre frequency of 85 kHz in the pitch–catch technique. The transducers were mounted for a V-shaped propagation path and acoustically shielded against each other using plates. The concrete surface was scanned on a planar grid, and the measurements were processed by three-dimensional SAFT reconstruction (Gräfe, 2009). Figure 22.22 shows two depth sections obtained from the volume data. In Fig. 22.22a, a direct indication of the side-drilled hole is visible (mean of depths from $z = 9.1$ to 9.9 cm), whereas Fig. 22.22b shows an indirect indication of the hole through back wall shadowing (mean of depths from $z = 18.8$ to 20.1 cm). This gives the perspective to develop an air-coupled ultrasonic method as a fast tool of tendon duct testing.

22.6.3 Tomographic imaging

The imaging methods presented so far use one-sided pulse–echo measurements. This approach faces problems if heavy reinforcement is encountered, leading to strong reflections that can mask objects and shadows regions behind. This is the case in the detection of honeycombing (unconsolidated concrete) in columns.

To overcome this problem, ultrasonic measurements can be conducted in transmission, where the pulse velocity is measured at different paths through the cross-section of the columns. If many such measurements are processed by a computed tomography algorithm, an image of the cross-
section of the column can be calculated that also yields an improved resolution.

The feasibility of this method is demonstrated in Fig. 22.23. The sample simulates a column with 38 cm outer diameter containing a cylindrical cavity with a circular cross-section and 7.6 cm diameter. A total of 440 measurements were carried out using a pair of pressure wave transducers at 250 kHz (Schickert, 2005). From the pulse velocities, a tomographic reconstruction was computed employing the Filtered Backprojection algorithm. A comparison with the outline of the sample shows that the cavity is correctly imaged. Additional research is necessary to include reinforcement and to investigate the resolution of this method.
22.7 Sources of further information and advice

As the field of ultrasonic testing of concrete is still developing, most information can be found in journal and proceedings articles. A comprehensive reference book is not known to the authors. Many experiences in the measurement of ultrasonic pulse velocity and its correlation to concrete material parameters are summarized in Galan (1990). The more recent book by Malhotra and Carino (2004) contains an overview of different NDT methods for concrete, also including ultrasonic testing. Ultrasonic imaging is not covered by these books.

National and international bodies such as RILEM (international), ASNT (USA), and DGZfP (Germany) maintain special committees for NDT of concrete, but few are specialized on ultrasonic testing. The German DGZfP Fachausschuss Zerstörungsfreie Prüfung im Bauwesen, Unterausschuss Ultraschalprüfung, issues the guideline B4 on ultrasonic testing of concrete including imaging. The current version (DGZfP, 1999) is expected to be revised in the near future. The RILEM committee TC 207 is working on recommendations for the combination of NDT methods for civil engineering. A publication is intended for 2010.

Current developments in ultrasonic testing of concrete are presented at international conferences, the most recognized being *Nondestructive testing in civil engineering (NDT-CE)* which is held every three years mostly in Europe. Other well known conferences are *Structural materials technology (SMT)* in the USA and *Structural faults and repair* in Scotland.

A comprehensive survey of NDT methods for concrete including ultrasonics contains the database *Zfp Bau-Kompendium* which can be accessed through the world wide web (ZfpBau-Kompendium, 2007). An English version is currently under construction.

### 22.8 References


Ultrasonic techniques


EN 13791 (2007), Assessment of in-situ compressive strength in structures and precast concrete components.


MALHOTRA, V M (ed.) (1984), In situ/nondestructive testing of concrete, Detroit, MI, American Concrete Institute, 1984.


RILEM (1972), RILEM NDT 1, Testing of concrete by the ultrasonic pulse method, Cachan, France, Réunion Internationale des Laboratoires d’Essais et de recherche sur les Matériaux et les Constructions (RILEM).


Schickert M and Hillger W (2010), ‘Automated ultrasonic scanning and imaging system for application at civil structures’, Proceedings of the 10th European conference on non-destructive testing, Moscow, 7–11 June 2010, accepted for publication.


Inspection of concrete retaining walls using ground penetrating radar (GPR): a case study

J. HUGENSCHMIDT, EMPA, Switzerland

Abstract: Radar data were acquired on a large retaining wall using three different antennae. An apparatus developed for this purpose enabled high precision antenna positioning and efficient data acquisition. Two layers of rebar and dowels were mapped with high quality. The localization of anchor heads posed a problem as it was difficult to decide whether reflections were caused by anchor heads or other structures.

Key words: ground penetrating radar (GPR), concrete, retaining wall, rebar, anchor heads, dowels.

23.1 Problem description

The Swiss motorway A9 was built in the early 1970s. It runs along the northern shore of Lake Geneva where there is a steep slope from the mountains towards the lake. As a result there are many retaining walls particularly on the uphill side of the motorway. After more than 30 years in service, many of those walls are in need of repair and/or inspection.

In order to evaluate the benefits of ground penetrating radar (GPR) as an inspection tool for those walls, a pilot study was carried out on one of the walls (Hugenschmidt and Kalogeropoulos, 2009). The choice of GPR seems to be rather obvious because it is a high-resolution method and many successful applications on reinforced concrete have been reported. The main objectives of the pilot study were to locate anchor heads and to gain information on the general structure of the wall.

The main challenges during this study were the development of an approach for data acquisition on the face of the walls as well as the trade-off between the quality of the final results and the effort required to carry out the study.

The inspected wall consists of four levels (Fig. 23.1) and is situated directly adjacent to the emergency lane of the motorway. The three bottom levels are each 6 m high, the top level has a height of 4 m. At the top of each level there is a narrow landing covered with grass and trees.
23.2 Data acquisition

One of the aims of this study was to test the capability to locate the heads of rock anchors, expected to be present in the wall, in order to enable subsequent destructive access to the anchor heads for visual inspection. Therefore, a high accuracy of the antenna position was required. In order to achieve this, an apparatus was developed. It consists of a rail system sitting on the copings of the different levels of the wall, an antenna box (ab) containing one or several antennae, a ladder-like guiding system for the ab, an electric motor for moving the ab up and down the face of the wall, a survey wheel for controlling the vertical position of the ab and an electronic protractor for monitoring the angle between the guiding system and the vertical line thus controlling the lateral position of the ab. In Fig. 23.2 the top of the apparatus is shown and in Fig. 23.3 the antenna box, marked with a white arrow, can be seen in the guiding system on the face of the wall.
23.2 Top of apparatus.

23.3 Guiding system with antenna box.
The coordinate system used is presented in Fig. 23.4. The vertical axis is the \( x \) axis with its origin on the coping of each wall level. All data presented will be named as ‘\( A \) slices’ with \( A \) being the axis perpendicular to the slice. For example, a \( Y \) slice is a slice perpendicular to the \( y \) axis with fixed \( y \) covering ranges in \( x \) and \( t \) (time).

Data were acquired along vertical lines in the \( x \) direction on the face of the retaining wall. The spacing between single lines (distance in \( y \) direction) was varied between 0.04 and 0.1 m. This is a compromise between the need for dense data and the time required for data acquisition. The sampling rate in the \( x \) direction was 0.005 m. No data processing was performed during acquisition. The equipment used was a GSSI-SIR20 radar unit and three different antennae with nominal frequencies of 1500, 900 and 400 MHz, respectively. After the equipment had been set up, up to 30 vertical lines were acquired per hour. Depending on the line spacing this corresponds to a horizontal distance between 1.2 and 3.0 m.

### 23.3 Data processing

Data were processed using a two-dimensional (2D) processing sequence consisting of bandpass filtering, static correction of direct wave, Kirchhoff migration and gain correction. This was followed by resampling to reduce the amount of data and a time cut to ensure that all traces have the same length. Two alternative processing sequences were applied to all datasets,
one with and one without background removal. During background removal a mean trace is computed followed by the subtraction of this mean trace from each single trace of a dataset thus removing signals that are constant over the whole length of the dataset. In Fig. 23.5 and 23.6, the same dataset (y slice) acquired with the 1500 MHz antenna before and after 2D processing is presented.

Finally, the processed data were merged into three-dimensional (3D) files as presented in Fig. 23.7, where a subset of the radargrams (y slices) is presented in their proper position in the 3D data cube.

23.5 Raw dataset, 1500 MHz antenna (Y slice).

23.6 Processed dataset, 1500 MHz antenna (Y slice).
23.4 Results

As this was a pilot study the client released no information on the structure of the wall. Therefore all interpretation and results are based on radar data alone.

In general, it can be assumed that high-frequency antennae provide better resolution but less depth of penetration than low frequency antennae. This assumption can be easily confirmed for the resolution by a comparison of the datasets obtained with the three antennae on level 2 of the retaining wall. In Fig. 23.8, the time range between 0 and 4 ns of a Y slice is shown for the 1500 MHz antenna. Three single bars can be clearly distinguished. In Fig. 23.9 the corresponding section acquired with the 900 MHz antenna is shown and Fig. 23.10 presents the 400 MHz data. Therefore, the assumption of higher frequency corresponding to higher resolution is valid in this instance. A similar comparison for later times showed that the 400 MHz antenna did not provide information from greater depths than the 900 MHz antenna. This is probably because of the abundance of rebar in the wall. It was therefore decided to restrict the interpretation to the 1500 and 900 MHz data.

In Fig. 23.11, a T (time) slice (t = 1.6 ns) from level 2 acquired with the 1500 MHz antenna is presented. Assuming a signal velocity of 0.09 m ns\(^{-1}\), this corresponds to a depth of 0.07 m. The top layer of rebar is mapped almost completely. In the lower left, at \(x = 4\) m, one bar has not been placed properly. Vertical bars (bars parallel to the x axis) cause weaker reflections when compared with horizontal bars because of the antenna orientation.
during data acquisition and because of the application of a background removal during data processing. At $t = 5.16$ ns (Fig. 23.12), corresponding to 0.23 m, a second layer of rebar is visible, but it is obvious that signal quality as well as resolution have deteriorated considerably.

As no additional information such as expected depth or approximate vertical position was available, it was very difficult to localize the position of anchor heads within the wall. This was mainly because of the presence of numerous reflections that made it impossible to decide, based on the radar data alone, whether they were caused by anchor heads or other structures. In Fig. 23.13 a $Y$ slice from level 1 that was recorded with the 1500 MHz antenna is presented. A prominent reflection is marked with a
23.10 Y slice, 400 MHz antenna, 0–4 ns.

23.11 Time slice, \( t = 1.6 \) ns, level 2, 1500 MHz antenna.

23.12 Time slice, \( t = 5.16 \) ns, level 2, 1500 MHz antenna.
Two sections, each 1.0 m long, of Y slices from level 3 (900 MHz) are presented in Fig. 23.14 and 23.15. The data from Fig. 23.14 were acquired on a vertical joint separating two different sections of the wall and the Fig. 23.15 data were recorded at a distance of 3.9 m from the joint. Away from the joint two layers of rebar are visible whereas on the joint there is only one layer of rebar but an additional reflection (arrow) that was interpreted as a dowel connecting the two sections of the wall.
23.5 Conclusions

This pilot study showed that data acquisition on large retaining walls can be carried out economically and with high precision. Because of the abundance of rebar in the wall, the 400 MHz data did not offer any benefit when compared with the 1500 and 900 MHz data. The survey provided information on structural elements of the wall such as rebar and dowels. One important aim of the survey, the locating of the positions of anchor heads, was not achieved completely. This was because it was not possible to distinguish between reflections caused by anchor heads and reflections caused by other structures based on the radar data alone. Additional information such as approximate positions or depths, would probably have rendered possible the distinction between anchor heads and other structures.

23.6 Reference

Acoustic emission and impact–echo techniques for evaluation of reinforced concrete structures: a case study

M. OHTSU, Kumamoto University, Japan

Abstract: Four applications of acoustic emission (AE) and impact–echo (IE) techniques are reported. AE parameter analysis is applied to crack detection for corrosion-induced cracks. In order to clarify fracture mechanisms of diagonal-shear failure in reinforced concrete beams, kinematics on crack location, crack type and orientation are quantitatively identified by AE moment tensor analysis. An imaging procedure for the impact–echo technique is developed. The procedure is successfully used to detect ungrouted post-tensioning ducts in prestressed concrete and to estimate surface-crack depths.

Key words: acoustic emission, corrosion, reinforced concrete, impact–echo, surface crack, tendon duct.

24.1 Introduction

Case studies on acoustic emission (AE) and impact–echo (IE) techniques applied to concrete structures are reported. By means of AE techniques, it has been reported that concrete cracking owing to reinforcement corrosion is effectively detected (Idrissi and Liman, 2003; Ohtsu and Tomoda, 2008; Yoon et al., 2000). Here, continuous AE measurement was conducted to clarify the corrosion process in reinforced concrete samples in a laboratory.

