Recommended Practice
Concrete Durability Series

Z7/07
Performance Tests to Assess Concrete Durability
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For information regarding permission, write to:
The Chief Executive Officer
Concrete Institute of Australia
PO Box 1227
North Sydney NSW 2059 Australia
Email: admin@concreteinstitute.com.au

Concrete Institute of Australia
National Office
Suite 401, Level 4
53 Walker Street
North Sydney NSW 2060 Australia
PO Box 1227
North Sydney NSW 2059 Australia

PHONE: +61 2 9955 1744
FACSIMILE: +61 2 9966 1871
EMAIL: admin@concreteinstitute.com.au
WEBSITE: www.concreteinstitute.com.au

For contact information on Institute Branches and networks in Queensland, New South Wales, Victoria, Tasmania, South Australia and Western Australia visit the web site at:

www.concreteinstitute.com.au

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**The Task Group** that developed this Recommended Practice was:

- W Green  Vinsi Partners (Chair)
- A Peek  GHD
- S Freitag  Opus
- M Dacre  AECOM
- R Barnes  PCTE

**Durability Committee** active contributors were:

- F Papworth  BCRC
- R Paull  GHD
- D Baweja  EMS

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Performance Tests to Assess Concrete Durability

The Durability Series is a set of Concrete Institute of Australia recommended practices that provide deemed to satisfy requirements applicable to all concrete structure types based on standard input parameters for design life, reliability and exposure. The series includes details on project planning and implementation which if followed will increase the likelihood that the specification, design detailing and construction will be optimal to achieving the developer and community expectations regarding the long term performance of concrete structures. Also included are methods for modelling degradation over time and for crack control design. Thus the series provides what is described as a unified durability design process.

Prior to around 1970 concrete was generally regarded by asset owners, designers and contractors as a reliable construction material that provided long term durability with relatively little maintenance. Subsequently, premature deterioration of concrete structures, arising from changing cement characteristics, quality management and other factors, damaged this reputation. Because concrete is a complex material, research into the cause of problems and development of appropriate new rules and operational methods has taken a long time. The durability series provides recommendations that if followed will largely eradicate premature deterioration.

While research into concrete durability continues, the knowledge on exposure significance, deterioration processes, materials properties and workmanship implications has developed significantly over the last 30 years. In addition, new cementitious materials, admixtures and additives have been widely introduced. Much more advanced concretes are now available. New durability design practices have also been developed, including durability modelling methods, and new methods of construction have been introduced. However, to an extent at least, these developments are not fully reflected in a clear and unified manner through the Australian Standards dealing with concrete durability requirements (e.g. modelling methods, use of fly ash, slag and silica fume, use of galvanised and stainless steel reinforcement). The durability series provides recommendations on durability design using a wider range of concretes and reinforcements, and details how to implement new durability design methods.

Durability requirements in Australian Standards are fragmented through different standards and their commentaries dealing with concrete durability requirements for different structure types (e.g. AS 2159, AS 3735, AS 4997 and AS 5100.5). Perceived conflicts between these documents (e.g. higher covers in AS 3735 than AS 3600 for the same life and exposure) might sometimes be explained by the different owner requirements (e.g. reliability required) but reasons for the differences are not given and the associated assessment methods not clearly stated. To some extent the concrete industries energy for contributing to development of durability codes is diluted through maintenance of the multitude of codes that cover the same topic in variable ways.

For many, concrete elements in mild exposures incorporating the recent durability related developments into a unified durability design process for all structure types may make little difference to their durability design because existing codes deemed to satisfy provisions often provide adequate performance. However, for elements in more severe exposures, guidelines that comprehensively detail how to assess owners’ needs, environmental exposures and materials requirements; how to specify performance or prescriptive materials properties; and how to ensure construction is appropriate to the design will provide structures that meet their durability requirements more consistently. The durability series provides the required guidelines.

The Concrete Institute of Australia first introduced Z7 “Durable Concrete Structures” in 1990 as an initial response to concerns about the poor durability performance of some concrete structures. This was revised in a second edition in 2001, which gave some excellent information on how to achieve durability but did not set out to provide a set of unified design guidelines as an alternative to the approach in the Australian Standards noted above.

The Concrete Institute of Australia’s Durability Committee was formed in late 2008 to review Z7. In view of the committee’s perceived need for a broader review of durability requirements it managed workshops around Australia in mid-2009 to review issues with concrete durability practices and standards.
in Australia. The outcome from these workshops, and other feedback from Concrete Institute of Australia members at the Concrete Institute of Australia National Conference in 2009, was that comprehensive and unified durability guidance was required. In response, the Durability Committee established Task Groups to produce a series of recommended practices as a major revision to Z7 that would form a durability series. The series comprises:

- Z7/01 Durability – Planning
- Z7/02 Durability – Exposure Classes
- Z7/03 Durability – Deemed to Comply Requirements
- Z7/04 Durability – Good Practice through Design, Concrete Supply and Construction
- Z7/05 Durability – Modelling
- Z7/06 Durability – Cracks and Crack Control
- Z7/07 Durability – Testing

The durability that the owner and community require from structures will only be obtained if specific consideration is given to how durability requirements impact on construction cost, inspections needs, maintenance requirements, aesthetics, and operational and community costs that unplanned maintenance brings. While strong emphasis is placed on achieving the design life, durability must be met long into the future, possibly well past the initial design life. The durability series will go a long way to providing the necessary tools for design and construction of durable structures based on the latest understanding of exposure, materials and deterioration process.

Frank Papworth
Durability Committee Chairman
All engineering materials, including concrete, deteriorate (corrode) with time, at rates dependent upon the type of material (concrete), the nature of the environment and the deterioration mechanisms involved.

In engineering terms, the objective is to select the most cost effective combination of materials (concretes, reinforcements, coatings, etc) to achieve the required design life. In doing so, it is critical to realise that the nature and rate of deterioration of materials is a function of their environment.

Accordingly, the environment is a “load” on a material (concrete) as a force is a “load” on a structural component. It is the combination of the structural (or non-structural) load and environment load in synergy which determines the performance of the concrete component.

The protective measures to be adopted for (concrete) structures/elements within a project depend on the risk of deterioration over the design life, the cost of preventative measures, the feasibility and cost of remedial actions and ongoing preventative maintenance. It is typical that these need to be balanced to arrive at the best whole-of-life cost and optimised value for money.

The Concrete Institute of Australia Durability Series provides the tools for managing durability through design, construction and maintenance. Z7/07 provides guidance on performance tests for durability design and implementation.

Test methods are available to assess various aspects of durability performance through a concrete structure’s life cycle including:

- Mix acceptance tests (including tests to validate values used in modelling).
- Tests for quality assurance.
- Tests where placed concrete is suspect.
- Tests for condition monitoring.

A wide range of tests designed to demonstrate the potential durability performance of concrete have been introduced over the years. This has caused some uncertainty for:

- Asset owners: To understand what methods are available, the appropriateness of those methods to the structures’ exposure, environment and life cycle, and the most cost effective testing regimes to achieve the required outcomes and level of certainty that they are looking to achieve.

- Designers: To know which tests are the most appropriate to specify and how much test data is required to ensure that the level of statistical confidence from the test results underpinning the design is appropriate.

- Contractors and material suppliers: To understand and have confidence in the consistency, repeatability and validity of trial data and quality control performance testing they are required to undertake for compliance with the project specification.

- Suppliers of laboratory testing services: To maintain and calibrate equipment, train staff, maintain third party accreditation for the tests (e.g. perform the tests to sufficient frequency, provide regular proficiency training of staff and keep detailed records) and competitively price test methods despite some being not often specified.

Often several test methods supply similar information. Combinations of tests may be necessary. The limitations and advantages of the methods are reviewed, and recommendations provided on which test(s) is the most suitable for project specifications.

The Durability Planning Recommended Practice Z7/01 provides guidance on performance tests for durability during design, construction and operation of a structure.

Design phase durability testing requirements are recommended to be clearly specified for four stages:

- Mix trials to confirm the mix is suitable.
- Quality assurance tests as construction proceeds.
- Tests at the end of the defects liability period to create a list of items for repair.
- Tests during the design and service life including monitoring.

Construction phase materials testing and selection requirements recommended are:

- Materials testing and selection must be completed in accordance with the project specifications prior to use in the works.
Additional testing is required prior to a change in supply of materials or a new source of materials.

- Verification of concrete mix designs to meet project specification durability requirements can take considerable time, and unscheduled changes in concrete supply during construction may result in program delays. Durability testing of concrete such as chloride diffusion, water permeability, drying shrinkage, etc, may have a long test period (e.g. up to 3 months).

- Variability of durability tests must be taken into account by the durability consultant, with specification test criteria allowing for alternative solutions to achieve the required durability if the test results do not achieve the specified values. This can be achieved by conservative durability design and/or provision for use of additional measures such as protective coatings or special additives or other measures.

Operation and maintenance phase monitoring and testing recommended are:

- **Practical completion inspection:** Prior to a structure going into service it is important to determine if any defects need to be contractor repaired and to document the initial structure characteristics and condition for future reference and comparison.

- **Periodic in-service visual inspection:** A reactive approach to on-going maintenance be limited to visual inspections only and these may be performed on a regular basis or ad-hoc. This may be adequate provided no major defects are found and may be sufficient to prevent minor defects from becoming major ones. If appropriate, follow up repairs are performed as required. This approach may be suitable for minor structures and/or structures with a short design life.

- **In-service condition monitoring and testing:** Proactive maintenance will involve early intervention to prevent or delay the onset of corrosion initiation. This will require regular inspections in conjunction with additional activities such as structural monitoring and non-destructive testing, as required. If significant repairs/strengthening have been carried out, then a post-intervention inspection should be carried out along similar lines to a new structure first inspection mentioned above.

This document is intended to inform all parties involved in design, construction and maintenance about the benefits of durability performance testing and how as part of a durability planning and implementation process will lead to an increased likelihood of achievement of design life of structures and buildings.

Warren Green
Z7/07 Task Group Chairman
Contents

Terminology

1 Introduction
   1.1 Scope of Z7/07
   1.2 Selection of Test Methods and Interpretation of Results
   1.3 Laboratory Accreditation

2 Summary Guide to Test Methods

3 Sampling and Sample Preparation
   3.1 General
   3.2 Standard Methods of Sample Preparation from Fresh Concrete
      3.2.1 Specification of testing requirements for fresh concrete
      3.2.2 Effect of sample concrete compaction on test results
      3.2.3 Curing of concrete samples
   3.3 Sampling Methods for In-situ Concrete
      3.3.1 Background
      3.3.2 Wet diamond coring
      3.3.3 Recommended practice for wet diamond coring
      3.3.4 Grinding core cuts
      3.3.5 Drilled dust samples

4 Concrete Mix Acceptance Tests for Durability Design and Construction Compliance
   4.1 General
   4.2 Chloride Penetration
      4.2.1 General
      4.2.2 Chloride diffusion tests
      4.2.3 Recommendations
      4.2.4 Atmospheric chloride content environmental assessment
   4.3 Carbonation Rate
      4.3.1 Introduction
      4.3.2 Recommendations
   4.4 Water Absorption and Sorptivity
      4.4.1 Introduction
      4.4.2 AS 1012.21 Volume of permeable voids
      4.4.3 Taywood/GHD/SGS sorptivity test
      4.4.4 ASTM C1585 sorptivity test
      4.4.5 RMS T362 sorptivity test
      4.4.6 Recommendations
   4.5 Water Permeability
      4.5.1 Introduction
      4.5.2 Recommendations
   4.6 CIA Z7/07 Semi Adiabatic Tests to predict Concrete Adiabatic Temperature Rise
      4.6.1 Semi-adiabatic concrete temperature monitoring methods
      4.6.2 Predicted concrete adiabatic temperature
   4.7 Alkali-Aggregate Reaction
   4.8 Delayed Ettringite Formation
   4.9 Sulfate Resistance
   4.10 Abrasion Resistance
      4.10.1 Introduction
      4.10.2 AS 4456.9 Abrasion test
      4.10.3 BS EN 13892-4 Abrasion test
      4.10.4 BS EN 13892-3 Abrasion test
      4.10.5 ASTM C779 Abrasion methods
      4.10.6 Recommendations
   4.11 Bleed Tests
   4.12 Between-Batch Variability

5 Tests for Quality Assurance During Construction
   5.1 Introduction
   5.2 Compressive Strength
   5.3 Cover
   5.4 Maturity/Temperature Matched Curing
   5.5 Cracks
   5.6 In-situ Temperature and Strain Measurements
   5.7 Electrical Resistivity
   5.8 Cross Hole Sonic Logging

6 Tests where Placed Concrete is Suspect
   6.1 Introduction
   6.2 Reinforcement Location and Cover Depth
      6.2.1 Instrument types and applicable standards
      6.2.2 Magnetic reluctance covermeters
      6.2.3 Pulsed eddy current covermeters
      6.2.4 Covermeter limitations
      6.2.5 Ground penetrating radar
      6.2.6 Ultrasonic pulse echo
   6.3 Compressive Strength
      6.3.1 Concrete core sample testing
      6.3.2 Windsor Probe and Capo tests
      6.3.3 Rebound hammer
      6.3.4 Ultrasonic pulse velocity
   6.4 Detecting Defects within the Concrete
      6.4.1 Ultrasonic pulse velocity
      6.4.2 Ultrasonic pulse echo
      6.4.3 Impact echo
      6.4.4 Impulse response
      6.4.5 Ground penetrating radar
   6.5 Assessment of Concrete Surface Quality
      6.5.1 Initial surface water absorption (ISAT)
      6.5.2 Torrent air permeability
   6.6 Mix Composition
      6.6.1 Cement (binder) content and composition
      6.6.2 Air content
      6.6.3 Water to cement ratio
6.7 SCM Content and Composition
6.8 Chloride and Sulfate Ion Content
6.9 Alkali Content

7. Condition Monitoring
7.1 Introduction
7.2 Test Locations
  7.2.1 Structure configuration
  7.2.2 Element configuration
  7.2.3 Construction influences
  7.2.4 Exposure
  7.2.5 Existing condition
  7.2.6 Type of monitoring
7.3 Visual Inspection
7.4 Intermittent Site Test
  7.4.1 Electrode (half cell) potential mapping
  7.4.2 Carbonation depth
  7.4.3 Chloride profile
  7.4.4 Resistivity
  7.4.5 Polarisation resistance
  7.4.6 Petrographic examination
  7.4.7 Microbial analysis
7.5 Permanent Surface Mounted and Embedded Monitoring Techniques
  7.5.1 Corrosion initiation
  7.5.2 Corrosion rate
  7.5.3 Strain, vibration and deflection

8. References

Tables
Table 2.1 Testing summary guide and report location of detailed description
Table 4.1 Recommended mix acceptance tests
Table 4.2 Outline of test methods used to measure chloride diffusion coefficient set up and analysis
Table 4.3 Common water absorption and sorptivity tests
Table 4.4 AVPV performance assessment criteria (VicRoads Specification 610 on structural concrete [Reference 13])
Table 4.5 RMS T362 water sorptivity test criteria
Table 4.6 Common concrete water permeability test methods
Table 4.7 Common concrete alkali-aggregate reaction test methods
Table 4.8 Expansion limits for ASTM C1012-13 test for fly ash blended cement
Table 4.9 Cement sulfate resistance tests
Table 4.10 Abrasion tests
Table 4.11 Criteria for acceptance of abrasion resistance of segmental pavers using the AS 4456.9 test
Table 4.12 BS8204-2:2003 and concrete society TR34 chaplin abrasion tests criteria
Table 5.1 Compressive strength listed in clause 4.5 of AS 5100.4
Table 5.2 Types of resistivity test
Table 5.3 Set of typical resistivity results for concrete cylinders
Table 5.4 Cross hole sonic logging tests
Table 6.1 Recommended tests for assessment of as placed concrete
Table 7.1 Guide for use of electrode potential mapping for condition monitoring
Table 7.2 Corrosion current criteria for surface applied corrosion rate measurements [Reference 55]
Table 7.3 Types of strain and displacement measurement

Figures
Figure 1 Phases in the life of a structure [Reference 2] (Terminology)
Figure 3.1 Wet diamond drilling to obtain concrete cores
Figure 3.2 Dry grinding core cuts to obtain concrete samples
Figure 3.3 Method of taking drilling dust samples
Figure 4.1 Typical carbonation tank where CO₂ level is controlled by injection of an air/CO₂ mixture and tank air/CO₂ mixture is re-circulated through a temperature and humidity controller
Figure 4.2 Taywood/GHD/SGS sorptivity test. A sample is stood in contact with water and the height rise and weight gain measured with time.
Figure 4.3 Pressure permeability test. One face of samples is kept under constant pressure by the compressed air bottle. The other side can be left open to witness time to water penetration or closed so that water flow rate can be measured.
Figure 4.4 Typical “hot box arrangement” and monitoring option 1 by one concrete position method [Reference 78]
Figure 4.5 Design of semi-adiabatic temperature monitoring option 2 by multiple concrete positions method [Reference 77]
Figure 4.6 Adiabatic temperature rise development for S50 concrete
Figure 4.7 Abrasion damage examples
Figure 4.8 Abrasion resistance vs compressive strength using AS 4456.9 test for off form finish [Reference 144]
Figure 4.9 Chaplin abrasion tests
Figure 5.1  Different types of crack measuring equipment
Figure 5.2  Schematic of 4 probe resistivity equipment
Figure 6.1  Schematic of a hand held GPR unit scanning rebar and an example of the on-screen real time image
Figure 6.2  Cover distribution from a series of GPR scans
Figure 6.3  Screen shot of the segment cross section generated by UPE with a matching sketch of reinforcement layout
Figure 6.4  Field measurements of water absorption
Figure 6.5  Torrent air permeability testing equipment
Figure 7.1  Measurement of electrode potentials
Figure 7.2  Establishing electrode potential criteria for corrosion assessment [Reference 73]
Definitions of terminology for many aspects of durability through a structure's service life are given in Z7/01. Definitions from Z7/01 that are relevant to testing are listed first, and additional definitions specific to this document listed second.

**Terminology extracted from Z7/01**

**Condition assessment:** A process of reviewing information gathered about the current condition of a structure or its components, its service environment and general circumstances, whereby its adequacy for future service may be established against specified performance requirements for a defined set of loadings and/or environmental circumstances.

**Condition control:** The overall through-life process for conserving the condition of a structure involving condition survey, condition assessment, condition evaluation, decision-making and the execution of any necessary interventions; performed as a part of the conservation process.

**Condition evaluation:** Similar to condition assessment, but may be applied more specifically for comparing the present condition rating with a particular criterion, such as a specified loading. Condition evaluation generally considers the requirement for any later intervention which may be needed to meet the performance requirements specified.

**Condition survey:** A process whereby information is acquired relating to the current condition of the structure with regard to its appearance, functionality and/or ability to meet specified performance requirements with the aim of recognising important limitations, defects and deterioration. A wide range of parameters may be included within a condition survey with data being obtained by activities such as visual inspection and various forms of testing. Condition survey would also seek to gain an understanding of the (previous) circumstances which led to the development of that state, together with the associated mechanisms causing damage or deterioration.

**Corrosion:** The destruction or deterioration of a material through reaction with its environment. The term “corrosion” can refer both to a process or the damage caused by such a process.

**Design service life or design life (specified):** The term “design life” is often used to convey the same intent as “design service life” and both terms are acceptable to convey the same intent. The period in which the required performance shall be achieved used in the design of new structures (see Figure 1).

The specified (design) service life is related to the

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**Figure 1: Phases in the life of a structure [Reference 2]**
required service life, as given by the stakeholders (i.e. owners, users, contractors, society) and to the other implications of service criteria agreement (e.g. with regard to structural analysis, maintenance and quality management). In this document the international definition is adopted. Refer Z7/01 for full definition.

**Deterioration:** Worsening of condition with time, or a progressive reduction in the ability of a structure or its components to perform according to their intended specifications.

**Deterioration model:** Mathematical model that describes a structure's performance as a function of time, taking deterioration into account.

**Durability:** The capability of structures, products or materials of continuing to be useful after an extended period of time and usage. In the context of performance-based design of structures, durability refers to the fulfilment of the performance requirements within the framework of the planned use and the foreseeable actions, without unforeseen expenditure on maintenance and repair. In this document the international definition is adopted. Refer Z7/01 for full definition.

**Durability consultant:** Person or group who completes the durability assessment and is the author of the durability assessment report and durability checklists. Intent is a person or group who can apply materials deterioration knowledge to construction materials and construction processes, additional to more common structural, civil, geotechnical and other engineering knowledge of design, construction and maintenance. Maybe an in-house employee of the design team, or an independent consultant engaged for the purpose. Intended to have a close working relationship with the asset owner, design team and construction team to ensure durability is provided to achieve the asset owner required service life. Practical experience is essential to ensure the durability assessment report and durability checklists do not become a research exercise. Contractor reviews are included to achieve a buildable final design for the asset owner service life. The durability consultant may be a person with relevant technical qualifications other than a qualified engineer (e.g. materials scientists), with the asset owner client (or authorised representative) responsible to approve the durability consultant for a project.

**Environment/exposure influences:** Physical, chemical and biological actions resulting from the atmospheric conditions or characteristics of the surroundings to the structure including macro and micro influences.

**Ingress:** The entry of substances (e.g. gas, liquid, ions) into structural and/or non-structural components of a structure. Often the term “ingress” is associated with the entry of substances that cause deterioration (e.g. chlorides into reinforced or prestressed concrete, water, sulphates and carbon-dioxide (CO₂) into concretes, etc).

**Inspection:** A primarily visual examination, often at close range, of a structure or its components with the objective of gathering information about their form, current condition, service environment and general circumstances.

**Investigation:** The process of inquiry into the cause or mechanism associated with some form of deterioration or degradation of the structure, and the evaluation of its significance in terms of its current and future performance. The term may also be employed during the assessment of defects and deficiencies. The process of inquiry might employ sampling, testing and various other means of gathering information about the structure, as well as theoretical studies to evaluate the importance of the findings in terms of the performance of the structure.

**Monitoring:** To keep watch over, recording progress and changes in materials properties or condition and/or structural properties or responses with time; possibly also controlling the functioning or working of an associated entity or process (e.g. warning alarms based upon parameters such as applied load, element deflection or some aspect of structural response).

**Monitoring plan:** Instructions for the monitoring specific to the structure, including all elements of the structure.

**Prolonged service life:** The period extended after the design service life by the owners with relevant maintenance completed (see Figure 1).

**Risk:** The combination of the likelihood of occurrence of a particular hazard and its consequences [Reference 2].

**Service life (operational):** The period in which the required performance of a structure or structural element is achieved, when it is used for its intended purpose and under the expected conditions of use. It comprises design service life and prolonged service lives (see Figure 1 and Design service life).
Service life (required): The stakeholders, i.e. owners, users, contractors, society) stated period in which the required performance shall be achieved after construction (see Figure 1 and Design service life).

Service life (residual): The remaining period in which the required performance shall be achieved from current time until the design service life is achieved (see Figure 1 and Design service life).

Survey: The process, often involving visual examination or utilising various forms of sampling and testing, aiming at collecting information about the shape and current condition of a structure or its components. Refer Z7/01 for full definition.

Terminology specific to Z7/07

Abrasion: The ability of the concrete surface to resist being worn away by rubbing and friction.

Absorption: A measure of the complete or partial filling of pores with a liquid (typically water) by any mechanism.

Diffusion: Movement of ions (e.g. chloride ions) due to a difference in the concentration gradient.

Durability performance test: A performance test method the results of which can be used to assess durability.

Penetrability: A term used to indicate general rate of ingress due to any mechanism (e.g. absorption, sorptivity, permeation, diffusion). It has no units and is a subjective term when used to convey a rate.

Permeability: A measure of flow of a liquid (typically water) or gas under pressure through a material. Permeability can be calculated from measurements of penetration depth at a certain time or flow rate for a known thickness.

Pressure gradient: The differential water pressure in metres divided by the concrete element thickness in metres that the pressure differential acts on

Sample: The pieces or “specimens” of concrete representing the concrete to be tested. A sample may comprise one specimen or several replicate specimens.

Sorptivity: Rate of absorption of a liquid (typically water) by capillary suction.

Specimen: The individual piece of concrete on which a test is performed. Several replicate specimens may be needed to comprise a representative sample of the concrete of interest.

Supplementary cementitious material (SCM): Fly ash, pulverised fuel ash (PFA); ground granulated blast furnace slag (GBFS); slag, ground granulated iron blast furnace slag (GGBFS); silica fume, condensed silica fume (CSF), microsilica, amorphous silica; or pozzolans.

Testing: The process of scientifically measuring a property of a material, element or structure to obtain information about its composition, condition or performance. Tests may be carried out directly on the structure itself (“in-situ testing”) or on test specimens made in the laboratory or taken from the structure/site. Testing may be classified as destructive or non-destructive. A “destructive” test is one that damages the specimen or structure, such that the specimen is destroyed or the structure needs repair. A test performed on a specimen taken from the structure is considered a destructive test because the sampling leaves damage that must be repaired. In contrast, a “non-destructive” test does not damage the structure or test specimen in any way. Destructive testing must not reduce the structural capacity of the element unless the element or structure is to be demolished or strengthened immediately following the sampling or testing.
1 Introduction

1.1 SCOPE OF Z7/07

The aims of this document are to consider the current design guides, codes and Australian practice to provide advice on preferred methods of testing concrete to demonstrate the durability performance of a structure to meet design life intent as set out in the project specification. There are a wide range of tests discussed in the various sections of this document and therefore as a quick reference all test methods considered are listed in Section 2.

In view of the above, this Recommended Practice provides discussion and recommendations on the appropriateness of test methods that can be used to assess various aspects of durability performance through a structure’s life cycle (see Terminology for design and service lives definitions and as shown in Figure 1). Before any testing can be undertaken test locations must be selected and in many cases samples taken. Section 3 provides recommendations on selecting test locations and methods of sampling. Testing is then discussed under the following main headings:

- **Mix Acceptance Tests**
  Testing during the design phase is conducted to support the design process, in particular to demonstrate that the combination of design details and practical concrete supply to the project will yield the desired performance. The test methods are primarily required to provide measurements that are relevant to the potential durability of structural elements in specific exposures, and these values are used in predictive modelling processes to inform design decisions or to show a certain performance level has been achieved. Prior to commencement of construction, trial mixes are generally required to produce samples for testing to demonstrate that the proposed mix design, using materials available to the project site, is suitable for the design life, exposure conditions and method of application. In Section 4 the principal concrete mix acceptance tests for durability design and construction are discussed.

- **Tests for Quality Assurance**
  Quality assurance/control tests are primarily required to demonstrate consistency of supply in conformance with the works specification and the properties established by acceptance trial mixes. Timeliness of results is therefore a highly important criterion, generally superseding the requirement to produce values for modelling purposes. Although strength tests of concrete are routine, durability tests are not, and the type and frequency of tests must be specified in the contract documents.
  In Section 5 tests for quality assurance of concrete durability are discussed.

- **Tests where Placed Concrete is Suspect**
  The concrete performance in the field will differ to that in the trial mixes. To assess the extent of this, performance tests on the as-placed concrete might be required. This is often by non-destructive testing, or testing cores for strength or other properties. Such testing may be undertaken where there is doubt that the construction method or materials will have provided the required performance (e.g. routine testing has revealed an anomaly or trend of concern), or to verify the efficacy of the routine quality control testing program.
  Section 6 reviews field testing that will provide information on the as-placed concrete durability.

- **Tests for Condition Monitoring**
  During the life of a structure it is necessary to check that the concrete is responding to the environment as expected. It is also often necessary to assess the residual life of the structure in order to determine what intervention is necessary (and when best performed) to reach the design life, or to assess the feasibility of extended operational service life. Some specifications now require that the maintenance manual for the structure be prepared at the time of design and that this include the type, frequency, extent and criteria for in-service testing.
  Section 7 deals with tests that can help establish the condition of a structure.
  In order to evaluate durability requirements a reasonable assessment of the exposure conditions is required. Tests for exposure assessment are not
included in this document, and it is recommended that this be developed in a future revision.

For all test methods the limitations and advantages are given and where possible the variance of the test results stated.

Applications for the tests are provided and where appropriate interpretation and use of results is included.

1.2 SELECTION OF TEST METHODS AND INTERPRETATION OF RESULTS

Laboratory test methods used to inform durability design often do not accurately reflect processes that occur in the field. Whether the results are used to compare different materials or to predict their performance, it’s important to account for the limitations of the method.

Therefore, it is strongly recommended that the selection of test methods, design of test programs, and interpretation of the results be performed by experienced professionals who have appropriate knowledge of both the testing processes and the deterioration processes that occur in structures exposed to various environments. This will ensure that test results, and the outcomes of predictive modelling based on the results, are interpreted appropriately.

It is also noted that the use of 2 or 3 different particular test methods can provide more complete data about the performance at various ages of a proposed concrete mix under different durability regimes.

1.3 LABORATORY ACCREDITATION

Testing by a NATA accredited laboratory is frequently specified. NATA accreditation applies to specific tests and a laboratory’s accreditation for one test does not necessarily mean it will be qualified to undertake a test for which it is not accredited. Many of the tests referenced in this document are not common, and there might not be a NATA accredited laboratory available for them. The designer should search NATA’s website for facilities that are accredited to undertake the test to be specified. Where the designer determines that the test can be undertaken by an accredited facility, they can specify testing by a NATA accredited facility. Where the designer determines that there is not a suitably accredited facility for the test, they can require that the test is undertaken by a facility to be approved by the designer. Approval can be based on specific experience with the test required and/or overall testing experience of a similar nature. Laboratories accredited by an organisation belonging to ILAC (the International Laboratory Accreditation Cooperation) for similar tests may also provide a suitable service.
A summary guide to the test methods and location in this document is given in Table 2.1. The more detailed description in this document must be considered for the evaluation and selection of appropriate methods for a particular structure, exposure, condition, etc. The comments below on reproducibility are indicative interpretations from nominal identical samples in one test location.