To identify fracture mechanisms in concrete, the moment tensor analysis of AE is a powerful procedure. In order to perform the analysis easily, SiGMA (simplified green’s functions for moment tensor analysis) procedure has been developed (Ohtsu, 1991). The tensor components are determined from two waveform parameters of the arrival time and the amplitude of the first motion of AE waveforms. An auto-picker based on Akaike information criterion (AIC) is developed to automatically measure these two parameters of detected AE waveforms. The SiGMA analysis with AIC picker was applied to four-point bending tests of reinforced concrete (RC) beams.
The IE method has been successfully applied to locate defects, flaws, inclusions and voids in concrete structures (Sansalone and Streett, 1998). By identifying the resonant frequency in a frequency spectrum, the location of a defect is estimated from a relationship between the wavelength of resonance and the travel distance via the defect. In actual cases, however, many peak frequencies are observed in the spectrum and the peak frequency responsible only for the location of the defect is not easily identified. In order to compensate for this drawback, stack imaging of spectral amplitudes based on impact–echo (SIBIE) has been developed, as an imaging procedure applied to the IE data in the frequency domain (Ohtsu and Watanabe, 2002). In pre-stressed concrete, the cases of ungrouted and imperfectly grouted ducts were investigated to demonstrate the applicability. In addition, depths of surface cracks were successfully evaluated for real cracks generated by bending tests of reinforced concrete beams, applying a scanning SIBIE procedure.

24.2 Applications of acoustic emission (AE) and impact–echo (IE) for concrete structures

24.2.1 AE parameter analysis

Characteristics of AE signals were estimated by using two indices of RA value (rise time divided by amplitude) and average frequency. These are defined from such waveform parameters as rise time, maximum amplitude, counts and duration shown in Fig. 24.1. According to the Japan Construc-
Acoustic emission and impact–echo techniques

Acoustic emission (AE) and impact–echo techniques (JCMS, 2003) are used to detect and classify active cracks in materials. AE sources of active cracks are classified, based on the relationship between the RA values and the average frequencies. Here, two indices are defined as:

\[
RA = \text{rise time/maximum amplitude} \quad [24.1]
\]

\[
\text{Average frequency} = \text{counts/duration} \quad [24.2]
\]

Although the relationship between two indices was originally proposed to classify cracks into tensile cracks and shear cracks, it is modified owing to recent findings as illustrated in Fig. 24.2. When the RA value is large and the average frequency is low, the AE source is classified as an other-type crack rather than a tensile crack. Otherwise, AE source is referred to as a tensile crack. This criterion is applied to classify AE events detected in the corrosion process.

To evaluate the size distribution of AE sources, the amplitude distribution of AE events was analyzed. A relationship between the number of AE events \(N\) and the amplitudes \(A\) is statistically represented as

\[
\log_{10} N = \alpha - b \log_{10} A \quad [24.3]
\]

where \(\alpha\) and \(b\) are empirical constants. The latter is called the \(b\) value, and is often applied to estimate the size distribution of AE sources (Shiotani et al., 2001). For large \(b\) values, small AE events are mostly generated. In contrast, when the \(b\) values are small, nucleation of large AE events takes place.

24.2 Classification of cracks by AE indices.

© Woodhead Publishing Limited, 2010
24.2.2 SiGMA analysis with AIC picker

The SiGMA analysis consists of a three-dimensional (3D) AE source location procedure and moment tensor analysis of AE source. Two parameters of the arrival time ($P_1$) and the amplitude of the first motion ($P_2$) shown in Fig. 24.3 are picked up and applied to the analysis. In the location procedure, the AE source is located from the arrival time differences $t_i$ between the observation point $x_i$ and $x_{i+1}$, by solving equations:

$$R_i - R_{i+1} = |x_i - x'| - |x_{i+1} - x'| = v_p t_i$$

where, $v_p$ is the velocity of the $P$ wave.

After determining the AE source location, the amplitudes of the first motion ($P_2$) are substituted into the following equation:

$$A(x) = C_s \frac{R \cdot \text{Ref}(t, \gamma)}{R} \gamma_p \gamma_q M_{pq} \Delta V$$

where, $A(x)$ is the amplitude of the first motion and $C_s$ is the calibration coefficient of the sensor sensitivity and material constants. The reflection coefficient $\text{Ref}(t, \gamma)$ is obtained as $t$ is the direction of sensor sensitivity. $\Delta V$ is the crack volume, $M_{pq}$ is the moment tensor and $\gamma$ is the direction vector of distance $R$ from the source to the observation point $x$. Since the moment tensor $M_{pq}$ is symmetric and of the second rank, the number of independent unknowns $M_{pq}$ is six. To determine the moment tensor components, waveforms are to be detected at more than six sensors. The classification of a crack is performed by the eigenvalue analysis of the moment tensor (Ohtsu, 1991). Eventually, microcracks are visualized by employing the Light Wave

![24.3 AE waveform and two parameters: P1 and P2.](image)
3D software (New Tek) as shown in Fig. 24.4. Here, an arrow vector indicates a crack motion vector, and a circular plate corresponds to a crack surface, which is perpendicular to a crack normal vector.

In the conventional SiGMA, determination of the two parameters of $P_1$ and $P_2$ for SiGMA analysis has been carried out manually via a software package named ‘wave-monitor’. In order to process many AE waveforms, easy and quick determination of the first motion is in great demand. Focusing on the intervals before and after the onset of seismic signal, autoregressive (AR) techniques have been developed (Sleeman and van Eck, 1999). Another auto-picker is also developed by combining the AR model with the AIC method (Kurz and Grosse, 2005). Here, a direct AIC method is applied to determination of the arrival time for the SiGMA analysis. As the number of amplitudes of a digitized AE wave is $N$ and values of amplitudes are $X_i$ ($i = 1, 2, \ldots, N$), $AIC_k$ at point $i = k$ is represented as:

$$AIC_k = k \log \{ \text{var}(X[1,k]) \} + (N - k) \log \{ \text{var}(X[k,N]) \}$$

where $\text{var}(X[1,k])$ is the variance between $X_1$ and $X_k$, and $\text{var}(X[k,N])$ is also the variance between $X_k$ and $X_N$. The AIC method defines the onset point as the global minimum.

The AIC function was applied to AE waveforms detected. One example is shown in Fig. 24.5a and a portion of the first motion enlarged is given in Fig. 24.5b. Here, the arrival time was determined at $k = 251$ manually. In Fig. 24.5b, the minimum value of AIC is shown at $k = 252$. In the automated detection of the first motion (auto-picker), this value was adopted. Then, the parameter $P_1$ of arrival time is determined by applying the following equation:

$$P_1 = [T_k[\text{Min}(AIC_k)] - 1] \times \Delta t$$

where $T_k[\text{Min}(AIC_k)]$ represents the sampling-time number when $AIC_k$ becomes the minimum value at $i = k$ and $\Delta t$ is the sampling time which is set to $1 \mu$s. As for the determination of the parameter $P_2$, amplitude $X_i$

24.4 Crack models in SiGMA analysis.
which satisfies the following condition is adopted, increasing the sampling-time number from \(k\) in equation [24.7].

\[
P2 = X_i \quad [24.8]
\]
if

\[
X_i(X_{i+1} - X_i) \leq 0 \quad [24.9]
\]

where the index \(i\) represents the sampling-time number between \(k + 1\) and \(N\).

### 24.2.3 SIBIE analysis

In order to generate a high-energy impact with a wide frequency range, a device for shooting a projectile with a tapered head by air-pressure was developed. An aluminum bullet of 8 mm diameter and 17 mm length in Fig. 24.6a is shot at the impact point by compressed air (0.05 MPa) through a steel pipe connected to a portable compressor shown in Fig. 24.6b. Elastic waves generated are detected at the surface by an accelerometer or the laser vibrometer.
For the analysis, a cross-section of a member measured is modeled and divided into square elements as shown in Fig. 24.7. Then, resonant frequencies owing to reflection at the element are calculated. The travel distance from the input point to the output via the center of the element is obtained as denoted in the Fig. 24.7:

\[ R = r_1 + r_2 \]  \[ 24.10 \]

Resonant frequencies owing to reflection at the element are theoretically calculated as

\[ f_2 = \frac{C_r}{r_2} \]  \[ 24.11 \]

and
where $C_p$ is the velocity of the P wave. In a frequency spectrum of a detected wave, spectral amplitudes corresponding to these two resonant frequencies are picked up and summed up at the element. Thus, a reflection intensity is estimated as a stack image at each element. Assigning stack images at all elements, a contour map on the reflection intensity is obtained at the cross-section. The minimum length of the square element $\Delta x$ is determined from:

$$\Delta x = C_p \Delta t / 2$$  \[24.13\]

where $C_p$ is the velocity of the P wave and $\Delta t$ is the sampling time of a recorded wave.

The current SIBIE method has a problem when dealing with real surface cracks. Because a two-dimensional (2D) image is expressed as symmetry owing to one-point impact and the other-point detection across the crack, sufficiently good information to resolve a crack-tip of a zig-zag configuration is not easily obtained. Thus, a scanning SIBIE method is developed. As shown in Fig. 24.8, a cross-section to be measured is divided into sections of the limited width. Then, the IE method is applied to the each section, and a frequency spectrum is calculated. The SIBIE analysis is conducted on each frequency spectrum, and a 2D image is obtained at each section. Eventually, all 2D images are superimposed crosswise. In Fig. 24.8, five pairs of impact and detection points are shown. When the spectrum was obtained via fast fourier transform (FFT) analysis, the maximum amplitude was normalized and then all spectra were superimposed as one image.
24.3 Case studies

24.3.1 Corrosion detection by AE in reinforced concrete

Outline of experiments

Reinforced concrete samples of dimensions 300 mm × 300 mm × 100 mm were tested. Deformed steel-bars (rebars) of 13 mm nominal diameter were embedded at 15 mm cover-thicknesses for longitudinal reinforcement. Configuration of the sample is illustrated in Fig. 24.9. When preparing samples, concrete was mixed with NaCl solution to investigate the threshold limit of chloride concentration for the corrosion. Here, the lower-bound threshold value (0.3 kg m\(^{-3}\) chloride in concrete; 0.093% mass of cement) prescribed in the codes (JSCE, 2001 and 2002) was taken into account. After standard curing in water for 28 days at 20°C, the chloride content of a standard cylindrical sample of 10 cm diameter and 20 cm height and was 0.125 kg m\(^{-3}\) (0.039% mass of cement), i.e. lower than the 0.3 kg m\(^{3}\) threshold value. A compressive strength at 28 days of the standard curing was 35.0 MPa. Before testing, all surfaces of the specimen were coated with epoxy, except the bottom surface for one-directional diffusion as illustrated in Fig. 24.9.
An accelerated corrosion test and a cyclic wet–dry test were conducted. In the accelerated corrosion test, the specimen was placed on a copper plate in a container filled with 3% NaCl solution. Between rebars and the copper plate, a 100 mA electric current (0.754 mA mm$^{-2}$) was continuously charged. In the cyclic wet–dry test, the samples were cyclically put into the container without charge for a week and subsequently dried under ambient temperature for another week. AE measurement was continuously conducted, by using the AE analyzer (LOCAN 320, PAC). Two broadband-type AE sensors (UT1000, PAC) of 1 MHz resonance were attached to the upper surface of the sample at the center of coring locations in Fig. 24.9. The frequency range of the measurement was 10 kHz to 1 MHz and total amplification was 60 dB gain. For counting AE hits, the dead-time was set to 2 ms and the threshold level was 40 dB gain. Half-cell potentials at the surface of the sample were measured by a portable corrosion meter (SRI-CM-II, Shikoku-Soken). In the accelerated corrosion test, the measurement was conducted twice a day, immediately after discontinuing the current. When the average potentials reached to −350 mV (CSE), which gives more than 90% probability of corrosion (ASTM, 1991), the test was terminated. In the cyclic test, the sample was measured weekly until the average potentials in dry conditions reached to −350 mV (CSE). During
the half-cell potential measurement, AE measurement was discontinued in both tests.

Chloride concentrations were measured over several periods. First, the initial concentration was measured by using a standard cylinder sample after 28-day moisture-curing as an initial value. For other periods, two core portions of 5 cm diameter were taken from the samples at the locations indicated in Fig. 24.9. After slicing the core into 5-mm-thick disks and crushing them, concentrations of total chloride ions were determined by the potentiometric titration method. At two stages during the cyclic test, rebars were taken out of the sample and cut into 10 mm portions. The ferrous ions on their surface layers were examined by the scanning electron microscope (JSM-5600, Nippon-Denshi).

Results and discussion

In the accelerated corrosion test, a relation between AE activity and half-cell potentials measured are shown in Fig. 24.10. The number of AE hits is plotted as a total of two channels counted for one hour. Two periods of high AE activities are clearly observed between 3 days elapsed and 7 days elapsed, the activity in the first period being even higher than in the second. It is noted that the half-cell potentials start to decrease after the first period of high activity. This may suggest the onset of corrosion in rebars, although the potential is still less negative than $-350 \text{ mV}$ (ASTM criterion denoted in the graph) at the second period of high activity. It is known that the half-cell potential lower than $-350 \text{ mV}$ is prescribed as more than 90% probability of the corrosion, and the probability is uncertain between $-200 \text{ mV}$.
and −350 mV by ASTM C876 (1991). This implies that the corrosion could be monitored from AE activities more confidently than the half-cell potentials, if the first period of high AE activity is associated with the corrosion activity.