Table 2.1: Testing summary guide and report location of detailed description

<table>
<thead>
<tr>
<th>Test Use of Test</th>
<th>Practical Application</th>
<th>Reproducibility</th>
<th>Experience Required</th>
<th>Test Time</th>
<th>Report Section</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>SAMPLING METHODS FOR IN-SITU CONCRETE</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wet diamond coring</td>
<td>Common</td>
<td>Easy</td>
<td>Good</td>
<td>Limited</td>
<td>N/A</td>
</tr>
<tr>
<td>Grinding core cuts</td>
<td>Occasional, for dry concrete sampling</td>
<td>Easy</td>
<td>Good</td>
<td>Limited</td>
<td>N/A</td>
</tr>
<tr>
<td>Drilled dust samples</td>
<td>Less common than coring</td>
<td>Easy</td>
<td>Good</td>
<td>Limited</td>
<td>No</td>
</tr>
<tr>
<td><strong>CONCRETE MIX ACCEPTANCE TESTS FOR DURABILITY DESIGN AND CONSTRUCTION COMPLIANCE</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Classical membrane cell</td>
<td>Not used, obsolete by other tests developed</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Nordtest NT Build 355</td>
<td>Not common</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Nordtest NT Build 443</td>
<td>Prequalification of concrete mixes</td>
<td>Moderate</td>
<td>Good</td>
<td>Fairly extensive</td>
<td>56 days</td>
</tr>
<tr>
<td>ASTM C1556-11 (derived from NT Build 443)</td>
<td>Prequalification of concrete mixes</td>
<td>Moderate</td>
<td>Good</td>
<td>Fairly extensive</td>
<td>56 days</td>
</tr>
<tr>
<td>AASHTO T259</td>
<td>Not common, prequalification of concrete mixes</td>
<td>Moderate</td>
<td>Fair</td>
<td>Fairly extensive</td>
<td>90 days</td>
</tr>
<tr>
<td>ASTM C1543-10a (derived from AASHTO T259)</td>
<td>Not common, prequalification of concrete mixes</td>
<td>Moderate</td>
<td>Fair</td>
<td>Fairly extensive</td>
<td>90 days</td>
</tr>
<tr>
<td>Nordtest NT Build 492</td>
<td>Used to derive “diffusion coefficients” when calibrated to individual mix</td>
<td>Moderate</td>
<td>Good</td>
<td>Fairly extensive</td>
<td>6-96 hours</td>
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</table>
### Performance Tests to Assess Concrete Durability

<table>
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<tr>
<th>Test</th>
<th>Use of Test</th>
<th>Practical Application</th>
<th>Reproducibility</th>
<th>Experience Required</th>
<th>Test Time</th>
<th>Report Section</th>
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<tbody>
<tr>
<td>Carbonation Rate</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>4.3</td>
</tr>
<tr>
<td>fib Bulletin 34 carbonation rate</td>
<td>Not common, typically for research and development</td>
<td>Difficult</td>
<td>Fair</td>
<td>Fairly extensive</td>
<td>28 days</td>
<td>4.3.1</td>
</tr>
<tr>
<td>Modified fib 34 accelerated carbonation test</td>
<td>fib 34 test modified to be consistent with standard Australian laboratory practice</td>
<td>Difficult</td>
<td>Fair</td>
<td>Fairly extensive</td>
<td>28 days</td>
<td>4.3.2</td>
</tr>
<tr>
<td>Water Absorption Tests</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td>4.4</td>
</tr>
<tr>
<td>AS 1012.21</td>
<td>Fairly common</td>
<td>Easy</td>
<td>Fair</td>
<td>Limited</td>
<td>2 days</td>
<td>4.4.2</td>
</tr>
<tr>
<td>ASTM C642-06</td>
<td>Not common</td>
<td>Easy</td>
<td>Fair</td>
<td>Limited</td>
<td>4 days</td>
<td>4.4.1</td>
</tr>
<tr>
<td>BS 1881 Part 122</td>
<td>Not common</td>
<td>Easy</td>
<td>Good</td>
<td>Limited</td>
<td>4 days</td>
<td>4.4.1</td>
</tr>
<tr>
<td>Water Sorptivity Tests</td>
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<td>4.4</td>
</tr>
<tr>
<td>Taywood/GHD/SGS sorptivity</td>
<td>Fairly common</td>
<td>Easy</td>
<td>Good</td>
<td>Limited</td>
<td>2-3 days</td>
<td>4.4.3</td>
</tr>
<tr>
<td>ASTM C1585-07</td>
<td>Becoming more common</td>
<td>Fairly easy</td>
<td>Good</td>
<td>Limited</td>
<td>28 days</td>
<td>4.4.4</td>
</tr>
<tr>
<td>RTA T362</td>
<td>Fairly common</td>
<td>Easy</td>
<td>Fair</td>
<td>Limited</td>
<td>36 days</td>
<td>4.4.5</td>
</tr>
<tr>
<td>BS 1881 Part 208</td>
<td>Not common, designed for use on in-situ concrete</td>
<td>Fairly easy</td>
<td>Poor</td>
<td>Fairly limited</td>
<td>Depends on sample conditioning</td>
<td>4.4.1</td>
</tr>
<tr>
<td>BS 1881 Part 124</td>
<td>Occasional</td>
<td>Fairly easy</td>
<td>Fair</td>
<td>Fairly limited</td>
<td>2-3 days</td>
<td>4.4.1</td>
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<tr>
<td>Water Permeability</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>4.5</td>
</tr>
<tr>
<td>Taywood/GHD/SGS water permeability</td>
<td>Occasional</td>
<td>Fairly easy</td>
<td>Good</td>
<td>Fairly limited</td>
<td>5 days</td>
<td>4.5</td>
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<tr>
<td>DIN 1048.5 Method 7.6</td>
<td>Not common</td>
<td>Fairly difficult</td>
<td>Fair</td>
<td>Fairly extensive</td>
<td>3 days</td>
<td>4.5</td>
</tr>
<tr>
<td>ASTM D5084-10 (Various methods)</td>
<td>Not common</td>
<td>Fairly difficult</td>
<td>Good</td>
<td>Fairly extensive</td>
<td>Varies</td>
<td>4.5</td>
</tr>
<tr>
<td>Main Roads Western Australia Test Method WA 625.1</td>
<td>Occasional</td>
<td>Fairly difficult</td>
<td>Good</td>
<td>Fairly extensive</td>
<td>Not defined</td>
<td>4.5</td>
</tr>
<tr>
<td>US Army Corps of Engineers CRD-C 49-92</td>
<td>Not common</td>
<td>Fairly difficult</td>
<td>Good</td>
<td>Fairly extensive</td>
<td>14-20 days</td>
<td>4.5</td>
</tr>
<tr>
<td>US Army Corps of Engineers CRD-C 163-92</td>
<td>Not common</td>
<td>Fairly difficult</td>
<td>Good</td>
<td>Fairly extensive</td>
<td>Not defined</td>
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</tr>
<tr>
<td>Test</td>
<td>Use of Test</td>
<td>Practical Application</td>
<td>Reproducibility</td>
<td>Experience Required</td>
<td>Test Time</td>
<td>Report Section</td>
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<tr>
<td>Semi Adiabatic Tests to predict Concrete Adiabatic Temperature</td>
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</tr>
<tr>
<td>Semi adiabatic test 1m³ concrete block – one concrete position method</td>
<td>Fairly common</td>
<td>Fairly easy</td>
<td>N/A</td>
<td>Fairly extensive</td>
<td>Up to 14 days</td>
<td>4.6.1</td>
</tr>
<tr>
<td>Semi adiabatic test 1m³ concrete block – multiple concrete positions method</td>
<td>Fairly common</td>
<td>Fairly easy</td>
<td>N/A</td>
<td>Fairly extensive</td>
<td>Up to 14 days</td>
<td>4.6.1</td>
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<tr>
<td>Alkali-Aggregate Reaction (AAR)</td>
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<td>Petrographic</td>
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<td>4.7</td>
</tr>
<tr>
<td>AS 1141.65</td>
<td>Fairly common</td>
<td>Easy</td>
<td>Good</td>
<td>Extensive</td>
<td>1-2 days</td>
<td>4.7</td>
</tr>
<tr>
<td>ASTM C295-12</td>
<td>Common</td>
<td>Easy</td>
<td>Good</td>
<td>Extensive</td>
<td>1-2 days</td>
<td>4.7</td>
</tr>
<tr>
<td>Mortar Bar Expansion</td>
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<td>4.7</td>
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<tr>
<td>ASTM C227-10</td>
<td>Not common</td>
<td>Difficult</td>
<td>Poor</td>
<td>Extensive</td>
<td>Up to 2 years</td>
<td>4.7</td>
</tr>
<tr>
<td>ASTM C441-11</td>
<td>Not common</td>
<td>Difficult</td>
<td>Poor</td>
<td>Extensive</td>
<td>&gt;1 year</td>
<td>4.7</td>
</tr>
<tr>
<td>ASTM C1260-07</td>
<td>Fairly common</td>
<td>Fairly easy</td>
<td>Fair</td>
<td>Fairly limited</td>
<td>Up to 2 months</td>
<td>4.7</td>
</tr>
<tr>
<td>ASTM C1567-13</td>
<td>Not common</td>
<td>Fairly easy</td>
<td>Fair</td>
<td>Fairly limited</td>
<td>Up to 2 months</td>
<td>4.7</td>
</tr>
<tr>
<td>RMS T363 &amp; VicRoads RC376.03</td>
<td>Fairly common</td>
<td>Fairly easy</td>
<td>Fair</td>
<td>Fairly limited</td>
<td>&gt;21 days</td>
<td>4.7</td>
</tr>
<tr>
<td>MRWA WA 624.1</td>
<td>Fairly common</td>
<td>Fairly easy</td>
<td>Fair</td>
<td>Fairly limited</td>
<td>&gt;22 days</td>
<td>4.7</td>
</tr>
<tr>
<td>AS 1141.60.1</td>
<td>Fairly common</td>
<td>Fairly easy</td>
<td>Fair</td>
<td>Fairly limited</td>
<td>22 days</td>
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<td>Aggregate</td>
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<td></td>
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<td>4.7</td>
</tr>
<tr>
<td>ASTM C289-07</td>
<td>Not common</td>
<td>Difficult</td>
<td>Poor</td>
<td>Extensive</td>
<td>24 hours</td>
<td>4.7</td>
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<td>Concrete Prism</td>
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<td>4.7</td>
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<tr>
<td>ASTM C1293-08</td>
<td>Fairly common</td>
<td>Fairly easy</td>
<td>Fair</td>
<td>Fairly limited</td>
<td>1-2 years</td>
<td>4.7</td>
</tr>
<tr>
<td>RMS T364 &amp; VicRoads RC 376.04</td>
<td>Fairly common</td>
<td>Fairly easy</td>
<td>Fair</td>
<td>Fairly limited</td>
<td>1 year</td>
<td>4.7</td>
</tr>
<tr>
<td>Qld Main Roads Q458</td>
<td>Fairly common</td>
<td>Fairly easy</td>
<td>Fair</td>
<td>Fairly limited</td>
<td>&gt;4 months</td>
<td>4.7</td>
</tr>
<tr>
<td>AS 1141.60.2</td>
<td>Fairly common</td>
<td>Fairly easy</td>
<td>Fair</td>
<td>Fairly limited</td>
<td>1-2 years</td>
<td>4.7</td>
</tr>
<tr>
<td>Delayed Ettringite Formation (DEF)</td>
<td></td>
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<td>4.8</td>
</tr>
<tr>
<td>There is no reliable concrete test to evaluate the risk of DEF. The risk of DEF is assessed by reviewing of the concrete mix composition, predicting the concrete peak temperature and specifying a maximum concrete peak temperature.</td>
<td></td>
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<td>4.8</td>
</tr>
<tr>
<td>Test</td>
<td>Use of Test</td>
<td>Practical Application</td>
<td>Reproducibility</td>
<td>Experience Required</td>
<td>Test Time</td>
<td>Report Section</td>
</tr>
<tr>
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<tr>
<td><strong>Sulfate Resistance</strong></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td>4.9</td>
</tr>
<tr>
<td>AS 2350.14</td>
<td>Used for benchmarking cement performance</td>
<td>Fairly difficult</td>
<td>Fair</td>
<td>Fairly extensive</td>
<td>16 weeks</td>
<td>4.9</td>
</tr>
<tr>
<td>ASTM C452-10</td>
<td>Used for benchmarking cement performance</td>
<td>Fairly difficult</td>
<td>Fair</td>
<td>Fairly extensive</td>
<td>14 days</td>
<td>4.9</td>
</tr>
<tr>
<td>ASTM C1012-13</td>
<td>Used for benchmarking cement performance</td>
<td>Fairly difficult</td>
<td>Fair</td>
<td>Fairly extensive</td>
<td>6-12 months</td>
<td>4.9</td>
</tr>
<tr>
<td>ASTM C1038-14</td>
<td>Used for benchmarking cement performance</td>
<td>Fairly difficult</td>
<td>Fair</td>
<td>Fairly extensive</td>
<td>14 days</td>
<td>4.9</td>
</tr>
<tr>
<td><strong>Abrasion Resistance</strong></td>
<td></td>
<td></td>
<td></td>
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<td>4.10</td>
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<tr>
<td>AS 4456.9</td>
<td>Test for segmental pavers</td>
<td>Fair easy</td>
<td>Fair</td>
<td>Fairly extensive</td>
<td>Hours</td>
<td>4.10.2</td>
</tr>
<tr>
<td>BS EN 13892-4</td>
<td>Test for floors</td>
<td>Fairly difficult</td>
<td>Fair</td>
<td>Fairly extensive</td>
<td>Hours</td>
<td>4.10.3</td>
</tr>
<tr>
<td>BS EN 13892-3</td>
<td>Test for floor screed materials</td>
<td>Fairly difficult</td>
<td>Poor</td>
<td>Extensive</td>
<td>Hours</td>
<td>4.10.4</td>
</tr>
<tr>
<td>ASTM C418-12</td>
<td>Concrete subject to abrasive wear with dry particles</td>
<td>Difficult</td>
<td>Fair</td>
<td>Extensive</td>
<td>Hours</td>
<td>4.10.1</td>
</tr>
<tr>
<td>ASTM C779/ C779M-12</td>
<td>Three alternative procedures for horizontal surfaces</td>
<td>Fairly difficult</td>
<td>Good to fair</td>
<td>Fairly extensive</td>
<td>Hours</td>
<td>4.10.5</td>
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<tr>
<td>ASTM C944/ C944M-12</td>
<td>Suited to cores</td>
<td>Fairy easy</td>
<td>Fair to poor</td>
<td>Fairly extensive</td>
<td>Hours</td>
<td>4.10.1</td>
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<tr>
<td>ASTM C1138M-05</td>
<td>Water-borne particles for concrete underwater</td>
<td>Difficult</td>
<td>Fair to poor</td>
<td>Extensive</td>
<td>Hours</td>
<td>4.10.1</td>
</tr>
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</table>

**TESTS FOR QUALITY ASSURANCE DURING CONSTRUCTION** 5

**Compressive Strength** 5.2

<table>
<thead>
<tr>
<th>Test</th>
<th>Use of Test</th>
<th>Practical Application</th>
<th>Reproducibility</th>
<th>Experience Required</th>
<th>Test Time</th>
<th>Report Section</th>
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<tbody>
<tr>
<td>AS 1379</td>
<td>Common</td>
<td>Easy</td>
<td>Good</td>
<td>Limited</td>
<td>Minutes</td>
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</table>

**Concrete Cover** Refer below in this Table 5.3

**Maturity/Matched Curing** 5.4

<table>
<thead>
<tr>
<th>Test</th>
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<th>Practical Application</th>
<th>Reproducibility</th>
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<th>Test Time</th>
<th>Report Section</th>
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<tbody>
<tr>
<td>ASTM C1074-11</td>
<td>Becoming more common</td>
<td>Fairly difficult</td>
<td>Fair</td>
<td>Fairly extensive</td>
<td>Up to 28 days</td>
<td>5.4</td>
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<td>Use of Test</td>
<td>Practical Application</td>
<td>Reproducibility</td>
<td>Experience Required</td>
<td>Test Time</td>
<td>Report Section</td>
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<td>Crack Measurements</td>
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<td>Crack width</td>
<td>Common</td>
<td>Easy</td>
<td>Good</td>
<td>Limited</td>
<td>Minutes</td>
<td>5.5</td>
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<tr>
<td>Crack movement</td>
<td>Occasional</td>
<td>Fairly easy</td>
<td>Good</td>
<td>Fairly limited</td>
<td>Days</td>
<td>5.5</td>
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<tr>
<td>In-situ Temperature and Strain Measurements</td>
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<td>In-situ temperature</td>
<td>Occasional</td>
<td>Fairly easy</td>
<td>Fair</td>
<td>Fairly limited</td>
<td>Days</td>
<td>5.6</td>
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<tr>
<td>In-situ strain</td>
<td>Occasional</td>
<td>Fairly difficult</td>
<td>Fair</td>
<td>Fairly extensive</td>
<td>Days</td>
<td>5.6</td>
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<tr>
<td>Electrical Resistivity</td>
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<td></td>
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<td>5.7</td>
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<tr>
<td>AASHTO TP95-11</td>
<td>Becoming more common</td>
<td>Easy</td>
<td>Good</td>
<td>Limited</td>
<td>Minutes</td>
<td>5.7</td>
</tr>
<tr>
<td>ASTM C1202</td>
<td>Occasional</td>
<td>Fairly easy</td>
<td>Fair</td>
<td>Fairly limited</td>
<td>6-96 hours</td>
<td>5.7</td>
</tr>
<tr>
<td>AASHTO T277</td>
<td>Occasional</td>
<td>Fairly easy</td>
<td>Fair</td>
<td>Fairly limited</td>
<td>6-96 hours</td>
<td>5.7</td>
</tr>
<tr>
<td>Cross Hole Sonic Logging</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>5.8</td>
</tr>
<tr>
<td>AS 2159</td>
<td>Not common</td>
<td>Difficult</td>
<td>Not known</td>
<td>Extensive</td>
<td>Hours</td>
<td>5.8</td>
</tr>
<tr>
<td>TESTS WHERE PLACED CONCRETE IS SUSPECT</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>6</td>
</tr>
<tr>
<td>Concrete Cover and Reinforcement Size</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>6.2</td>
</tr>
<tr>
<td>Magnetic reluctance covermeter</td>
<td>Common</td>
<td>Fairly easy</td>
<td>Good</td>
<td>Fairly limited</td>
<td>Minutes</td>
<td>6.2.2 &amp; 6.2.4</td>
</tr>
<tr>
<td>Pulsed eddy current covermeter</td>
<td>Common</td>
<td>Fairly easy</td>
<td>Good</td>
<td>Fairly extensive</td>
<td>Minutes</td>
<td>6.2.3 &amp; 6.2.4</td>
</tr>
<tr>
<td>Ground penetrating radar</td>
<td>Not common</td>
<td>Fairly difficult</td>
<td>Fair</td>
<td>Extensive</td>
<td>Minutes</td>
<td>6.2.5</td>
</tr>
<tr>
<td>Ultrasonic pulse echo</td>
<td>Not common</td>
<td>Difficult</td>
<td>Fair to poor</td>
<td>Extensive</td>
<td>Minutes</td>
<td>6.2.6</td>
</tr>
<tr>
<td>Compressive Strength</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>6.3</td>
</tr>
<tr>
<td>Concrete core sample testing</td>
<td>Common</td>
<td>Easy</td>
<td>Good</td>
<td>Limited</td>
<td>1 day</td>
<td>6.3.1</td>
</tr>
<tr>
<td>Windsor Probe and Capo tests</td>
<td>Not common</td>
<td>Fairly difficult</td>
<td>Fair to poor</td>
<td>Fairly extensive</td>
<td>Hours</td>
<td>6.3.2</td>
</tr>
<tr>
<td>Rebound hammer</td>
<td>Common (as an indicator of concrete surface hardness)</td>
<td>Easy</td>
<td>Fair to poor</td>
<td>Limited</td>
<td>Minutes</td>
<td>6.3.3</td>
</tr>
<tr>
<td>Ultrasonic pulse velocity</td>
<td>Not common</td>
<td>Fairly easy</td>
<td>Fair to poor</td>
<td>Fairly extensive</td>
<td>Minutes</td>
<td>6.3.4</td>
</tr>
<tr>
<td>Assessment of Defects</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>6.4</td>
</tr>
<tr>
<td>Ultrasonic pulse velocity</td>
<td>Occasional</td>
<td>Fairly easy</td>
<td>Fair to poor</td>
<td>Fairly extensive</td>
<td>Minutes</td>
<td>6.4.1</td>
</tr>
<tr>
<td>Ultrasonic pulse echo</td>
<td>Not common</td>
<td>Fairly difficult</td>
<td>Fair to poor</td>
<td>Fairly extensive</td>
<td>Minutes</td>
<td>6.4.2</td>
</tr>
<tr>
<td>Test</td>
<td>Use of Test</td>
<td>Practical Application</td>
<td>Reproducibility</td>
<td>Experience Required</td>
<td>Test Time</td>
<td>Report Section</td>
</tr>
<tr>
<td>-------------------------------------</td>
<td>---------------</td>
<td>-----------------------</td>
<td>------------------</td>
<td>---------------------</td>
<td>-----------</td>
<td>----------------</td>
</tr>
<tr>
<td>Impact echo</td>
<td>Not common</td>
<td>Fairly difficult</td>
<td>Fair to poor</td>
<td>Fairly extensive</td>
<td>Minutes</td>
<td>6.4.3</td>
</tr>
<tr>
<td>Impulse response</td>
<td>Not common</td>
<td>Difficult</td>
<td>Fair</td>
<td>Extensive</td>
<td>Minutes</td>
<td>6.4.4</td>
</tr>
<tr>
<td>Ground penetrating radar</td>
<td>Not common</td>
<td>Fairly difficult</td>
<td>Fair</td>
<td>Extensive</td>
<td>Minutes</td>
<td>6.4.5</td>
</tr>
</tbody>
</table>

### Assessment of Concrete Surface Quality

| BS 1881-208 initial surface water absorption (ISAT) | Not common | Fairly easy | Poor | Fairly limited | Hours | 6.5.1 |
| Karsten Tube, RILEM 25-PEM, initial surface water absorption (ISAT) | Not common | Fairly easy | Poor | Fairly limited | Hours | 6.5.1 |
| Torrent air permeability             | Not common   | Fairly difficult | Poor | Extensive     | Minutes | 6.5.2 |

### Mix Composition

| Cement (binder) content and composition | Occasional | Fairly easy | Good | Extensive | 1 day | 6.6.1 |
| Air content                           | Petrographic examination | Fairly easy | Good | Extensive | 1 day | 6.6.2 |
| Water to cement ratio                 | Not common   | Fairly easy  | Fair | Extensive | 1 day | 6.6.3 |

### SCM Content and Composition

| Petrographic examination | Occasional | Fairly easy | Good | Extensive | 3-5 weeks | 6.7 |
| Electrical resistivity     | Occasional | Easy        | Fair | Limited   | Minutes   | 6.7 |
| Chemical analysis          | Occasional | Fairly easy | Good | Extensive | 1 day     | 6.7 |

### Chloride and Sulfate Ion Content

| Chemical analysis          | Common      | Fairly easy | Good | Extensive | 1 day | 6.8 |

### Alkali Content

| Chemical analysis          | Occasional  | Fairly easy | Good | Extensive | 1 day | 6.9 |

### Condition Monitoring

| Visual inspection          | Common      | Fairly easy | Good | Fairly limited | Inspector & site dependent | 7.3 |

### Intermittent Site Tests

<p>| Electrode (half cell) potential mapping | Common | Fairly easy | Good | Limited | Site dependent | 7.4.1 |
| Carbonation depth             | Common   | Fairly easy | Good | Limited  | Site dependent | 7.4.2 |</p>
<table>
<thead>
<tr>
<th>Test</th>
<th>Use of Test</th>
<th>Practical Application</th>
<th>Reproducibility</th>
<th>Experience Required</th>
<th>Test Time</th>
<th>Report Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chloride profile</td>
<td>Common</td>
<td>Fairly easy</td>
<td>Good</td>
<td>Limited</td>
<td>Site dependent</td>
<td>7.4.3</td>
</tr>
<tr>
<td>Resistivity</td>
<td>Common</td>
<td>Fairly easy</td>
<td>Good</td>
<td>Limited</td>
<td>Site dependent</td>
<td>7.4.4</td>
</tr>
<tr>
<td>Polarisation resistance</td>
<td>Occasional when defect justify</td>
<td>Fairly difficult</td>
<td>Fair</td>
<td>Fairly limited</td>
<td>Site dependent</td>
<td>7.4.5</td>
</tr>
<tr>
<td>Petrographic examination</td>
<td>Occasional when defect justify</td>
<td>Fairly easy</td>
<td>Good</td>
<td>Extensive</td>
<td>3-5 weeks</td>
<td>7.4.6</td>
</tr>
<tr>
<td>Microbial analysis</td>
<td>Occasional when defect justify</td>
<td>Fairly easy</td>
<td>Good</td>
<td>Extensive</td>
<td>3-5 weeks</td>
<td>7.4.7</td>
</tr>
</tbody>
</table>

**Permanent Surface Mounted and Embedded Monitoring**

<table>
<thead>
<tr>
<th>Test</th>
<th>Use of Test</th>
<th>Practical Application</th>
<th>Reproducibility</th>
<th>Experience Required</th>
<th>Test Time</th>
<th>Report Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Corrosion initiation</td>
<td>Occasional for high corrosion risk</td>
<td>Fairly easy</td>
<td>Fair</td>
<td>Fairly limited</td>
<td>1-20+ years</td>
<td>7.5.1</td>
</tr>
<tr>
<td>Corrosion rate</td>
<td>Occasional for high corrosion risk</td>
<td>Fairly difficult</td>
<td>Fair</td>
<td>Fairly limited</td>
<td>1-20+ years</td>
<td>7.5.2</td>
</tr>
</tbody>
</table>

**Strain, Vibration and Deflection**

<table>
<thead>
<tr>
<th>Test</th>
<th>Use of Test</th>
<th>Practical Application</th>
<th>Reproducibility</th>
<th>Experience Required</th>
<th>Test Time</th>
<th>Report Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Resistive strain gauge</td>
<td>Occasional when defect justify</td>
<td>Fairly difficult</td>
<td>Fair</td>
<td>Fairly extensive</td>
<td>Site dependent</td>
<td>7.5.3</td>
</tr>
<tr>
<td>Vibrating wire strain gauge</td>
<td>Occasional when defect justify</td>
<td>Fairly easy</td>
<td>Fair</td>
<td>Fairly limited</td>
<td>Site dependent</td>
<td>7.5.3</td>
</tr>
<tr>
<td>Crack meter</td>
<td>Occasional when defect justify</td>
<td>Easy</td>
<td>Fair</td>
<td>Limited</td>
<td>Site dependent</td>
<td>7.5.3</td>
</tr>
<tr>
<td>Mechanical strain gauge</td>
<td>Occasional when defect justify</td>
<td>Fairly easy</td>
<td>Fair</td>
<td>Fairly limited</td>
<td>Site dependent</td>
<td>7.5.3</td>
</tr>
<tr>
<td>Linear variable displacement transducer</td>
<td>Occasional when defect justify</td>
<td>Fairly difficult</td>
<td>Fair</td>
<td>Fairly extensive</td>
<td>Site dependent</td>
<td>7.5.3</td>
</tr>
<tr>
<td>Accelerometers</td>
<td>Rarely used</td>
<td>Difficult</td>
<td>Fair</td>
<td>Extensive</td>
<td>Site dependent</td>
<td>7.5.3</td>
</tr>
<tr>
<td>Interferometric radar</td>
<td>Occasional when defect justify</td>
<td>Difficult</td>
<td>Fair</td>
<td>Extensive</td>
<td>Site dependent</td>
<td>7.5.3</td>
</tr>
</tbody>
</table>
Performance Tests to Assess Concrete Durability

3

Sampling and Sample Preparation

3.1 GENERAL

Sampling and sample preparation requirements for fresh concrete and hardened concrete (including laboratory specimens and samples taken from in-situ concrete) and the frequency and extent of sampling are all necessary considerations prior to any durability testing whether it be concrete mix acceptance testing, quality assurance testing, testing where placed concrete is suspect or condition monitoring testing.

The accuracy of any test result is strongly influenced by the quality of the sample tested. Thus representative sampling, sample preparation, and the repeatability and reproducibility of the method are critical features. A sample may comprise one specimen or several replicate specimens. Several replicate specimens may be needed to comprise a representative sample of the concrete of interest.

3.2 STANDARD METHODS OF SAMPLE PREPARATION FROM FRESH CONCRETE

3.2.1 Specification of testing requirements for fresh concrete

The specification of durability related parameters other than compressive strength for concrete means that the concrete has to be specified as “special class” in accordance with AS 1379:2007 [Reference 7]. In addition, where special class concrete is specified, the project concrete specification will often call for quality control testing to be performed under “Project Assessment”. Under the provisions of AS 1379:2007 [Reference 7], this triggers a higher frequency of routine compressive strength testing from more restricted sample pools than the “production assessment” applied to “normal class” concrete.

The method of preparing compressive strength test samples from production concrete is referenced to AS 1012.8.1:2000 [Reference 95], which describes various compaction procedures deemed acceptable for preparation of test samples.

Samples for durability performance related tests are most commonly prepared as standard 1000x200 mm compressive strength test cylinders. However, other geometries such as cubes and beams of various sizes can also be required for particular test methods.

As AS 1379:2007 [Reference 7] only nominates requirements for compressive strength samples, the project concrete specification must nominate the number and type of samples required for durability related testing in addition to the frequency of sampling.

3.2.2 Effect of sample concrete compaction on test results

While the standard compaction procedures described in AS 1012.8.1:2000 (cylinders) [Reference 95] and AS 1012.8.2:2000 (beams) [Reference 96] are satisfactorily consistent for measurement of macroscopic properties such as compressive strength, testing of durability related properties such as water penetration, chloride ingress, carbonation or chemical resistance is inherently more sensitive to the microstructure and continuity of voids within the concrete.

Published systematic repeatability studies of water permeability testing [Reference 6] have shown that consistency of concrete compaction of the sample is the primary factor that influences the often perceived poor repeatability/reproducibility of durability related tests. Where due care is taken in compacting the concrete when casting test specimens, coefficients of variation for water permeability and absorption tests, for example, that are comparable to those for compressive strength testing can be obtained.

Compaction of test specimens using a vibrating table is preferred to other methods because it consolidates concrete with relatively little effort; therefore, is the easiest way to consistently achieve good compaction in all samples. However, concrete can be consolidated effectively with other methods if care is taken to avoid over- or under-compaction.

“Durability” samples should not be given “special attention” to try and produce particularly high levels of compaction in order to secure favourable test results. For example, forensic investigation of past instances of failure to comply with specified water permeability criteria has found that over-zealous compaction can result in the formation of microstructural features such as segregation and bleed channels that adversely affect test results, even to the extent of compromising specification compliance.
3.2.3 Curing of concrete samples

The way in which concrete is cured significantly affects its potential durability, both through the extent of cement hydration and the nature of the pore structure that develops. Similarly, the way in which concrete samples are cured will significantly affect the results of durability tests.

AS 1379:2007 [Reference 7] nominates curing of compressive strength samples in accordance with AS 1012.8.1:2000 [Reference 95]. Test cylinders are required to be capped or covered and initially left at site, but transferred to a controlled wet curing environment within 18 to 36 hours from moulding.

Standard wet curing conditions by lime saturated water bath immersion or, much less commonly, fog room, are 23 ± 2 °C for sites in temperate climates (defined as Australian Capital Territory, New South Wales, South Australia, Tasmania, Victoria, and Western Australia south of latitude 25° S) or 27 ± 2 °C for tropical climates (defined as Queensland, Northern Territory, and Western Australia north of latitude 25° S). In NZ, the standard curing temperature is 21 +/- 2 ºC.

It is sometimes necessary to employ non-standard curing conditions in the preparation of samples for durability related tests, in order to reflect the conditions under which the concrete will cure on-site. Examples of this include:

- Early age steam curing of precast elements.
- Temperature matched curing for maturity calculations.
- Unusually cold or hot conditions.
- Insulated conditions.
- Trial mixes that are performed in a different climatic zone to the project site.

Where non-standard curing conditions are required, these must be described in the project concrete specification.

3.3 SAMPLING METHODS FOR IN-SITU CONCRETE

3.3.1 Background

Sampling from a concrete structure is often required to assess one or more properties. The sample type, size, locations and frequency of collection will depend on the properties to be determined, the reliability required for the results and the practicalities and cost of testing. The methods of sampling for in-situ concrete are reviewed in this section.

Many test methods have specific requirements for the size and condition of samples. It is necessary to be familiar with such requirements before specifying a sampling program. Unless sampling particular anomalies, a primary requirement is often to ensure that the extracted samples represent the bulk of the concrete in terms of properties such as aggregate distribution and voids. Certain structures, such as dams and some historical structures, and elements with large cross sections and little or no reinforcement can contain abnormally large aggregate particles, to greater than 200 mm, which makes “representative” sampling difficult or impractical. In such cases, experience is required in assessing sampling methods and sample size to facilitate gathering the required information.

The most common method for taking samples is by wet diamond coring. Where this is not convenient, drilled dust samples or dry grinding core cuts may be taken. This Section describes standard procedures for taking all three sample types.

Before taking the sample, the reinforcement position should be located and marked so that the sample can be taken over the reinforcement to inspect the physical condition of the reinforcement and measure the cover depth for verification of cover meter readings, or to avoid the reinforcement as required.

Furthermore, before taking the sample the concrete surface should be inspected and any defects recorded.

3.3.2 Wet diamond coring

Extracted cores have the same diameter as the internal diameter of the core bit. In contrast, coring contractors typically refer to the “core size” as the size of the hole left behind, i.e. the outside diameter of the core bit. The difference is usually about 5-6 mm, hence the designer must clearly define the diameter when specifying cores.

Diamond drilled cores for chemical analysis of concrete commonly have diameters from 30 mm to 150 mm. A 30 mm core is adequate for visual examination of deterioration induced by chemical attack. If the concrete maximum aggregate size is 20 mm or larger, a 30 mm core will not be suitable for chemical analysis (e.g. chloride or sulfate ion content) as the sample aggregate:cement ratio might not be representative of the concrete. For example, a single piece of coarse
aggregate could occupy the entire cross section of the core at one or more locations along the core length. If 30 mm diameter cores are the largest that can be taken for chemical analysis, three such cores should be extracted and combined to provide a representative sample. Cores of 50 mm diameter provide sufficient area to give representative samples for chemical analysis provided that the depth increments selected are at least 10 mm thick. Often it is preferable to cut a core in half longitudinally and measure chloride content on one half and cement content or carbonation on the other half. Cores of 75 mm diameter provide sufficient cross sectional surface area for both halves of the core to give representative samples.

Many physical test methods prescribe requirements for the geometry of the core sample. For example, for compressive strength testing AS 1012.14:1991 [Reference 97] states that “The diameter of cores shall be not less than the greater of 75 mm or 3 times the nominal size of coarse aggregate in the concrete …” Other test methods specify the sample size by the face area of the core, or the volume of concrete comprising the core.

Cores of 100 mm diameter are often considered to be the standard size for diffusion, penetrability and strength testing. However, 75 mm diameter cores can often be used as the minimum size for concrete with aggregate size up to 20 mm. However, to ensure compliance with minimum requirements a tolerance should be allowed and 80-85 mm diameter cores provide the ideal balance between minimum damage to the structure (less concrete extracted and less risk of damaging reinforcement) and acceptable assurance of meeting minimum diameter requirements.

3.3.3 Recommended practice for wet diamond coring

AS1012.14:1991 [Reference 97] provides limited information on the method of taking cores. Z7/07 recommends the following procedures when coring:

- Locate reinforcement with by a covermeter or ground penetrating radar (GPR), see Section 6.2, before coring. GPR will typically show reinforcement to around 300-500 mm whereas covermeters only show the top reinforcement. Cores containing reinforcement could be unsuitable for testing. Cutting the reinforcement damages the structure, and cut reinforcement cannot be easily repaired.
- Bolt (or attach by vacuum pad) diamond tipped coring drills to the concrete surface and lock them in place with the core bit aligned perpendicular to the surface, refer Figure 3.1c. This will minimise damage to the concrete core sample by the core bit vibrating during coring. If the core drill is not firmly locked in place, ridges can form on the core surface, which might make the core sample unsuitable for testing. Failure to properly secure the drill can also result in it detaching from the surface, with risks of injury to the operator and damage to the equipment.
- Use mains water or a battery operated pump on the water reservoir for continuous water supply. This avoids damaging the core bit, and hence the core, by interrupted water supply.
- Coring to be undertaken at a constant rate with relatively constant and not undue force. If possible do not interrupt coring. This will help minimise ridges on the core and core breakage.
- Before coring, mark the core’s exposed outer surface to indicate orientation and designation.
- Immediately after coring, mark the core sides to indicate orientation and designation, refer Figure 3.1a and Figure 3.1b.

After coring, check the cores to ensure that they satisfy testing requirements and clearly identify cores that should be rejected. Record the following details about the core samples:

- Length and whether it is acceptable for testing, (e.g. for length/diameter ratio after ends trimmed for strength tests) or to reach the required depth into the structure.
- Reinforcement present, size, cover depth, corrosion condition before flash rusting, and whether any reinforcement not removed by trimming the core will prevent testing.
- Cracks, delaminations or cold joints present.
- Voids quantity and size.
- Compaction variations or segregation of the concrete.
- Core ridges greater than 2 mm.
- Photograph all concrete surfaces.

Useful information might also be gained from inspecting the core hole. For example record:

- Depth of carbonation.
- Reinforcement present, size, cover depth and
Performance Tests to Assess Concrete Durability

3.3.4 Grinding core cuts

Dry grinding core cuts are not ideal to obtain concrete samples because they do not provide suitable samples for some test methods (see below), and sampling to depths exceeding the cover zone can be difficult. However, they are a useful compromise where access is difficult or wet diamond coring equipment is impractical (e.g. overhead) or not available (e.g. remote areas).

In some cases the cored concrete sample is obtained intact while in other cases the concrete comes out as fractured pieces. In some cases it is virtually dust with some pieces of aggregate.

A hammer drill fitted with a tungsten carbide tipped core cutter, refer Figure 3.2, is used to cut the concrete but will not cut through reinforcement. No coring machine frame or cooling water is required. The drill is hand held, refer Figure 3.2a and 3.2b, and if not held firmly the core can break while being cut, jamming the bit. A pilot drill bit makes a hole in the centre of the core over the first 20 mm. A nominal 82 mm outside diameter grinding core is typically used and is probably the maximum practical size to cut.

A) Diamond Core. 100 mm cores are often considered standard for strength but 80-85 mm diameter is generally preferable as it causes less damage, is considerably faster to take and has less risk of cutting reinforcement.

B) Diamond Core. 30 mm cores are quick to take and can provide a lot of information on materials and concrete quality to considerable depth. 50 mm cores are more suited for chemical analysis.

C) Diamond Coring with 82 mm core bit with battery operated water pump in the foreground.

Features such as delamination planes or cracks.

Voids quantity and size.

Cracks, delaminations or cold joints present.

Reinforcement present, size, cover depth, corrosion condition before flash rusting.

Compaction variations or segregation of the concrete.

Excessive voids.

Immediately after coring, mark the core sides to indicate orientation and designation, refer Figure 2a) and Figure 2b).

Figure 2a) and Figure 2b).

Core ridges greater than 2 mm.

Compaction variations or segregation of the concrete.

Photograph all concrete surfaces.

Use mains water or a battery operated pump on the water reservoir for continuous water supply. This avoids damaging the core bit, and hence the core, by interrupted water supply.

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Grinding core cut samples can be used for visual examination, reinforcement presence, size, cover depth and corrosion condition, refer Figure 3.2c and Figure 3.2d, petrographic examination, chemical tests, etc. However, extracted cores might contain cracks and micro-cracks induced by the percussion drilling process that could affect test results.

Information can also be obtained from the core hole visual inspection, similar to that described for wet diamond drilled cores.

3.3.5 Drilled dust samples

This method is often regarded as the option of last resort where other concrete sampling methods are impractical or not available. However, when conducted in accordance with the recommendations below, the method does provide concrete samples suitable for chemical testing.

A hand held percussion drill with a 20-25 mm diameter tungsten carbide tipped drill bit is used for taking drilled dust samples. A 60 mm diameter clear pipe cut off at 45 degrees and with a drill bit entry hole is used for collecting dust, as shown in Figure 3.3.

Dust is usually collected from several sequential depth increments from the outer surface to the depth of the outer reinforcement or slightly beyond it.

Drilled dust samples extracted using a 20-25 mm bit will not provide a representative sample unless a number of adjacent holes are drilled and the dust combined. The number of drill holes required is typically:

- 15 mm depth intervals − 6 to 8 No.
- 20 mm depth intervals − 4 to 6 No.

The number of holes might need to be increased if drilling soffits, due to less efficient dust collection, or if smaller diameter drill bits are used.

Fewer holes can be drilled if larger drill bits are used however, drilling with greater than 25 mm diameter drill bits can be more difficult.

A minimum weight of concrete sample can be required for a specific test that will determine the number of holes at specific depth increments.

The depth interval will depend on the cover depth, but should be a maximum of cover x 0.3, if the concrete is thought to be heavily contaminated. Sampling beyond the cover depth will help determine the depth of contamination as well as background levels of the contaminant.

Select the concrete surface area and drill the first hole to the required depth collecting the dust. When complete move to the second hole and collect the dust in the same as the first. Repeat the process until the sample bag contains dust from the prescribed number of holes. Wrap the dust collected tightly in the polythene bag and record the sample location and the depth increment on the bag with a permanent marker pen.

Clean the collection tube with a brush and remove dust remaining in the drill hole. Then take the sample from the second depth increment using the same drill holes used for the first depth increment.

Use the same routine to collect samples for each depth increment. Wrap all the samples in one bag and label the bag with the sample number.

Guidance on drilled dust samples for chloride ion content testing is given in Concrete Society Technical Report TR 60 [Reference 189] that requires a minimum of 25 grams of concrete dust for each depth increment from multiple drill positions.
- 20 mm depth intervals – 4 to 6 No.
- The number of holes might need to be increased if drilling soffits, due to less efficient dust collection, or if smaller diameter drill bits are used.
- Fewer holes can be drilled if larger drill bits are used; however, drilling with greater than 25 mm diameter drill bits can be more difficult.
- A minimum weight of concrete sample can be required for a specific test that will determine the number of holes at specific depth increments. The depth interval will depend on the cover depth, but should be a maximum of cover x 0.3, if the concrete is thought to be heavily contaminated. Sampling beyond the cover depth will help determine the depth of contamination as well as background levels of the contaminant.

Select the concrete surface area and drill the first hole to the required depth collecting the dust. When complete move to the second hole and collect the dust in the same as the first. Repeat the process until the sample bag contains dust from the prescribed number of holes. Wrap the dust collected tightly in the polythene bag and record the sample location and the depth increment on the bag with a permanent marker pen.

Clean the collection tube with a brush and remove dust remaining in the drill hole. Then take the sample from the second depth increment using the same drill holes used for the first depth increment.

Use the same routine to collect samples for each depth increment. Wrap all the samples in one bag and label the bag with the sample number.

Guidance on drilled dust samples for chloride ion content testing is given in Concrete Society Technical Report TR 80 [Reference 189] that requires a minimum of 25 grams of concrete dust for each depth increment from multiple drill positions.
Concrete Mix Acceptance Tests for Durability Design and Construction Compliance

4.1 GENERAL

This section reviews the primary types of durability performance tests used for concrete mix acceptance. The designer and/or durability consultant will need to assess whether specific durability tests are required for the project (see Section 1). If durability testing is required, the project specification would include:

- Test method according to some publicly available method.
- Sampling method (including curing), sample geometry and size (unless included in the method).
- Frequency of test and number of test specimens to be used to give one result (unless included in the method of test).
- Acceptance criteria (including tolerances) or details to be reported (if not clear in the method of test).

Testing during the project’s design and construction phases is described in Z7/01 and Z7/04. Tests to provide data for design life modelling are listed in Z7/05, and tests related to early age crack control and crack measurement are referred to in Z7/06. Table 4.1 summarises the recommended durability performance tests for mix acceptance. Additional details of these tests, and other tests that give similar data, are given later in this section. The method of using test results in modelling and the criteria applicable to test results for mix assessment are provided in Z7/01 and/or other documents.

<table>
<thead>
<tr>
<th>Common Reference</th>
<th>Parameter Measured</th>
<th>Key Testing Method Aspects</th>
<th>Applicable Exposures</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>NT Build 443:1995 Chloride penetration [Reference 98], Section 4.2</td>
<td>Effective chloride transport coefficient for mix as used in modelling</td>
<td>Result is average of 3 results from different cylinders from bulk of cylinders. Typically 15 weeks from completion of curing to actual chloride diffusion coefficient (Dc) results.</td>
<td>Severe chloride exposures, e.g. splash and tidal zone to determine acceptability of the mix at the nominated cover.</td>
<td>Results from trial mix are generally required to confirm assumed values or indicative results from NT Build 492.</td>
</tr>
<tr>
<td>NT Build 492:1999 Chloride migration test [Reference 99], Section 4.2</td>
<td>Chloride migration coefficient when ingress accelerated under an applied electrical field</td>
<td>Result is average of 3 results from different cylinders from bulk of cylinders. Typically 3 weeks from completion of curing to actual migration coefficient results.</td>
<td>Intermittent QA.</td>
<td>Used to give an early indication of chloride migration (diffusion) coefficient (results need correlation factor). Can be used as intermittent QA test once calibrated against NT Build 443.</td>
</tr>
<tr>
<td>fib 34:2006 Carbonation rate test [Reference 1], Section 4.3</td>
<td>Depth of pH change due to ingress of CO₂</td>
<td>Carbonation accelerated by use of high CO₂ content. Test exposure condition modified to Australian temperate conditions. Typically 4 weeks exposure after curing.</td>
<td>Where environment is not dry enough to restrict corrosion rate and: (a) CO₂ levels are high, (b) concrete quality is not clear (e.g. new materials), or (c) cover is low.</td>
<td>Results can be used in a model that allows for other aspects of exposure (e.g. wetting) to give the depth of carbonation with time. Results can also be used to compare different mixes.</td>
</tr>
</tbody>
</table>

Table 4.1: Recommended mix acceptance tests
<table>
<thead>
<tr>
<th>Common Reference</th>
<th>Parameter Measured</th>
<th>Key Testing Method Aspects</th>
<th>Applicable Exposures</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASTM C1585:13 Water sorptivity test [Reference 18], Section 4.4</td>
<td>Rate of water absorbed in a standard environment</td>
<td>Conditioning of sample to a standard (not oven dried) environment. Typically 4 weeks to complete (excluding presaturation stage).</td>
<td>In any severe exposure where ingress of contaminants by capillary absorption is a risk.</td>
<td>Used where a rate of absorption is required for modelling or where absorption is not deemed adequate to define performance for durability assessment.</td>
</tr>
<tr>
<td>AS 1012.21:1999 Volume of permeable voids [Reference 12], Section 4.4</td>
<td>Water absorption test that is likely to saturate all pores</td>
<td>Boiled absorption of standard sized samples. Typically 2-3 weeks to complete.</td>
<td>The test can be used for any exposures as different criteria are set for different concrete grades.</td>
<td>Used extensively in Victoria by VicRoads to show that concrete mixes of specified characteristics will give a minimum level of durability.</td>
</tr>
<tr>
<td>Water permeability test, Section 4.5</td>
<td>Water transport under a pressure head</td>
<td>Ensure no leakage around edges of sample. Allows for measurement of flow at different times.</td>
<td>Where water transport is under a pressure head.</td>
<td>Test can be specified where penetration under a high pressure gradient is a concern.</td>
</tr>
<tr>
<td>Semi-adiabatic test, Section 4.6</td>
<td>Temperature rise under insulated conditions with minimal heat loss</td>
<td>1 m³ insulated block that approaches adiabatic conditions. Minimises energy interchange with the environment, e.g. solar gain and losses due to wind.</td>
<td>Elements where limits on rebar stress might not be an adequate means of crack control: (a) thick sections (&gt;500 mm) (b) high heat output concrete (c) elements with high restraint.</td>
<td>Used to calculate adiabatic temperature rise, not as a direct indication of in-situ temperature rise. Include strain measurement to give coefficient of thermal expansion when required. Used wherever early age thermal cracking or excessive maximum temperatures are an issue.</td>
</tr>
<tr>
<td>Alkali aggregate reactivity (AAR), Section 4.7</td>
<td>Various aspects to different tests</td>
<td>Process of determining testing requirements is defined in AS HB79 [Reference 63]. Rapid test on mortar bars takes approximately 23 days after manufacture of specimens. Concrete prism testing takes 3-24 months depending on test method.</td>
<td>Areas where sufficient moisture is available to fuel reaction with potentially reactive aggregates.</td>
<td>The experience in assessing results is as important as the test method. Test methods and in-situ aggregate performance vary from state to state and expertise on local conditions should be employed.</td>
</tr>
<tr>
<td>Common Reference</td>
<td>Parameter Measured</td>
<td>Key Testing Method Aspects</td>
<td>Applicable Exposures</td>
<td>Comments</td>
</tr>
<tr>
<td>-------------------</td>
<td>--------------------</td>
<td>---------------------------</td>
<td>----------------------</td>
<td>----------</td>
</tr>
<tr>
<td>Delayed ettringite formation (DEF), Section 4.8</td>
<td>Concrete mix composition is reviewed combined with a concrete peak temperature criterion</td>
<td>Study review as a reliable test method does not exist (see Section 4.8).</td>
<td>DEF may occur if concrete temperature during early stages of curing exceeds about 70 ºC and the concrete is subsequently kept in damp or wet service conditions. Temperature up to 80 ºC is allowed for concrete with specific proportions of GGBFS or FA.</td>
<td>The risk of DEF is assessed by reviewing the concrete mix composition, predicting the concrete peak temperature (see Section 4.6) and specifying a maximum concrete peak temperature.</td>
</tr>
<tr>
<td>Sulfate resistance, Section 4.9</td>
<td>Cement system is assessed and combined with a penetrability requirement</td>
<td>Expansion of mortar bars stored in sulphate solution for 16 weeks minimum. No specific requirement given to concrete penetrability.</td>
<td>Most mild-moderate sulfate exposures but not more extreme exposures (e.g. acid sulphate soil conditions).</td>
<td>Further work is required to define test requirements for use of uncoated concrete in more extreme sulfate exposures.</td>
</tr>
<tr>
<td>AS 4456.9:2003 Abrasion test [Reference 139], Section 4.10</td>
<td>% volume loss when concrete surface subjected to impact and the rolling action of steel ball bearings</td>
<td>Equipment is not commonly available. Test takes about a week including preconditioning of specimens.</td>
<td>For general application, strength is basis of acceptance. Test is specifically designed for segmental pavers but use has been extended.</td>
<td>Calibration of test is required before extending to application not previously assessed. Does not relate to actual wear rates in service. Results are strongly influenced by finishing and curing processes.</td>
</tr>
<tr>
<td>BS EN 13892-4:2002 Abrasion resistance [Reference 180], Section 4.10</td>
<td>Depth of wear due to rotating wheels.</td>
<td>Equipment is not commonly available. Test is carried out on in-situ concrete.</td>
<td>For most slabs, specification of concrete by strength grade is adequate. Use BS EN 13892-4 where risk of failure due to abrasion is moderate or above.</td>
<td>Consider limitations found in real implications of test results to actual abrasion resistance. Interpretation of results can be difficult where impact damage arises from wheels “skipping” over exposed coarse aggregate particles.</td>
</tr>
</tbody>
</table>
4.2 CHLORIDE PENETRATION

4.2.1 General

Ingress of chloride ions is a common cause of reinforcement corrosion. Hence resistance to chloride ion penetration is an important measure of concrete durability, particularly in marine environments.

In saturated concrete, the mechanism of chloride ingress is by diffusion through saturated pores, voids and cracks. In concrete that is not permanently saturated, the mechanism is a combination of absorption of water into partly dry concrete, concentration of chlorides when the concrete dries, and diffusion through saturated pores.

Although it does not account for cyclic wetting and drying processes, chloride diffusion is relatively easy to measure accurately and model, therefore it has become a parameter often used in modelling. Consequently, chloride diffusion tests are frequently used in specifications for concrete in marine and coastal structures.

4.2.2 Chloride diffusion tests

While test methods have been developed that effectively measure chloride diffusion, there is still great debate about how these results should be applied. Tests that give the most precise estimate of chloride diffusion take a long time to complete. The original “steady state diffusion” (SSD) method of measuring chloride diffusion involved placing a sample in a cell with a chloride solution on one side and a solution with no chlorides on the other. The chloride concentration in the receiving cell was measured over time until a constant chloride ion concentration was achieved. The chloride diffusion coefficient was then calculated. However, the diffusion coefficient could take six months or more to measure, and require many chloride analyses. Because of the slow rate of increase in chlorides in the receiving side of the cell, it is easy to assume a steady state has been reached when it has not. This can lead to reporting of optimistic diffusion coefficients.

Rapid tests have been developed whereby the diffusion of chlorides is driven by a potential difference. The property measured under these conditions is often described as chloride migration rather than diffusion. The relationship between the results from rapid tests and the longer term tests is determined by the pore water chemistry of the individual concrete, which in turn is determined by the cement and SCM chemistry, batch water supply and admixtures. Therefore there is no universal relationship between results from each type of test, although calibration curves can be determined for individual concretes if required.

A range of test methods also exist for measuring or inferring the sample’s chloride ion diffusion coefficient. These include “non-steady state diffusion” (NSSD) and “non-steady state migration” (NSSM) methods. The resistance to chloride ingress is typically expressed as the chloride diffusion coefficient. Because the test result is influenced by other ions present in the concrete, the result is often referred to as the apparent diffusion coefficient.

Table 4.2 compares the various chloride penetration test methods and provides an outline of the test method set up. It also provides comparative results for a sample concrete using the various tests. These are a general comparison for one concrete and should not be taken to apply to all concretes. However, they indicate the order of magnitude of results, and demonstrate clearly the difference between steady-state and non-steady state test results.

More details on chloride penetration test methods can be found in references such as Stanish et al [Reference 43], Tang and Sørensen [Reference 46], Lane [Reference 44], Vivas et al [Reference 47], Narsilio et al [Reference 45] and Peek et al [Reference 6].

In Australia, the NordTest NT Build 443 [Reference 98] method is commonly used to test and pre-qualify concrete mixes for use in chloride-bearing environments. An alternative standard, ASTM C1556-11a [Reference 50], is based on NT Build 443. Although there are procedural differences between the methods, any effect on the test results has not been published at the time of writing, the two methods are expected to produce similar results for practical purposes.

Both methods require appropriate curing of concrete, NT Build 443 [Reference 98] requires “28 maturity days” while ASTM C1556-11a [Reference 50] requires “28 days of laboratory standard moist curing”. This has implications for the “age at test”, because the standard European laboratory curing temperature is 20 °C (e.g. NT Build 201 [Reference 101] is 20 ± 4 °C), compared to 23 ± 2 °C as in the USA and Australia and 21 +/- 2 °C in NZ.
<table>
<thead>
<tr>
<th>Test Method</th>
<th>Test Set Up</th>
<th>Equation</th>
<th>NaCl/NaOH</th>
<th>Volts</th>
<th>Time (t)</th>
<th>$D_c \times 10^{-12}$ m²/s</th>
<th>Precision (CoV %)</th>
<th>Mechanism</th>
<th>When to use</th>
</tr>
</thead>
<tbody>
<tr>
<td>Classical Membrane Cell Test</td>
<td></td>
<td>Fick’s 1st Law</td>
<td>1.2 M/0.3 M</td>
<td>—</td>
<td>150-300 d</td>
<td>0.61</td>
<td></td>
<td>Measures the steady state diffusion coefficient.</td>
<td>With the development of the other tests to measure diffusion coefficient, this test is considered to be obsolete. Not recommended for use.</td>
</tr>
<tr>
<td>NordTest NT Build 355</td>
<td></td>
<td>Nernst Plank</td>
<td>5%/0.3 M  200</td>
<td>5 d</td>
<td></td>
<td>0.83</td>
<td></td>
<td>This is also a steady state test but it is accelerated by application of a driving force, i.e. 200V charge. Originally developed as a rapid alternative to a classical SSD.</td>
<td>Has not been commonly used in Australia. Not recommended for use.</td>
</tr>
<tr>
<td>Nordtest NT Build 443 (ASTM C1556-11a is derived from NT Build 443)</td>
<td></td>
<td>Fick’s 2nd Law</td>
<td>2.8 M (168 g/l)/0.3 M</td>
<td>—</td>
<td>35+ d</td>
<td>13.6</td>
<td>15% according to NT Build 443</td>
<td>NSSD test. Concrete cylinders or cores exposed to sodium chloride solution for minimum of 35 days. Specimens sliced and analysed for chloride concentration. Effective diffusion coefficient in m²/s calculated based on best fit of Fick’s Second Law of Diffusion.</td>
<td>Used for prequalification of concrete mixes. High quality concrete mixes require at least 56 day exposure duration. Useful for durability modelling.</td>
</tr>
<tr>
<td>NordTest NT Build 492</td>
<td></td>
<td>Nernst Plank</td>
<td>10%/0.3 M 200-1200</td>
<td>6hr to 96 hr (usually 24 hr)</td>
<td>16.6</td>
<td></td>
<td>9% according to NT Build 492</td>
<td>NSSM test. Slices from concrete cylinders or cores placed in migration cell and electrical field applied. Depth of chloride penetration measured by colourimetric method and migration coefficient in m²/s is calculated.</td>
<td>Quicker than NT Build 443. Results affected by binder composition and admixtures due to their effects on pore water chemistry. Only used to derive “diffusion coefficients” when calibrated to individual mix.</td>
</tr>
<tr>
<td>AASHTO T259 (ASTM C1543-10a is derived from T259)</td>
<td></td>
<td>Average chloride concentration from several 12 mm slices at specified depths. Profile affected by sorptivity, vapour transmission and diffusion.</td>
<td>3%/ -</td>
<td>—</td>
<td>90 d</td>
<td>13.6</td>
<td>13.6 if sorption and vapour effects minimal</td>
<td>Salt Ponding of dried samples. Slabs exposed to sodium chloride solution for at least 90 days and up to 12 months or more. Powder samples at specific depths collected and analysed for chloride concentration.</td>
<td>Can require exposure greater than 90 days to achieve measurable results.</td>
</tr>
</tbody>
</table>

"Time" is the duration under test and does not include curing time or time required for chemical analysis of concrete slices after exposure.
The diffusion coefficient will change with time as the concrete matures, therefore the age at which the test is carried out will determine the results. Thus apparent diffusion coefficients determined at early ages do not represent ultimate diffusion rates, particularly for concretes containing fly ash or slag.

Curing is followed by exposure to a chloride solution for at least 35 days. For a reliable result, the exposure period must be long enough for a measurable chloride ion concentration profile to develop. For high performance concretes, particularly those containing SCM, exposure for 56 days or more is necessary to give time for sufficient chloride ions to penetrate.

NordTest NT Build 492 [Reference 99] also requires appropriate curing of the samples prior to test by reference to NT Build 201 (cylinders) [Reference 101] and NT Build 202 (cores) [Reference 102]. The test involves driving chloride ions through the concrete under an applied potential difference and measuring the depth of penetration after a specified time (6-96 hours depending on current passed with applied voltage) determined by the concrete quality.

The rapid chloride permeability test (RCPT), originally AASHTO T277 [Reference 48] and later ASTM C1202 [Reference 49], is a misnomer as it doesn’t give an indication of chloride diffusion directly. It was originally developed as a quick test to indicate the chloride diffusion coefficient, but it has subsequently been found to be primarily a measure of electrical resistivity. Internationally the test is used primarily for quality control, but is being displaced in favour of resistivity tests. ASTM has also introduced a bulk electrical conductivity test for hardened concrete, ASTM C1790 [Reference 100] as a rapid indicator of chloride ion diffusion resistance as measured by ASTM C1556-11a [Reference 50]. These rapid tests are not discussed here, but are referred to in Section 5.7 (Electrical resistivity).

4.2.3 Recommendations

NT Build 443 [Reference 98] or ASTM C1556-11a [Reference 50], modified in terms of a specific sample curing regime and to be consistent with standard Australian laboratory practice, is recommended.