Chloride concentration was initially measured by using a standard cylindrical sample for the compression test. For accelerated tests of three specimens simultaneously, core samples were taken at three periods, following two periods of high AE activities and at the final stage. Thus, total chloride ions were determined in depths. Based on chloride concentrations at the initial and after the first period of high AE activity, chloride concentration at the cover thickness was analytically estimated, assuming one-dimensional diffusion. The results are plotted in Fig. 24.11a. A broken curve shows the analytical values, which are compared with measured values (open circles in the figure) at the cover thickness in the tests. Agreement between analyzed and measured values after the second period of high activity and at the final stage is reasonable. In Fig. 24.11, two horizontal lines are denoted. One is the lower boundary of threshold value for the onset of corrosion (0.3 kg m\(^{-3}\) in volume, 0.093% mass of cement volume) and the other is the threshold value (1.2 kg m\(^{-3}\) in volume) for performance-based design (JSCE, 2001). In this design code, concrete with a chloride content over 1.2 kg m\(^{-3}\) (0.372% mass of cement) is not allowed in construction to prevent corrosion. In the figure, the total number of AE hits observed during the acceleration test is also given by a solid curve. It is interesting that the curve of the total number of AE hits is in remarkable agreement with the typical corrosion loss in the phenomenological model in Fig. 24.11b by Melchers and Li (2006). This suggests that AE activity observed could phenomenologically correspond to the corrosion activity in rebar. Comparing AE activity with chloride concentration, it is found that chloride concentration becomes higher than 0.3 kg m\(^{-3}\) around the first period of AE activity, and it reaches more than 1.2 kg m\(^{-3}\), around the second period of AE activity. Consequently, it could be summarized that two high AE activities reasonably correspond to two periods of the onset of corrosion and the nucleation of cracking.

In the cyclic wet and dry test, the RA values and the average frequency were determined from equations [24.1] and [24.2], averaging the data in two weeks of wet and dry. Results are shown in Fig. 24.12a. Corresponding to the first period, the RA value becomes large and the average frequency is low. According to Fig. 24.2, this implies that AE sources are classified as other-type cracks. After 100 days, the RA values are low and the average frequencies are fairly high. It is suggested from Fig. 24.2 that tensile cracks were nucleated at this period. In addition, the \(b\) values were determined during each wet–dry cycle as the average values as shown in Fig. 24.12b. It is observed that \(b\) values become high during the first period and then \(b\)
values keep fairly low. Results suggest generation of small other-type cracks in the first period and nucleation of fairly large tensile cracks in the second period. Combining these results, it is reasonable to conclude that the first high AE activity is associated with generation of small other-type cracks and could be related with the onset of corrosion in reinforcement. The second high AE activity results from the nucleation of fairly large tensile cracks, which could be generated by the expansion of rebar cased by corrosion product.

24.11 Results of the accelerated corrosion test and a corrosion loss mode (Melchers and Li 2006): (a) total number of AE hits and chloride concentration, (b) increase of corrosion loss for steel.
In the cyclic wet–dry test, it was found that chloride concentration at rebar reaches to the lower-bound threshold of 0.3 kg m\(^{-3}\) after 40 days. At around 100 days elapsed, the concentration became higher than 1.2 kg m\(^{-3}\). Accordingly, at 42 days elapsed and 126 days elapsed, rebars were removed and visually inspected. As shown in Fig. 24.13, no corrosion was observed on rebar surface after 42 days, whereas rebar was fully corroded after 126 days. These results imply that rebars could corrode after chloride concentration reaches over 1.2 kg m\(^{-3}\) in concrete. AE activities after 100 days

24.12 Results of AE parameter analysis in the cyclic wet–dry test: (a) RA values and average frequencies, (b) variation of b values.
could reasonably result from concrete cracking owing to the expansion of corrosive products in rebars.

In order to investigate the condition of rebar at the first AE activity, the surface of rebar was examined by the scanning electron micrograph (SEM). Distributions of ferrous ions at the initial as-received condition and after 42 days has elapsed are compared in Fig. 24.14. In the as-received condition, a reinforcing steel bar was cut into a fragment around 10 mm long and then the cross-section of the rebar fragment was examined. In Fig. 24.14a, from the left, white, gray and black zones are observed. The white zone is observed inside steel and the gray zone corresponds to the surface layer.
As shown, homogeneous distribution of ferrous ions is observed at the surface as the gray zone. After 42 days had elapsed, rebars removed were cut into 10 mm fragments, and the circumferential surfaces were examined. In Fig. 24.14b, it is shown that at some portions of the surface layer, ferrous ions (gray) vanish and black areas appear as non-ferrous zones. This demonstrates that the onset of corrosion occurs in rebars in the first period of AE activities. Although no corrosion was visually observed, rebars were actually corroded as found in the SEM observation.
24.3.2 Mechanisms of shear failure in reinforced concrete by AE-SiGMA

Outline of experiment

A reinforced concrete beam of dimensions $150 \times 250 \times 2000$ mm is shown in Fig. 24.15a. The compressive strength of concrete after 28 days of standard curing was 29.7 MPa. The velocity of the P wave was $4230 \text{ m s}^{-1}$ and the Poisson’s ratio was 0.2. Two reinforcement bars (13 mm in diameter) were installed at a cover thickness 46.5 mm from the bottom. To focus only on the diagonal-shear failure, eight AE sensors were attached using electron wax on the surface of shear span as illustrated in Fig. 24.15b. AE signals were detected by 150 kHz resonant AE sensors (R15, PAC), and the sampling frequency for recording waveforms was 1 MHz by 8-channel DiSP system (PAC). AE waves were pre-amplified by 40 dB and amplified by 20 dB in main system (DiSP). The threshold level was set to 42 dB. In the four-point bending test, 3D source location was carried out by the DiSP system (PAC) in real time. AE sources are located from the difference times of all eight channels when amplitudes were higher than a threshold level at each channel.

![Diagram of reinforced concrete beam and AE sensor array](image_url)
Results and discussion

During the test, AE generation behaviors observed are shown in Fig. 24.16. Thus, a loading process is divided into three stages. AE events are determined from AE waveforms detected by all eight AE sensors. More than 700 AE events were recorded within the event definition time (EDT) of 120 \( \mu \)s. From 4230 m s\(^{-1}\) of P wave velocity, the value of EDT resulted in the fact that the distances from AE sensors at the area shorter than 507.6 mm could cover the shear span. Table 24.1 shows the number of AE events classified by the SiGMA analysis with hand-picker (manual) and auto-picker (automated). By directly reading the first motions on the CRT monitor (hand-picker), 445 events were analyzed, whereas 370 events were analyzed by the auto-picker. However, the number of AE events classified into crack modes of tensile, mixed, and shear by the auto-picker are in reasonable agreement with those of hand-picker. Time consumed in the auto-picker analysis is ten times shorter than that of hand-picking, even

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Stage 1</td>
<td>4</td>
<td>3</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>6</td>
<td>4</td>
</tr>
<tr>
<td>Stage 2</td>
<td>60</td>
<td>50</td>
<td>70</td>
<td>62</td>
<td>161</td>
<td>147</td>
<td>291</td>
<td>259</td>
</tr>
<tr>
<td>Stage 3</td>
<td>46</td>
<td>35</td>
<td>28</td>
<td>19</td>
<td>74</td>
<td>53</td>
<td>148</td>
<td>107</td>
</tr>
<tr>
<td>Total</td>
<td>110</td>
<td>88</td>
<td>99</td>
<td>82</td>
<td>236</td>
<td>200</td>
<td>445</td>
<td>370</td>
</tr>
</tbody>
</table>

24.16 AE generation behavior in the bending test of reinforced concrete beam.
though dominant motions vary through the stages from the tensile crack to the shear crack. These trends are reasonably analyzed by the auto-picker and are similar to those by hand-picker.

The 370 AE events analyzed by the auto-picker are plotted in the shear span as illustrated in Fig. 24.17. At Stage 2, all types of microcracks are observed along the area of a final diagonal-shear macrocrack. It is noted that no visual cracks were observed at that stage. At Stage 3, most of tensile cracks are in loading and supporting locations, suggesting that the crack extends from the central zone of the shear span to both ends of the final macrocrack. The area of AE clusters is in remarkable agreement with the location of the final failure plane indicated by the line in the graph. Thus, it is demonstrated that AE-SiGMA with the automated first-motion picker could provide a ready means of identifying mechanisms of concrete failure.

24.3.3 Identification of ungrouted tendon-duct by SIBIE

Outline of experiments

A test sample is illustrated in Fig. 24.18a. The compressive strength of concrete after 28 days of standard curing was 32.9 MPa. A concrete block of dimensions 260 mm × 400 mm × 1000 mm contains two ribbed sheaths
of steel (galvanized tin: 60 mm outer diameter and 55 mm inner diameter) and polyethylene (65 mm outer diameter and 55 mm inner diameter). Here, a non-metal sheath is investigated. This is because the peak frequency corresponding to an ungrouted duct is reported to be not easily identified (Sansalone and Streett, 1997), as the acoustic impedance of the polyethylene sheath is similar to that of concrete.

The impact tests were conducted as shown in Fig. 24.18b. A downward arrow shows the impact point, and two upward arrows denote the locations of accelerometers. The distances (intervals) between the accelerometer and the impact point, which is located right over the duct, are varied as 20, 50, and 90 mm.

Immediately before the test, the P wave velocity was determined by employing an ultrasonic tester (MIN-02, Marui). The time of flight was
measured ten times in the axis direction of a cylindrical sample, and the averaged P wave velocity ($C_P$) was obtained as 4025 m s$^{-1}$. A storage-type oscilloscope (TDS2014, Tektronix) was employed to store the waveform data in time domain. A waveform was digitized at 4 μs sampling time ($\Delta t$) of 2048 recording words ($N = 2048$).

The case where the sheath was imperfectly grouted was also tested. After the impact tests were conducted, two ducts in Fig. 24.18a were grouted by grouting cement as illustrated in Fig. 24.19a. Grouting with an inclined angle as shown, one quarter, half and three quarters of the depth were filled with cement at the cross-sections (1), (2) and (3), respectively, in Fig. 24.19b. The impact test was carried out one day after the sheaths were grouted.

![Sketch of imperfectly grouted sheath](image-url)

24.19 Sketch of imperfectly grouted sheath: (a) sheath to be grouted, (b) elevation view of grouted sheath and cross-sections.
Results and discussion

Frequency spectra obtained at two accelerometers in the case of 50 mm intervals in the metal sheath are shown in Fig. 24.20. Theoretical resonant frequencies \( f_T(\frac{C_P}{2T}) \), \( f_{\text{void}}(\frac{C_P}{2d}) \) and \( f'_{\text{void}}(\frac{C_P}{d}) \) are denoted in the figure, where \( T \) is the thickness of the specimen and \( d \) is the depth of the duct. Although corresponding peak frequencies are observed, other peaks are also found. This implies that identification of only the peak frequencies responsible for the location of the sheath is not always easy. By employing these spectra, the SIBIE analysis of two spectra was conducted and two images of reflection intensity are superimposed as one image.

Typical results are given in Fig. 24.21 and 24.22. For 20 mm intervals, reflection zones of high reflection intensity are clearly observed at the top of the sheath, whereas both reflection zones at the top and the bottom are realized in the case of 50 mm intervals. This suggests that the diameter of the duct might be identified when the interval distance is similar to the diameter of the sheath (duct). This is because the reflected waves are generated by deformation at the bottom and they are readily detected at the accelerometers located at the surface with similar interval distances. When the interval is shorter than the diameter of the sheath, in contrast, reflected waves are hardly detected by the accelerometers.

It is also noted that results for polyethylene sheaths are similar to those of metal sheaths, suggesting that the effect of the acoustic impedance is minor in the SIBIE analysis. This implies that the SIBIE procedure is also suitable for identifying ungrouted ducts in polyethylene and plastic sheaths.
24.21 SIBIE results in metal sheath: (a) at 20 mm intervals and (b) at 50 mm intervals.
The SIBIE results of imperfectly grouted ducts are given in Figs 24.23 and 24.24. For imperfectly grouted ducts, high reflection zones are only observed at the top of the sheath for both metal and polyethylene. For fully grouted ducts, no high-reflection zones are observed as shown in
Fig. 24.24b. Thus, the applicability of the SIBIE procedure to identify imperfectly grouted ducts is successfully demonstrated.

24.3.4 Evaluation of surface-crack depth by SIBIE

Outline of experiments

During the bending test of the beam shown in Fig. 24.15a, several surface cracks were observed in the bending span. Cracks were extended in a zigzag
manner with a crack depth of approximately 200 mm. Here, two surface cracks are selected as illustrated in Fig. 24.25. One is a multiple crack, and the other is a zigzag crack. Before the test, the velocity of the P wave was measured as 4065 m s$^{-1}$. After shooting the aluminium bullet against the top surface of the specimen with a compressed pressure of 0.05 MPa, surface displacements resulting from the impact were recorded by an accelerometer on the top.

24.24 Examples of imperfectly grouted polyethylene sheath: 
(a) three-quarter section grouted and (b) fully grouted.
Firstly, the impact test of the conventional SIBIE method was conducted by one-point detection. The distance between input and output was set at 100 mm across the crack. Secondly, the impact test for the scanning SIBIE method was conducted with three pairs of impact and detection points with 5 cm interval. Fourier spectra of accelerations were analyzed by FFT. Sampling time was 4 $\mu$s and the number of digitized data for each waveform was 2048.

Results and discussion

Results of the conventional SIBIE are shown in Fig. 24.26. For a multiple crack, a high-intensity region is observed at around 150 mm depth, whereas the crack tip visually observed is located approximately at 200 mm depth. Although the depth visually identified is also around 200 mm for a zigzag crack, the high-intensity region is observed at around 165 mm depth. From the one-point detection conducted, the image by the SIBIE analysis is symmetric, and the high-intensity regions are not in good agreement with locations of the crack tip.