The “modified NT Build 443 or ASTM C1556 method” is as follows:

- Curing of concrete cylinders to AS 1012.8.1 [Reference 95] to 28 days. This curing applies only to Portland and blended Portland cement concretes. The curing of other non-Portland cement binders is to be separately specified.
- At the age of 28 days, the specimens shall be exposed to water containing sodium chloride in accordance with NT Build 443 or ASTM C1556 at 23 ± 2 °C.
- The period of exposure to sodium chloride solution shall be 56 days typically.
- For core samples they shall be kept wet until delivery to the test laboratory. They must be representative of the concrete and/or structure in question and the concrete must be hardened to a minimum of 28 maturity-days.

There are no universal acceptance criteria for this test, as the required value is calculated based on modelling analysis.

The following two-step process can be followed when specifying the chloride penetration resistance:

- Establish a mean diffusion coefficient that gives the design life required by diffusion modelling and any allowances considered appropriate to provide a margin for error. It is essential that model values are reviewed by an experienced professional in terms of what can reasonably be expected to be achieved in practical concrete production prior to incorporation in a works (project) specification, otherwise there is a risk of specifying impractically low values that cannot be achieved in practice.

- Specify the maximum diffusion coefficient to be achieved, or note the desired value and require that the actual value be reported. It is recommended that a specified value only be set where local suppliers have experience in meeting the proposed specified value. If local suppliers do not have the necessary previous experience in making this type of concrete then they will need to carry out trial batching to demonstrate they can produce this type of concrete with satisfactory consistency.

4.2.4 Atmospheric chloride content environmental assessment

In many cases, the environmental chloride load can be assessed using the guidelines in AS 3600 and AS 5100.5, categorising the exposure conditions as A1,
A2, B1, B2, C1 or C2 accordingly. If a more detailed assessment of local exposure conditions at the site is required, rates of salt deposition can be estimated by methods such as a "wet candle method" or "coupon exposure". These methods are considered beyond the scope of this document at this time.

### 4.3 CARBONATION RATE

#### 4.3.1 Introduction

Carbonation is the reaction between atmospheric carbon dioxide (CO₂) gas and cement hydration products dissolved in the concrete pore water, in particular calcium hydroxide. Carbonation reduces the pH of concrete pore solution from 13 to less than 9. This change destabilises the passive oxide film on the surface of the steel reinforcement, allowing the reinforcement to corrode.

Under normal atmospheric conditions the deemed-to-comply concrete grade and cover requirements of AS 3600 [Reference 177] and AS 5100.5 [Reference 178] will generally provide adequate protection. Because carbonation alone rarely causes significant corrosion, carbonation rate tests are not generally undertaken. However, carbonation rate testing is sometimes required, for example when:

- A mix does not perform well in general quality tests and the actual carbonation resistance of the mix is required.
- Where non-standard materials are used and their performance has to be specifically assessed.
- Where the in-situ carbonation rates are high and/or the design life is in excess of 100 years, and a specific assessment of the concrete needs to be used to determine cover requirements.

Testing for carbonation rate is relatively simple. Concrete samples are cured for an appropriate time and are then placed in an airtight tank, refer Figure 4.1. The carbonation rate is accelerated by using a higher than normal CO₂ content in the tank while temperature and humidity are held at a standard level. The CO₂ content must not be so high that it affects the chemical reactions that occur, or the morphology of the reaction products. When the results are used in modelling, allowance is made for the effects of actual exposure conditions compared to those in the tank.

Although carbonation rate tests have been undertaken in Australia using different tank conditions, none have been standardised. Extensive research on carbonation was undertaken by CSIRO in the 1980s, involving both accelerated and field exposure tests (e.g. Reference 105). The accelerated conditions thus developed (25 °C, 50% RH, 4% CO₂) were empirically correlated with exterior exposure in Melbourne to give an approximately 50-fold acceleration, and therefore have some direct relevance to Australian conditions.

Accelerated test methods have been proposed in Europe and the UK. An “Accelerated carbonation test” is detailed in fib Bulletin 34 [Reference 1]. A draft test method was developed by CEN and has been evaluated by researchers including Harrison et al [Reference 106] and Jones et al [Reference 107]. An accelerated test method is under development in the form of prBS 1881-131:2011 [Reference 108]. NordTest NT Build 357 [Reference 109] also describes a methodology for accelerated carbonation testing.

#### 4.3.2 Recommendations

The “Accelerated carbonation test” as detailed in fib Bulletin 34 [Reference 1], modified to be consistent with standard Australian laboratory practice, is recommended.

In the “fib method”, the carbonation rate is accelerated by testing at a carbon dioxide concentration of 2% which is approximately 50 times the normal atmospheric CO₂ concentration but not so high as to change the reaction mechanisms or the morphology of the reaction products. After curing, concrete samples are exposed for 28 days. Thus, ignoring other factors the carbonation depth achieved during the test will be achieved in-situ in approximately

![Figure 4.1: Typical carbonation tank where CO₂ level is controlled by injection of an air/CO₂ mixture and tank air/CO₂ mixture is re-circulated through a temperature and humidity controller](image)
four to five years (depending on wet/dry cycles in-situ). The benefits of the “fib method” are:

- The buffering capacity of the concrete does not have to be considered separately.
- Changes of the carbonation resistance due to carbonation do not have to be considered additionally.
- Good reproducibility of the test results.
- Short duration. Usually the factor that determines the duration of carbonation testing is the time required to condition the test specimens to a specified constant relative humidity prior to starting the carbonation phase.

The “modified fib method” (to be consistent with standard Australian laboratory practice) is as follows:

- Production of concrete prism specimens of dimensions 100 x 100 x 350 mm as used for AS 1012.8.2 [Reference 96] standard flexural strength test.
- After removing the formwork, the specimens are stored in water at 23 ± 2-3 °C for six days.
- After water storage, the specimens are stored for 21 further days at 23 ± 2-3 °C and 60 ± 5% RH.
- At the age of 28 days, the specimens are placed in a carbonation chamber at 23 ± 2-3 °C, 60 ± 5% RH. In the chamber the specimens are exposed to a CO2 concentration of 2.0 ± 0.2 vol. % for 28 days. CO2 concentration may be increased up to 4% and additional exposure periods such as 7, 56, 90 and 112 days added if required, e.g. to obtain more reliable data or enable comparison with existing data. However, CO2 concentrations must not be elevated beyond 4% because this can affect the nature of the carbonation reaction products.
- After removal, the concrete specimens are split longitudinally and the carbonation depth is measured at the plane of rupture with an indicator solution consisting of 1.0 g phenolphthalein per litre (dissolved in a 50:50 methylated spirits/distilled water mixture). The phenolphthalein solution will stain un-carbonated concrete magenta, while carbonated concrete will not be stained (see also Section 7.4.2).
- The inverse carbonation resistance factor, \( R_{\text{ACC},0}^{-1} \), is calculated as:
  \[
  R_{\text{ACC},0}^{-1} = \left( \frac{x_c}{\tau} \right)^2 \text{ units are } x 10^{-11} \text{ (m}^2\text{/s)/(kg/m}^3\text{).}
  \]
  Where \( x_c \) = mean carbonation value from the test in metres
  \( \tau \) = time constant = 420 (from fib Bulletin 34).

CIA Recommended Practice Z7/05 provides further details of the modelling procedure using the “fib 34 method”.

### 4.4 WATER ABSORPTION AND SORPTIVITY

#### 4.4.1 Introduction

Water absorption and sorptivity are measures of concrete pore volume (porosity) and structure as indicated by the volume and rate of uptake of water by capillary suction respectively [Reference 5]. Solvent absorption can also be used as a measure of the capillary porosity of concrete [Reference 6].

Permeability is also a measure of pore structure, but measures the influence of pore volume and connectivity on the rate of transport of water applied under pressure. Permeability tests are described in Section 4.5.

Absorption tests measure the total weight gained by concrete immersed in water. Absorption tests focus on filling the voids in concrete without reference to the rate at which this occurs, although some tests might only partially fill the voids if the duration of the absorption period is fixed. Results can be used as mix acceptance tests, but they cannot be used for modelling. Absorption test results are often referred to as the “volume of permeable voids (VPV)” or the “apparent volume of permeable voids (AVPV)”. Absorption values are expressed as percentage void space by volume of sample, or less commonly as the mass of absorbed water as a percentage of the dry mass of sample.

In contrast, Sorptivity tests measure the rate of water absorption. The results can be used in formulae for assessing the depth of penetration of water with time under capillary action. Sorptivity is measured by placing concrete in contact with water with no pressure head and measuring weight gain with time to obtain a sorptivity value. The corresponding visible height rise is often also measured; however, this is a less accurate method for calculation of a sorptivity value. The result can be used for modelling of water ingress due to capillary rise, and can also be used as a quality assurance test. Sorptivity values are calculated as...
volume absorbed per unit surface area related to the square root of time, e.g. $\text{mm}^3/\text{mm}^2/\text{min}^{1/2}$ which is often reduced to $\text{mm}/\text{min}^{1/2}$.

Both absorption tests and sorptivity tests are widely used in Australia for mix acceptance and quality assurance. Absorption tests are simple, therefore preferable where data for modelling is not required, while sorptivity tests are used where modelling of water ingress is required.

Table 4.3 summarises the various test methods in common use in Australian concrete specifications. It includes information on the repeatability of the test method where available, typically as the coefficient of variation (CoV). Table 4.3 does not include certain tests such as the water absorption tests in AS 4058:2007 [Reference 110], AS 4198:1994 [Reference 111], or AS/NZS 4676:2000 [Reference 112]. These are tests specified for particular precast concrete products, and are not used outside of relevant manufacturing standards [Reference 6].

Since compressive strength monitoring has a long track history, and is routinely performed for all grades of concrete regardless of other parameters that might be specified for a given project, it is sensible to use the same statistics for tests related to concrete porosity when using them to assess compliance [Reference 6]. Modern computer controlled concrete batch plants can achieve CoVs in the order of 4-6% for compressive strength values taken over a significant number of production batches. This level of consistency can be used as a minimum value when evaluating water absorption and sorptivity test results, although in practice the actual variability of the water absorption and sorptivity tests may be higher due to the inherently greater complexity of the test methods.

<table>
<thead>
<tr>
<th>Common Reference</th>
<th>Test Type</th>
<th>Drying Temp</th>
<th>Precision (CoV %)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Absorption Tests</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>AS 1012.21</td>
<td>Essentially an Australian adaptation of ASTM C642-90, with minor procedural changes. Also known as the “Apparent volume of permeable voids” test or “AVPV” test.</td>
<td>105 ºC</td>
<td>No precision statement in standard</td>
<td>Measures the AVPV as a percentage of the volume of the bulk materials, i.e. solid and voids. A standard VicRoads “durability criterion” used on road projects in Victoria.</td>
</tr>
<tr>
<td>ASTM C642</td>
<td>Oven dry sample of defined size at least 24 hrs and to weight change tolerance, immerse in water not less than 48 hrs and to weight change tolerance, boil 5 hrs and cool not less 14 hrs in water. Measures the volume of permeable voids (VPV) as a percentage of the solid.</td>
<td>105 ºC</td>
<td>No precision statement in standard</td>
<td>Values of absorption and density calculated at different’s stages. Popular in the late 1990s, now sometimes superseded in Australia by AS 1012.21, or often by quicker and less expensive BS 1881:Part 122 test.</td>
</tr>
<tr>
<td>BS 1881 Part 122</td>
<td>Oven dry samples of specified geometry for 72 hrs, equilibrate at standard room temperature for 24 hrs, immerse in water for 30 mins and calculate water absorption from mass gain ($\Delta m$).</td>
<td>105 ºC</td>
<td>5-5.5% [Reference 6]</td>
<td>Essentially a 30 minute single-point sorptivity test, often colloquially referred to as the “BSAT” test.</td>
</tr>
<tr>
<td>Common Reference</td>
<td>Test Type</td>
<td>Drying Temp</td>
<td>Precision (CoV %)</td>
<td>Comments</td>
</tr>
<tr>
<td>------------------</td>
<td>-----------</td>
<td>-------------</td>
<td>-------------------</td>
<td>----------</td>
</tr>
<tr>
<td>Sorptivity Tests</td>
<td>“Taywood/ GHD/SGS Sorptivity” test</td>
<td>Oven dry to constant weight, expose one planar face to water and measure uniaxial water absorption by weight gain at intervals over 4 hrs. Assessment by linear regression analysis of $\Delta m$ vs $\sqrt{t}$. Tests can be completed in 7 days.</td>
<td>105 ºC</td>
<td>5.2-16.8% [Reference 6]</td>
</tr>
<tr>
<td></td>
<td>ASTM C1585-13</td>
<td>Dry under specified conditions for 3 days, equilibrate at standard room temperature for at least 15 days, seal non-exposed surfaces and measure uniaxial water absorption by weight gain at specified intervals over 9 days. Assessment by linear regression analysis of $\Delta m$ vs $\sqrt{t}$. Tests can be completed in 14 days.</td>
<td>50 ºC 80% RH</td>
<td>6.0%</td>
</tr>
<tr>
<td></td>
<td>RTA T362</td>
<td>Dry prism samples in specified controlled environment up to 35 days, immerse in water up to 24 hrs, measure depth of water penetration by dye stain method.</td>
<td>23 ºC 50% RH</td>
<td>No precision statement available</td>
</tr>
<tr>
<td></td>
<td>BS 1881 Part 208</td>
<td>Initial surface absorption test (ISAT). Measure rate of flow into defined area at 200 ± 20 mm head pressure at intervals over 1 hour. Geometry means that non-uniaxial flow occurs, making comparison of results to other methods complex.</td>
<td>N/A</td>
<td>Not known</td>
</tr>
<tr>
<td></td>
<td>BS 1881 Part 124</td>
<td>Solvent absorption to saturation into oven-dry sample, initially at reduced pressure. Calculate porosity from weight gain. Part of original free water content determination in standard.</td>
<td>105 ºC</td>
<td>Not known</td>
</tr>
</tbody>
</table>

Notes:
1. Water absorption tests are not suitable for concretes containing water repellents.
2. Water sorptivity tests are suitable for concretes containing water repellents.
Table 4.4: AVPV performance assessment criteria (VicRoads Specification 610 on structural concrete [Reference 13])

<table>
<thead>
<tr>
<th>Concrete Grade</th>
<th>Max AVPV at 28 days (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Vibrated Cylinder</td>
</tr>
<tr>
<td>VR330/32</td>
<td>14</td>
</tr>
<tr>
<td>VR400/40</td>
<td>13</td>
</tr>
<tr>
<td>VR450/50</td>
<td>12</td>
</tr>
<tr>
<td>VR470/55</td>
<td>11</td>
</tr>
</tbody>
</table>

4.4.2 AS 1012.21 Volume of permeable voids

AS 1012.21:1999 [Reference 12] is an Australian adaptation of ASTM C642-06 [Reference 113]. In the AS1012.21:1999 test method cylinders or cores are cut into four equal slices so that several results are obtained from one test specimen. The slices are oven dried to a constant weight then immersed and boiled in water in order to saturate the concrete’s permeable voids and thus give a measure of the total pore volume.

Extensive work by VicRoads [Reference 65] has enabled them to assess the potential durability of concrete, as indicated by AVPV results, for concrete of different grades to ensure a high reliability that the mixes will be durable in the exposure for which they are intended. Criteria developed for VicRoads are shown in Table 4.4.

Whiting [Reference 66] compared AVPV results measured using ASTM C642-06 [Reference 113] with results from water permeability and air permeability on mixes with different water/binder (w/b) ratio and a mix with silica fume. The results showed that AVPV differentiated better than air permeability between the concretes at w/b ratios less than 0.4 but neither method differentiated between concretes at w/b ratios over 0.4. In comparison, water permeability differentiated the high w/c ratio concretes but measurements at low w/c ratios could not be made.

Andrews-Phaedonos [Reference 65] reported on the relationship between AVPV and various concrete properties as follows:

- Relationship to strength is poor, reflecting that strength is a poor indicator of durability.
- AVPV detects the improved performance of slag, fly ash and silica fume in a similar fashion to other durability tests.
- Some admixtures reduce AVPV very significantly while others did not reduce AVPV significantly.

While CC&AA [Reference 5] notes that there is a poor relationship between some absorption tests and chloride diffusion, others [Reference 65] quote a reasonable relationship between AVPV and chloride diffusion and propose AVPV as possibly the best overall assessment method for durability performance.

4.4.3 Taywood/GHD/SGS sorptivity test

The Taywood sorptivity test [Reference 69], refer Figure 4.2, was originally a brick test. Recognising the significance of the rate of water ingress due to capillary action, as opposed to tests which only measure the volume of pores, the original brick test was adapted for concrete in the late 1970s. The test is mentioned in Concrete Society TR31:1988 [Reference 69] but was never issued in a standard.

The test has continued to be used by many people who had developed an understanding of concrete quality through the extensive use of this method. A significant quantity of tests have been conducted on Australian concrete using the method named as Taywood Sorptivity, GHD Sorptivity 2001 to 2008 and SGS Sorptivity since 2008.

4.4.4 ASTM C1585 sorptivity test

The ASTM C1585-13 [Reference 117] test is relatively simple to undertake. Although it takes slightly longer than the Taywood sorptivity test and with shorter history of usage in Australia, it gives results that are more representative of in-situ concrete, and is an international standard.

The fact that it is more representative of in-situ concrete is significant for modelling, which is a major advantage over alternative tests.
4.4.5 **RMS T362 sorptivity test**

In the RMS (formerly RTA) T362 water sorptivity test [Reference 16], 100 x 100 x 350 mm samples are pre-conditioned at 50% RH and then are placed under 50 mm of water for 6-24 hours before being broken open in flexure using a beam test rig and the depth of water penetration measured. As the water head is low the results are a measure of the concrete capillary suction.

This method is similar in many respects to the test developed at CSIRO [Reference 105].

The RMS and CSIRO methods are seldom used commercially because each test specimen yields only one result, the test specimens are relatively large, and many specimens are required if results at different test ages are required.

The RMS T362 water sorptivity test [Reference 16] can be used in different ways depending on the application of the concrete, (Table 4.5). It is used by RMS as an acceptance method where contractors decide to deviate from deemed-to-comply curing requirements, and as a specified requirement in various exposures for bridges, refer Table 4.5.

4.4.6 **Recommendations**

1. Where a general test of concrete quality is required for mix assessment or quality control it is recommended that the AS 1012.21 [Reference 12] AVPV test is used because it is already standardised and guidelines for interpretation of test results have been published, albeit for specific concrete mix designs.

2. Where a sorptivity rate is required for use in modelling, the ASTM C1585 [Reference 117] test should be specified. This is because its method of specimen preconditioning means its results will more closely reflect the in-situ sorptivity.

3. As the RMS sorptivity test offers no...

### Table 4.5: RMS T362 water sorptivity test criteria

<table>
<thead>
<tr>
<th>Exp. Class</th>
<th>Deemed-to-comply or prove curing</th>
<th>Sorptivity Depth (mm)</th>
<th>Used for specification of durability performance e.g. RMS B80</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Deemed to Comply Curing (days test)</td>
<td></td>
<td>Max (mm) for GP Cement</td>
</tr>
<tr>
<td>A</td>
<td>7 (7)</td>
<td>45</td>
<td>35</td>
</tr>
<tr>
<td>B1</td>
<td>7 (7)</td>
<td>35</td>
<td>25</td>
</tr>
<tr>
<td>B2</td>
<td>9 (14)</td>
<td>17</td>
<td>17</td>
</tr>
<tr>
<td>C</td>
<td>14 (21)</td>
<td>11</td>
<td>N/A</td>
</tr>
</tbody>
</table>
advantages over the ASTM C1585
[Reference 117]. Z07/07 recommends that
in the interest of industry-wide consistency,
that industry endeavours to develop ASTM
C1585 as an eventual replacement for RMS
sorptivity.
4. AS 4198 [Reference 111], AS 4676
[Reference 112] and AS 4058 [Reference
110] are specific to quality control for
particular manufacturing processes and
should not be used for purposes other than
those for which they are intended. Neither
should they be adapted for other purposes,
instead the tests described above should be
used.

4.5 WATER PERMEABILITY

4.5.1 Introduction

The penetration of water into concrete under a
pressure head (permeability) is an important durability
performance parameter for concrete exposed to water
pressure. The coefficient of water permeability is also a
very sensitive indicator of durability for other exposure
conditions, as the factors controlling permeability also
control other penetration modes.

The coefficient of permeability or hydraulic
conductivity is defined by Darcy’s Law, and has the
units of length/time (typically m/s).

There are no Australian Standard test methods
for measuring the coefficient of water permeability of
concrete. Some test methods include ASTM D5084-10
[Reference 120], Main Roads Western Australia Test
Method WA 625.1 [Reference 121], US Army Corps of
Engineers CRD-C 48-92 [Reference 122] or US Army
Corps of Engineers CRD-C 163-92 [Reference 123].
These methods differ in the type of permeameter used,
which can be either rigid or flexible wall. The methods
are similar in principle and all are suitable for measuring
water permeability.

A common method used for measuring water
permeability in Australia is the method developed by
Taywood Engineering, refer Figure 4.3. The method was
published as an in-house procedure and was included
[Reference 69] and TR31:2008 [Reference 70].

Testing for coefficient of water permeability
by any method requires appropriate equipment for

specimen preparation and the test itself. The test can
take several days to complete.

DIN 1048.5 [Reference 124] gives a test method
for measuring water penetration of concrete under
a constant head of water. However, the results are
measured in terms of depth of penetration rather than
coefficient of permeability. The distorted hemispherical
flow pattern through the concrete means the depth of
penetration cannot be easily converted to permeability.

Table 7 summarises the concrete water permeability
test methods.

The coefficient of water permeability for good
quality concrete is typically of the order of 10-12 m/s
or lower. The coefficient of variation associated with
laboratory tests for coefficient of water permeability is
usually around 10% [Reference 6].

4.5.2 Recommendations

The Concrete Society TR 31 water permeability
test [References 69 and 70] is a simple test and
can be undertaken using equipment that is easily
manufactured. Results using this method have been
compared with tests using the MRWA 625.1 method
and results were found to be comparable [Reference 6].
Hence it is recommended that this method is specified
for water permeability testing of concrete.
4.6 CIA Z7/07 SEMI ADIABATIC TESTS TO PREDICT CONCRETE ADIABATIC TEMPERATURE RISE

4.6.1 Semi-adiabatic concrete temperature monitoring methods

The chemical reaction that takes place when cement is mixed with water is exothermic. The volume of the concrete will expand and contract as it heats up and then cools back down to the ambient temperature. If the concrete is restrained (unable to move freely) in certain locations, this expansion and contraction may result in the concrete cracking. Drying shrinkage that starts after effective curing ceases will add to the risk of concrete cracking. The concrete’s early age temperature will also determine the risk of delayed ettringite formation (DEF, see Section 4.8) and strength development (i.e. high early concrete temperature may result in lower ultimate strength).

Models are used during design to predict concrete peak temperature, maximum temperature differentials, concrete strain and concrete crack risk, refer to CIA Z7/06.

To predict the in-situ concrete temperature of a concrete pour, the heat generating capacity of the concrete must be known. Historically, adiabatic temperature rise (i.e. the temperature rise of a piece of concrete if there was no heat loss) was measured directly by placing a concrete sample in an insulated chamber and measuring the heat input required to maintain it at the same temperature as the concrete pour it represents. However, this test is available from a limited quantity of materials testing laboratories, equipment is not commonly available for site testing and it is logistically more difficult to arrange at a laboratory by comparison with on-site or at a concrete batch plant. Therefore, adiabatic temperature rise tests are possible but they are rarely conducted except for research purposes. Instead, semi-adiabatic temperature rise is more commonly measured and the data obtained is used to predict the concrete adiabatic temperature.

To calculate thermal strains, the coefficient of thermal expansion of the concrete will need to be used. Values can be assumed based on aggregate type; however, actual values can vary significantly from one source to another for the same aggregate type. Hence,
when conducting semi-adiabatic temperature rise tests, it is recommended that embeddable vibrating wire strain gauges (VWGs) are used to measure heat-up and cool-down strain, noting use of a conservative coefficient of thermal expansion based on published data might have a significant effect to modelling outcomes. This data is used to derive actual concrete thermal coefficients of expansion and contraction.

The most reliable approach to predict the adiabatic temperature rise from a semi-adiabatic test is to test concrete using the same mix constituents and proportions as in the concrete pour. In addition to mix proportions, the adiabatic temperature rise is significantly influenced by the binder type (e.g. proportions and composition of ordinary Portland cement, ground granulated blast furnace slag, flyash and silica fume) and the aggregate type (e.g. granite, limestone, basalt, etc.).

A semi-adiabatic test based on a 1 m$^3$ block of concrete has been commonly used for 25+ years [References 79 and 125]. The concrete temperature is measured by thermocouples. Vibrating wire strain gauges (VWGs) can be installed for strain measurement that allows the concrete coefficient of expansion and contraction to be calculated. The approach is also known as the “hot box” test or “thermal block” test.

There is no Australian Standard for measuring the semi-adiabatic temperature rise; however, generally the sample consists of a 1.0 x 1.0 x 1.0 m cube of unreinforced concrete. The block (Figure 4.4a) is cast within 18 mm plywood formwork, which is lined on all interior surfaces with 100 mm-thick polystyrene to allow free expansion and contraction of the concrete and to minimise temperature losses from conduction, convection and radiation. It is important that the polystyrene be inside the plywood as this reduces errors due to the energy absorption of the plywood. The box is typically placed on two 100 mm deep timbers to reduce ground effects. A polythene sheet 0.2 mm thick is generally placed on the inside of the polystyrene to prevent leakage through joints in the polystyrene and plywood. The assembly should be sheltered from sun, wind and rain to minimise temperature variations outside of the box.

The concrete semi-adiabatic temperature and ambient temperature are measured and an allowance for heat loss made to predict the concrete adiabatic temperature rise. Two measurement approaches have been adopted in Australia as described below:

(a) Semi-adiabatic temperature monitoring option 1 by one concrete position method [Reference 78]: In this test method a thermocouple is located at the centre of the box (Figure 4.4b), which has been found to enable an accurate assessment of adiabatic temperature rise by appropriate allowance of heat loss. A strain gauge with thermocouple can be used instead of just a thermocouple where the coefficient of thermal expansion is required.

(b) Semi-adiabatic temperature monitoring option 2 by multiple concrete positions method [Reference 77]: The concrete monitoring is completed at multiple concrete positions to detect any heat loss and the positions can be selected to coincide with a

---

Figure 4.4: Typical “hot box arrangement” and monitoring option 1 by one concrete position method [Reference 78]
calculation method to predict the adiabatic temperature by Ng et al [References 79, 80] with details at Section 4.6.2. This approach has been used successfully in Australia by Jong et al [Reference 77] and Paull & Jong [Reference 78] using the hot box in Figure 4.4a. The concrete block contains two horizontal, perpendicular placed, lengths of threaded rod at mid-height that are sleeved with PVC pipe to minimise any impact on VWGs placed in the middle. The VWGs have integral temperature sensors suitable to measure concrete early age temperature and strain. The strain gauges are attached with zip-ties and polystyrene spacer blocks to prevent the strain gauge platens from contacting the PVC sleeves.

Probes that can accurately measure early age concrete temperature (e.g. thermocouples or thermistors) are placed to record temperature at recommended positions as shown on Figure 4.5:

- Centre of the box, \( T_m \) (an extra probe approximately 75 mm directly underneath \( T_m \) position will provide a back-up given this is a critical measurement location).
- Mid-depth, in middle of face, 50 mm in from face, \( T_f \).
- Mid-depth, on an edge, 50 mm in from adjacent faces, \( T_e \).
- Top-corner, 50 mm in from 3 adjacent faces, \( T_c \).
- Middle of top face, directly above \( T_m \), 50 mm in from the top face, \( T_t \).
- Air ambient temperature \( T_a \), 300 mm away from the box.

The thermocouples are positioned using a wooden dowel and all other thermocouples are positioned using the threaded rod. Figure 4.5 shows the VWG and thermocouple positions in the thermal block. The strain and temperature data is recorded by a multi-channel data logger.

Whichever measurement method is used subsequent concrete placing and reporting requirements are the same. The concrete is poured and vibrated as required, avoiding impact with the gauges. Lifting hooks are installed to allow the concrete block to be moved following completion of monitoring and the polystyrene lined lid is fitted.

The following information should be reported for each test:

- Mix design identification, mix design and actual batched records provided by the concrete supplier to record the specific concrete tested. A minimum of 3 m\(^3\)

**Figure 4.5:** Design of semi-adiabatic temperature monitoring option 2 by multiple concrete positions method [Reference 77]
should be batched.
- Location the test was conducted.
- Shelter provided for the test sample.
- Time and temperature of batched concrete taken with the compressive strength samples.
- Compressive strength test at 3, 7, 28 and 56 (for blended cement types only) days.
- Time at commencement of filling concrete into the box and time the lid is placed.
- Concrete temperature, concrete strain and air ambient temperature readings for 60 minutes prior to pour, at time of pouring and then continue at 30 minute intervals until 7 to 10 days after the peak temperature.
- Name and type of all probes used and the datalogger.
- Name of people who conducted the test.

### 4.6.2 Predicted concrete adiabatic temperature

The data generated by the semi-adiabatic temperature rise method requires calculations to predict the adiabatic temperature rise to compensate for heat loss that is dependent on the specimen size, ambient conditions, energy absorbed by the mould and the heat insulation provided. One calculation method is provided by Ng et al [References 79 and 80]. Prior to calculation methods being developed, a method used was a curve-estimation drawn on a graph of the semi-adiabatic temperature rise test data by people with concrete thermal behaviour experience.

CIRIA C660 [Reference 125] details the assessment of thermal expansion and cracking, including a spreadsheet that automatically provides an adiabatic temperature rise based on concrete mix designs and cementitious binders. The spreadsheet was developed specifically for United Kingdom binders. It should not be used for any other binders without appropriate modification for the specific binder product to be used. Even the performance of the same binder product may vary with time, therefore for large projects and for update information on local binder performance, thermal behaviour is recommended to be assessed whenever the binder product, type or composition changes.

An example of adiabatic temperature rise prediction from a semi adiabatic test, plus CIRIA C660 [Reference 125] estimation is shown in Figure 4.6. The S50 mix for low-heat (LH) cement type with ground granulated blast furnace slag in 2014 behaves more like a GP cement probably due to the specific clinker and the grind which gave high 7 day compressive strengths. The adiabatic temperature rise of the concrete mix is very different to other S50 concretes with LH cement and CIRIA C660 [Reference 125] predictions for the mix, which illustrates the importance of local cement testing.

When a semi-adiabatic test is not conducted for a project, the adiabatic temperature rise would need to be estimated by a person with appropriate local experience in concrete thermal behaviour who can justify the prediction.

**Figure 4.6: Adiabatic temperature rise development for S50 concrete**

![Adiabatic temperature rise development for S50 concrete](image-url)
4.7 **ALKALI-AGGREGATE REACTION**

Alkali-aggregate reaction (AAR) refers to the chemical reaction between alkalis released by hydrating cement and certain silica and carbonate minerals that can be present in aggregates. Various different tests are available for assessing the reactivity of aggregates prior to selection for use in concrete. These include petrographic examination of the aggregate, chemical analysis and mortar bar or concrete prism expansion tests based on standard mix designs. For mortar bar and concrete prism tests, expansion limits have been set to indicate the reactivity of the aggregate under test. Different AAR test methods can give contradictory results. There is ongoing research to improve test methods so that they better predict field performance of aggregates. However, interpretation must be based on local knowledge about the alkali reactivity of similar aggregates in the specific test used and in concrete structures.

Table 4.7 summarises laboratory test methods available in Australia for assessing AAR. Further information on the suitability and limitations of test methods can be found in relevant references such as Carse and Dux [Reference 23], Shayan [Reference 28], Standards Australia HB79 [Reference 63], Guirguis and Clarke [Reference 27], Shayan and Morris [Reference 26], Thomas et al Reference 24], Lindgård et al [Reference 25] and Ideker et al [Reference 29]. HB79 [Reference 63] summarises current Australian best practice.

For Australian aggregates, Z7/07 recommends the following tests:

- **Mortar Bar Expansion**: AS 1141.60.1 [Reference 190].

Where previous use or petrographic tests do not provide sufficient evidence that the aggregates will be acceptable it is recommended that this test is undertaken to determine the degree to which the aggregate is reactive.

The RMS (RTA) Test Method T363 [Reference 37]; VicRoads RC 376.03 [Reference 41] or Main Roads Western Australia Test Method WA 624.1 [Reference 39] are sufficiently similar that results are comparable. Hence they might be used as an alternative to the AS 1141.60.1 method but this is not recommended as it leads to a perpetuation of unnecessary multiple tests and this is likely to lead to industry confusion.

ASTM C1260-07 [Reference 34] is based on the South African NBRI test [Reference 128], and can give different results to the AS 1141.60.1 test because the early-age curing regimen is quite different. Hence this method is not recommended.

- **Concrete Prism**: AS 1141.60.2 [Reference 191], RMS (RTA) Test Method T364 [Reference 38], VicRoads RC 376.04 [Reference 42] or Queensland Department of Transport and Main Roads Q458 [Reference 40].

Where petrographic or mortar bar expansion tests are inconclusive these tests can be used to provide specific evidence as to whether a proposed concrete mix and cement system will have acceptable resistance to AAR. These methods are similar but differ in detail and are not necessarily directly comparable. Usage varies by state and the individual methods were developed along similar lines but with variation designed to detect perceived differences in aggregate reactivity. The Q458 test [Reference 40] is covered by a patent, so can only be performed by a laboratory with a licence from the patent holder. As aggregate reactivity does vary from state to state it is recommended that the common local test is undertaken to assess concrete until a more universal test and criteria are developed.

Results of accelerated testing can occasionally be misleading, and more than one test method might be needed to resolve the potential AAR risk. For example, in WA a widely used aggregate that is classified “potentially reactive” by petrography, gives “highly reactive” expansions by mortar bar test, but is classified “innocuous” by concrete prism and has 40+ years ASR problem free track record in marine and water retaining structures. Thus wherever possible, the specific tests used should be those upon which knowledge about reactivity of local aggregates is based, and local experience should be utilised in interpreting test results.

Whichever test methods are used, the art of the testing is in designing a test programme that best meets project needs, rather than simply applying a pass/fail criterion associated with a particular test.
method. For example, with use of appropriate control mixes, test results can be evaluated by comparison with aggregates or concrete with known site performance. Laboratories or consultants experienced in this type of testing will be able to advise on a suitable approach for a given application.

Because significant variations in mineralogy can exist within quarries, it is recommended that the material be characterised by petrography annually, and AAR testing be repeated at a frequency that reflects changes in the source material. For some sources this may be as often as yearly.

4.8 DELAYED ETTRINGITE FORMATION

Ettringite is a normal early product of Portland cement hydration at ambient temperatures and does not damage the concrete when formed in this way. But if the temperature of the concrete exceeds about 70 °C during the early stages of curing, for example in mass concrete, or in accelerated curing, a different hydration product forms. Once normal ambient temperatures are restored, it will convert back to ettringite if sufficient water is available. This process is known as Delayed Ettringite Formation (DEF). Ettringite takes up more space than the original hydration products, and so the conversion generates internal stress, which, like ASR, can be enough to crack the concrete. Cracking may take 20 or more years to appear, but can also be rapid and dramatic. Like other forms of cracking, it can increase the risk of secondary forms of deterioration such as reinforcement corrosion by allowing ingress of aggressive agents.

Reported cases have included precast railway sleepers, and precast piles immersed in water. In these cases, accelerated heat curing is likely to be the cause.

DEF has also been reported in mass concrete, where heat of hydration has elevated the early age concrete temperature.

The risk of DEF is determined primarily by the early age curing temperature, although some cement compositions and aggregate types will further increase the risk [References 85, 86, 87, 88 & 129]. It is very sensitive to small changes in chemical and physical conditions so is often localised. This leads to inconsistent observations that make its cause difficult to identify [Reference 130].

AAR is also induced by high early age curing temperatures. It is often associated with DEF, and there has been considerable debate about the relationship between the two reactions. Evidence suggests DEF is unlikely to cause damage unless preceded by AAR (or other mechanisms that produce microcracking in which the ettringite can crystallise), although it can occur by itself if all high-risk factors are present [Reference 131].

DEF can almost always be prevented by limiting the maximum peak concrete hydration temperature to less than 70 °C during initial early age. This can be achieved directly via the project specification. Concrete with specific proportions of ground granulated blast furnace slag or flyash can allow the initial early age peak concrete hydration temperature up to 80 °C.

There is no reliable concrete test to evaluate the risk of DEF. Thus the risk of DEF is assessed by reviewing of the concrete mix composition, predicting the concrete peak temperature (see Section 4.6) and specifying a maximum concrete peak temperature.

4.9 SULFATE RESISTANCE

Concrete is often exposed to sulfates in ground water or in industrial chemicals. Commonly used standard test methods use mortars of standard proportions for the purpose of benchmarking cement performance. The sulfate resistance of concrete is assessed in three ways:

(a) For compliance with concrete durability requirements in the Australian Standard, sulfate resistance is obtained by using the specified cover, concrete grade and a sulfate resisting cement (AS 3972 Type SR). The cements’ sulfate resistance is measured in accordance with AS 2350.14:2006 [Reference 132], in which standard mortar bars are immersed in a 5% sodium sulfate solution. Expansion for Type SR cement is limited to 750 microstrain at 16 weeks as defined in AS 3972:2010 [Reference 133]. The test criteria is deemed to take account for the variance of the test method when the required number of samples are tested.

(b) The cements’ sulfate resistance might not adequately describe the sulfate resistance of the concrete proposed. Sirivivatnanon and Lucas [Reference 134] investigated methods of assessing concrete resistance to sulfates over a three year exposure period, and proposed two testing criteria when samples
<table>
<thead>
<tr>
<th>Common Reference</th>
<th>Exposure</th>
<th>Time</th>
<th>Test Type</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Petrographic</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>AS 1141.65</td>
<td>None</td>
<td>Hours</td>
<td>Microscopic examination of aggregates using polarised light.</td>
<td>Identifies potentially reactive components in aggregates. Requires a skilled petrographer for accurate assessment. Can be subjective. Best used in conjunction with mortar bar or concrete prism expansion tests such as AS 1141.60.1/ASTM C1260 and/or AS 1141.60.2/ASTM C1293.</td>
</tr>
<tr>
<td>ASTM C295-12</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Mortar bar expansion</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ASTM C227-10</td>
<td></td>
<td>3-6 mths</td>
<td>Up to 2 years for reliable results</td>
<td>Inability to detect slow reactivity, long duration and excessive leaching of alkalis. Unreliable.</td>
</tr>
<tr>
<td>38 °C, high humidity</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ASTM C441-11</td>
<td></td>
<td>&gt;1 year</td>
<td>To determine effectiveness of supplementary cementitious materials. High alkali cement is used in matrix and crushed Pyrex (borosilicate) glass is used as aggregate.</td>
<td>Pyrex glass chemistry differs from natural aggregates and test data does not necessarily represent performance of mixes with natural aggregates. Variability in sources of glass affects results. Reactivity and alkalis in glass, in addition to severe limits, might give false positives. Unreliable.</td>
</tr>
<tr>
<td>ASTM C1260-07</td>
<td></td>
<td>&gt; 14 days</td>
<td>Up to two months. Water/binder material ratio = 0.47.</td>
<td>Useful as screening test. Best used in conjunction with other tests such as ASTM C1293 due to false negatives/positives.</td>
</tr>
<tr>
<td>ASTM C1567</td>
<td></td>
<td></td>
<td></td>
<td>Similar method to ASTM C1260. Used to evaluate mixes containing pozzolans or blast furnace slag.</td>
</tr>
<tr>
<td>RMS T363 &amp; VicRoads RC376.03</td>
<td></td>
<td></td>
<td></td>
<td>Specimen preparation as per ASTM C1260 Clause 7. Similar to RMS Test Method T363 but with different expansion limits and additional classifications for reactivity of fine aggregate.</td>
</tr>
<tr>
<td>&gt; 21 days</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MPWA WA 624.1</td>
<td>Acceleated in 1 N NaOH at 80 °C</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Common Reference Exposure Time Test Type</td>
<td>Comments</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>----------------------------------------</td>
<td>----------</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Petrographic Examination</td>
<td>Requires a skilled petrographer for accurate assessment. Can be subjective. Best used in conjunction with mortar bar or concrete prism expansion tests such as AS 1141.60.1/ASTM C1260 and/or AS 1141.60.2/ASTM C1293.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mortar bar expansion</td>
<td>Inability to detect slow reactivity, long duration and excessive leaching of alkalis. Unreliable.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Accelerated Concrete Expansion Test in 1 N NaOH at 80 °C</td>
<td>Up to two months. Water/binder material ratio = 0.47. Useful as screening test. Best used in conjunction with other tests such as ASTM C1293 due to false negatives/positives.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Performance Test Method AS 1141.60.2</td>
<td>Modified ASTM C1293. Cement content 420±10 kg/m³ and NaOH added to mixing water to give total alkali content of 0.9% Na₂O equivalent. Water/cement ratio 0.42-0.45. The principal modification is to allow the use of cement with lower alkali content as obtaining Australian cement with an alkali content of 0.9%, as used in ASTM C1293, is difficult.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>AS 1141.60.1</td>
<td>Modified ASTM C1260. Classification limits for the test have been drawn from local methods based on Australian experience of ASR. The method has been modified to include procedures, based on Australian and international research, intended to improve repeatability and reproducibility.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Aggregate</td>
<td>Crushed aggregate immersed in sodium hydroxide solution at elevated temperature. Amount of dissolved silica and reduction of alkalinity is measured.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ASTM C289-07</td>
<td>None Hours</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Classification limits for the test have been drawn from local methods based on Australian experience of ASR.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete Prism</td>
<td>Use high alkali cement. Minimum cement content equivalent to 420 kg/m³ and NaOH added to mixing water to give cement alkali content of 1.38% Na₂O equivalent. Water/binder ratio 0.42-0.45. Long duration. Can also be used to test effectiveness of supplementary cementitious materials over test duration of two years.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ASTM C1293</td>
<td>38 °C, high humidity</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RMS T364 &amp; VicRoads RC 376.04</td>
<td>1 year</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Qld Main Roads Q458</td>
<td>&gt;4 months</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>AS 1141.60.2</td>
<td>38 °C, 100% RH</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Minimum cement content equivalent to 420 kg/m³ and NaOH added to mixing water to give cement alkali content of 1.38% Na₂O equivalent.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Inability to detect slow reactivity, reported poor reliability and false negatives, operator-sensitive. Aggressive test conditions not necessarily representative of field conditions. Does not account for contribution of other phases in aggregate that might affect reactions. Unreliable for Australian aggregates. Results have been found to correlate more reliably with the in situ performance of highly reactive fresh volcanic aggregates such as rhyolites and andesites found in NZ.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Concrete prism test. Similar to ASTM C1293 but with higher alkali content. Shown to have good correlation with field performance for Queensland aggregates. Discussed further in Carse and Dux [Reference 23].</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Minimum cement content equivalent to 420 kg/m³ and NaOH added to mixing water to give cement alkali content of 1.0% Na₂O equivalent. Specimens are steam cured prior to testing.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Minimum cement content equivalent to 520 kg/m³ and NaOH added to mixing water to give cement alkali content of 1.0% Na₂O equivalent. Specimens are steam cured prior to testing.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Modified ASTM C1293. Cement content 420±10 kg/m³ and NaOH added to mixing water to give total alkali content of 0.9% Na₂O equivalent. Water/cement ratio 0.42-0.45.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Shown to have good correlation with field performance for Queensland aggregates. Discussed further in Carse and Dux [Reference 23].</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
were exposed to 5% NaSO4 solution as follows:
- An expansion limit of 220 microstrain per year in the first 3 years and a maximum expansion in 3 years of 500 microstrain.
- 90% retention of 28 day strength.

(c) In some situations the composition or concentration of the in-situ sulfate solution might not be adequately addressed using the standard sulfate exposures. In such cases a project-specific sulfate exposure might be undertaken. Such testing should be designed and overseen by experts in relevant test procedures and should include control specimens so that the results can be assessed with direct correlation to the performance of samples of known performance. For example, comparison of performance with samples in seawater can identify if the weight or strength loss measurements are significant by comparing the results to exposure to a common environment. This helps account for the accelerating effects of sample size and geometry.

Laboratory testing of concrete for sulfate resistance is described in Xu et al [Reference 93]. Tests involve measuring the expansion of mortar bars exposed to a sulfate solution. Sulfate resistance can also be assessed by measuring the change in other physical properties of the concrete with time while exposed to sulfate, e.g. dynamic elastic modulus, ultrasonic pulse velocity or residual compressive strength.

ASTM standards for measuring the sulfate resistance of cementitious binders include ASTM C452-10 [Reference 135], ASTM C1012-13 [Reference 136] and ASTM C1038-14a [Reference 137]. ASTM C1012-13 [Reference 136] is similar to AS 2350.14:2006 [Reference 132] except that the specimen size and curing period are different. International Standards detail sulphate resisting cements differently to Australia. For example, ASTM C150-12 [Reference 184] only permits use of Portland cement (i.e. it does not include the use of SCMs) and lists:
- Type II cement as “For general use, more especially when moderate sulfate resistance or moderate heat of hydration is required”.
- Type V cement as “For use when high sulfate resistance is desired”.

ASTM C150-12 [Reference 184] is a prescriptive specification for cement and lists specific chemical composition limits for Type II and Type V including limits on C3A content of 8% and 5% which are designed to provide the required sulphate resistance. A permitted alternative to various chemical limits for Type II and Type V cements, including the C3A, is to limit 14 day expansion of 0.04% when tested in accordance with ASTM C452-10 [Reference 135]. SO3 is limited to 3.0 and 2.3% respectively, but is permitted to be higher if ASTM test C1038-14a [Reference 137] (mortar bar immersed in water for 14 days) gives an expansion of less than 0.2%.

ASTM C452-10 [Reference 135] is not suitable for testing blended cement but ASTM C1012-13 [Reference 136] can be used to verify that a blended cement will be sulphate resistant. This method is listed in ASTM C1157-11 [Reference 186] which is a performance specification for cements. In this test, mortar bars are immersed in 5% sodium sulphate for a specified period (typically 6-12 months) and if the expansion is less than a designated amount (Table 4.8) the cement is considered sulphate resistant. The test has been criticised for the time it takes and because expansion occurs predominantly at the ends of the specimen where the test pins are located.

Monteiro et al [Reference 183] lists various criticisms of ASTM C452-10 [Reference 135] and ASTM C1012-13 [Reference 136] accelerated sulphate durability tests, i.e:

### Table 4.8: Expansion limits for ASTM C1012-13 test for fly ash blended cement

<table>
<thead>
<tr>
<th>Sulfate Resistance of Blended Cement</th>
<th>Expansion (maximum) at 6 months %</th>
<th>Expansion (maximum) at 12 months %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moderate</td>
<td>0.10</td>
<td>–</td>
</tr>
<tr>
<td>High</td>
<td>0.05</td>
<td>0.10</td>
</tr>
<tr>
<td>Very high</td>
<td>–</td>
<td>0.05</td>
</tr>
</tbody>
</table>
Table 4.9: Cement sulfate resistance tests

<table>
<thead>
<tr>
<th>Common Reference</th>
<th>Test Type</th>
<th>Precision (CoV %)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASTM C 452/ C452M-10</td>
<td>Measures expansion of mortar bars based on type GP Portland cement. Mortar bars are immersed in water.</td>
<td></td>
<td>Used to determine cement compliance as a permitted alternative to otherwise prescriptive requirements of ASTM C150. Not suited to blended cements.</td>
</tr>
<tr>
<td>ASTM C1012</td>
<td>Measures expansion of mortar bars based on Type GP Portland cement and blends of Type GP Portland cement with pozzolans or slags. Specimens are immersed in 50 g/L Na₂SO₄ solution for 6-12 months.</td>
<td></td>
<td>Used to show compliance with cement performance requirements of ASTM C1157 Other sulfate solutions might be used to simulate exposure environment.</td>
</tr>
<tr>
<td>ASTM C 1038</td>
<td>Measures expansion of mortar bars of Portland cement including sulfates when exposed to water.</td>
<td></td>
<td>Used to determine limit on sulfate content in ASTM C150.</td>
</tr>
<tr>
<td>AS 2350.14</td>
<td>Similar to ASTM C1012 but test length is 16 weeks.</td>
<td></td>
<td>Other sulfate solutions might be used to simulate exposure environment.</td>
</tr>
</tbody>
</table>

Figure 4.7: Abrasion damage examples

- Too sensitive to specimen size and geometry.
- Sulphate addition not representative of field conditions.
- Tests do not account for different forms of attack at different sulphate concentrations.
- pH not representative of field conditions.

Monteiro et al [Reference 183] recommends control of pH and testing for strength loss as a means of overcoming some of these issues. As yet a standard test based on strength loss and expansion has not been produced.

Table 4.9 summarises the test methods, noting that all of the methods shown are primarily designed for benchmarking cement performance under standardised conditions.

4.10 ABRASION RESISTANCE

4.10.1 Introduction

Abrasion is potentially an issue for:
- Pavements and slabs exposed to vehicle and foot traffic, particularly where inadequate finishing and curing leads to a weak or delaminating surface layer.
- Pipelines, spillways, retained waterways, seawalls and other hydraulic structures, exposed to particulate matter carried at high flow rates and/or turbulent flow causing cavitation, refer Figure 4.7.

However, in general abrasion resistance has not been found to be an issue, provided Australian code
requirements for concrete compressive strength for different applications are achieved.

There are many abrasion tests used internationally for a variety of purposes, but most are not used in Australia. In general, the results of abrasion tests cannot be correlated directly with in-situ durability performance, therefore their application for modelling is limited to comparative testing of (for example) different mix designs and construction techniques. This is discussed for the different tests.

Test types include rotating discs under pressure, rotating steel balls, dressing wheels or blasting with sand. Abrasion resistance test methods are summarised in Table 4.10.

4.10.2 AS 4456.9 Abrasion test

The abrasion test specified for segmental pavers, AS 4456.9:2003 [Reference 139] also known as the Sydney City Council Test, is useful because acceptance criteria for different applications have been developed based on this method by correlation with in-situ performance. In this test the % volume loss after subjecting the concrete surface to impact and the rolling action of steel ball bearings for 3,600 tumbling cycles, is measured. The result is expressed as the “abrasion index”.

Table 4.11 presents criteria for selecting segmental pavers for different applications based on their abrasion resistance measured by this test.

Table 4.10: Abrasion tests

<table>
<thead>
<tr>
<th>Common Reference</th>
<th>Test Type</th>
<th>Precision (CoV %)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>AS 4456.9</td>
<td>Test for segmental pavers</td>
<td></td>
<td>Simulates wear under high impact loads with scraping. Has associated acceptance criteria.</td>
</tr>
<tr>
<td>BS EN 13892-4</td>
<td>In-situ test for floor surfaces</td>
<td></td>
<td>Depth of wear by rotating wheel measured. Has associated acceptance criteria. Also makes reference to relevance of BS 8204-2 test and criteria.</td>
</tr>
<tr>
<td>BS EN 13892-3</td>
<td>Laboratory test known as the “Bohme test”. Used for testing screed materials.</td>
<td></td>
<td>Usually relevant for dry-shake toppings used on floors. Difficult to adapt to other applications as the manufactured specimen cannot be representative of the actual floor.</td>
</tr>
<tr>
<td>ASTM C418-12</td>
<td>Sandblasting method</td>
<td>10</td>
<td>Measures loss of volume due to sandblasting. Applicable to concrete subjected to abrasive wear with dry particles</td>
</tr>
<tr>
<td>ASTM C779/C779M-12</td>
<td>(a) Revolving disk with silicon carbide abrasive</td>
<td>5.5</td>
<td>Three alternative procedures for horizontal surfaces. Procedure (a) uses sliding and scuffing of revolving steel disks in conjunction with abrasive grit. Procedure (b) uses a dressing wheel machine to give impact and sliding friction of three sets of steel dressing wheels. Procedure (c) involves high contact stresses, impact and sliding friction from steel balls and water flushing.</td>
</tr>
<tr>
<td></td>
<td>(b) Rotating dressing wheels</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(c) Steel ball abrasion with water</td>
<td>18</td>
<td></td>
</tr>
<tr>
<td>ASTM C944/C944M-12</td>
<td>Rotating dressing wheels</td>
<td>12-21</td>
<td>Suited to cores. Similar to Procedure (b) of ASTM C779.</td>
</tr>
<tr>
<td>ASTM C1138M-05</td>
<td>Steel grinding balls in water</td>
<td>14</td>
<td>Depth of wear measured. Applicable to abrasion by water-borne particles experienced by concrete underwater.</td>
</tr>
</tbody>
</table>
The method is not suitable for measuring the performance of concrete cast in-situ because abrasion resistance is largely determined by the method of construction, which cannot necessarily be replicated in laboratory-made specimens. However, the relative performance of different mixes, including the effect of aggregate, can be measured by this test.

CC&AA [Reference 144] investigated the performance of various concretes using this test and found a reasonable correlation between strength and abrasion resistance, refer Figure 4.8, but noted: The durability of the macro-texture is likely to be influenced by the surface compaction and finishing technique and the associated bleeding characteristics of the concrete. Similarly, inadequate curing is likely to reduce abrasion resistance. Hence, specifying the appropriate strength grade might not necessarily lead to an acceptable abrasion resistance and is a sound reason for some level of QA testing on critical floors.

CC&AA [Reference 144] also compared results from saw cut and off form finishes from the same mixes. Saw cut surfaces had approximately half the

<table>
<thead>
<tr>
<th>Application</th>
<th>Max Abrasion Resistance (Mean Abrasion Index)</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residential and industrial pavements</td>
<td>7</td>
<td>ACT and Municipal Services, Standard Specification for Urban Infrastructure, 2002 [Reference 147]</td>
</tr>
<tr>
<td>Low volume public footpaths, minor &amp; collector roads</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>High volume public footpaths</td>
<td>3.5</td>
<td></td>
</tr>
<tr>
<td>Public footpaths – low impact¹</td>
<td>6.0</td>
<td>NZS 3116:2002 [Reference 148]</td>
</tr>
<tr>
<td>Public footpaths – high impact¹</td>
<td>3.5</td>
<td></td>
</tr>
<tr>
<td>Public space – low traffic volume¹</td>
<td>7</td>
<td>AS/NZS 4455:2010 [Reference 149]</td>
</tr>
<tr>
<td>Public space – medium traffic volume¹</td>
<td>5.5</td>
<td></td>
</tr>
<tr>
<td>Public space – high traffic volume¹</td>
<td>3.5</td>
<td></td>
</tr>
</tbody>
</table>

¹ Refer to the relevant Standards for specific details about these applications and applications that do not have an associated abrasion resistance criterion.

Figure 4.8: Abrasion resistance vs compressive strength using AS 4456.9 Test for off form finish [Reference 144]
4.10.3 BS EN 13892-4 Abrasion test

Hulett [Reference 185] reviewed abrasion resistance of floors and in particular the BS EN 13892-4 [Reference 146] test method as follows:

The most commonly used method for testing in-situ floor surfaces is described in BS EN 13892-4 [Reference 146]. This Standard prescribes a machine, known as the BCA (British Cement Association) test, which creates a wearing process. The machine, shown in Figure 4.9, simulates a wearing mechanism by the use of three hardened-steel wheels mounted on a revolving plate. The plate revolves at a set speed for a set time under a prescribed load. The resultant annulus of wear is measured at eight points and the average depth of wear is reported to the nearest 0.01 mm.

Historically, floors have been classified in accordance with Tables 3 and 4 of BS 8204-2 [Reference 140]. It is now appreciated that those classifications are of little value as the difference between the applications related to each of the classifications is totally subjective and therefore unhelpful. It has also been shown that in practice, the difference between for example an AR1 and AR2 (see Table 4.12) floor has not been reflected in real long-term wear rates of the floor.

For UK Concrete Society Technical Report TR34 [Reference 149], it was concluded that the test method

Table 4.12: BS8204-2:2003 and concrete society TR34 chaplin abrasion tests criteria

<table>
<thead>
<tr>
<th>BS8204 Class</th>
<th>Duty</th>
<th>Type of Concrete</th>
<th>Concrete Grade N/mm³</th>
<th>Minimum Cement Content kg/m³</th>
<th>Maximum Wear Depth mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Special</td>
<td>Severe abrasion</td>
<td>Special mixes and resins</td>
<td>Special mixes and dry-shake or sprinkle finishes, resins, etc</td>
<td>0.05</td>
<td></td>
</tr>
<tr>
<td>AR1</td>
<td>Very high abrasion</td>
<td>High strength toppings</td>
<td>60-85</td>
<td>475</td>
<td>0.1</td>
</tr>
<tr>
<td>AR2</td>
<td>High abrasion</td>
<td>Direct finish concrete</td>
<td>C50</td>
<td>400</td>
<td>0.2</td>
</tr>
<tr>
<td>AR3</td>
<td>Moderate abrasion</td>
<td>Direct finish concrete</td>
<td>C40</td>
<td>325</td>
<td>0.4</td>
</tr>
</tbody>
</table>
was not useful in characterising long-term wear rates under typical warehouse use. This is to say that there are no data to show that, for example, a floor with a test classification of AR1 will have a longer service life than a floor that is compliant with AR2. However, the test was accepted as being a useful indicator of a minimum acceptable floor surface for this type of application. For that reason a maximum test limit of abrasion of 0.2 mm is required using BS EN 13892-4 [Reference 146].

TR34 [Reference 149] notes that resin-based curing compounds create a layer or “skin” on the surface of the floor that can be impenetrable to the test machine. Caution should therefore be exercised when interpreting results.

4.10.4 BS EN 13892-3 Abrasion test

Hulett [Reference 185] notes that: in parts of Europe, BS EN 13892-3:2004 [Reference 180] is commonly cited in specifications for floors. This is a laboratory test known as the Böhme test and is used for testing screed materials. It is usually cited in respect of dry-shake toppings used on floors. It is difficult to see the relevance of this test method as the manufactured test specimen used in the laboratory cannot be representative of the actual floor.

4.10.5 ASTM C779 Abrasion methods

Hulett [Reference 185] also notes that: ASTM C779M-12 [Reference 142] prescribes three methods using different machines, each of which apply a different abrasion mechanism. It can be assumed that the three different methods are intended to simulate different types of abrasive action. However, no guidance is given in this standard to suggest the appropriate method for simulating any particular use of a floor. It is understood that floors are rarely tested by any of these methods.

The rotating cutter method, however, could be used to test core samples to compare the abrasion resistance at different locations on a floor as well as the strengths of the concrete at that location.

4.10.6 Recommendations

CIA Z7/07 recommends:

- Where abrasion resistance is critical, the materials and mix be assessed using AS 4456.9:2003 [Reference 139] to ensure that the abrasion resistance expected will be in accordance with expectations for the concrete grade.
- Where the performance of an industrial pavement is critical, the specification should include BS EN 13892-4:2012 [Reference 146] tests on a trial slab to demonstrate the performance of the mix and finishing methods. Three tests should be undertaken on the trial slab and the abrasion resistance calculated as the average of the three results.

4.11 BLEED TESTS

Bleed of concrete can have a major effect on the properties of a concrete surface that determine whether it will be durable. This is discussed at length in CIA Z7/04.

It is recommended that where the bleed of a concrete mix is not known and bleed could have an adverse effect on durability (see Z7/04), that its measurement on a trial mix is included in the project specification.

For very deep pours where the concrete will be fluid over the full height at the same time, AS1012.6:1999 [Reference 150] method will not provide adequate guidance on bleed due to pressure effects. CIA Z17 [Reference 151] details appropriate test procedures that should be followed for such pours.

4.12 BETWEEN-BATCH VARIABILITY

Samples for determining durability performance for mix acceptance would usually be cast from trial mixes. Although more than one trial might be produced to demonstrate that the plant is capable of consistent supply of the proposed mix, it is unlikely that properties other than slump, air, density and compressive strength would be measured on more than one trial batch. Instead, additional durability testing may be requested as part of the project quality assurance testing (see Section 5).
5 Tests for Quality Assurance During Construction

5.1 INTRODUCTION

Testing for assurance that durability achieves expectations is undertaken throughout construction to verify durability is not compromised by unforeseen construction or material matters that may influence concrete performance. This Section outlines tests that are used for durability quality assurance during construction.

5.2 COMPRESSIVE STRENGTH

Z7/04 notes that: One of the reasons that compressive strength is such a useful tool is that it is relatively simple to do, has a well-established methodology and hence low variance and established criteria for acceptance and sharing of risk. As such it is an essential part of the durability testing suite of tests. However, it also notes that maintenance of the trial mix actual strength should be considered and provides a recommendation to that effect.