Results of the scanning SIBIE are shown in Fig. 24.27, which shows that, the images are in better agreement with the locations of the crack-tips of actual cracks, and the symmetry of images was improved. In both cases, high-intensity regions are clearly observed at around 200 mm depths near the crack tip. Thus, an applicability of the scanning SIBIE procedure is demonstrated.
24.26 Images by the conventional SIBIE for (a) multiple and (b) zigzag surface cracks.

24.27 Scanning SIBIE images for (a) multiple and (b) zigzag surface cracks.
24.4 Conclusions and future trends for on-site application

Continuous AE monitoring was performed during the chloride-induced corrosion of reinforcing steel (rebar) in concrete. High AE activities were observed twice. During the first period of high AE activity, AE sources were of small amplitudes and classified as other-type cracks. At this stage, the chloride concentration at the level of rebar was just higher than the lower-bound level for the initiation of corrosion. Although rebars were removed, no corrosion products were visually observed. According to examination of rebar surfaces by the SEM observation, ferrous ions on the rebar surface are found to vanish at some areas, which could result from the first period of high AE activity owing to generation of small other-type cracks.

During the second period of high AE activity, the chloride concentration was already higher than the upper-bound level for corrosion in the code. When rebars were removed, corrosion products were visually observed at the surface. Here, AE events were associated with fairly large tensile cracks and thus could result from concrete cracking owing to rebar expansion caused by corrosion product. It was confirmed that both periods of high AE activity were observed earlier than the stage where the potentials became more negative than −350 mV. The results clearly show that both the onset of rebar corrosion and the nucleation of concrete cracking can be detected as high AE activities during the process.

For the on-site application of this method, the second period of high AE activity should be taken as indicative of rebar corrosion. This is because the first period of high AE activity is primarily associated with other-type micro-cracks and might not be readily detected in real structures.

The SiGMA analysis is useful for identification of fracture mechanisms in concrete. In order to process a large number of AE waveforms, the auto-picker based on AIC is developed and applied to detected AE waveforms in a four-point bending a reinforced concrete beam. Results of the SiGMA analysis with a hand-picker are in reasonable agreement with those of the auto-picker. Thus, the AE-SiGMA with automated-picker could be readily applied in identifying mechanisms of concrete failure. For existing structures, the SiGMA analysis would be applied in situ to identify cracking mechanisms in a particular element that is structurally important and should be carefully monitored for internal cracking.

The applicability of the procedure to identify ungrouted tendon ducts in post-tensioned concrete girders has been examined both for polyethylene and steel sheaths. Successful results were obtained for both. In addition, imperfectly grouted sheaths were tested. All results show that the method has great potential for inspecting imperfectly-grouted ducts in pre-stressed
concrete bridges. For on-site application, the SIBIE procedure was originally developed to identify ungrouted tendon ducts in existing pre-stressed concrete structures. Based on the results of the case study, it is demonstrated that the procedure is readily applicable for existing bridges and buildings.

Using the conventional SIBIE method, surface-crack depths were visually identified. However, because of the one-point detection, the image became symmetric, and the crack-tip was not accurately estimated. By applying the scanning SIBIE method, surface-crack depths were successfully identified and the problem of a symmetric image was also solved. When surface cracks are observed in real structures, repair work is called for. So far, the confirmation of repair for the grouted surface cracks has been conventionally performed by core-boring. Introducing the SIBIE method, the determination of the surface-crack depths and confirmation of repair could be easily made in on-site applications.

24.5 References


IACS-III B5706 (2003), Monitoring method for active cracks in concrete by AE, Japan Construction Materials and Housing Equipments Industries Federation.

JSCE (2001), Standard specifications for concrete structures ‘version of maintenance’, Japan Society for Civil Engineers.

JSCE (2002), Standard specifications for concrete structures ‘version of construction’, Japan Society for Civil Engineers.


© Woodhead Publishing Limited, 2010

Using ground-penetrating radar (GPR) to assess an eight-span post-tensioned viaduct: a case study

X. DéROBERT, LCPC, France; B. BERENGER, LRPC Angers, France

Abstract: An eight-span post-tensioned viaduct built in the 1960s showed some significant cracks as soon as it was open. Over the past few decades, periodical inspections have shown that these cracks were progressive. In 2002, an expert investigation was recommended, focusing on the state of the cables, and followed by a new structural design. The assessment of the structure was achieved by a combination of ground-penetrating radar (GPR) and gammagraphic techniques, followed by some windowing for the location and evaluation of post-tensioned cables. The results have informed the design of strengthening works, using complementary external post-tension. In a second step, GPR mapping was performed for the layout of coring corresponding to the position of the deviators. The methodology of non-destructive testing (NDT) techniques is discussed, showing the complementarity of NDT and how it could be optimized in this application.

Key words: ground-penetrating radar (GPR), gammagraphy, non-destructive testing (NDT), post-tensioned bridge.

25.1 Introduction

The structure being assessed is an eight-span post-tensioned viaduct built in the 1960s, 342 m long and composed of two independent 14.8-m-wide decks. The decks are Homberg-type post-tensioned ribbed slabs 2.5 m high, which were poured on a launching beam.

As early as the opening of this bridge, cracks appeared in the axis of the coupling sections. An inspection done in 1976 revealed that these cracks were progressive and, in 1980, another inspection recommended the silting up of the cracks.

In 2002, the progression of the cracks was confirmed and the products used for silting up appeared to have deteriorated. The conclusions of this inspection recommended an expert investigation of the post-tensioning followed by a new structural design. The underlying question concerned the state of the cables with regard to any water ingress and their residual stress.
The particular zones to be surveyed were the coupling sections, which presented a high level of cracking, and the bearing zones. The appraisal of the structure and its reinforcement consisted of the following methodology:

- localization of the ducts by ground-penetrating radar (GPR) in order to limit the number of gammadigraphic images required while centering the ducts on the films,
- windowing on areas with missing grouting, visual evaluation of cable corrosion and testing of strand tension for residual stress estimation,
- evaluation of the post-tension of the structure,
- design for strengthening procedures, using complementary post-tension,
- layout of coring by GPR for the positioning of the deviators, and
- strengthening procedures.

Figure 25.1 is a diagram of two spans of the bridge, showing the location of the deviators in relation to complementary post-tension.

### 25.2 Localization of post-tensioned ducts

As the reinforcement maps from the archives could not give sufficiently accurate information for the location of the post-tension, some non-destructive (ND) electromagnetic techniques were used.
Initially, a covermeter device was used to position the rebars close to the surface, while confirming the validity of the reinforcement maps, and locating the ducts. As this technique has a limited depth accuracy, not guaranteed at depths more than 10 cm, the covermeter was unsuitable for use in many places, because various networks of ducts crossed each other in the concrete walls.

Then, a complementary survey with GPR was used instead of the first method. More vertical profiles (Fig. 25.2) were obtained using a high-frequency antenna (1.5 GHz), selecting the positions of the tops of the hyperbolas corresponding to the ducts on the wall, while avoiding the collection of data directly over a reinforcement. The GPR parameter settings were 5 mm for the spatial spacing, and 15 ns for the range, ducts being expected between 3 and 6 ns, depending on the zones.

Finally, the implementation was validated using a similar procedure on the opposite facing. This step is very useful for the face-to-face positioning of the gammagraphic transmitter and receiver.

This approach minimizes the number of films required, while centering them on the most useful places to obtain the maximum information. Indeed, if the ducts were not located by the survey, the typical differences that could exist between the real situation and the reinforcement maps could lead to the duct being displayed on the edge of the gammagraphic film, reducing the surveyed length of the duct to 30%.

Example of vertical GPR profile, carried out at 1.5 GHz, on a concrete wall.
25.3 Gammagraphic imaging

For this appraisal, 12 films were required as representative samplings per risk area, using a GMA-2500 with a cobalt-60 source as transmitter, and AGFA type F8 film, 0.80 m away from the opposite facing (Fig. 25.3). These measurements were performed by the French Public Works Regional Laboratory (LRPC) of Lyon in accordance with the French Standard NF A09-202, using an inspection platform after stopping the traffic in the slow lane. In total, 24 films were taken, corresponding to ~20% of the 68 bearing zones and ~30% of the 32 damaged coupling zones.

25.4 Windowing

The locations of the windows are based on the results of the gammagraphic imaging, focusing on areas showing signs of suspected failure in the post-tension (wire or strand breaking, lack of grouting).

The windowing is achieved in four steps. First, the concrete is cut with a disk grinder and the concrete destroyed by a light air-chisel for the depth remaining below the depth of the duct. The surface involved is about 5–10 cm wide and 20–30 cm long, depending on the diameter of the duct. The last covering of concrete (1–2 cm) is removed manually with a hammer and a hand chisel.

Finally, the duct is cut with a mini-drill combined with a small grinder, and the grout removed (Fig. 25.4a). During all these operations, photographs are taken to study the state of the materials (concrete, rebars, duct, grout, cable).
Some windows are chosen for a complementary crossbow test, and then enlarged to a length of ~60 cm. This technique, tested over many decades in the French Technical Network of Public Works laboratories, enables a tension test to be performed using the direct tensile strength of an isolated wire. The device is equipped with two rollers, 25 cm apart, which are in contact with the wire (Fig. 25.4b). A central hook is inserted under the wire to be tested. Strain and displacement sensors record the ‘strain–stress’ curve in order to obtain the local tension of the wire.
For practical reasons, as it is a destructive technique, only five tests are performed on areas considered doubtful and two on areas considered sound.

Finally, the windows are closed: the strands are protected by an anticorrosive curing, the duct is closed, and the window is back-filled by a resurfacing product adapted for vertical use, with no shrinkage and a good bonding to the existing concrete.

25.5 Evaluation of the structure and reinforcement proposal

The results of the appraisal confirmed the suspicions of the structural engineers, showing missing grouting and suspected cable corrosion (no broken wire being detected). Moreover, the crossbow test revealed values 10% lower than the expected residual stress in the sound areas, and 30% lower in the doubtful areas.

A complete design calculation, using the commercial software PYTHAGORE®, allowed the internal stresses of the bridge to be estimated for every construction phase, and then confirmed the conclusion of the appraisal and explained the origin of the detected cracks.

Therefore, it was proposed that the bridge should be reinforced by adding some external post-tension to it. This additional post-tension is to be achieved by eight series of seven cables (10T15S and 9T15S types) positioned on the sides of the ribs, as shown in the diagram of Fig. 25.1.

The anchors and deviators of the additional post-tension will be added to the structure using reinforced concrete elements fixed by transverse crossing ties (four per element). Therefore, to position the drilling jig for these concrete elements, maps of the reinforcement need to be drawn directly on the structure in order to avoid damaging any of the internal post-tension.

25.6 Localization of post-tensioned ducts and coring

In this phase of reinforcing the structure, GPR was used in order to position the four cores necessary to anchor each deviator directly on the facings using crossing ties. Over 260 drawing maps (1.5 m × 2 m) were required. For the whole operation, it was recommended that 12 maps per scaffolding position were produced each day, over a four-month period.

The GPR parameter settings were 5 mm for the spatial spacing, and 15 ns for the range, ducts being expected between 3 and 7 ns depending on the zones, using a 1.5 GHz antenna. The profile directions were vertical and horizontal in order to remain similar to the axis of the rebars, while avoiding the collection of data directly over a reinforcement.

© Woodhead Publishing Limited, 2010
The first stage of the GPR survey was dedicated to validation of the protocol. The method consisted of drawing the reinforcements during their acquisition, while positioning the cursor, corresponding to the center of the antenna, on the top of every hyperbola. We then had to minimize the number of profiles, while still guaranteeing that each selection drawn on the facing could be linked correctly and actually correspond to a duct or a rebar.

About four profiles per direction were sufficient (>30 cm spacing) for each drawing map, once the expected slopes of the ducts had been confirmed in the first GPR feasibility stage. As various networks of ducts crossed each other, each with different slopes, any ambiguity in their positioning led to several (three or four) intermediate vertical profiles.

Figure 25.5a and 25.5b present some examples of complex reinforcement drawings, with various duct slopes. Ducts are depicted by two lines linked by small waves, in order to represent their 5-cm diameter, and the rebars are depicted by dashed lines. In Fig. 25.5a, a linking strand is visible in the lower right part of the image, localized with a continuous line. The area marked by the dashed line in Fig. 25.5a, corresponds to the expected future position of the deviator.

Depending on the position of the post-tension, the layout of the deviators had a margin of ±10 cm. As four cores are necessary for the anchorage of the deviators with crossing ties, the limitation of the margin for their positioning required some cutting of the rebars (Fig. 25.6). This was especially the case for the deviators close to the piers, owing to the high density of vertical rebars.

Once the cores had been made, it was possible to build the deviators with surface concrete drilling, reinforcement, formworking, and pouring. Finally, after 28 days, while the nominal strengthening was increasing, the post-tension was added (Fig. 25.7).

25.7 Discussion of the applied methodology

The NDT methodology employed both for the appraisal and the positioning of the cores appeared to be efficient and well integrated during the work stages. To our knowledge, no other techniques could have performed as well; in fact, no duct was cut or damaged, which was the first requirement. Moreover, although some rebars were cut, this fact was accepted before making the cores, all of them being correctly localized by GPR.

Nevertheless, a question remains about the possible use of GPR 3D mapping, as proposed by the manufacturers. This kind of approach, together with specific software modules, allows the visualization and location of all kinds of reinforcement. Figure 25.8 presents an example of 3D imaging. The left side shows the reinforcement during processing: a manual selection
25.5 (a) and (b) Examples of drawing maps of the reinforcement for complex cases. In panel (a), the expected position of the deviator is within the dashed line.

of the GPR profiles. The right side shows the rebar localization. A reorientation of the 3D image is possible to obtain a vertical view of the reinforcement.

Such an approach is interesting for obtaining a good understanding of the reinforcement mapping as well as for an accurate positioning of the ducts. The drawbacks that remain are the acquisition duration, and the time needed to the draw the result on the concrete facings.
For our particular application, a 3D-GPR mapping could have been useful for the validation of the feasibility step, in one or two areas, for a quick understanding of the reinforcement. But increasing by fourfold the time spent on each drawing would not have added enough value, when only four types of reinforcement were encountered in over 250 mappings. Moreover, this technique could become incompatible with this type of works management.

25.6 (a) and (b) Example of core, revealing a cut rebar.
25.7 Deviators (a) after uniformworking and nailing, and (b) before post-tensioning.

25.8 Acknowledgements

The authors would like to thank Entreprise VSL-France, the main works contractor of this bridge, who allowed the distribution of this information. Many thanks also to the French Network of Public Works Laboratories: Lille for the crossbow technique, Lyon for the gammagraphic imaging and Saint-Brieuc for the radar survey.
Example of 3D-GPR mapping during processing with a specific software module (courtesy MDS – Le Matériel de Sondage).

25.9 References


French Standard NF A09-202 ‘Principes généraux de l’examen radiographique à l’aide de rayons X et gamma des matériaux béton, béton armé et béton précontraint’.


Index

A1040, 509
A1220, 508
A1220 Monolith, 51
A scan processing, 127
A scans, 7, 477
Acoustic Control Systems, 508, 509
acoustic–elastic waves
   non-destructive testing of concrete, 125–42
   future trends, 142
   sophisticated data processing, 136–42
   standard data processing, 127–35
   wave propagation and implications on data processing, 125–7
acoustic emission analysis, 121
acoustic emission array processing, 202
acoustic emission sensors, 191–2
acoustic emission technique
   AE waveform
      and AIC values, 548
      and two parameters, 546
      parameters, 544
   and impact–echo techniques for reinforced concrete structures evaluation, 543–72
applications for concrete structures, 544–50
   parameter analysis, 544–5
   SiGMA analysis with AIC picker, 546–8
corrosion detection, 551–69
   accelerated corrosion test results and corrosion loss mode, 555
   AE activities and half-cell potentials, 553
   AE parameter analysis results in cyclic wet–dry test, 556
   ferrous ions distributions on rebar surface, 558
   rebars visual observation, 557
   reinforced concrete specimen, 552
   crack models in SiGMA analysis, 547
   cracks classification, 545
   future trends for on-site application, 571–2
   limitations and accuracy, 206–10
   accuracy picking signal onset times, 209–10
   activity measured with sensors, 208–9
   Kaiser effect, 207
   localisation accuracy, 209
   standard errors for localisation, 210
   parametric and signal-based analysis, 187–91
   comparison, 190
   parameter-based techniques, 188–9
signal-based techniques, 189–91
reinforced concrete structures evaluation, 185–210
burst signals compared with continuous acoustic waves emission, 187
sensors and instruments, 191–3
recording devices, 192–3
sensors, 191–2
shear failure mechanisms in reinforced concrete by AE-SiGMA, 559–61
AE events analysed by SiGMA, 560
observed AE generation behaviours, 560
reinforced concrete beam sketch and AE sensor array, 559
SiGMA analysis results by auto-picker, 561
source localisation, 193–7
3D localisation, 194–7
3D visualisation, 196
onset time extraction, 196
planar location principle, 194
seismograms recorded by triaxial sensor, 195
zone and planar location methods, 193–4
source mechanisms and moment tensor analysis, 197–9
inversion techniques principle, 198
shear crack radiation pattern, 199
acoustic impedance
pressure wave and reflection factors relative to cement matrix, 494
received signals, 495
acoustic methods, 6
acoustic wave equation, 126
acoustic waves
non-destructive testing of concrete, 144–59
constitutive equations, 147–8
fundamental waves, 148–51
numerical wave field modelling, 151–4
underlying physics, 146–7
wave field inversion and imaging, 154–8
FT-SAFT, 156–8
SAFT, 154–5
wave field inversion as non-linear problem, 155–6
ACSYS A1220, 55
active thermography, 5, 397–9
applications, 386–96
automatic crack detection in thermograms, 394
CFRP on concrete, 390
cement parameters for different test specimens, 387
crack signature increase owing to capillary water transport through the crack, 393
cracks characterisation, 390–1, 393
grey scale coded temporal crack growth, 395
large area with voids after heating with IR radiators, 391
limits and requirements, 393–6
material properties investigation, 386–9
maximum temperature contrast and time, 388
plaster on concrete, 389
temperature transient curves, 388
thermograms of an image sequence during cooling down, 392
voids location in industrial precast concrete beams, 389
data processing, 380–6
combination with other methods, 385–6
histogram of thermogram recorded at a concrete test specimen, 383
pulse-phase thermography, 381–3
reconstruction, 384–5
segmentation, 383–4
superpositioning of photogrammetric equalised digital photo, 386
surface temperature as function of time for each pixel, 380
thermal contrast, 380–1
thermogram with maximum temperature contrast for heating time, 384
thermograms, amplitude images, phase images and radar depth slices, 382
direct heating sources applied in civil engineering, 378
experimental equipment and calibration, 376–80
calibration, 379–80
characteristic data of various objective lens used for SC1000 IR camera, 379
heating sources, 377–8
infrared camera, 378–9
schematic principle of the method and typical example, 377
future trends, 396
guidelines and sources of further information and advice, 397–9
national (German) and international standards and guidelines, 398
physical principle and theoretical background, 372–4
concrete, red brick and granite spectra, 374
heat transfer theoretical background, 372–3
temperature measurement physical principle, 373–4
reinforced concrete structures evaluation, 370–99
state of the art, 375–6
advanced infrared technology and application, 397
ageing phenomena, 52
AGFA type F8 film, 577
Akaike information criterion, 543
alkali metals, 176–8
alkali–silica reaction, 176–8, 255
ALOK, 44
alternating current impedance spectroscopy, 268–9, 270
American Society for Testing and Materials, 397, 399
Ampère’s law, 145
amplitude, 476
amplitude tomography, 337, 342–3, 344, 353
AMUS-P, 44
32-channel unit, 45
anisotropy, 266
aperture, 506
Archie’s law, 245, 252, 255
Association of German Engineers, 399
ASTM C403, 425
ASTM C597, 421
ASTM C786, 286
ASTM C876, 554
ASTM C1202–97, 243
ASTM C1383–4, 467
ASTM C876–87 standard, 303
ASTM C403M, 425
ASTM E251, 90
ASTM E610, 187
ASTM E 976–99, 204
AURA see automated ultrasonic rail
Aura Kaiserslautern, 39, 40
AURA Krefeld, 39, 40
AURA Paderborn, 39
AUROPA, 36–8
integration into inspection rail, 38
train before in-motion inspection, 39
transducer side view with probe housing, 38
transducer top view, 37
AUROPA II, 37
automated non-destructive testing case studies of successful innovations, 36–46
in-line pipeline-inspection gauges, 43–6
railway safety, 36–43

© Woodhead Publishing Limited, 2010
construction engineering, 46–54
ageing phenomena diagnosis, 52
analysis procedure schematic, 53
BetoScan, 47–51
BetoScan system features, 51–2
driving modes, 51
life prediction, 53
project partners, 48
quality management, 54
repair planning, 54
service life management, 54
data acquisition, control and evaluation
multisensor data acquisition and analysis, 35
multiple-sensor data acquisition by OSSCAR scanner, 54–9
application for bridge testing according to German Standard DIN 1076, 56
development and features, 56–7
genral remarks, 54–6
method combination, 58
scanner frame, 55
scanner software screen shot, 59
reinforced concrete structures and other applications, 30–60
innovation cycles, 31–4
automated ultrasonic rail, 38–41, 43
autoregressive models, 119, 120

B scan, 7, 477–8
B scan processing, 127
bandpass filtering, 127–8
beamforming acoustic emission, 203
Berlin Central Railway Station, 88
Berlin Westendbrücke, 87
BetoScan, 31
features, 51–2
data evaluation, 52
non-destructive diagnosis of reinforced concrete structures, 47–51
demonstrator system, 49–50
NDT methods, 50
project partners, 48
robotic system, 49
sensors, 50–1
borehole GPR antennae, 341
Born approximation, 364
boundary plane, 492
Bragg wavelength, 70–1
bridge testing, 56
Brillouin scattering, 69
British Institute of Non-Destructive Testing, 397
building diagnosis
NDT state of the art and future trends, 14–29
applications examples, 18–27
efficient testing methods, 15–17
future trends, 28–9
tasks, 14–15
C scan, 7, 477–8
calcium/oxygen ratio, 170
Canin +, 50
capacimetry
applications, 281–2
radar and capacitive measurements vs volumetric salted water content, 282
calibration, 280
capacitance probe prototype, 279
capacitive calibration, 280
capacitive measurements vs volumetric water content, 281
data acquisition and interpretation, 280–1
equipment, 279
physical principle and theory, 276–9
equivalent capacitive scheme of sensor and its electrodes, 277
relative permittivity, 278
reinforced concrete structures evaluation, 276–83
limitations and reliability, 282–3
capacitance, 276
capacitance value, 277
capacitive calibration, 280
capacitive measurements
vs volumetric salted water content on homogenous concrete, 282
vs volumetric water content of concrete mixtures, 281
capacitive sensor, 278
capacitive techniques, 276
capillary sorption, 411
carbon-fibre-reinforced plastic, 371
on concrete, 390
test specimens that have no contact at hatched areas, 392
thermogram after infrared radiator heating, 392
carbonation, 255, 353
cell factor, 256
cement, 167–71, 247
CFRP see carbon-fibre-reinforced plastic
chlorine, 173–5
Cole–Cole model, 335–6
compressional wave, 443
compressive strength, 499, 502
concrete, 167–71, 247, 250
see also concrete structures;
  reinforced concrete structures
non-destructive testing with
  electromagnetic and acoustic–elastic waves, 125–42
  future trends, 142
  seismic, ultrasonic and electromagnetic wave propagation, 125–7
  sophisticated data processing, 136–42
  standard data processing, 127–5
concrete cover, 19, 53
concrete resistance, 256
concrete structures
active thermography measurements
  for voids location inside precasted concrete beams, 390
  impact–echo techniques evaluation, 466–85
  applications, 481–4
  basic principles, 467–9
  data interpretation, 469–74
  equipment, 480–1
  future trends, 484–5
  method development history, 466–7
  numerical simulations, 474–5
  signal processing, data presentation and imaging, 475–80
  material properties characterisation, 499, 501–2
  compressive strength, 502
  elastic moduli, 501–2
  other properties, 502
surface wave techniques evaluation, 441–60
  basic principles, 443–8
  equipment, 457–9
  field application, 459–60
  signal processing and data presentation, 449–57
conductance, 244
conductivity, 244
constitutive equations, 147–8
contact resistance, 261
continuous wavelet transform, 449, 477
corrosion, 261–2, 264–5, 284
corrosion evaluation
  how to interpret measurements, 303–5
  corrosion potential and electrical resistance, 303
  corrosion rate, 303–5
measurement methods, 293–302
  commercial corrosion rate meter and sensors, 303
  corrosion potential and polarisation resistance, 293–4
  corrosion rate on site in concrete structures, 297–302
  devices for on-site determinations, 302
  obtaining polarisation resistance, 294–7
measuring corrosion rate and corrosion potential of reinforced concrete structures, 284–313
future trends, 311–12
monitoring systems, 310–11
embedded and surface sensors, 311
monitoring results in concrete structure, 312
practical application, 306–10
corrosion potential measurements, 306–7
corrosion rate measurement, 307–10
principles, 285–92
corrosion potential, 285–8
corrosion rate, 289–92
resistivity, 288–9
values of resistivity and corrosion risk, 289
corrosion penetration, 291–2, 304
corrosion potential
how to interpret measurements, 303
measurement, 293–4
practical application, 306–7
principles, 285–8
chloride content, 287
concrete moisture content, 286–7
cover thickness, 287
isopotential lines generated by corroding zone, 286
macrocell polarisation effects, 287–8
oxygen content, 288
pH influence, 287
ranges of potential and corrosion risk, 287
corrosion rate
how to interpret measurements, 303–5
log–log diagram, 305
measurement on site in concrete structures, 297–302
critical length reached by signal, 297–9
current attenuation with distance from current electrode, 301
error factor, 299
localised corrosion spots on current lines, 300
modulated or sensorised confinement of polarising current, 300–1
multiple electrode or potential attenuation method, 301–2
non-uniform distribution of current, 298
theoretical confinement of applied current, 300
transmission line model, 298
practical application, 306–10
accurate measurement, 309
auxiliary probe placement, 309
before starting the survey, 307
equipment and reinforcement connection, 309
location of the bars, 308
preparation of concrete surface, 308
principles, 289–92
accumulated corrosion penetration depth, 291–2
corrosion rate vs time, 292
polarisation resistance, 289–90
progressive accumulated corrosion depths, 293
Randles circuit, 291
ranges and relation to velocity, 291
ranges of values, 291
corrosion risk, 287, 289
corrosion sensor, 86–7
COST 299, 89
counter electrode, 294
covermeter, 5, 17
crack intensity, 267
cracks, 267–8
critical length, 298
crossbow test, 579
current density see corrosion rate
current pulse–echo methods, 502