Compressive strength testing in accordance with AS 1379:2007 [Reference 7] is considered appropriate as one of the measures for assurance of durability. For non-severe exposures it might be the only concrete test required to assure durability. SCMs are commonly used for severe exposures, which reduces the correlation between compressive strength and durability. Durability tests other than compressive strength are more likely to be required when SCMs are used. However, compressive strength remains an important consistency test for durability assessment purposes.

5.3 COVER

Cover is defined as the minimum distance from the concrete surface to the closest face of the steel reinforcement.

AS 3600 [Reference 177] and AS 5100.5 [Reference 178] specify minimum cover depths for different exposure conditions and concrete grades. These form the basis for durability design. However, the intended durability will not be achieved if the cover depths are less than specified. Indeed, Marrosszeky [Reference 152] and Sirivivatanon [Reference 153] report a large proportion of cover depths being significantly less than the specified value. This could lead to durability problems becoming widespread in many structures before the end of their design service life. Therefore it is important to ensure the design cover is actually achieved.

The specified cover is achieved by fixing the steel reinforcing within the formwork and physically spacing it from the formwork using spacers or bar chairs. Z7/04 provides information on suitable bar chair types and quantity required. However, reinforcement sagging between bar chairs placed too far apart, crushing of bar chairs, and general displacement of the reinforcement cage during construction can reduce the cover achieved. Low cover on unformed items can be particularly irregular (high or low) due to the lack of a fixed surface to guide the concrete placer and the difficulty of matching top bar locations to the falls proposed on the slab.

After fixing the reinforcement and having largely completed formwork and screed rails in place the cover can be physically measured to ensure that it is within specification. This checking is particularly important as in some locations cover checking will not be practical after the concrete is placed. In these locations it also will not be possible to establish the cover distribution for residual life assessments later in the structure’s service life. Consequently, the following procedures are recommended:

(a) (i) On the first pour of each type, undertake a detailed pre-pour cover survey, completed by taking sufficient random cover measurements (i.e. a minimum of 30 measurements) to establish the mean cover and its variance in each zone where these values might differ. A set of gauge blocks that fit between the bar and formwork might be a more convenient way of measuring the cover than by tape measure (latter more difficult). On the top surface where there is no formwork, cover can be checked by measuring between the reinforcement and a string line stretched between screed rails.

(ii) The measured cover depths can be used to accept or reject the method of construction to achieve specified cover. If accepted then the data should be provided to the designer.
and contractor as a permanent record of the cover distribution expected for that type of element. Where cover is found to be deficient it is to be remedied. This process should be repeated on subsequent pours until the cover is found to be acceptable on the first assessment.

(b) Once a pour of a given type has been found to have no cover deficiencies it is reasonable to assume that the method of construction is appropriate and the pre-pour cover check on subsequent pours might be limited to a general check using a block passed between the reinforcement and formwork or reinforcement and stringline to ensure that there are no apparent areas with a lower cover than specified. The block size used would be the same as the specified minimum cover, i.e. the reinforcement spacer size less the negative tolerance on cover. The placement of concrete can move the reinforcing within the formwork. It can either be weighed down or shift because of vibration, insufficient spacers, etc.

Confirming that the correct cover has been achieved is important to ensure that the construction process is properly managed. Where errors are made during construction of the first element, catching this early can prevent the problem persisting through every element. It is in the contractors’ interest to undertake post–pour checks on early pours to ensure the construction method will provide the cover specified, rather than checking cover at the end of a project and thus risk finding all pours have a cover deficiency due to a systematic error. Through the life of the structure the cover distribution will also be important for residual life assessments. Hence the following procedure is recommended for post pour cover checks:

(a) On the first pour, measure the cover over a wide area using a non-destructive test (NDT) method as described in Section 6.2. Record the results as a mean cover and standard deviation. Then proceed as in (a) (i) above.

(b) Once a pour type is found to have an appropriate cover distribution then the cover checks by NDT can be reduced. The extent should be included in the specification as appropriate for the project, but as a minimum at 10 locations on every fifth pour.

5.4 Maturity/Temperature Matched Curing

Maturity measurement and matched curing are means of assessing in-situ concrete strength. They take account of in-situ temperatures that might be higher or lower than the standard curing temperature of cylinders. When strength gain is faster than in standard curing conditions, the tests can enable early stripping or early transfer of prestress. When strength gain is slower they can prevent stripping, loading or stressing before concrete has achieved adequate strength. In terms of durability, however, the primary interest relates to curing requirements.

It is commonly accepted that curing can cease once the concrete reaches 65% of its characteristic strength. This is demonstrated for accelerated curing in Table 5.1.

By monitoring the maturity (maturity or matched cure methods) of the cover zone concrete to determine when the concrete has reached 65% \( f'_{c} \), it may be possible to curtail curing at an early age without adversely affecting the concrete durability. Conversely, cold weather temperatures that retard strength development can result in longer curing duration.

<table>
<thead>
<tr>
<th>Exposure Class</th>
<th>Strength Requirements (MPa)</th>
<th>B as a % of A</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minimum Characteristic A</td>
<td>At Completion of Accelerated Curing B</td>
</tr>
<tr>
<td>A</td>
<td>25</td>
<td>15</td>
</tr>
<tr>
<td>B1</td>
<td>32</td>
<td>20</td>
</tr>
<tr>
<td>B2</td>
<td>40</td>
<td>25</td>
</tr>
<tr>
<td>C</td>
<td>50</td>
<td>32</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Exposure Class</th>
<th>Strength Requirements (MPa)</th>
<th>B as a % of A</th>
</tr>
</thead>
<tbody>
<tr>
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<td>Minimum Characteristic A</td>
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</tr>
<tr>
<td>B1</td>
<td>32</td>
<td>20</td>
</tr>
<tr>
<td>B2</td>
<td>40</td>
<td>25</td>
</tr>
<tr>
<td>C</td>
<td>50</td>
<td>32</td>
</tr>
</tbody>
</table>

Table 5.1: Compressive strength listed in clause 4.5 of AS 5100.4

5:2
Performance Tests to Assess Concrete Durability
ASTM C1074-11 [Reference 154] details maturity measurement. The major limitations of the maturity method are:

(a) The concrete must be maintained in a condition that permits cement hydration.
(b) The method does not take into account the effects of early-age concrete temperature on the long-term strength.
(c) The method needs to be supplemented by other indications of the potential strength of the concrete mixture.

The accuracy of the estimated strength depends, in part, on using the appropriate maturity function for the particular concrete mixture. There are a number of commonly used maturity calculation methods, namely:

<table>
<thead>
<tr>
<th>Method</th>
<th>Equation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arrhenius maturity</td>
<td>$M = \sum_{0}^{t} e^{-\frac{k}{R} \left[ \frac{1}{273+T_c} - \frac{1}{273+T_r} \right]} \cdot \Delta t$</td>
<td>This method accounts for nonlinearity in the rate of cement hydration. The Arrhenius method yields a maturity index in terms of an “equivalent age”, which represents the equivalent duration of curing at the reference temperature that would result in the same value of maturity as the curing period for the given average temperature.</td>
</tr>
<tr>
<td>Nurse-Saul maturity</td>
<td>$M = \sum_{0}^{t} (T_t - T_e) \cdot \Delta t$</td>
<td>The Nurse-Saul maturity function is the sum of the average temperature for the time interval minus the datum temperature multiplied by the time interval of interest.</td>
</tr>
</tbody>
</table>

Temperature matched curing involves storing cylinders in a container in which the temperature is controlled to match the temperature of the in-situ concrete the sample represents. The cylinders are tested for strength at various ages after casting, until the specified strength is attained.

It is recommended that:

(1) Specifications base the required curing time on the strength of concrete in the cover zone and allow curing to be curtailed when the strength reaches 65% $f'_c$.
(2) Maturity or matched curing be permitted by specifications as a means of establishing the in-situ strength of concrete.
(3) Stripping and stressing times in cold weather (°C hours in-situ less than standard curing °C hours) be based on maturity showing that the required in-situ strength has been met.
(4) Alternatively, match curing may be used to measure the actual strength of the in-situ concrete at specified ages after placement.

5.5 CRACKS

Cracks in concrete are often inevitable. The impact of cracks on durability can be a contentious matter for all stakeholders during asset design, construction and operation. Therefore, all stakeholders should have awareness of potential crack formation, location and size. The concrete crack risk should be proactively evaluated during design, and any unexpected cracks evaluated reactively. The importance of concrete cracks is described in Z7/06.
Not all concrete crack widths that exceed the limit used in design calculations and/or stated in specifications will need to be repaired. For example, some cracks may exceed the design and/or specification crack width at the surface but the crack may not penetrate to any great depth. Hence a pragmatic approach to cracking is required.

If cracks form, investigation is required to determine whether they are acceptable or should be repaired. While the design crack widths may be a guide, the actual cracks require specific evaluation of their likely significance over the design life. The cause and significance of cracks formed should be evaluated using technical publications on concrete cracks such as CIA Z15 [Reference 81], Z7/06, CIRIA C660 [Reference 125], Concrete Society Technical Report TR 22 [Reference 82] and ACI 224.1R-07 [Reference 83].

Measuring crack width at the concrete surface to determine if the crack exceeds a given limit is not as straightforward as it seems because the width varies along the crack length. In addition, the crack width changes with concrete temperature (i.e. cracks close as the temperature increases) and moisture content (cracks close as concrete moisture content increases).

Crack measurements (width, depth and orientation) during construction will be used together with other information to determine responsibility for the crack formation. For example, plastic cracks are usually related to construction practice but early age restrained thermal and drying shrinkage cracks might be design influenced or due to construction not being in accordance with the specification. Whoever is responsible, cracks that are clearly near or over the limit of acceptability, will need to be measured so that they can be assessed objectively. Measurement is generally based on preparing a crack map and measuring the crack width at various points. A representative number of measurements must be taken including additional positions near maximum crack widths. The broken top corner arris of a crack must be identified separately to the crack width. Cracks can quickly taper with depth, and hence depth of crack should also be established to determine whether the crack presents a significant corrosion risk. Repairs to cracks that are still opening might fail and hence it might be important to measure crack movement. A selection of crack measuring instruments is shown in Figure 5.1.

When the crack width is to be measured it is recommended that:

a) The width values should be reported as maximum for each crack. A mean and distribution (with variance) can then be assessed for comparison with the design values. The mean and variance values require taking sufficient measurements to obtain a statistically representative sample (e.g. 30 measurements are likely to be sufficient). Planning for crack measurements is critical as the quantity needs to consider the number and length of cracks on specific structure elements or portions of large sized elements.

b) A measuring magnifier be used to give an accuracy of ±0.05 mm. Crack width meters are suitable for first estimates of crack width (e.g. to assess which cracks need to be measured) but accuracy is only about ±0.1 mm so they are not suited to more accurate assessments. With an appropriate crack width distribution the characteristic crack width can be assessed and used as the crack width.

c) Crack widths should be measured during the coolest part of the day when they have a maximum width. This will make decision making conservative. The impact of cooler weather crack widening needs to be considered when measurements are taken in hot weather conditions.

d) Concrete core samples of 50 mm diameter can be taken through representative crack positions to the interior reinforcement depth to evaluate the crack depth and width beneath the concrete surface. This approach is recommended where the concrete crack width measured at the surface is a concern or is non-compliant. When the core is removed from the structure, the loss of restraint from the surrounding concrete may result in widening of the crack, which requires evaluation for the specific crack and the measured crack width impact on durability, structural adequacy and contract requirements.

![Different types of crack measuring equipment](image)

**Figure 5.1: Different types of crack measuring equipment**

- **Crack width meter**
  - 10 reduction scales
  - Up to 5 mm width

- **Measuring magnifier**
  - 8 x magnification
  - Up to 15 mm width
  - Accuracy ± 0.05 mm

- **Crack monitor**
  - Horizontal and vertical scales
  - For flat areas and corners
  - Accuracy ± 0.10 mm
  - Fix with glue or screws

- **Deformation meter**
  - Cracks are measured at various times
  - Accuracies ± 0.001 mm to ± 0.1 mm
  - Analogue or digital display
  - 5 mm max. crack width
Performance Tests to Assess Concrete Durability

recommended that:

(a) The width values should be reported as maximum for each crack. A mean and distribution (with variance) can then be assessed for comparison with the design values. The mean and variance values require taking sufficient measurements to obtain a statistically representative sample (e.g. 30 measurements are likely to be sufficient). Planning for crack measurements is critical as the quantity needs to consider the number and length of cracks on specific structure elements or portions of large sized elements.

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(c) Crack widths should be measured during the coolest part of the day when they have a maximum width. This will make decision making conservative. The impact of cooler weather crack widening needs to be considered when measurements are taken in hot weather conditions.

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Measuring microscopes are useful for more precise work such as in laboratory research, but their accuracy is often greater than the crack movement and the measuring process is too slow to be practical for use on construction sites.

5.6 IN-SITU TEMPERATURE AND STRAIN MEASUREMENTS

Site measurement of concrete temperature is used to determine the concrete maximum temperature and differential temperature for comparison with technical specification requirements for compliance and design predicted cracks (i.e. predictive calculations of crack risk and likely positions for cracks to form). Site measurement of strain is used to determine actual restraint within the concrete (i.e. restraint factor and differential strain) and presence of any internal cracks. Temperature and strain measurements, where required, are normally taken on representative initial cast concrete elements. The test results are used to interpolate the expected temperature and strain for the remaining concrete pours on the project, plus the need for any additional testing. Design crack risk assessment can be re-evaluated with the initial construction feedback data to minimise the risk of any unexpected high concrete temperatures or unexpected cracks during the project.

The position of thermocouples will be selected to determine the concrete peak temperature and maximum differential temperature for structural components as directed by the designer and/or durability consultant. Similarly strain gauges will be positioned to determine maximum restraint and any likely internal cracks.

An example of site temperature monitoring positions is listed below.

- Centre of the thickest section of the element, where heat loss to the environment will be slowest.
- Top, centre and bottom of the pour at locations where the edge effects for temperature loss will be minimal.
- Side of the pour at mid-height and well away from other sides.
- Near a corner where heat loss to the environment will be highest, and thus the largest temperature differential to the centre of the pour will exist.
- Ambient temperature.

In the case of the top, bottom, side and corner
measurements the thermocouple or thermistor can be tied to the front face of reinforcement so that measurements are effectively made at the depth of cover. Alternative depths of the thermocouple from the surface up to 100 mm may be used, which is the typical maximum depth to monitor surface effects. The reinforcement also provides a degree of protection of the sensor from damage during concrete placement. The thermocouple end sensor should not be in direct contact with the reinforcement to avoid an electrical and thermal connection.

Monitoring at 30 minute or hourly intervals is typically satisfactory. Measurements are continued after stripping the formwork until the concrete temperature complies with the specified requirements, which will vary depending on the concrete minimum dimension.

Strain monitoring uses embeddable type vibrating wire strain gauges (VWGs), which must be the type appropriate for concrete early age movement (i.e. there are many strain gauge products and only specific products are acceptable). The strain test positions are selected where design crack assessment indicates high strain levels or a significant probability of internal cracking. For example, positioned adjacent to construction cold joints to allow verification of restraint factors, differential strain development between pours and crack formation.

The concrete site testing of temperature and strain is complementary with the testing to determine the concrete adiabatic temperature outlined at Section 4.6.

5.7 ELECTRICAL RESISTIVITY

Electrical resistivity of unsaturated concrete primarily reflects moisture content. Electrical resistivity of saturated concrete is influenced by the nature of the cement system and in particular the use of SCMs, certain admixtures and w/b ratio. Changes to binder composition and admixtures may not have a significant effect on strength, and hence resistivity is a useful quality assurance test on the concrete mix. There are two common types of resistivity test:

- Tests that are designed to measure resistivity, such as the 4 probe Wenner resistivity test.
- Tests that were designed to measure chloride ingress rate but in fact measure resistivity, e.g. rapid chloride permeability test, RCPT (i.e. AASHTO T277 [Reference 48] and ASTM C1202-12 [Reference 49]).

Both types of test are listed in Table 5.2 but as RCPT is a complex test compared to resistivity it is only included for completeness. Z7/07 recommends that this test not be included in specifications.

The four probe Wenner method of measuring concrete resistivity was developed for soils investigation and is used extensively to assess the resistivity of different soil layers. In this method four probes are spaced at equal distances apart. A current is passed between the outer probes and the voltage generated is measured between the inner two. In the adaptation of the test for concrete the probes are placed closer together than in soil tests. Based on the idealised potential field created, the resistivity of the concrete can be calculated, see Figure 5.2.

The probe spacing can be any distance. In concrete it is generally the cover zone concrete that is of concern, and a probe spacing that will not be influenced by reinforcement is desired. Initial work

\[ \rho = \frac{2\pi a V}{I} \]

\( \rho \) = Resistivity
\( a \) = Probe spacing
\( V \) = Potential difference between inner probes
\( I \) = AC current passed between outer probes

Figure 5.2: Schematic of 4 probe resistivity equipment
30 years ago focused on a probe spacing of 50 mm such that the probe length was 150 mm. This could sometimes not be fitted within a rebar grid and hence narrower probe spacing has been used (i.e. 30 mm). As a universal measuring device a spacing of 30 mm is preferred but this is more influenced by the surface concrete and hence might be more susceptible to low readings caused by bleed or surface drying for example. It will also be susceptible to small defects and the effects of larger aggregates. AASHTO TP95-11 [Reference 155] recommends a spacing of 1.5" or 38 mm. Different models of test equipment offer different probe spacings.

The comparison of results of resistivity measurements from different probe spacings and specimen sizes (i.e. cylinder lab sample to on-site measurement) means that correction factors are required to convert these readings.

<table>
<thead>
<tr>
<th>Common Reference</th>
<th>Test type</th>
<th>Precision (CoV %)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>AASHTO TP95-11</td>
<td>4 probe Wenner surface resistivity test, designed for use on cylindrical specimens or in situ concrete</td>
<td>The single operator CoV of a single test result = 6.3%. Two tests by the same operator on concrete samples from the same batch should not differ by more than 21%.</td>
<td>This test is a direct measure of electrical resistivity, which is one of the factors that affect potential reinforcement corrosion rate. Although the method is increasingly used in the US to indicate resistance to chloride ion penetration, it can only be used to monitor chloride penetration resistance if calibrated against chloride ion diffusion tests on the same concrete. It will, however, readily detect changes in concrete composition.</td>
</tr>
<tr>
<td>ASTM C1760-12</td>
<td>Standard test method for bulk electrical conductivity of hardened concrete. It uses the same cell as ASTM C1202-12, and the same specimen type, though specimen sides do not need to be sealed and both upstream and downstream cells are filled with 3% NaCl. Current passed is measured 1 min after applying a 60 V potential difference across the cell, and conductivity calculated from specimen dimensions, V and I.</td>
<td>Not Known</td>
<td>Conductivity can be related to ASTM C1556 apparent chloride diffusion via references given in the test method.</td>
</tr>
<tr>
<td>ASTM C1202-12</td>
<td>Chloride penetration accelerated by electrical charge. Results reported in terms of charge passed (coulombs) and rated on scale from negligible to high.</td>
<td>Repeatability 12.3% Reproducibility 18% [Reference 66]</td>
<td>Often referred to as RCPT, these tests were developed as a rapid test for measuring chloride ion penetration. Widely used in USA. Has been used in Australia for monitoring concrete quality during construction (e.g. Queensland). It is essentially a complex and long resistivity test.</td>
</tr>
<tr>
<td>AASHTO T277</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 5.2: Types of resistivity test
A set of typical resistivity results on concrete cylinder samples is shown in Table 5.3.

Carrying out this test as part of a mix evaluation or to assess variation in concrete during a construction project is simple. The test is fast and non-destructive and the samples already collected for standard 7 and 28 day age compressive strengths are suitably and consistently conditioned. Adding a resistivity check before crushing has a very low cost. The sheer volume of samples and hence data produced is itself a strong argument for such an approach, which is being used increasingly by US state road authorities.

When used as a check on concrete variation, there are no established criteria for the test results. It is the variation in resistivity that is significant and the variation should only be used as an indication that there might be something untoward with the concrete that requires further investigation.

Temperature has a significant effect, therefore laboratory tests must be undertaken at a standard temperature (e.g. 23 ± 2 °C). Moisture content also has a significant effect, and hence samples should be tested for resistivity within 15 minutes of removal from the water bath. If core samples are to be tested, they should be preconditioned to a saturated state in accordance with AS 1012.14:1991 [Reference 97].

For structures in severe exposures it is recommended that:

1. 4 probe Wenner resistivity be measured in accordance with AASHTO TP95-11 [Reference 155] or equivalent to determine the mean resistivity for the concrete based on results from multiple batches with tests on three cylinders from each batch.

2. 4 probe Wenner resistivity be measured in accordance with AASHTO TP95-11 [Reference 155] or equivalent on all 28 day age compressive strength tests undertaken. If resistivity trends up and results fall to 20% below the average result in the trial mix, then the designer and/or durability consultant should consider what actions should be undertaken to bring the mix back to the performance of the trial mix.

### 5.8 CROSS HOLE SONIC LOGGING

Cross hole sonic logging is used to assess voids and cave-ins in bored piles and diaphragm walls as either of these defects might reduce the final durability of the pile. Testing involves measuring the time of flight of an ultrasonic wave between a transmitter and receiver lowered to the same height down different tubes. The tubes are installed during construction and are usually plastic or steel. The tubes are capped and filled with water. The transducers are hydrophones and use the water as couplant for the transmission of waves.
ultrasonic waves. Using results from multiple tubes and multiple heights, a large proportion of a pile or wall can be assessed. On occasion tests might be between two sonic transducers within a single tube. Transmission is direct between transmitter and receiver and no data can be obtained for regions outside the perimeter of the tubes. Hence design of the tube layout is important. The test is included in AS 2159:2009 [Reference 156] as shown in Table 5.4.

Cross hole sonic logging is recommended for general usage in piles and diaphragm walls as it can detect defects that are not uncommon and can have a significant impact on durability. Coverage of all piles and walls on a project is generally not economical, hence testing on selected elements is undertaken. The extent of testing might depend on the significance of the element, the ground conditions and the element geometry. It is recommended that the first element cast is tested and each element thereafter until no problems are found in a pour. Thereafter, testing at the rate of 5-30% of elements is recommended. The frequency of testing can be relaxed as the system of construction is proven, returning to a higher frequency when problems are identified.

Cross hole sonic logging services are provided by contractors specialising in pile integrity testing and specialised geotechnical and foundation testing, rather than by materials testing laboratories.
6.1 INTRODUCTION

The performance of a concrete in situ will differ from its performance in trial mixes or laboratory tests on quality control samples taken during construction. Testing of as-placed concrete is not a common requirement, as in-situ concrete quality is generally accepted as being adequate if the mix supplied is in accordance with the specification and there is no reason to suspect that placing, finishing and curing were substandard.

Testing of the as-placed concrete during construction might be required for assurance of durability when the concrete mix, placing, or curing is suspect. This is analogous to non-destructive testing, or testing of cores for strength where the designer or contractor is not satisfied that the method of construction will provide the performance that would be expected.

The main tests that might be undertaken to assess the adequacy of new construction are summarised in Table 6.1 and are described in more detail in this Section.

6.2 REINFORCEMENT LOCATION AND COVER DEPTH

Both the reinforcement location/spacing/position and the depth of cover over reinforcement are important, and may need to be determined when investigating the quality of a concrete placement. The importance of cover depth was described in Section 5.3.

6.2.1 Instrument types and applicable standards

The types of equipment available for non-destructively locating reinforcement and measuring its cover depth include magnetic reluctance, pulsed eddy current, ground penetrating radar and ultrasonic pulse echo. The applications of these and recommended procedures are described in this Section.

There are few published standards on the performance and use of covermeters. BS1881-204:1988 [Reference 21] is probably the most comprehensive in that it specifies accuracy, although it only relates to electromagnetic (magnetic reluctance) type meters, as follows:

- For use in lab: ±5% or 2 mm whichever is greater.
- For use in field: ±15% or 5 mm whichever is greater.

6.2.2 Magnetic reluctance covermeters

Covermeters have undergone a generational change since their inception. The first “covermeter” was of the magnetic reluctance type, developed in 1955 at the (then) C&CA in the UK. Covermeter manufacturers used this principle for the next thirty years, and it is still found in use today. This method measures small changes in the electromagnetic field of the core, and it can be strongly affected by variations in the core, e.g. temperature, external magnetic fields. The field is also affected by magnetic aggregates. Devices might have different heads with varying coil arrangements for specific purposes such as determining cover of congested reinforcement, measuring cover in different depth ranges and for bar diameter estimation.

In terms of the accuracy of electromagnetic covermeters, BS 1881:Part 124:1988 [Reference 21] requirements apply:

- Covers <80 mm, accuracy = ±1 mm.
- Covers 80-120 mm, accuracy = ±2 mm.
- Covers 120-160 mm, accuracy = ±3 mm.
- Covers 160-180 mm, accuracy = ±4 mm.

6.2.3 Pulsed eddy current covermeters

In the pulse eddy current method, a pulse of current creates a magnetic field through the coils in the instrument, which induces an eddy current in the reinforcement. The eddy current induced in the bar produces a second magnetic field that creates a decay time signal in the coils proportional to the bar diameter and cover. This process occurs in less than a millisecond, enabling reasonably fast scanning rates. Devices might have different heads with varying coil arrangements for specific purposes such as determining cover around core holes, measuring cover in different depth ranges and for bar diameter estimation. Alternatively, various coil types might be housed in one measuring head and are tuned for sensitivity to bar spacing or cover depth. These coils
### Table 6.1: Recommended tests for assessment of as placed concrete

<table>
<thead>
<tr>
<th>Suspect Aspect</th>
<th>Parameter Measured</th>
<th>Key Testing Method Aspects</th>
<th>Applicable Circumstance</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforcement Position and Cover Depth (see Section 6.2)</td>
<td>Strength of magnetic reluctance or pulsed eddy current induced in the reinforcement, i.e. covermeter</td>
<td>Use equipment that will give accurate cover measurement.</td>
<td>Simple and known accuracy for spot measurements where reinforcement details known.</td>
<td>Measurements by either method must be calibrated by exposing the reinforcement at representative sites and measuring its cover depth by ruler or tape measure. It takes many hours of practice on different types of concrete and structure to master the use of a cover meter or radar unit. Inadequate cover is one of the major causes of premature deterioration. Therefore the actual cover distribution should be measured and kept as a permanent record on all projects.</td>
</tr>
<tr>
<td></td>
<td>Flight time of radio wave reflected by the reinforcement, i.e. ground penetrating radar</td>
<td>Result affected by permittivity of the concrete which is moisture sensitive.</td>
<td>Fast scanning to give image of variation of reinforcement cover. Wherever unknown bar size, lap locations or closely spaced rebar invalidate covermeter measurements. Can measure covers deeper than covermeter.</td>
<td></td>
</tr>
<tr>
<td>Compressive Strength (see Section 6.2)</td>
<td>Core compressive strength</td>
<td>Take &amp; condition cores to AS1012.14. Test to avoid rebar.</td>
<td>Where cylinder strengths are low or deemed unrepresentative of in-situ strength.</td>
<td>Only method accepted in Australian codes for assessment of in-situ strength.</td>
</tr>
<tr>
<td></td>
<td>Hardness using a rebound hammer</td>
<td>Proper surface preparation. Appropriate statistical analysis and calibration against strength measured by core sample.</td>
<td>Use NDT to define variations in strength across an element and hence guide core locations.</td>
<td>Ultrasonic pulse velocity and rebound relationship to strength varies from mix to mix so calibration to same concrete mix of known strength required. Rebound is a measure of surface hardness only, hence results might be unrepresentative of bulk or unformed or deteriorated surfaces.</td>
</tr>
<tr>
<td></td>
<td>Flight time over known distance to give ultrasonic wave velocity</td>
<td>Accurate measurement of ultrasonic pulse flight path.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Presence or Depth of Defects (see Section 6.4)</td>
<td>Time of flight of direct compression wave i.e. Ultrasonic Pulse Velocity</td>
<td>Ability to accurately locate transmitter and receiver on opposite sides of the element.</td>
<td>Where delaminations, cold joints or significant voids are suspected.</td>
<td>In-expensive equipment but high labour cost. Results influenced by presence of reinforcing steel.</td>
</tr>
<tr>
<td></td>
<td>Flight time of reflected shear wave i.e. Ultrasonic Pulse Echo</td>
<td>Various multiple transmitter/receiver devices available.</td>
<td></td>
<td>Advanced equipment gives ability to see cross sections through the concrete.</td>
</tr>
<tr>
<td></td>
<td>Frequency of reflected compression wave from rear face or defect i.e. Impact Echo</td>
<td>Simple impact echo equipment available but requires experienced operator.</td>
<td></td>
<td>Simple method for assessing element thickness and identifying defect risk within a section at a single point on the surface.</td>
</tr>
<tr>
<td></td>
<td>Flight time of radar wave reflected by the rear face or defect</td>
<td>Wide range of Ground Penetrating Radar equipment available.</td>
<td>If voids present that are likely to be water filled.</td>
<td>Only identifies voids if saturated. Might not show air-filled voids or defects.</td>
</tr>
<tr>
<td>Quality of Surface Concrete (see Section 6.5)</td>
<td>Initial surface water absorption (ISAT)</td>
<td>BS 1881-208 ISAT</td>
<td>Water penetration into concrete surface under a small head pressure (&lt;200 mm).</td>
<td>Not recommended as a means of assessment of in-situ concrete quality. Results are affected too much by concrete moisture content.</td>
</tr>
<tr>
<td></td>
<td>Karsten Tube ISAT</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Air permeability</td>
<td>Torrent Test</td>
<td>Air flow measurement into surface concrete and determination of air permeability coefficient (m/s) and depth of penetration (mm)</td>
<td>Not frequently used. Results are affected too much by concrete moisture content.</td>
</tr>
</tbody>
</table>
Performance Tests to Assess Concrete Durability

can then be interpreted conjunctionally to give greater functionality.

In terms of the accuracy of pulsed eddy current meters, suppliers claim:
- ±1 mm up to 60 mm cover
- ±2 mm at 60-80 mm cover
- ±3 mm at 80-160 mm cover
- ±4 mm at 160-180 mm cover

6.2.4 Covermeter limitations

Like all devices, electromagnetic (magnetic reluctance) and pulsed eddy current covermeters have limitations. The following are the most common:

(a) Determining reinforcement diameter:
   The strength of the field depends on the bar diameter, hence for accurate cover measurements the bar diameter must be known. Some covermeters give an approximate bar diameter through the use of multiple sensors in the instrument; however, these are generally of low accuracy, particularly at high covers or areas of congested reinforcement. This is a good reason for recording the cover distribution as part of the construction quality assurance process, as the reinforcement diameters will then be known. Even where designed reinforcement diameters are known, it is recommended that reinforcement diameter and depth be established by exposing the reinforcement at one or more representative sites and measuring its cover depth and diameter by ruler or tape measure.

(b) Laps increase the field strength relative to single reinforcing bar, the effect being analogous to increasing the diameter of the reinforcement, consequently the cover indicated at a lap is lower than the true cover. This can be tested by sliding the covermeter probe along the reinforcement and looking for a sudden jump or drop in cover. It is important to consult the drawings of the reinforcing detail to choose positions where cover measurements will be accurate. Where drawings are not available, extensive tests might be necessary to resolve where laps occur. If the location of laps is not known, it might be preferable to use ground penetrating radar (GPR).

(c) Interference from adjacent reinforcing bars:
   When measuring cover of closely spaced reinforcement, interfering signals will be picked up from the reinforcement either side of the target reinforcing bar, increasing the total signal strength and therefore decreasing the displayed cover. This effect is also a function of reinforcement diameter. Some covermeters have in-built adjustment for near reinforcement correction, these covermeters generally give better accuracy for congested reinforcement. However, a point is reached where the cover is so high and the reinforcement so close that there is no detectable signal strength change as the probe is swept over the surface.

(d) Reinforcing bar diameter affects sensitivity:
   The field strength induced in the reinforcement depends on reinforcement diameter and hence the maximum depth of measurement decreases with decreasing reinforcement size. The depth of measurement is typically up to 150 mm for larger reinforcement but might be less than half that for smaller reinforcement.

(e) Reinforcement orientation: The measurement head when located midway between certain reinforcement arrangements might falsely indicate the existence of reinforcement. Follow the manufacturer’s advice for scanning in these circumstances. Reinforcement cannot be assumed to be straight, parallel or remaining in plane. It is recommended that measurements be taken at different positions of the same reinforcement to ensure that relevant cover measurements are made.

6.2.5 Ground penetrating radar

Ground Penetrating Radar (GPR) uses radio waves to pick up changes in the dielectric properties inside an element. This technology makes use of a transmitter sending radio waves into the substrate and a receiver recording the reflected waves. The waves are reflected where the dielectric constant changes. The depth of such changes can be calculated using the time of flight and an estimation of wave velocity. GPR is generally used for the location of steel embedments in concrete, but has many and varied applications in civil engineering.

Many of the limitations described above for covermeters do not apply to GPR, although GPR has its own limitations. Often covermeters and GPR can
be used synergistically to get a true impression of the cover.

Where reinforcing is particularly congested or where there are laps, use of covermeters is not recommended as erroneous readings are likely. GPR can achieve greater accuracy in these circumstances. The time of flight of the radar wave is highly sensitive to the concrete permittivity, which is largely controlled by the concrete moisture content. Hence some means of “ground truthing” (calibration) is required in order to gain accurate cover values. Calibration can be achieved by:

(a) Comparing a dimension measured by tape with a measurement made by GPR, e.g. a wall thickness.
(b) Comparing covermeter measurements with GPR measurement.
(c) Measuring actual cover by exposing reinforcement in one or more locations.

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**Figure 6.1:** Schematic of a hand held GPR unit scanning rebar and an example of the on-screen real time image

**Figure 6.2:** Cover distribution from a series of GPR scans

Mean (mm) = 52.8
Std. Dev. (mm) = 8.3
Charac. (mm) = 39
6.3 COMPRRESSIVE STRENGTH

Testing the strength of concrete in the field might be required for controlling the construction process quality assurance, repair of the structure, or auditing a structure’s final form.

Once the dielectric constant has been set for the concrete being measured, the GPR unit can rapidly scan the concrete surface. This is the ideal method of establishing both the cover distribution and any patterns to cover, as many covers can be visualised and measured in a short time. Thus the dielectric constant of a particular concrete mix will vary between different placements or positions within a single placement depending on exposure to moisture, and the dielectric constant at a particular site may vary with time due to cyclic wetting and drying.

Widespread cover surveys are possible with small hand held GPR units, refer Figure 6.1. The onscreen display enables real time review of the scan along a line. Continuous scans of 10-20 m are possible, the review of the full scan makes it easy to see rise and falls in the reinforcement, and the cover depth can be quickly read. Sufficient scans then need to be taken to give a clear impression of the pattern of cover on an element. This might enable the element to be broken into different zones with different covers due to the geometry or method of construction. Several elements can then be scanned to give a few scans representing each zone. A cover distribution can then be established for each zone based on several scans, refer Figure 6.2.

Higher performance GPR equipment is available that can detect reinforcement at greater depths, but its use is rarely warranted for development of cover distributions as the units are more cumbersome and more expensive.

6.2.6 Ultrasonic pulse echo

Ultrasonic pulse echo (UPE) was introduced for assessment of concrete using a device with numerous (48 or more) transmitter/receivers mounted in one head. The small diameter shear wave transmitters are spaced at up to 30 mm centres and when triggered the array of signals produces a cross sectional image through the concrete. The frequency of the waves emitted can be increased or decreased dependent on the geometry of the scenario (i.e. thick element, large targets etc.). In general use, the low frequency waves pass through the reinforcement without significant interference and the reflected waves are taken to represent flaws, voids or the concrete rear surface as these represent the largest change in impedance (which causes the waves to reflect). In high frequency mode, the reinforcement is detected and in some cases the device can be used to give a reasonable estimate of cover.

Although not commonly used for cover measurements due to the equipment’s expense and unsuitability for general cover measurements or bar location, it has been used successfully to assess the cover of edge bars in steel fibre reinforced tunnel lining segments (Figure 6.3). The cover to edge bars was a concern as it was known that the anti-burst edge cages had twisted leading to very low cover in some cases, and the edge bars were particularly vulnerable should joint leakage occur. The other types of covermeter noted in this section were unable to measure the cover due to the interference of the steel fibres.

In situations where cover is critical and reinforcement bars are masked by steel fibres, UPE is a consideration for post-pour quality assurance checks.
Tests for concrete strength fall into two main categories, destructive tests and non-destructive tests. A destructive test is defined as one that will leave damage requiring repair. A non-destructive test will not leave any damage.

Both forms of testing have their strengths and weaknesses. Destructive tests give a definitive value directly measured from the structure, but as they damage the concrete they are generally restricted and only provide information about the small area tested. Non-destructive tests can be carried out over wide areas on the structure. However, they do not definitively measure strength, and accurate interpretation of results requires calibration against results from tests on laboratory specimens or destructive in-situ tests.


Measuring compressive strength of concrete in an existing structure will generally involve a combination of non-destructive tests (i.e. no concrete removal) and destructive tests (e.g. core samples extracted). The amount of concrete removed must not reduce the structural capacity or other performance parameters of the element.

6.3.1 Concrete core sample testing

Compressive strength testing of core samples from hardened concrete is described in AS 1012.14:1991 [Reference 97]. This standard specifies that the core sample should have a length to diameter ratio close to 2:1, with a minimum diameter of 75 mm or 3 times the maximum nominal aggregate size, whichever is greater. It should be taken from the bulk of the concrete rather than the top surface where concrete is known to be of a lower strength. The standard also touches on correction for differing dimensions, treatment of sample prior to testing, precision and rejection criteria. The compressive strength test itself is to be completed accordance with AS 1012.9:1999 [Reference 181].

BS EN 13791:2007 [Reference 157] (and Standards cited therein) provides detailed guidance on evaluating compressive strength from core compressive strength tests. CIA Z11 [Reference 159] gives more general guidance on evaluating concrete strength from the results of testing cores taken from the structure. It describes the reasons for strength evaluation by the use of cores, and how to obtain cores from concrete structures so that they will be suitable for testing and the test results will be “significant” in the true statistical sense. Steps in the determination and evaluation of concrete strength from the results include providing for corrections to the indicated compressive strength of the core for length to diameter ratio of the core; presence of reinforcement in the core; position of the core axis in relation to the standard cylinder axis; age of the concrete; quality of compaction of the concrete; and, the curing regime experienced. A feature of the correction for quality of compaction of the cores is a series of photographs of concrete of known void contents. This visual means of correcting for excessive voids in the cores is presented with an alternative procedure using actual density measurements of the cores [Reference 161].

It is recommended that the practices in CIA Z11:2002 [Reference 159] be followed, with extra interpretation as in BS EN 13791:2007 [Reference 157] and associated documents.

6.3.2 Windsor Probe and Capo tests

The Windsor Probe and Capo tests are proprietary tests. The Windsor Probe fires a projectile into the concrete using gun powder. The depth the probe penetrates reflects the quality of the surface concrete. Users need to be licenced to operate the equipment.

The Capo test is a form of pull out test, in which an expandable fitting is inserted into a hole drilled in the hardened concrete. After expanding the fitting in the hole, the force required to pull it out is measured. The design of the fitting means that the primary mode of resistance to the tensile force is the compressive strength of the concrete.

These tests are not generally used for site investigations. For further information see CIA CPN 22 [Reference 161].

6.3.3 Rebound hammer

A rebound hammer test measures the surface hardness or more correctly the rebound of the concrete. During the test a steel spring driven mass is released from its maximum height to impact on the surface of a concrete specimen. The perpendicular
Performance Tests to Assess Concrete Durability

distance from the concrete surface that the mass rebounds or bounces is then measured and called the “rebound number”. This rebound distance is a function of the elastic rebound of the concrete specimen. Newer rebound hammers measure the velocity of the mass instead to alleviate the effect of gravity on the test.

The rebound number of the concrete depends not only on the compressive strength of the concrete but also the local hardness at the point of impact. Thus, it is influenced by the presence of aggregate or paste at the point of impact; surface texture/finish, cleanliness, moisture condition and carbonation; concrete age, and type of coarse aggregate. Although a measure of the quality and condition of the concrete surface, it may be affected by whether the element is supported or suspended, and on the thickness of suspended elements. Consequently, different concrete mixes of the same nominal strength, or the same concrete in different placements, may give different rebound numbers.

Because it is influenced by so many factors other than strength, rebound number must be calibrated against compressive strength. Three methods commonly used to correlate rebound number are as follows:

(a) Rebound number is not calibrated or converted to a compressive strength value, and instead the readings are only used comparatively over a site or element.
(b) Rebound numbers are compared with compressive strength test results obtained from cylinder or core samples. In some cases, such as in precasting large numbers of units, a specific concrete mix calibration can be created in the laboratory prior to use in the field. For site investigations, the rebound hammer testing is completed and then a core sample is extracted at the same position for compressive strength testing. This is the most accurate method.
(c) Rebound hammer manufacturer’s calibrations are used. The calibration is not specific to the project concrete, therefore this approach cannot be used as anything but a rough guide, and certainly not to accept or reject concrete. Guidance can be found in BS EN 12504-2:2012 [Reference 162] (replacing BS 1881-202:1986) and ASTM C805/C805M-13a [Reference 163]. These standards give step by step instructions on the use and upkeep of the test equipment and on the statistical treatment of results. CIA CPN 22 [Reference 161] considers that coefficients of variation of compressive strength estimated from rebound number can range between 18 to 30%.

To achieve reliable results, rebound hammers require regular checking on standard test blocks, with cleaning and lubrication of the mechanism when deviation from the calibration value is found.

6.3.4 Ultrasonic pulse velocity

Ultrasonic pulse velocity (UPV) typically requires access to two faces of the element under test – one for a source transducer and the other for a receiver. It measures average velocity of a compression wave over the path between the two transducers. This velocity has a direct relationship to the compressive strength, elastic modulus and density of concrete, thus can be used to assess concrete quality. It is also affected by the presence of cracks, voids and other discontinuities, therefore it may also be used to detect the extent of such defects.

The test can be carried out in several ways:
- Direct transmission, where the source transducer is opposite the receiver.
- Semi-direct transmission, where the source and the receiver do not face each other but are located on different faces of an element such as across a corner.
- Indirect transmission, where the source and receiver are placed on the same surface.

The most critical aspect in obtaining useful UPV data is the ability to measure the direct linear distance between the transducers accurately.

Correlation of the UPV to concrete compressive strength is carried out in a similar manner as outlined for rebound hammer testing. Readings can be treated purely for comparative purposes or calibrations can be formulated using destructive testing. Generally the results are interpreted comparatively, i.e. by looking at the pattern of results over a surface, rather than the absolute values. Deviations from a consistent pattern may indicate a possible defect.

International standards providing guidance on both the use of and interpretation of UPV readings are BS EN 12504-4:2004 [Reference 164] (replacing BS 1881-203:1986), ASTM C597–09 [Reference 165] and

6:7
Performance Tests to Assess Concrete Durability

6.4 **DETECTING DEFECTS WITHIN THE CONCRETE**

Defects may be detected to a limited extent by visual inspection of the surface and/or core samples. A range of non-destructive test methods is available for determining whether defects are present over wider areas. These are listed in Table 6.1.

6.4.1 **Ultrasonic pulse velocity**

See Section 6.3.4.

6.4.2 **Ultrasonic pulse echo**

See Section 6.2.6.

6.4.3 **Impact echo**

Elastic, low frequency transient stress waves are produced at a test point by tapping a small weight against the test element’s concrete surface. The waves travel through the concrete and are reflected by any discontinuities caused by changes in acoustic impedance (partially affected by concrete density). A transducer held on the surface adjacent to the impact point detects the reflected waves as they rebound between the concrete surface and discontinuities.

Major discontinuities (such as defects) reflect the most energy. For in-plate concrete elements such as slabs and walls with no defects, the most significant discontinuity is the rear face of the element. ASTM C1383-04a [Reference 182] refers to the measurement of thickness in concrete slabs and plates using the impact echo method. Where defects such as voids or delaminations provide a significant barrier to the wave, reflections of the wave energy cause an amplitude peak at the corresponding frequency and the distance to the defect can be calculated.

6.4.4 **Impulse response**

Originally, the impulse response method was developed for deep foundation evaluation, but it is now widely used in the United States, Europe and Singapore to diagnose the integrity of concrete structural elements.

A low-strain hammer (rubber mallet) sends a stress wave into the test element. Plate-like structures respond to the impact in a bending mode, i.e. the structure vibrates at low frequency. The force of the hammer impact is measured by the hammer’s built-in load cell as a force vs time plot (the “force spectrum”). The concrete’s surface vibration is measured as a velocity vs time plot by a geophone. A fast fourier transform (FFT) algorithm is used to process both signals to give force vs frequency for the hammer impact and velocity vs frequency for the geophone responses (the “velocity spectrum”). As the response is dependent on the impact force, a “transfer function” is derived by dividing the velocity spectrum by the force spectrum. This transfer function, known as the “mobility”, has units of m/s/N. For each test point mobility vs frequency is obtained. The average mobility, which is the averaged reading between 0 to 800 kHz, is generally used for analysis. The average mobility is then indirectly proportional to the concrete element thickness.

For large structures, tests are conducted on a grid pattern to cover the area of concern. Sound concrete in plate structures (slabs, walls, bridge decks and arches) typically gives constant values of impulse response (average mobility) across the element. Poor consolidation and honeycombing causes a rise in average mobility.

6.4.5 **Ground penetrating radar**

See Section 6.2.5.

6.5 **ASSESSMENT OF CONCRETE SURFACE QUALITY**

The quality of the concrete surface greatly influences the durability of the concrete. In turn, the surface quality is determined largely by curing method and duration. Inadequate curing may reduce compressive strength, but the effect on durability is likely to be greater. When the quality of curing is suspect, measurement of the surface’s resistance to ingress of water or air by methods such as those described below is possible but is not frequently done.

The biggest issue with the types of tests described below is that the test result is affected by
the concrete moisture content. Consequently, care is required when interpreting the results. They are also influenced by the surface finish/texture, which may affect the correct functioning of the test equipment. The most common methods of assessing the quality of as cast concrete is to take samples and measure the performance using a laboratory tests such as sorptivity or volume of permeable voids (VPV) although a test related directly to the mechanism of deterioration may be considered. For field testing, the Torrent test could be considered but only if an allowance for concrete moisture content is possible. However, this method is not commonly available in Australia and initial surface water absorption tests are more likely to be available.

6.5.1 Initial surface water absorption (ISAT)

The BS 1881-208:1996 [Reference 167] ISAT, refer Figure 6.4b, is rarely used in on-site investigations. A container of defined size is bolted to the concrete surface such that the container’s open surface is in contact with the concrete. The container is filled with water and the water absorbed by the concrete under a defined pressure head of 200 mm is measured. This pressure head is taken to be higher than the average pressure of driving rain.

The equipment is simple to set up in laboratory tests, but more cumbersome for in-field tests. Results are affected by concrete moisture content. Therefore, the test is not recommended as a means of assessment of in-situ concrete surface quality.

The Karsten Tube, RILEM 25-PEM test 11.4:1986 [Reference 168], refer Figure 6.4a, has been developed as a simple form of ISAT. It does not require bolting to the concrete surface. However, with little exposure to the research outside Europe, it has failed to become internationally specified. A study by Nwaubani [Reference 169] comparing in-situ methods of ISAT rated the Karsten Tube method highly because of the non-destructive nature and low cost. In the US, the method has found favour in slightly modified form, with larger diameter reservoirs being used to increase the test area.

Like the BS 1881-208:1996 ISAT, the primary limitation of the Karsten Tube ISAT is that the test result is affected markedly by the concrete moisture content. Hence, the test is not frequently used.

Furthermore, the BS 1881-208:1996 ISAT cannot be used where a surface treatment has been applied, although the Karsten Tube ISAT might be useful in assessing the quality of surface treatments.

6.5.2 Torrent air permeability

An indication of the air permeability of surface concrete can be determined using a Torrent

Figure 6.4: Field measurements of water absorption

(a) ISAT - Karsten Tube method.
(b) BS 1881-208 ISAT test equipment.
6.6 MIX COMPOSITION

6.6.1 Cement (binder) content and composition

Chemical analysis of concrete by BS 1881:Part 124:1988 [Reference 21] can be used to determine the acid-soluble calcium and silicon contents of the concrete, from which the approximate cement content can be determined. For cementitious binders consisting of type GP cement alone, the calcium content alone will be generally be sufficient, but if the concrete also contains shell, limestone aggregate, or other sources of calcium carbonate then the silicon content also needs to be measured.

BS 1881:Part 124:1988 [Reference 21] reports a repeatability of 40 kg/m³ and a reproducibility of 60 kg/m³ for cement content determinations of flint-based concrete. Findings of an inter-laboratory trial involving four mixes each with different binder type indicated an average repeatability of 60 kg/m³ and an average reproducibility of 120 kg/m³ [Reference 91]. Although the authors suggest the accuracy is closer to that reported by BS 1881: Part 124: 1988 [Reference 21] once outliers are removed, they recommend caution in using the results of cement contents determined by this method, in particular when using them to express the results from other analyses as a proportion of cement content.

Chemical analysis by these methods may be less accurate for concrete containing SCM unless a reference sample is available for comparison.

Chemical analysis must be carried out by a laboratory with appropriate experience and understanding of analysing concrete by these methods, preferably with NATA or equivalent accreditation (see Section 1.4).

Petrographic examination (see Section 7.4.5) can determine the type of cement used (e.g. Type GP, SR, GB, white cement, calcium aluminate cement) and the volume percent of binder [References 21 & 90].
6.6.2 **Air content**

The entrained air content of hardened concrete can be estimated by petrographic examination [Reference 90].

6.6.3 **Water to cement ratio**

The water to cement ratio of hardened concrete is difficult to measure accurately, but can be estimated by chemical analysis [Reference 21] or petrographic examination [Reference 90].

Chemical analysis [e.g. Reference 21] involves combining measurements of cement content, bound water in hydrated cement, and porosity. The estimated accuracy (expressed as reproducibility) is +/- 0.1 of the actual water to cement ratio for the range 0.4-0.8 [Reference 89]. It is less accurate for concrete that is poorly compacted, carbonated, air entrained or cracked. Results reported by Barnes and Ingham [Reference 91] varied by up to +/- 0.2 from the actual value.

Petrographic examination is more reliable as it also provides information about carbonation, air entrainment and voids. For accuracy it requires comparison with reference materials of known water to cement ratio. Provided the sample is representative of the placement (rather than of a defective area), the results may be accurate within an error of +/- 0.05 of the actual water to cement ratio for the range 0.4-0.8. Greater accuracy is achievable for relatively new concrete where reference materials consists of exactly the same concrete cured in the same way as the in-situ concrete [Reference 90]. This approach may be appropriate for measuring water to cement ratio on large projects where reference samples are match-cured for other testing such as compressive strength.

6.7 **SCM CONTENT AND COMPOSITION**

Electrical resistivity can be utilised to indicate whether SCMs have been used in accordance with the project specification. Section 5.7 recommends that for concrete in severe exposure conditions, resistivity of all 28-day compressive strength test specimens be measured as a check on the consistency of concrete composition. In general, such testing is unnecessary, as batch records and QA testing will provide adequate evidence of correct use of SCMs. However, if there is doubt about the use, type or quantity of SCMs in the concrete, the significantly higher saturated resistivity of SCM concrete compared to concrete with GP cement can be used to indicate if SCMs have been used correctly.

The test may be carried out on cylinders, cores or in situ. Results must be compared with results from a reference sample of concrete of the same composition.

Because the resistivity of concrete is highly affected by moisture content, the resistivity test should be undertaken on saturated concrete (e.g. by applying a sprinkler to the element’s surface for several hours), or by wet conditioning core samples in accordance with AS 1012.14:1991 [Reference 97].

If a low resistivity relative to the saturated cylinder resistivity is obtained it might be inferred that SCM usage is not as would be expected. Further analysis of batch records may reveal the cause of the difference. If not, petrographic examination may be able to identify whether the binder contained the intended SCM.

The presence and nature of SCM in relatively new concrete can be determined by petrographic examination or scanning electron microscopy (SEM) [Reference 90], provided the concrete is sampled before the SCM has reacted significantly. Fully reacted SCM cannot be physically differentiated from normal cement phases. Residual particles of fly ash and slag may remain visible for more than a year after casting. For amorphous silica and other SCMs that react rapidly, evidence may be limited to agglomerates of poorly dispersed SCM, or indirect evidence such as the distribution and size of voids and calcium hydroxide in the hardened cement paste. Chemical analysis for species such as calcium, silicon, aluminium and iron can sometimes provide indicative information if compared to reference samples of known composition.

6.8 **CHLORIDE AND SULFATE ION CONTENT**

To minimise the risk of chloride-induced reinforcement corrosion, maximum limits on the acid soluble chloride ion content of fresh concrete are set by Standards such as AS 1379:2007 [Reference 7]. The chloride ion content may be determined by adding together the chloride ion contents of all concrete constituents, or by chemical analysis of the hardened concrete.

The acid-soluble chloride content of concrete
is determined by methods such as AS1012.20:1992 [Reference 175], ASTM C1152/C1152M-04 [Reference 177] or BS 1881:Part 124:1988 [Reference 21]. These analyses must be carried out by suitably experienced laboratories (ideally ones that hold NATA or equivalent accreditation for these or related analyses). The methods differ slightly in the way the samples are prepared, but the actual analyses are all based on sound analytical procedures. Methods such as AS 1012.20:1992 [Reference 175] allow for a variety of analytical procedures to be used provided they are calibrated against a reference procedure and meet the repeatability requirements of the standard.

Sulfate ion content is determined largely from the sulphate content of the binder. It may be measured by methods such as AS 1012.20:1992 [Reference 175] or BS 1881:Part 124:1988 [Reference 21]. An inter-laboratory trial of the BS 1881:Part 124:1988 [Reference 21] method found that it consistently overestimated the chloride content, often significantly, while the results from the sulfate analysis showed no consistent trend between actual and measured values [Reference 91]. Clear [Reference 92] subsequently recommended that for marine structures the method of summation be used instead of chemical analysis of hardened concrete.

Sirivatnanon et al [Reference 171] showed the repeatability of AS 1012.20:1992 [Reference 175] when used to test sand was good but its reproducibility was poor. They concluded it could mean a difference of 0.05% chloride (by weight of sample) between results from two different labs when used to test concrete.

Freitag et al [Reference 72] considered the precision of the AS 1012.20:1992 [Reference 175] method as stated in the test method and quoted by a testing laboratory. AS 1012.20’s required repeatability was considered equivalent to an uncertainty and precision of +/-0.2 kg Cl/m³ concrete. In the context of a maximum Cl content of 0.5 kg Cl/m³ for prestressed concrete in NZS 3101:2006 [Reference 148], this was considered significant. Therefore, it was suggested that when evaluating compliance with NZS 3101, concrete chloride contents not exceeding 0.7 kg/m³ Cl be accepted as complying with the 0.5 kg Cl/m³ requirement.

Freitag et al [Reference 72] also considered the accuracy of x-ray fluorescence (XRF) spectroscopy for determining chloride ion contents. Results suggested that the relationship between XRF and AS 1012.20:1992 [Reference 175] was not close enough to directly convert XRF results to “AS 1012 equivalent” results. In addition, the accuracy of XRF was found to depend heavily on the use of appropriate reference materials. It was concluded that XRF would only be suitable for assessing compliance if initially calibrated against wet chemistry methods using the concrete in question, and if XRF analyses for a specific project included a reference material comprising a sample of the concrete on which the calibration had been determined.

Because of the inherent inaccuracy in the chemical analyses, Z7/07 recommends that where measurements of chloride and sulfate ion content exceed the specified limits then the values determined by summing of the constituents shall take precedence.

### 6.9 ALKALI CONTENT

In some circumstances, the risk of AAR is managed by specifying a maximum concrete alkali content. For example, in NZ a maximum alkali limit of 2.5 kg/m³ is used for many concretes when potentially reactive aggregate is used. Determination of the alkali content of hardened concrete may be required to assess compliance with such specifications, or to evaluate the cause of AAR in an existing structure to help identify appropriate precautions for future construction.

BS 1881-124:1988 [Reference 21] provides a method for determining acid soluble sodium and potassium contents. Barnes and Ingham [Reference 91] reported that inter-laboratory test results from this method were consistently higher than the target, which may reflect extraction of alkalis that would not normally be available for AAR, such as alkalis bound within minerals in the aggregate. Of potentially greater concern is that the range of results they reported exceeded 1.0 kg/m³ for all for concretes tested. Thus, this method may not be accurate enough to be of practical use.

Other in-house methods may be more appropriate, such as methods that measure water soluble alkalis, and/or that separate the binder fraction from the aggregate and analyse only the binder fraction, but were considered beyond the scope of this document.
7.1 **INTRODUCTION**

Condition monitoring is an asset management function undertaken to assess whether a structure is still able to fulfil its levels of service and how it is performing with respect to its design life. This informs the asset owner on future maintenance expenditure and when either major renewals and/or replacement of the asset should be planned.

Monitoring is not solely collecting and storing data. It requires the data to be interpreted, and decisions made about when intervention is necessary to maintain serviceability or structural performance. Criteria for interpreting data and signalling a need to consider changing the monitoring methods or frequency intervention must be established at the same time as the methods for collecting and storing data.

Condition monitoring can be undertaken in various ways. The project owner needs to be clear on the advantages and disadvantages of the approaches including:

(a) **Intermittent visual inspection:** Visual inspection will only detect visible damage. Unless conducted regularly, by the time damage occurs, deterioration might be advanced and options for preventative maintenance might be reduced or lost. Visual inspection can be undertaken relatively cheaply over the whole structure at frequent time intervals.

(b) **Intermittent site testing:** Site testing can be undertaken to monitor the rate of deterioration, and predict residual life based on the observed deterioration rate. Calculating residual life often requires various assumptions to be made as it is impractical to measure all parameters in the models. This can lead to large errors in predictions.

(c) **Permanent surface mounted and embedded monitoring:** These are installed preferably at the time of construction but can also be retrofitted. Some monitoring methods facilitate the assessment of residual life and because they establish a base line at construction for items like strain they give more extensive information not obtained from intermittent site testing that starts once the deterioration is imminent. This type of monitoring has a significant capital outlay and ongoing maintenance and operating costs, therefore should only be installed where the structure’s owner will commit to undertaking the monitoring throughout its service life. Where monitoring of this type is undertaken it must be considered throughout the design phase and into the construction phase. Experienced contractors must be used during the installation to ensure it is given the greatest chance of being successfully operational.

7.2 **TEST LOCATIONS**

In order to obtain representative test results, the locations tested must be representative of the population. In determining the locations to be tested careful consideration should be given to the configuration of the structure/element, the macro and micro exposure and the method of construction. When developing the testing plan, the item to be assessed (structure or exposure) should be split into zones where the results can reasonably be expected to be similar. In that way the sample results expressed as a mean and variance will give a true statistical representation of the zone. In this section examples of zone selection are discussed.

7.2.1 **Structure configuration**

Even though a structure might be constructed from the same concrete mix throughout, the in-situ performance of the concrete might vary depending on placing, finishing and curing. For example:

(a) There might be many areas of different elements of similar construction and materials that can be treated as the same zone. For example, all soffits might be considered as one zone.

(b) The structure will have been built from concrete batched at different times. This might lead to variability. The method of batching quality control might suggest a pattern in quality (e.g. control of water content can be difficult after rain).

(c) Apart from mix and placing variability, materials also vary. For example, in Western Australia, the performance of slag blended cements was affected by ageing so concrete produced at
the end of the slag stockpile might have lower strengths than the fresh slag. It might be valuable to check historical records of compressive strength and locations of concrete placement by batch to determine any pattern that might help direct the zoning for tests.

(d) Prestressed concrete requires specific consideration.

7.2.2 Element configuration
Even though an element might be constructed from the same concrete mix or even the same concrete batch, the in-situ performance of the concrete might vary depending on placing, finishing and curing. For example:

(a) Formed surfaces have different finishing and curing to unformed surfaces. Test results on an unformed concrete surface might have no relationship to the performance of formed surface and neither necessarily relate to the performance of bulk concrete. Where cores are taken for strength assessment they are normally taken from the bulk of the concrete. Samples are taken of sufficient length so that the top 50 mm can be cut off so strength results are not affected by surface effects. Conversely, when assessing the durability of concrete the cover zone is critical and surface samples are valuable.