cyclic wet–dry test, 556

3D GPR mapping, 580, 582
during processing with specific software module, 584
3D localisation, 194–7
3D migration, 139
3D SAFT, 103
data acquisition system, 192
data cleaning, 344
data fusion, 28, 35, 96, 98–100
pixel level, 98–100
schematic view, 99
terms/definitions, 98
data processing, 127
implication of seismic, ultrasonic and electronic wave propagation, 125–7
sophisticated, 136–42
deconvolution/shaping, 136–8
frequency-wavenumber filter, 140–1
migration, 138–40
time-depth conversion, 141–2
standard, 127–35
clutter reduction, background removal, 132–3
dewowing and standard bandpass filtering, 127–8
profile energy balancing, 134–5
time-base shift correction, 128–9
time varying gain, 131–2
time zero, 129–31
deay constants, 423
deformation, 147
deformation rate equation, 146
deformation sensors, 88–9
Dempster–Shafer theory, 96
detection spatial resolution, 69
Deutsche Gesellschaft für Zerstörungsfreie Prüfung, 14, 318
dewowing, 127–8
DGZfP see Deutsche Gesellschaft für Zerstörungsfreie Prüfung
dielectric displacement, 145
diffraction tomography, 364–5
dilatation, 147
dilatation rate equation, 147
DIN 54162, 397
DIN 54190-1, 397
DIN 54190-2, 397
DIN 54190-3, 397
DIN EN 473, 8
DIN EN 13187, 397
DIN EN 1330-9, 187
DIN/ISO 17025, 8
direct reflections, 374
8-channel DiSP system, 559
displacement current, 145
distributed computing strategy, 118
dry point contact, 508, 513
echelle spectrometer, 167
EFIT see elastodynamic finite integration technique
EFPI see extrinsic fibre Fabry–Perot interferometer sensors
eight-span post-tensioned viaduct applied methodology, 580–2
3D-GPR mapping during processing with specific software module, 584
evaluation of structure and reinforcement proposal, 579
gammagraphic film showing lack of grouting, 577
gammagraphic imaging, 577
ground-penetrating radar assessment, 574–84
post-tensioned ducts and coring localisation, 579–80
core revealing a cut rebar, 582
deviators after unformworking and nailing and before post-tensioning, 583
drawing maps of reinforcement for complex cases, 581
post-tensioned ducts localisation, 575–6
two spans of the structure diagram, 575
vertical GPR profile, 576
windowing, 577–9
crossbow device in position on wire, 578
window revealing the state of grout and wires, 578
elastic moduli, 499, 501–2
elastic waves, 146
non-destructive testing of concrete, 144–59
fundamental waves, 148–51
numerical wave field modelling, 151–4
underlying physics, 146
wave field inversion, 158–9
elastodynamic finite integration technique, 151, 154, 499
electric field, 335
electric flux density, 145
electrical anisotropy
characterisation, 266–8
electrical resistivity tomography on concrete, 269
fibre orientation in steel-fibre reinforced concrete, 266–7
logarithm of variations, 268
study of cracks in concrete, 267–8
synthetic polar diagram, 267
electrical resistance, 244, 289, 303
electrical resistivity
application, 255–64
calibration and reliability, 263
equipment and measurement, 255–63
corrosion current-resistivity, 262
four-probe devices optimum locations, 261
four-probe resistivity measurement technique, 259–63
four-probes square device, 260
measurement cell for laboratory resistivity assessment, 256–8
resistivity cell device, 256
resistivity range related to risk of corrosion, 262
resistivity vs apparent resistivity, 260
sample surface and length, 257
single-probe device, 258
single-probe resistivity measurement technique, 258–9
two-probe resistance measurement technique, 259
impedance spectroscopy, 268–70
Nyquist diagram, 269
influencing factors, 246–55
aggregate, 248
chloride concentration, 252
cement resistivity evolution, 250
edge on resistivity measurement, 254
external factors, 252–5
fluids, 251–2
intrinsic factors, 246–50
range of variation, 249
saturation rate, 251
variations with temperature, 253
water/cement ratio, 247
other developments, 264–8
electrical anisotropy characterisation, 266–8
electrical resistivity tomography, 268
monitoring for durability, 264–6
MRE probe, 266
physical principles and theory, 244–55
Archie’s law parameters, 245
electrical conduction in concrete, 244–6
reinforced concrete structures evaluation, 243–70
cement resistivity values, 244
electrical resistivity tomography, 268, 269
electrochemical impedance spectroscopy, 268, 297
electrochemical noise, 296
electromagnetic waves
non-destructive testing of concrete, 125–42, 144–59
constitutive equations, 147–8
fundamental waves, 148–51
future trends, 142
numerical wave field modelling, 151–4
sophisticated data processing, 136–42
standard data processing, 127–35
underlying physics, 145–7
wave field inversion, 158–9
wave propagation and implications on data processing, 125–7
electromagnetically generated and received ultrasound, 37
electronic scanning, 521–3
emissivity, 374
EN 473, 397
EN 13791:2007, 502, 526
extrinsic fibre Fabry-Perot interferometer sensors, 72, 80
measured restraining effect of steel bar, 81
silicon mould with EFPI before casting, 78

Faraday’s law, 291
Faraday’s law of induction, 145
fast Fourier transform, 475, 550
FBG see fibre Bragg grating sensors
ferritic steels, 217
fibre Bragg grating sensors, 70–2, 77
schematic, 70
fibre Fabry-Perot interferometer sensors, 70, 72–3, 78, 80, 81, 82–3
microsection, 79
sensors fixed on reinforcement cage, 82
fibre-optic pH sensor, 86
head design, 85

fibre-optic point sensors, 73, 83–7
prefabricated anchors with fixed pH sensors, 86
water-filled topcoat, 85
fibre-optic relative humidity sensor, 77
fibre-optic sensors, 67–8, 91
long-gauge-length, 68–70
short-gauge-length, 70
types, 68
vs laser vibrometer results, 80
finite difference time domain, 137, 139, 142, 365
finite integration technique, 146, 151
fk-filter see frequency-wavenumber filter
focal plane array, 373, 375
four-probe resistivity measurement technique, 259–63
Fourier Slice Theorem, 339, 349
Fourier transform, 423
Fourier transform-SAFT, 99, 103, 156–8, 506
FPI see fibre Fabry-Perot interferometer sensors
Fraunhofer-Gessellschaft, 30
Fraunhofer IZFP, 408
free induction decay, 406
French Standard NF A09–202, 577
frequency downshift method, 363–4
frequency-wavenumber filter, 133, 140–1
Fresnel rays, 339
Fresnel region, 322
Fresnel theory, 347
Fresnel zone, 338
full matrix reconstruction, 509
fundamental waves, 148–51
homogeneous materials, 148
inhomogeneous materials, 151
plane waves, 148–9
spherical waves, 150–1
galvanostatic pulses, 295–6
Gecor 08, 302
Geiger method, 194
GEOPSY, 446
German directive RI-ZFP-TU, 526
German Rail, 37
German Standard DIN 1076, 56
giant magnetoresistance sensor, 234
global capacitance, 277
GMA-2500, 577
GPR see ground penetrating radar
GPR ProEX system, 50
Green's function approach, 385
ground penetrating radar, 127
case study of concrete retaining walls
  inspection, 533–42
  data acquisition, 534–6
  data processing, 536–7
  problem description, 533
  results, 537–41
current and future trends, 327–31
  bridge girder, 328
  3D radar surveys, 328
  inversion, 329–31
  inversion and modelling, 330
  inversion result from reinforced concrete, 330
  material properties, 331
  quotient of amplitudes and chloride/moisture content of specimens, 331
  radargram from bridge girder, 329
  time slice from bridge girder, 329
data display formats, 322–3
  data cube acquisition and time slice display, 323
  measurement in single position, 322
  radargram acquisition, 323
  data processing and interpretation, 323–5
  processed dataset, 324
  processed dataset with interpretation, 325
  raw dataset, 324
eight-span post-tensioned viaduct assessment, 574–84
  applied methodology, 580–2
gammagographic imaging, 577
localisation of post-tensioned ducts and coring, 579–80
post-tensioned ducts localisation, 575–6
structure and reinforcement proposal evaluation, 579
windowing, 577–9
equipment, 325–6
  accessories, 326
  antennae, 325
  central unit, 326
  mobile GPR acquisition system, 327
limitations and reliability, 326–7
main advantages, 317
physical principles and theory, 318–22
  electromagnetic wave with area, 320
  sources and sinks in longitudinal wave, 319
principles, 318
reinforced concrete structures evaluation, 317–32
  symbols and constants, 331–2
GSSI-SIR20 radar unit, 536
gyromagnetic ratio, 405
Hertzian dipole, 345
Hooke's law, 147
hydrogen, 178–80, 405
ICE1, 37
ICE III, 39
Imote2, 120
impact-echo techniques, 17, 23, 421
  and acoustic emission techniques for reinforced concrete structures evaluation, 543–72
  applications, 481–4
  applications for concrete structures, 544–50
  SIBIE analysis, 548–50
  basic principles, 467–9
  impact-echo method test principle, 468
© Woodhead Publishing Limited, 2010
vibrational resonances, 469
wave propagation, 467–8
concrete structures evaluation, 466–85
data interpretation, 469–74
compression wave velocity time
domain determination, 473–4
conventional interpretation of
spectral response in plate-like
structures, 469–70
guided-wave interpretation of
spectral response in plate-like
structures, 470–2
impact-echo shape factor as
function of Poisson’s ratio, 472
interpretation of spectral response
in other structures, 472–3
normalised phase velocity
dispersion curves, 471
double-leaf wall thickness
determination, 23–5
view of remaining layer after
demolition, 25
wall cross section with distortion
after concreting, 23
wall thickness as a height profi le,
24
wall thickness measurement, 24
equipment, 480–1
automation equipment, 481
conventional testing systems, 481
future trends, 484–5
future trends for on-site application,
571–2
imperfectly grouted sheath examples
1/4 and 3/4 section grouted, 567
3/4 section and full grouted, 568
internal defect detection, 481–4
delamination, 481–3
impact-echo method for small
void detection, 484
internal voids including post-
tensioning duct voids, 483
other applications, 483–4
method development history, 466–7
numerical simulations, 474–5
reference manuals, guidelines and
standards, 467
setting time measurement, 428
signal processing, data presentation
and imaging, 475–80
B scan above an empty tendon
duct, 478
basic Fourier transform, 475–6
C scan measured with laser
interferometer, 479
measurement carried out on
0.25 m thick slab, 475
other 2-D and 3-D image
construction, 477–80
S1 Lamb mode extraction in noisy
signal with wavelet transform,
477
time-frequency analysis, 476–7
slab/wall thickness determination,
481
strength development measurement,
430
surface-crack depth evaluation by
SIBIE, 567–9
conventional SIBIE images, 570
observed bending cracks, 569
scanning SIBIE images, 570
surface response fi nite element
modelling, 474
ungrouted tendon-duct identifi cation
by SIBIE, 561–7
frequency spectra detected, 564
impact test in concrete block with
polyethylene and steel sheaths,
562
metal sheath results, 565
polyethylene sheath results, 566
sketch of imperfectly grouted
sheath, 563
impulse-echo, 157
impulse thermography, 376
indirect refl ections, 374
Iner City train Experimental, 37
infrared thermography, 370
innovation cycles, 31–4
inside-out NMR devices, 404
inverse media, 447
inversion, 329–31
ionic ingress, 265–6
ISO/IEC 17025/2000, 89
isotropy, 266