(b) The performance of concrete at the bottom of deep pours will differ from that at the top due to bleed and pressure effects.

(c) Tension zones of an element are likely to contain micro-cracks that could affect the concrete performance.

7.2.3 Construction influences
When assessing cover, the configuration of the reinforcement might be important. For example, cover might be low at starter bars or where ligatures overlap.

7.2.4 Exposure
Exposure variations will lead to differences in condition of parts of an element constructed in identical ways and with the same materials. For example, one side of a beam might be exposed to direct wind-blown salts while the other side is sheltered. Once a structure is broken down into zones of similar configuration, the zones should be further subdivided based on exposure. Care should be taken not to assume too much about exposure severity of micro climates. While one side of a beam might seem to be sheltered, wind eddies can lead to salt deposition in what appears to be sheltered locations.

7.2.5 Existing condition
If deterioration has already started, it is important to determine whether the visible damage is related to local defects or represents the beginning of a more widespread problem. If obviously related to a local defect (such as honeycombing, formwork defects, embedded items or displaced reinforcement), then sample locations representing apparently undamaged concrete in zones potentially more likely to deteriorate (such as the outer edges of a bridge deck soffit) will better represent the risk to the overall durability of the structure or element.

7.2.6 Type of monitoring
The type of condition monitoring will also play a role in collecting data. Methods such as electrode (half-cell) potential can be fast scanning techniques (e.g. using a wheel type electrode) and can be used to identify areas of reinforcement corrosion induced deterioration in undamaged concrete areas. It is practical to use this type of technique globally. Embedded reference electrodes (half cells) can also be used to measure electrode potential but they only measure at a single point and are therefore more useful to measure the change of condition of a more globally similar yet difficult access situation.

Each technique has its own limitations and advantages, as discussed below, and should be considered as part of a system for condition monitoring.

7.3 VISUAL INSPECTION
As noted, visual inspection only provides evidence of visible deterioration damage and once such damage has occurred it might be too late to apply cost effective preventative measures. However, visual inspections might detect the development of deterioration or structural issues that would not have otherwise been detected. They can also point to areas of more serious exposure, local deterioration related to construction defects or design features, or less resistant
concrete. Hence they can be usefully incorporated into an overall condition monitoring programme to detect potential deterioration before it becomes more serious.

Inspection of distant surfaces by naked eye or binoculars may fail to observe defects, particularly in poor lighting conditions. Therefore as with physical testing, good access must be provided to surfaces being inspected. This is likely to involve managing hazards related to working at heights, in confined spaces, traffic, industrial processes, or environmental hazards.

The biggest issue for visual inspections is their subjectivity: they are undertaken by different people who even with training will give different ratings to the scale of deterioration. Hence, excellent training and provision of standard defect descriptions and ratings is required. Major asset owners may have their own inspection manuals and inspector accreditation schemes. General guides to assessing the visible condition of concrete structures are found in technical publications such as CIA Z15 [Reference 81], CIRIA C660 [Reference 125], UK Concrete Society Technical Report TR 22 [Reference 82] and American Concrete Institute ACI 224.1R-07 [Reference 83]. Details with references specific to concrete cracks are in Z7/06.

Visual inspection involves examining surfaces for evidence of damage such as:

- **Cracking**: distinguishing between cracks caused by construction processes, structural actions, single events (e.g. fire, seismic events or other ground movements, impact) and durability issues such as reinforcement corrosion, alkali aggregate reaction, sulfate attack, delayed ettringite formation, unsound aggregates or binder.

- **Surface scaling or delamination**: may be related to traffic or natural weathering combined with freeze-thaw cycles, crystallisation of salts or possibly construction-related defects.

- **Surface softening**: may be related to attack by soft or aggressive water, carbonation, acid, sulfates, sugars, carbonation.

- **Surface erosion**: related to trafficking or water flow over an otherwise sound surface.

- **Surface deposits**: that may harbour bacteria and other microorganisms

Visual examination also involves quantifying visible features such as:

- The location of defects and the area or length of concrete affected.
- The depth of surface scaling, delamination or softening.
- The orientation, length and approximate width of cracks and their relationship to the position of reinforcement.

Core samples may be taken to investigate the depth of surface damage. This may initially simply involve visual examination of the core and cutting further sub-samples to measure carbonation depth and examine by microscope under reflected light. It may also involve petrographic examination in more detail, see Section 7.4.6.

Records of such observations and measurements at successive inspections can then be compared to assess the rate of deterioration. Alternatively, results from a single visual inspection may be used to estimate the remaining service life, from which the timing of future inspection/maintenance can be determined based on the actual risk. This can be more cost effective than basing inspections and more detailed instrumental monitoring simply on age or a regular time interval.

The presence of environmental factors or design features contributing to the damage should also be recorded; for example, water draining over the surface, traffic patterns, local turbulence in water flow, other evidence of acidic groundwater or soil, operational aspects and use of chemicals in an industrial process. It may be possible to address these before extensive damage is incurred.

### 7.4 INTERMITTENT SITE TESTS

In addition to visual examination described above, non-destructive test methods and methods of sample analysis can be employed at intervals during the structure’s life. The structure should deteriorate slowly if well-designed and constructed. Consequently, many of these methods will not provide any significant information other than establishing an initial baseline performance until at least 20% of the time into the design life. This approach is attractive to design and construct projects because the testing can often be specified in the maintenance programme and have no impact on the design and construction cost.

Some tests can be used to scan the concrete surface to help identify what other tests should be
undertaken in different areas, for example:

- Electrode (half cell) potential mapping used to identify areas of corrosion (see Section 7.4.1).
- Rebound hammer used to identify areas of low surface strength (see Section 6.3.3).

Where the rate of ingress of a critical deterioration related front is required, e.g. chloride profiles to give the depth of corrosion activation front (see Section 7.4.3) and carbonation depth (see Section 7.4.2), test results can be used in conjunction with cover distributions established during construction to calculate the residual life. Electrode potential mapping results might indicate that the deterioration front should be measured in areas that are consistently passive, while further information on resistivity and corrosion rate (see Section 7.4.5) might be indicated for active corrosion areas. Sampling for petrographic examination (see Section 7.4.6) or wet chemical analysis (see Section 6.6 and 6.7) might be called for where chemical attack is of concern due to the deterioration risk or due to low apparent surface strengths identified using rebound hammer scans (see Section 6.3.3).

7.4.1 Electrode (half cell) potential mapping

The process of corrosion in reinforced concrete involves the establishment of anodic (more negative potential) and cathodic (more positive potential) sites on the embedded steel. Since differences in electrical potential exist between anodic (“corroding”) and cathodic (“passive”) sites, then by measuring potentials on the surface of the concrete with respect to the potential of a stable reference electrode the size and location of anodic and cathodic areas can be established [Reference 71 and Reference 189]. As design life is often defined as time to corrosion initiation, the method gives an immediate impression of whether the design life has been reached. Results can also be used to define what further testing is required. At anodic areas the extent of corrosion then needs to be assessed. If not actively corroding, the time to corrosion initiation needs to be evaluated. If actively corroding, the section loss and corrosion rate might need to be evaluated.

The equipment set up incorporating a portable reference electrode is shown in Figure 7.1. A positive electrical connection is made between the steel reinforcement and a high impedance voltmeter (multimeter). The negative return in the voltmeter is connected to a standard reference electrode (for example, copper in a saturated copper sulfate solution, silver/silver chloride/potassium chloride, manganese/manganese dioxide). The electrical circuit is completed when the reference electrode is placed on a wetted concrete surface (via the concrete electrolyte). Due to the high order of the electrical contact resistance and also (in some cases) the high resistivity of the concrete, a high impedance voltmeter is essential (input impedance >10 Mohms) [References 71 and 189].

A predetermined grid pattern is developed and marked on the concrete surface. As the reference electrode is touched to each grid point, the potential is read on the voltmeter and recorded at the corresponding location on a grid map of the surface [References 71 and 189]. Recording might be manual or automated. The testing continues until the grid is complete. Plotting of potential contours or isopotential values is then conducted. ASTM C876-09 [Reference 172] gives a method for the taking and displaying of
potential measurements. For rapid scans of large areas a wheel electrode can be used [Reference 74]. The required spacing of the wheel runs is marked on the concrete surface and the measurement interval along each run set in the instrument. Scans are then made along each run. Whichever method of recording results interpretation is similar.

ASTM C876-09 [Reference 172] provides corrosion probability criteria based on absolute potential values as developed from statistical analysis of potential surveys conducted on US bridge decks suffering from de-icing salt attack. It considers a potential value more negative than -350 mV with respect to a copper/copper sulfate reference electrode (CSE) to indicate a greater than 90% probability of corrosion and a potential value more positive than -200 mV CSE to indicate a less than 10% probability of corrosion. At intermediate potentials corrosion activity is uncertain. The ASTM C876-09 [Reference 172] criteria must be used with caution as they are generally not applicable to other situations. As can be seen from Figure 7.1, cover for example, determines where in the potential field the surface potential is measured and this can significantly affect the measured value, as can concrete resistivity, amongst other factors, see below [References 71 and 189].

Gulikers [Reference 73] provides an approach that can be used to establish “ASTM type criteria” for an individual structure. In this method all the potential measurements are treated as being from bi-modal distributions representing active (anodic areas) and passive (cathodic areas) states. Results between the clear distributions are considered as uncertain, see Figure 7.2. In order to use the method, results need to include significant areas of active (anodic) and passive (cathodic) reinforcement.

Individual absolute values of potential can also provide an indication of corrosion activity but only when correlated with the visual condition of the reinforcement subsequent to concrete breakout.

A further alternative analysis method is to use potential differences and the rate of potential change (potential gradients) to provide a measure of the location and extent of reinforcement corrosion [Reference 75].

Although the electrode potential technique serves in most cases as a reasonable prediction of the thermodynamic tendency of steel to corrode, it provides no information about the extent or rate of corrosion. This is illustrated for reinforced concrete immersed in seawater where although the potential might reach -600 to -900 mV CSE (indicating active steel), the corrosion rate is typically negligible due to lack of oxygen to fuel the cathodic reaction in the corrosion process [References 71 and 189].

There are many traps for the unwary, including the following [References 71 and 189]:
Performance Tests to Assess Concrete Durability

- **Junction potentials:** Carbonated concrete tends to produce potential readings less negative than expected. This is due to the generation of a liquid junction potential at the carbonated/uncarbonated concrete interface due to the difference in hydroxyl ion concentration between the almost neutral carbonated layer and the highly alkaline concrete. A junction potential effect could also occur where chloride has penetrated into the concrete but has not yet reached the reinforcement. A more concentrated solution is near the surface leading to a more negative potential reading than would be expected.

- **Concrete cover:** With increasing concrete cover, the potential values at the concrete surface over anodic and cathodic areas become more similar, and localised active areas more difficult to detect, see Figure 7.1.

- **Highly resistive concrete surface layers:** A highly resistive surface layer tends to record a less negative potential over anodic areas than expected. The macro-cell current paths tend to avoid the highly resistant concrete.

- **Polarisation fields:** Anodic areas could polarise adjacent cathodic areas due to macro-cell effects. This effect has been recorded, for example, on marine structures where that part of a reinforced concrete element which was immersed was polarising the above-water part of the element.

- **Interference:** Stray electrical direct currents (DC) can lead to polarisation of the steel reinforcement. It is recommended that the asset maintenance manual could include options for the use of electrode potential mapping for condition monitoring. Like all condition monitoring, electrode potential mapping is used to reduce the risk of failure by providing an early warning of more serious deterioration that might occur if preventative measures are not taken. Based on the risk associated with corrosion in the elements design life, a guide that might be considered is given in Table 7.1.

When electrode (half cell) potentials are used it is recommended that an expert in the taking of measurements and the interpretation of the results be utilised. They may also require additional testing based on the results.

### 7.4.2 Carbonation depth

Carbonation causes concrete to lose its alkalinity and it can be measured by applying a pH indicator to a freshly exposed surface. Solutions of phenolphthalein in a distilled water/methylated spirits mixture such as that specified in Section 4.3 produce a bright purple stain when applied to un-carbonated concrete with pH higher than 10.5, but do not alter the concrete colour when applied to carbonated concrete with pH lower than 9. Thus the depth of carbonation can be clearly indicated. Other pH indicators might also be used, but their colour changes are less distinct.

A method for measuring concrete carbonation by phenolphthalein test is described in BS 14630:2006 [Reference 174] that was based on the UK Concrete Society Technical Report TR 60 2004 [Reference 189] and prior Current Practice Sheet No 131:2003 [Reference 175].

Concrete carbonates quickly after exposure to air, therefore carbonation must be measured on a freshly-exposed surface. Cutting might expose un-carbonated cement grains that have not previously

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**Table 7.1: Guide for use of electrode potential mapping for condition monitoring**

<table>
<thead>
<tr>
<th>Condition</th>
<th>Condition Monitoring Using Electrode Potentials</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low risk</td>
<td>Only if there are signs of corrosion induced distress</td>
</tr>
<tr>
<td>Moderate risk</td>
<td>At 20% of the design life</td>
</tr>
<tr>
<td>Very high risk (or embedded reference electrodes indicate potentials shifting more negative)</td>
<td>At 50% of the design life</td>
</tr>
</tbody>
</table>
been exposed to the air, and phenolphthalein staining associated with these grains might mask carbonated areas. Fracture surfaces are therefore considered to provide more accurate measures of carbonation depth. However, the appearance of cut surfaces can reveal other features that could affect carbonation, and if all cutting debris is removed from the cut surface prior to applying the phenolphthalein it can give meaningful results. If only a very small sample is available, the only way to generate a fresh surface in the desired orientation might be to cut the sample. When testing a cut or drilled surface, the depth of carbonation is determined while flushing the surface with indicator. When testing a fracture surface, the depth of carbonation is determined by lightly spraying the surface with phenolphthalein, avoiding run-off.

Concrete is not a homogeneous material, therefore carbonation depth varies over a surface. The maximum carbonation depth might represent a localised physical feature such as a crack or void that could render the concrete more susceptible to chemical or physical attack at that point. Thus, maximum carbonation depth should be recorded in addition to the mean or typical carbonation depth.

Phenolphthalein testing allows a large surface area to be assessed quickly and relatively cheaply. It can be carried out in-situ, or on samples such as cores or lumps extracted from the structure for examination in the laboratory.

Phenolphthalein might produce a pale discolouration on concrete that is partly carbonated. A decrease in the apparent depth of carbonation observed once the indicator solution has dried may indicate a zone of partial carbonation. The possibility of partial carbonation can be confirmed by petrographic examination if necessary.

Concrete can lose alkalinity not only by carbonation caused by exposure to atmospheric carbon dioxide gas, but also by exposure to acids or soft water. Determination of carbonation depth by phenolphthalein testing will not distinguish between alkalinity loss from carbonation or by other causes. Alternative methods of analysis such as by petrographic examination are needed to determine the presence of other such mechanisms.

When the depth of carbonation reaches the cover depth, the steel reinforcement might start to corrode. If it hasn’t already reached cover depth, the time at which it might do so can be estimated, see Section 4.3. The rate of corrosion will then be determined predominantly by the availability of moisture and oxygen. It might be very slow.

7.4.3 Chloride profile

The depth and amount of chloride ion contamination is determined by analysing samples from increasing depth from the exposed concrete surface to at least the outermost cover depth (BS 14629 [Reference 189] that was based on the UK Concrete Society Technical Report TR 60 [Reference 189]). Typically 10-25 mm increments would be tested, depending on cover depth, refer Section 3.3.

Samples might be drilled powder (Section 3.3.5) or slices cut from core samples (Sections 3.3.2 and 3.3.4).

Samples are usually analysed for total chloride content, determined as acid soluble chlorides [References 21, 175 & 177], or as total chlorine by instrumental methods such as x-ray fluorescence (XRF). These analyses must be carried out by suitably experienced laboratories (see Section 6.8). The methods differ slightly in the way the samples are prepared, but the actual analyses are all based on sound analytical procedures. Methods such as AS 1012.20:1992 [Reference 175] allow for a variety of analytical procedures to be used provided they are calibrated against a reference procedure and meet the repeatability requirements of the standard, refer Section 6.8. The accuracy of the methods is discussed in Section 6.8.

A semi-quantitative indication of chloride content can be obtained from test kits. These might be suitable when an instant decision is needed or the site is remote, but do require some skill and care to carry out, and are much less accurate than laboratory testing.

Chloride concentrations are used to determine the risk of chloride-induced reinforcement corrosion. If the chloride concentration at cover depth exceeds a threshold value, the steel reinforcement might start to corrode. If the chloride concentration at cover depth hasn’t reached threshold level, the time at which it is likely to do so can be calculated, refer Section 4.2. The rate of corrosion will then be determined predominantly by the availability of moisture and oxygen.

A decrease in chloride ion concentration with depth from the exposed surface indicates the chlorides
were introduced from external sources such as exposure to seawater or seaspray. A constant chloride content with increasing depth indicates chlorides were present from the time the concrete was cast, perhaps added via site-contaminated aggregate or as a chloride-based accelerating admixture.

7.4.4 Resistivity

A method for measuring concrete resistivity on-site is described in the UK Concrete Society Technical Report TR 60 [Reference 189]. Water saturated concrete with no reinforcement corrosion will give a low resistivity value and some references (e.g. TR 60 [Reference 189]) do not adequately highlight this matter. Similarly, water saturated to dry concrete with no reinforcement corrosion will have increasing resistivity to greater than 100,000 ohm.cm.

If electrode potential mapping does not indicate that the reinforcement is activity corroding, there is little point in taking resistivity measurements.

7.4.5 Polarisation resistance

The polarisation resistance or (linear) polarisation resistance (LPR) method is commonly used in research to measure the corrosion rate of steel in concrete. The method is also used for field measurement of corrosion rate and commercial equipment is available. Numerous references describe the method and its limitations. Examples include Green and Grapiglia [Reference 94], Bertolini et al [Reference 55], Andrade and Alonso [Reference 52], Carino [Reference 57], Nygaard [Reference 58], So and Millard [Reference 59], Berke et al [Reference 53] and UK Concrete Society TR 60 [Reference 189]. Polarisation resistance testing involves either a potential shift (potentiostatic from an auxiliary electrode) or current perturbation (galvanostatic from an auxiliary electrode) [References 94 and 189].

Field polarisation resistance tests are usually carried out in areas identified as probably corroding (anodic areas) by electrode (half-cell) potential measurements. Since the corrosion rate is dependent on variables such as temperature and moisture content, it is important to recognise that the rate measured by field polarisation resistance tests is instantaneous and reflects the conditions at that particular moment in time.

For potentiostatic polarisation resistance measurements, a small potential, $\Delta E$ in the range 10-20 mV is applied to the reinforcement (working electrode) to perturb it from the corrosion potential, $E_{corr}$. This results in a current, $\Delta I$, passing between the reinforcement (working electrode) and auxiliary (counter) electrode, which is measured after a suitable delay time to allow equilibrium to be established. From this the (linear) polarisation resistance, $R_p$, can be determined and then related to the corrosion current density using the Stern-Geary equation. The corrosion current density can then be converted into a direct weight-loss of iron in terms of a corrosion penetration rate, as shown in Table 7.2.

While an official standard test method for polarisation resistance on concrete has not yet become available, recommended procedures exist. These include RILEM TC 154-EMC [Reference 52] and SHRP-S-330 [Reference 56].

A key consideration for on-site measurements is the knowledge and determination of the area of reinforcement that is polarised. Limitations and potential errors arising from non-uniform current distribution can be reduced by confining the polarised area through the use of a guard ring auxiliary electrode around a central auxiliary (counter) electrode. Alternatively, the critical length reached by the electrical field needs to be calculated. The RILEM TC 154-EMC document [Reference 52] describes other means of confinement and calculation of critical length. This can include use of additional reference electrodes aligned with the auxiliary

<table>
<thead>
<tr>
<th>Corrosion Current Density ($\mu$A/cm²)</th>
<th>Corrosion Rate (µm/year)</th>
<th>Corrosion Rate Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Up to 0.1 to 0.2</td>
<td>Up to 1-2</td>
<td>Very low or passive</td>
</tr>
<tr>
<td>0.2 to 0.5</td>
<td>2-6</td>
<td>Low to moderate</td>
</tr>
<tr>
<td>0.5 to 1.0</td>
<td>6-12</td>
<td>Moderate to high</td>
</tr>
<tr>
<td>&gt;1.0</td>
<td>&gt;12</td>
<td>High</td>
</tr>
</tbody>
</table>

Table 7.2: Corrosion current criteria for surface applied corrosion rate measurements [Reference 55]
electrode and reference electrode of the polarisation resistance device.

For condition monitoring, polarisation resistance using portable probes is recommended where electrode potentials taken during condition monitoring indicate possible active corrosion, but it is uncertain whether the corrosion rate will be significant (e.g. saturated concrete).

7.4.6 **Petrographic examination**

Examination of concrete samples by microscope techniques will reveal information about concrete composition, aggregate type, concrete quality and the causes and extent of damage.

Petrographic examination is a technique used by geologists to investigate rocks and aggregates. It involves visual examination of concrete surfaces under reflected light in hand specimen or by binocular microscope, or of thin sections under polarised transmitted light. Scanning electron microscopy (SEM) can be used to detect further detail, and is usually augmented by spot chemical analyses by x-ray techniques to identify the composition of specific features. A geologist will be able to analyse concrete samples to identify features related to the aggregates, but if information about the concrete is required, then the examination should be carried out by a petrographer with specialist knowledge about concrete materials and technology.

Samples for petrographic examination are generally prepared from core samples. Other means of sampling may causing microcracking or other damage.

Thin section microscope examination and SEM is valuable to identify the cause and extent of chemical attack both from external aggressive agents such as acids or sulfates and from within the concrete itself such as alkali aggregate reaction (AAR) and delayed ettringite formation (DEF). It is also useful for identifying the cause and extent of physical attack such as by freezing and thawing, surface defects, cracking, or penetration of the concrete by materials such as surface treatments (protective or remedial), and can distinguish zones of complete and partial carbonation. The UK Concrete Society Technical Report TR71 [Reference 90] describes petrographic examination of concrete in more detail.

Petrographic examination of retrieved samples might be specified as part of condition monitoring when:

(a) Aggregates used are potentially alkali reactive, and the means of mitigation are unknown or in question.

(b) DEF is a potential risk due to concrete peak hydration temperature being close to or higher than specified limits.

(c) Surface deterioration due to some form of chemical attack is expected to occur at a rate that leads to a greater than moderate risk in the life of the structure.

Testing at 20% of the structures life will indicate if the potential deterioration of concern is commencing or not.

7.4.7 **Microbial analysis**

Where microbial attack is a potential concern or suspected, such as in water retaining structures, samples of biological slime or other surface deposits can be analysed by a microbiologist to identify the presence of bacteria, algae, fungi, etc, that could attack the underlying concrete or steel (or other metal components).

Specialist microbiological analytical services are available for this purpose. Samples must be collected, stored and transported in accordance with their recommendations.

7.5 **PERMANENT SURFACE MOUNTED AND EMBEDDED MONITORING TECHNIQUES**

It is important that installation of surface mounted or permanently embedded sensors is carried out by experienced persons. Improper installation of sensors or damage to sensors, cables or termination boxes can cause long term problems that are difficult to fix and can also cause difficulties in data interpretation. Furthermore, testing should be carried out on sensors before concrete is placed and again immediately after concrete pours to ensure they are still operational.

Similarly, monitoring and interpretation of results must be carried out by personnel with appropriate experience.

Permanent monitoring devices should not be installed unless the owner of the structure is committed to the monitoring process. To ensure it is undertaken correctly, the nature of the instrumentation, its maintenance requirements and the monitoring process itself must be documented in detail in the asset.
management and maintenance manual. The monitoring must be part of the asset’s maintenance programme. Monitoring may be carried out in-house, or by specialist consultants.

7.5.1 **Corrosion initiation**

The point in time when corrosion of steel in concrete begins can be detected by installing a reference electrode into the concrete structure. A reference electrode can be cast into new or existing structures to raise an alarm when potentials shift substantially and possibly indicate a change in corrosion state of the reinforcement. Typically the reference electrode is of silver/silver chloride or manganese/manganese dioxide composition specifically designed for permanent installation into concrete.

Probes such as “macro-cell probes” can be utilised to predict the corrosion initiation of steel in concrete. The probe is cast into the cover concrete, of newly constructed concrete structures with the capability of measuring most of the relevant corrosion parameters. Macro-cell probes are a multi-sensor system, which typically consist of three or four steel anodes and one noble metal cathode. The anodes are placed in varying distances from the exposed concrete surface. By measuring the corrosion current and electrode potential in different depths in the concrete cover it is possible to predict when the corrosion front (i.e. chlorides or carbonation) will reach the reinforcement and thus prepare the necessary maintenance measures in time before damage occurs.

7.5.2 **Corrosion rate**

Monitoring of corrosion rate is as important to define when corrosion starts on reinforcement set at different covers as it is to measure the corrosion rate after corrosion initiation of the actual reinforcement.

Corrosion rate is most widely determined by the polarisation resistance technique as discussed in Section 7.4.5. For a typical permanent condition monitoring system, a polarisation resistance probe can be embedded in concrete quite often with the capability of detecting corrosion potential, concrete resistance and concrete temperature.

Permanent corrosion rate monitoring systems can be installed for either existing structures or new structures. Installation for new structures is carried out pre-pour by tying to the rebar and connections to the rebar allow corrosion rate data to be collected. Polarisation resistance probes can also be cast into existing concrete elements to provide information about corrosion rate over a varying depth profile. Several independent polarisation resistance electrodes at varying depth within concrete cover, allow an accurate estimation of the rate at which the corrosion front is penetrating the concrete.

An automated data-logging system or a portable data collection system is often employed to collect, store and transfer polarisation resistance data in a permanent monitoring system.

7.5.3 **Strain, vibration and deflection**

Strain, vibration and deflection measurement might be employed as part of a condition monitoring programme for several reasons:

(a) Concern over collapse of ageing infrastructure due to unnoticed deterioration.

(b) The appearance of new cracks as an indicator of untoward structural actions. Inspectors are often briefed on critical areas of potential high stress for more thorough inspection. Strain can be monitored in critical areas using embedded or surface mounted strain gauges and these would provide a much clearer impression of the actions occurring than intermittent visual inspection.

(c) Long term structural performance can be tracked by monitoring the change in strain response to load with age. Load can be from normal traffic conditions or wind load (e.g. to obtain natural frequency) or from artificial loads (e.g. for load testing) such as weight trucks. Signature strains collected during commissioning can be compared to periodic re-measurements and both can be compared to structural predictions.

(d) Determination of frequency/tension of cables – vibration under wind loads (e.g. cables).

(e) Measure the effectiveness of strengthening – take deflection readings before and after construction to monitor outcomes for projects and match modelling.

The types of strain, vibration and deflection measurement instruments are outlined in Table 7.3. The measurements from some of these instruments can be linked into the system, if necessary, used for monitoring of corrosion.
### Table 7.3: Types of strain and displacement measurement

<table>
<thead>
<tr>
<th>Instrument</th>
<th>Monitoring Speed</th>
<th>Monitoring Region</th>
<th>Description</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Resistive strain gauge</td>
<td>Fast, sufficient to allow fully dynamic monitoring</td>
<td>Can be affixed to outside of structure, or smaller gauges used to obtain local reinforcing strain or similar parameters</td>
<td>Metal and plastic leaf bonded to monitoring region. Measured resistance varies with elongation.</td>
<td>Resistive strain gauges require temperature compensation and are often used in pairs. When fixing to concrete it is important to use a strain gauge relatively larger than aggregate size to ensure a wholesale strain is measured, rather than local strain in paste or aggregate.</td>
</tr>
<tr>
<td>Vibrating wire strain gauge</td>
<td>Slow, can monitor long term strain changes</td>
<td>Can be affixed to surface or embed within structure</td>
<td>Steel gauge sealed with end points mounted to structure, either flanges embedded or grouted or bonded.</td>
<td>VW strain gauges when heat treated to remove residual strain of construction offer long term stability superior to other strain measurement.</td>
</tr>
<tr>
<td>Crack meter</td>
<td>Slow, can monitor long term strain changes</td>
<td>Affixed to surface, usually over joints or cracks</td>
<td>Similar construction to VW gauge, but will monitor elongation of tens of millimetres</td>
<td>Generally larger than strain gauges. Up to 3 sensors required for triaxial movement.</td>
</tr>
<tr>
<td>Mechanical strain gauge</td>
<td>Manual, data collected by hand</td>
<td>Discs bonded to surface, change in distance measured by precision gauge</td>
<td>Precision gauge, usually mounted to low temperature variable metal. Metal points engaged with studs to monitor change in position.</td>
<td>Used where long term performance monitoring required.</td>
</tr>
<tr>
<td>Linear variable displacement transducer</td>
<td>Slow to fast, depending on type used</td>
<td>Measures displacement between fixed reference points</td>
<td>Analogous to an electrical transformer, as magnitude of deflection increases in positive or negative direction the AC voltage will increase.</td>
<td>Types available for static and dynamic monitoring. Can only determine the structural deformation vs the reference point.</td>
</tr>
<tr>
<td>Accelerometers</td>
<td>Fast, 10 times maximum sampled frequency required for 5% accuracy. (e.g. 1 Hz sampling rate for measurement of 0.1 Hz vibration frequency signal)</td>
<td>Cables, questionable elements, varied heights on structures to match vibration modes</td>
<td>Piezoelectric sensor that directly measures acceleration. Usually uniaxial although tri-axial options can be sought. Low frequency large sensors required for civil applications.</td>
<td>Accelerometers can produce a large sample of data very quickly. This can prevent analysis on long term projects and cause issue in data storage and transfer. Event type data logging can be more advantageous, whereby the data logger is programmed with a threshold, whereby only major events are logged.</td>
</tr>
<tr>
<td>Interferometric radar</td>
<td>Very fast, e.g. can monitor cable vibration frequency</td>
<td>Any location observable remotely</td>
<td>The change in phase of a reflected radar wave is monitored over various points on the structure with one instrument to give displacements.</td>
<td>No contact needed with the structure. Can detect displacement with a resolution of 0.01 mm and frequencies of 100 Hz.</td>
</tr>
</tbody>
</table>
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