Japan Construction Materials Standards, 545
joint inversion, 35
JSM-5600, 553

Kaiser effect, 206–7
Kirchhoff migration, 536
Kirchhoff’s law, 374

Lamb wave analysis, 455
Lamb wave dispersion theory, 470
Lamb wave ZGV frequency, 474
Lamb waves, 447–8, 470
Lamé constants, 148, 468
Large Concrete Slab, 516
Larmor frequency, 405
laser-induced breakdown spectroscopy, 7, 167
cement, mortar and concrete characterisation, 167–71
ablation crater visualisation, 169
calcium and sodium distribution, 172
hydrogen distribution in a concrete sample, 174
melt mineralogical structures on crater walls, 170
mineral aggregate and cement matrix distribution, 171
multi-component system concrete, 168
normalised intensity levels differentiation, 173
fundamentals and measurement, 164–7
plasma ignition and concrete sample with LIBS measurement, 166
typical result, 167
typical set-up, 165

limitations and reliability, 183–4
calibration function characteristics and sections, 183
process limits in hardened cement paste, 184
mobile set-up: on-site applications, 180–3
horizontal and vertical measuring mode, 181
on-site use, 182
schematic diagram, 181
reinforced concrete structures evaluation, 163–84
specific elements detection, 173–80
alkali metals, 176–8
change in spectrum with wet building material surfaces, 179
chlorine, 173–5
chlorine distribution imaging, 176
chlorine gradient, 175
cement, mortar and concrete characterisation, 167–71
ablation crater visualisation, 169
calcium and sodium distribution, 172
hydrogen distribution in a concrete sample, 174
melt mineralogical structures on crater walls, 170
mineral aggregate and cement matrix distribution, 171
multi-component system concrete, 168
normalised intensity levels differentiation, 173
fundamentals and measurement, 164–7
plasma ignition and concrete sample with LIBS measurement, 166
typical result, 167
typical set-up, 165
limit of detection, 183
limit of quantification, 183
linear variable differential transformers, 67
lithium, 178
LOCAN 320, 552
long-gauge-length sensors, 68–70, 74–6
longitudinal waves, 492
magnetic field method, 17
detection of cracks in pre-stressed steel rods, 25–7
imaging of cracks of two tendons in highway slab, 27
investigation of single transverse tendons, 26
magnetic field scanner application, 27
post-tensioned concrete, 26–7
pre-stressed concrete in the direct bond, 25–6
pre-stressed concrete roof trusts, 25
magnetic flux density, 145
magnetic flux leakage
applications, 235–9
equipment, 237
MFL signal, 237
pre-stressed ceiling beams, 236
residual field signal of damaged and intact beam, 236
residual field signals of sensors, 238
non-destructive evaluation of pre-stressed concrete structures, 215–41
recommendations for application, 240–1
recent developments, 239–40
joint bridge bearings measurement result, 240
system on-site, 240
reinforced concrete structures inspection, 215–33
amplitude of rupture signals, 224
analysis for measured signals evaluation, 225–9
axial component, 220
damaged tendon residual field signal, 233
detection of pre-stressed tendons ruptures, 215–16
empirical parameter for rupture amplitudes estimation, 224
laboratory tests arrangement, 230
limit of detection, 229–33
long term test, 234
magnetic leakage signal, 221, 222
magnetisation measurement scheme, 221
modelling, 218
on-site test, 232
physical basis, 216–25
pre-stressing steel magnetisation curve, 218
principle, 217
residual field measurements, 223
rupture signals calibration, 223
signal analysis, 228
synthetic rupture and filtered residual field signal, 228
required equipment, 233–5
exciting magnetic field components, 235
probe, 234
magnetic induction see magnetic flux density
magnetic resonance imaging, 403
magnetisation, 217
magnitude, 476
MALÅ ProEx, 55–6
MASW see multi-channel analysis of surface waves
Matrix techniques, 447
Maxwell equations, 126, 145, 318, 319
MDT see minor destructive testing
measuring spatial resolution, 69
mechanical wave see stress wave
MEMS see microelectromechanical systems sensors
MFL see magnetic flux leakage
microbolometers, 375
microelectromechanical systems sensors, 114, 121
Middleware, 117, 118
migration, 138–40
MIN-02, 562
minimum norm stabiliser, 348
minimum support stabiliser, 348
minor destructive testing, 95, 96–8
modulated confinement method, 300–1
moisture, 265–6
moment tensor, 197–9
moment tensor inversion methods, 197, 198
mortar, 167–71
motes, 112, 113, 114
MSP430, 120
multi-channel analysis of surface waves, 442, 453–5, 459–60, 474
multi-channel simulation with one receiver procedure, 458
multi-resonance transducers, 192
multihop network, 112, 113
multiple electrode method, 301–2
multisensor systems, 34–6

N-colour tomography, 348
NAS 942, 90
NDT Systems and Services, 43–4
Newton-Cauchy equation, 146
NMR-INSPECT, 404, 408, 409
NMR logging tools, 404
NMR-MOLE, 404
NMR-MOUSE, 404
noise-equivalent temperature difference, 375
non-contacting corrosion method, 313
non-destructive evaluation
magnetic flux leakage for prestressed concrete structures, 215–41
application, 235–9
description of equipment required, 233–5
recent developments for reinforced concrete inspection, 239–40
recommendations for application, 240–1
reinforced concrete structures inspection, 215–33
non-destructive testing, 418
acoustic waves field inversion and imaging, 154–8
heuristic inversion, 154–5
processing alternatives to SAFT, 156–8
wave field inversion as a non-linear problem, 155–6
application examples, 18–27
air-filled ducts and layer borders, 22
concrete cover measured values, 21
displacement bodies, 22
exposed pile head, 19
exposed pile wall area, 20
magnetic field method, 25–7
other testing problems with radar, 21–2
radargrams of measuring lines, 20
reinforcement location and concrete cover measurement with radar, 18–21
thickness of wall as height profile checked with impact-echo, 24
tunnel cross-section schematic, 18
vertical measuring lines per position, 19
building diagnosis, 14–29
automated data recording, 28
efficient testing methods, 15–17
future trends, 28–9
task, 14–15
test tasks, 16
combining results for reinforced concrete, 95–106
combination of methods for concrete structures investigation, 97
future trends, 105–6
NDT and MDT methods, 96–8
data fusion, 98–100
pixel-level, 98–100
processing schematic, 99
terms/definition, 98
electromagnetic, acoustic and elastic waves modelling and imaging, 144–59, 145–51
constitutive equations, 147–8
fundamental waves, 148–51
shear-vertical and shear-horizontal plane waves, 150
tendon duct below reinforcement, 145
underlying physics, 145–7
Index

electromagnetic and acoustic–elastic waves, 125–42
future trends, 142
seismic, ultrasonic and electromagnetic wave propagation, 125–7
electromagnetic and elastic waves
field inversion, 158–9
elastic wave field modelling and inversion, 160
electromagnetic vector wave data, 159
guidelines and recommendations, 11–12
impact-echo
double-leaf wall cross section after concreting, 23
double-leaf wall thickness, 23–5
thickness of wall as height profile, 24
view of remaining wall layer after demolition of second layer, 25
wall thickness measurement, 24
international, European and National Standards, 9–10
methods for concrete structures, 419
numerical wave field modelling, 151–4
computer model of a concrete specimen, 153
2D acoustic wave scattering, 152
2D elastic wave propagation, 154
2D elastic wave scattering, 153
overview, 6–7
acoustic methods, 6
electromagnetic methods, 6–7
optical methods, 7
 qualification/validation of methods
methods validation, 8
qualification of personnel, 8
radar and ultrasonic data fusion
along a beam of a box girder bridge, 101–2
C scans, 103
cross beam inside a box girder bridge, 103–5
radar data recording, 102
radar results of vertical slice at cantilever, 104
results with and without velocity correction, 104
radar data fusion, 100–1
radar trace, 100
recommended 3D FT-SAFT B scan along concrete floor plate, 101
reinforced concrete structures, 3–12
strategies for application, 4–6
sophisticated data processing, 136–42
2D profile and fk-filter application, 141
deconvolution/shaping, 136–8
diffractors and steep reflectors, 139
frequency-wavenumber filter, 140–1
migration, 138–40
radargram with strong ghosts, 138
radargram with strong multiples, 137
raw and migrated data, 140
time-depth conversion, 141–2
wavelet shaping filter application, 136
standard data processing, 127–35
clutter reduction, background removal, 132–3
continuous gain function, 131
2D line before correction, 129
2D raw profiles from 3D grid survey, 134
dewowing and standard bandpass filtering, 127–8
dewowing process, 128
errors for time zero correction, 130
incoherent clutter reduction methods, 133
noise ringing, 132
profile energy balancing, 134–5
time-base shift correction, 128–9
time slice for 3D grid survey, 135
time varying gain, 131–2

time zero, 129–31

transmitter receiver configuration, 130

Non-destructive testing in civil engineering, 397, 526

normal moveout correction, 126

nuclear magnetic resonance imaging

application possibilities, 409–13

early-age concrete hardening, 412–13

liquid transport coefficients determination, 410–12

moisture depth profiles determination, 409–10

future trends, 414–15

hardware, 407–9

conventional equipment with enclosing magnet and coil, 408

OSA–NMR equipment with open magnet and coil, 408

physical background, 404–7

physical principle schematic description, 405

two NMR signals schematic presentation, 406

reinforced concrete structures evaluation, 403–15

reliability and limitations, 413–14

numerical wave field modelling, 151–4

Ohm’s law, 288

On-Site SCAnneRs, 31

multiple-sensor data acquisition, 54–9

application for bridge testing, 56

development and features, 56–7

method combination, 58

scanner frame, 55

scanner software, 59

one-side access nuclear magnetic resonance, 404

calibration curve, 409

concrete hardening monitoring by relaxation curve measurement, 413

equipment with open magnet and coil, 408

liquid transport coefficient, 412

moisture profile measurement on lightweight concrete pillar, 410

one-side access technique, 404, 410

Ontario Hydro Canada, 421

optical methods, 7

optical time domain reflectometry, 74–5, 76

OSA-NMR see one-side access nuclear magnetic resonance

OSSCAR see On-Site SCAnneRs

p-omega transform, 454

P-wave, 419–20

see also compressional wave

parameter-based acoustic emission techniques, 188–9

PCUS 40, 39

permeation, 411

phase match filtering, 449

phase velocity, 497

piezo-resistive cement-based strain sensors, 74

piezoelectric sensors, 191, 192

pile integrity tests, 82–3

pipeline-inspection gauges, 43–6

10"-PIG backplane, 46

56" PIG launching into a pipe, 45

crack detection PIG with 20" sensor carrier, 46

pit factor $\alpha$, 304

pixel-level data fusion, 98–100

planar location method, 193–4

plane pressure waves, 149

plane waves, 148–9

Poisson’s Ratio, 443, 470, 472, 485

polarisation phenomena, 335

polarisation plane, 492

polarisation resistance, 289–90

measurement, 293–4

methods of obtaining polarisation resistance, 294–7

electrochemical impedance spectroscopy, 297
Index

 electrochemical noise, 296
 galvanostatic and potentiostatic
 pulses, 295–6
 linear plot of polarisation curve,
 295
 potentiodynamic techniques,
 294–5
 response of potential to a
galvanostatic pulse, 296
 portable seismic pavement analyser,
 452
 potential attenuation method, 301–2
 potentiodynamic techniques, 294–5
 potentiometric titration method, 553
 potentiostatic pulses, 295–6
 pre-stressed concrete structures,
 215–41
 pressure waves, 492
 profile energy balancing, 134–5
 Profometer, 55
 Profometer 5+, 50
 pulse-phase thermography, 376, 381–3
 pulse velocity, 497
 PYTHAGORE, 579
 quantitative acoustic emission
 technique, 188
 quantitative infrared thermography,
 397
 R-wave, 419–20
 radar, 6–7, 17, 318
 other testing problems, 21–2
 reinforcement location and concrete
 cover measurement, 18–21
 radar data, 101–5
 radar data fusion, 100–1
 radar thermography, 5
 radar tomography
 acquisition procedures, 341–3
 amplitude calibration set-up, 343
 time calibration, 342
 artefacts, 349–50
 crosshole geometry, 350
 double crosshole geometry, 351
 full geometry, 352
 bridge column test, 360–1
 attenuation map from amplitude
tomography on selected column,
 362
 bridge built in 1994 on eight
 concrete columns, 360
 column selected for tomographic
 test, 361
 full tomographic dataset recorded
 on selected column, 361
 velocity map from traveltime
tomography on selected column,
 362
 data inversion, 346–9
 iterative approach to tomographic
 inversion, 347
 data pre-processing, 344–5
 GPR antenna real radiation
 patterns, 346
 pre-processing sequence for
tomographic inversion
 preparation, 344
 traveltime and amplitude picking,
 345
 equipment, 339–41
 antenna frequency selection
 according to TX–RX distance
 range, 340
 shielded antenna operating at very
 high frequencies, 340
 examples, 354–61
 Fresnel rays, 339
 fundamental equations, 337–8
 hints on advanced algorithms, 363–5
 diffraction tomography, 364–5
 frequency downshift method,
 363–4
 laboratory test, 354–8
 attenuation map from amplitude
tomography on concrete sample,
 358, 359
 concrete blocks reinforcement
 structure, 357
 full tomographic dataset recorded,
 356
 laboratory specimen, 354
tomographic acquisitions, 355
velocity map from traveltime
tomography on concrete sample,
357, 359
physical principles, 335–7
reinforced concrete structures
evaluation, 334–65
resolution, 338–9
results interpretation, 350–4
radargram, 328, 334
rail wheel inspection, 42–3
Ilsenburg, Germany, 42
Nishny Tagil, Russia, 43
railway safety, 36–43
Raman scattering, 69
Randles circuit, 291, 294
Randles method, 290
rate of disturbance, 254
ray-superposition theory, 447
Rayleigh scattering, 69
Rayleigh surface wave, 443
Rayleigh wave, 36, 492
Rayleigh wave velocity, 473
reference electrode (RE), 294
reference electrodes, 310
reflection mode, 126
reinforced concrete structures
acoustic emission and impact–echo
techniques evaluation, 543–72
applications for concrete
structures, 544–50
case studies, 551–69
future trends for on-site
application, 571–2
acoustic emission evaluation,
185–210
applications, 199–206
limitations and accuracy, 206–10
parametric and signal-based
analysis, 187–91
sensors and instruments, 191–3
source localisation, 193–7
source mechanisms and moment
tensor analysis, 197–9
active thermography evaluation,
370–99
applications, 386–96
data processing, 380–6
experimental equipment and
calibration, 376–80
future trends, 396
physical principle and theoretical
background, 372–4
state of the art, 375–6
automated non-destructive
evaluation, 30–60
case studies of successful
innovations, 36–46
construction engineering, 46–54
data acquisition, control and
evaluation, 34–6
innovation cycles, 31–4
multiple-sensor data acquisition,
54–9
capacimetry, 276–83
applications, 281–2
calibration, 280
data acquisition and
interpretation, 280–1
equipment, 279
limitations and reliability, 282–3
physical principle and theory,
276–9
combining results of non-destructive
evaluation techniques, 95–106
data fusion, 98–100
future trends, 105–6
NDT and MDT, 96–8
radar data and ultrasonic data,
101–5
radar data fusion, 100–1
corrosion rate and corrosion
potential measurement, 284–313
future trends, 310–12
how to interpret measurements,
303–5
methods, 293–302
monitoring systems, 310–11
practical application, 306–10
principles, 285–92
deficiencies found in concrete
infrastructure, 431
electrical resistivity, 243–70
  application, 255–64
  impedance spectroscopy, 268–70
  other developments, 264–8
  physical principles and theory, 244–55
ground penetrating radar, 317–32
  current and future trends, 327–31
  data display formats, 322–3
  data processing and interpretation, 323–5
  equipment, 325–6
  limitations and reliability, 326–7
  physical principles and theory, 318–22
  symbols and constants, 331–2
honeycombing and voids close to reinforced concrete beam surface, 390
laser-induced breakdown spectroscopy, 163–84
  cement, mortar and concrete characterisation, 167–71
  fundamentals and measurement, 164–7
  limitations and reliability, 183–4
  mobile set-up: on-site applications, 180–3
  specific elements detection, 173–80
life cycle of civil infrastructure system, 418
NDT methods for concrete structures, 419
non-destructive test program planning, 3–12
  overview, 6–7
  qualification/validation of methods, 8
  strategies for application of NDT methods, 4–6
nuclear magnetic resonance imaging, 403–15
  application possibilities, 409–13
  future trends, 414–15
  hardware, 407–9
physical background, 404–7
  reliability and limitations, 413–14
radar tomography evaluation, 334–65
  acquisition procedures, 341–3
  artefacts, 349–50
  data inversion, 346–9
  data pre-processing, 344–5
  equipment, 339–41
  examples, 354–61
  fundamental equations, 337–8
  hints on advanced algorithms, 363–5
  physical principles, 335–7
  resolution, 338–9
  results interpretation, 350–4
stress wave propagation evaluation, 417–36
  applications, 424–34
  future trends, 435–6
  methods, 419–24
structural health monitoring system, 63–91
  effective and innovative technologies, 74–87
  future trends, 90–1
  innovative monitoring methods, 67–73
  monitoring capabilities, 64–7
  reliability and standardisation, 87–90
ultrasonic techniques evaluation, 490–526
  applications and requirements, 499–500
  concrete elements imaging, 503–20
  future trends, 521–4
  transmission methods, 500–3
  ultrasonic wave propagation in concrete, 491–9
wireless monitoring, 111–22
  application, 119–22
  basic principles, 112–14
  future trends, 122
  monitoring task, 114–16
  system design and assembly, 116–19
  systems in operation, 119
relative humidity, 251
relative permittivity, 278
relaxation time, 407
residual field, 225
resistivity, 243, 288–9
values and corrosion risk, 289
resistivity electrodes, 310
resolution, 350–1
resonant transducers, 193
retaining walls, 534
co-ordinate system, 536
ground penetrating radar inspection, 533–42
data acquisition, 534–6
data processing, 536–7
problem description, 533
processed dataset, 537
raw dataset, 537
results, 537–41
guiding system with antenna box, 535
selection of Y slice in data cube, 538
time slice, t = 1.6 ns, level 2,
1500 MHz antenna, 540
time slice, t = 5.16 ns, level 2,
1500 MHz antenna, 540
top of apparatus, 535
Y slice
400 MHz antenna, 540
900 MHz antenna, 539
1500 MHz antenna, 539
recorded 3.9 m off the joint,
900 MHz antenna, 542
recorded on joint, 900 MHz antenna, 541
with prominent reflector, level 1,
1500 MHz antenna, 541
Réunion Internationale des Laboratoires et Experts des Matériaux, 399
Reynolds numbers, 448
RILEM see Réunion Internationale des Laboratoires et Experts des Matériaux
RILEM committee TC 207, 526
RILEM TC154EMC, 261, 263
robots, 31, 47–51
Runge–Kutta direct integration, 447
rupture amplitude, 227
rupture signals, 223, 226, 227
Rytov approximation, 364
S-wave, 419–20
see also shear waves
SAFT see synthetic aperture focusing technique
SASW see spectral analysis of surface waves method
saturation, 256–7
saturation rate, 251
scalar Green function, 150
scanning SIBIE method, 550
Scholte wave, 448
seismic refraction, 450
seismic wave, 125–7
SensCore system, 87
sensorised confinement method, 300–1
sensors, 64, 310–11
shear-horizontal plane waves, 149, 150
shear modulus, 443, 468
shear-vertical plane waves, 149, 150
shear waves, 443, 492
SHM see structural health monitoring
short-gauge-length sensors, 70, 76–83
SIBIE see stack imaging of spectral amplitudes based on impact–echo
SiGMA see simplified green’s functions for moment tensor analysis
signal-based acoustic emission techniques, 189–91
signal to noise ratio, 453
signal velocity, 321
simplified green’s functions for moment tensor analysis, 543
analysis with AIC picker, 546–8
crack models in SiGMA analysis, 547
shear failure mechanisms in reinforced concrete by AE-SiGMA, 559–61
AE events analysed by SiGMA, 560
observed AE generation behaviours, 560
reinforced concrete beam sketch and AE sensor array, 559
SiGMA analysis results by auto-picker, 561
single-probe resistivity measurement technique, 258–9
SIR GSSI 20, 18
slack-stack transform, 454
smart sensors, 113
SMARTape, 76
Sobel filter, 391
Sobel operator, 384
SOFO see surveillance d’ouvrages par fibres optiques
Sommerfeld integral, 345
sound velocity, 497
spatial resolution, 69
spectral analysis of surface waves method, 442, 450–3, 459–60
spherical waves, 150–1
spiking filter, 137
spin echo signal, 406
splitting test, 200
SQUID sensors, 234
SRI-CM-II, 552
stack imaging of spectral amplitudes based on impact–echo, 544, 548–50
conventional SIBIE and scanning procedure for zigzag crack, 551
cross-section divided into meshes and travel distance, 550
projectile and impacting system, 549
surface-crack depth evaluation, 567–9
conventional SIBIE images, 570
observed bending cracks, 569
scanning SIBIE images, 570
ungrouted tendon-duct identification, 561–7
frequency spectra detected, 564
impact test in concrete block with polyethylene and steel sheaths, 562
SIBIE results in metal sheath, 565
SIBIE results in polyethylene sheath, 566
sketch of imperfectly grouted sheath, 563
standard hydrogen electrode, 293
Stefan–Boltzmann law, 373
Stirling cooler, 375
stress wave, 443
stress wave propagation applications, 424–34
ageing/deteriorating concrete infrastructure, 433
concrete infrastructure ageing and deterioration, 428–34
freshly cast concrete, 427
hardened concrete, 429
initial time of set and strength development for newly cast concrete, 424–8
repaired concrete infrastructure, 434, 435
expansion over time owing to alkali–aggregate reaction and UTT measurements, 432
future trends, 435–6
impact-echo
setting time measurement, 428
strength development measurement, 430
methods, 419–24
attenuation coefficient, 421–3
attenuation coefficient and damage correlation, 423
attenuation coefficient in the time domain, 422
pulse velocity and structure condition correlation, 422
quality factor, 423–4
quality factor in the frequency domain, 424
velocity, 421
reinforced concrete structures evaluation, 417–36
ultrasonic through transmission set-up, 420
set-up for setting time measurement, 426
setting time measurement, 427
strength development measurement, 430

Structural faults and repair, 526
structural health monitoring, 112, 113, 114, 115, 122
definition, 64, 65
effective and innovative monitoring technologies, 74–87
fibre-optic point sensor, 83–7
fibre-optic RH probe, 77
fibre-optic sensors and laser vibrometer results, 80
long-gauge-length sensors, 74–6
precasted sensor-equipped piles, 83
quasi-distributed fibre-sensor, 75
sensors fixed on reinforcement cage, 82
short-gauge-length sensors, 76–83
silicon mould with EFPI sensor before casting, 78
innovative monitoring methods, 67–73
Fibre Bragg grating sensors, 70–2
Fibre Fabry-Perot interferometer sensors, 72–3
fibre-optic point sensors, 73
high-performance sensors for deformation measurement, 67–70
types of fibre-optic sensors, 68
reinforced concrete structures, 63–91
demands on monitoring systems, 64–7
future trends, 90–1
reliability and standardisation, 87–90

Structural materials technology, 526
structural noise, 494

sulphur, 175–6
surface wave
see also Rayleigh wave
detection using laser interferometer, 459
transmission coefficient and crack depth relationship, 457
surface wave attenuation analysis, 455–7
surface wave techniques
basic principles, 443–8
homogeneous elastic halfspace, 443–5
layered elastic media, 445–8
concrete structure evaluation, 441–60
equipment, 457–9
field application, 459–60
multiple time domain signals x-t waterfall plot presentation, 450
one possible MASW testing configuration, 453
real-valued phase and group velocity dispersion curves for single solid layer, 446
SASW test method testing configuration, 451
signal processing and data presentation, 449–57
multi-channel analysis of surface waves method, 453–5
spectral analysis of surface waves method, 450–3
surface wave attenuation analysis, 455–7
time domain analysis for homogeneous media, 449–50
transient wave field in solid half-space, 444
surveillance d’ouvrages par fibres optiques, 68–9
synthetic aperture focusing technique, 28, 99, 138, 154–5, 499
see also Fourier transform-SAFT box girder 3D SAFT reconstruction, 515
2D reconstruction of sample containing two side drilled holes, 507
elastic wave scattering, 520
foundation test slab cross-section, 513
partly grouted tendon duct examination, 516
pipe targets 3D reconstruction, 519
reinforcement bars 3D reconstruction, 514
tendon ducts in box girder, 517

TC RILEM154EMC, 243
TDS2014, 563
telegraph equation, 126
tendon ducts, 151
tendons, 222
theory of electromagnetism, 145
thermal conductivity, 372
thermal diffusivity, 372
thermal effusivity, 372
thermogram, 370
ThermoSense, 397
time-depth conversion, 141–2
time-of-flight technique, 455, 473
time signal clipping, 476
time synchronisation, 118
tomography, 334
transmission mode, 126
transversal wave, 492
travelling tomography, 337
taveltime calibration, 343
taveltime picking, 344
9T15S, 579
10T15S, 579
two-probe resistance measurement technique, 259
two-way-traveltime, 321
ultrasonic data, 101–5
ultrasonic echo, 17
ultrasonic pitch–catch transducer M2502, 508
ultrasonic pulse velocity, 420, 423

ultrasonic techniques, 185–6
A-scan measurement using handheld instrument, 509
application examples, 512–18
3D imaging of concrete regions, 517–18
detection and localisation of voids in tendon ducts, 515–17
reinforcement imaging, 513–14
tendon ducts localisation, 514–15
test site containing pipes, 518
thickness measurement, 512–13
applications and requirements, 499–500
automated scanning measurement systems, 510–12
BAM measurement equipment, 510
BAM scanners for post-tensioned concrete bridge ultrasonic testing, 511
MFPA measurement equipment, 510, 512
MFPA Weimer automated measurement system, 512
common inspection tasks at concrete element, 499
concepts and notions, 503–7
B-scan line measurements, 505
B-scan of sample containing two drilled holes, 506
C-scan planar measurements, 506
SAFT imaging, 506–7
single A-scan measurements, 503–5
concrete elements imaging, 503–20
3D co-ordinate system used for measurements and images, 503
future trends, 521–4
air-coupled imaging, 523
concrete column test specimen, 525
electronic scanning, 521–3
side drilled hole air-coupled scanning measurement, 524
tomographic imaging, 523–4
ultrasonic array with 48 transducers, 522
void and back wall displacement iso-surface view, 522
measurement equipment, 508–12
measurement instruments and transducers, 508–9
handheld instrument, 508–9
line scanner, 509
pulse–echo experiment setting, 504
reinforced concrete structures evaluation, 490–526
simulation, 518–20
transmission methods, 500–3
concrete material properties characterisation, 501–2
flaw detection in transmission, 502–3
measurement devices, 501
ultrasonic wave propagation in concrete, 491–9
acoustic impedance for pressure wave and reflection factors, 494
elastic wave, 491–4
elastic wave propagation in concrete, 494–9
influence of concrete properties on signal shape, 498
received signals, 495
testing parameters typical ranges, 496
transmitted signals, 497
ultrasonic testing, 35
ultrasonic through transmission set-up, 420
set-up for setting time measurement, 426
setting time measurement, 427
strength development measurement, 430
ultrasonic wave, 125–7
ultrasound testing, 185
under-floor inspection unit (UFPE), 43
CAD drawing, 41
Frankfurt, 42
UT1000, 552
validation, 8
variable damping factor, 348
VDI/VDE/GESA 2635, 90
VDI/VDE guideline 2660, 90
water/cement ratio, 246
wave equation, 148
wave field inversion, 154–8, 158–9
wave-monitor, 547
wave-shaping filter, 136
Weiner filter, 136
Wenner probes, 259
wheel set inspection, 40–1
wireless monitoring
basic principles, 112–14
flowchart, 115
multihop network with sensor clustering, 112
robust wireless sensor node, 114
reinforced concrete structures, 111–22
application, 119–22
future trends, 122
monitoring system design and assembly, 116–19
monitoring task, 114–16
systems in operation, 119
wireless sensors, 111, 112, 113, 122
working electrode, 294
yoke magnet, 219
Young’s modulus, 443
zero offsets, 127
ZfpBau-Kompendium, 526
zone location method, 193–4