Structures Engineering

Detailed Non-Destructive Bridge Inspection Guidelines

Concrete and Steel Bridges (Level 3 Inspections)

Document No. 6706-02-2241

DETAILED NON-DESTRUCTIVE BRIDGE INSPECTION GUIDELINES CONCRETE AND STEEL BRIDGES (LEVEL 3 INSPECTIONS)

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AUTHORISATION

As head of Structures Engineering of Main Roads Western Australia, I authorise the issue and use of this manual.

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1.0 GENERAL

1.1 Introduction

This document is one of a set of documents, which together, prescribe and detail the management processes and procedures used by Main Roads Western Australia (MRWA) to manage its bridges. The types of bridge inspections and their associated scope and general management policy are documented in the *Structures Inspection & Information Management Policy Doc.No. 6706-01-202*.

Detailed Non-Destructive Inspections for Concrete & Steel Bridges (to be referred to *as* Level 3 Inspections) are special inspections involving material condition assessment which may be instigated by request for a specific reason. They are not typically scheduled but may be required due to concerns over a structure's safety, its condition or load capacity or for a structure subject to complex associated repair, strengthening or widening works. They may also result from scheduled follow-up material surveys or be required to inspect those components that are not accessible during a detailed visual inspection (a Level 2 Inspection).

These Guidelines provide information on material condition assessment, particularly non-destructive testing (methods to examine materials or components in ways that do not impair their future usefulness and serviceability) that may be used to provide an assessment of the structures' material condition and in-service durability.

1.2 Qualification of Inspector

At this time there is no formal qualification identified for the personnel executing Level 3 Inspection activities and Main Roads will make individual qualification assessments as and if required. In general, the Inspector shall be very knowledgeable in non-destructive testing methods and techniques as well as various aspects of construction materials and bridge engineering including design, load rating, construction, rehabilitation and maintenance.

2.0 PURPOSE AND SCOPE

The purpose of this document is to provide a consolidated reference on the standard non-destructive testing techniques available to asset managers and more importantly, guidance on their appropriate use and interpretation of results.

This document is intended to assist Main Roads staff in the scoping of appropriate testing and investigation works such that the output received will be of a high standard and consistency and be a positive contribution to the management of structures. It provides the following:

- Information on deterioration mechanisms and common defects
- Guidance on planning for a Level 3 Inspection
- Guidance on the preparation of a scope of works
- Guidance on what tests are appropriate
- Detailed investigation techniques to identity the cause, extent and rate of deterioration of concrete and steel
- Guidance on the output required, presentation of results
- Template for the Report

3.0 OTHER REFERENCES

- Structures Inspection and Information Management Policy

  Document No. 6706-01-202
- Detailed Visual Bridge Inspection Guidelines for Concrete and Steel Bridges (Level 2 Inspections)
- Document No: 6706-02-2233

4.0 OBJECTIVE AND EXTENT OF A LEVEL 3 INSPECTION

Level 3 Inspections may be instigated for different reasons and may have differing purposes and methodologies. However the focus of these Guidelines is the non-destructive testing of materials and the objectives and extent are a reflection of that.

4.1 Objective of a Level 3 Inspection

The main objectives are:

- To establish and record the current physical and functional condition of a structure;
- To identify likely future problems and the approximate timing of those problems;
- To determine and measure the type and extent of the maintenance needs;
- To establish a history of material performance;
- To provide feedback to design, construction and maintenance engineers;

4.2 Extent of a Level 3 Inspection

The extent of an inspection will be defined in the investigation brief. The extent may be very broad and will depend on the purpose of the inspection. For example the purpose may be testing of material condition to establish a reference from which to measure and monitor deterioration (establishing a benchmark), or to establishment extent of maintenance works, (defect identification) or to provide information on components that are not accessible during a Level 2 Inspection.

4.3 Outputs of a Level 3 Inspection

The outputs of a Level 3 Inspection include:

- Summary of purpose and scope
- Description of test plan and test methods utilised
- Diagrammatic and photographic information on test locations
- Test results with analysis and interpretation where required
- Photographic records of all deteriorated materials observed on site.
- Recommended maintenance options including intervention schedule for use by the Asset Management Structure (AMS)
- Recommended repair materials
- Quantification of the extent of repairs suitable for comparison of alternatives and also for preliminary budgetary purposes.

Section 7.4 covers the expected reporting requirements in more detail

5.0 LEVEL 3 INSPECTION PROCEDURE

Level 3 Inspections of bridges are associated with fully identifying the cause and extent of observed, indicated or suspected deterioration and the severity of structural or material distress, which could all affect the load-bearing capacity or service life of the structure. This section provides guidance concerning the available test methods for determining the cause of the deterioration and quantifying its extent.

A Level 3 Inspection to assess the cause, extent and rate of deterioration (where possible) is preferable before repair work is undertaken on structures. If a repair is to be successful it must address the cause, seek to repair the whole extent of the damage and seek to ensure that the structure is protected from further deterioration or recurrence of the original cause for the whole of its projected lifespan.

The testing and monitoring of structures seeks to both locate and identify the various defects that can occur in structures and subsequently allow a rational engineering assessment to be made of the need for repair or maintenance. Prior to treating any structure showing signs of distress it is vital to establish with certainty the cause of defects. The purpose of testing should be to determine whether the distress is attributed to deterioration of the concrete or other materials, direct corrosion of the reinforcement in concrete, or to corrosion of metallic components.

The following Figure 1 presents the general approach adopted in Level 3 Inspections for the investigation and assessment of concrete and steel structures:

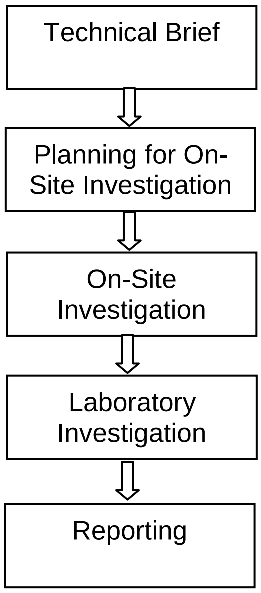


Figure 1 - GENERAL APPROACH TO LEVEL 3 INSPECTIONS

5.1 Technical Brief

The Technical Brief is prepared to identify and agree with MRWA the objective and scope of the investigation. The technical brief consists of background information, the investigation and testing schedule required at each bridge to be inspected, comment on any special knowledge of the bridge and/or site, and identify any special reporting. The investigation and testing schedule should be devised with reference to the bridge location, the environment (or environments) the bridge components are likely to be exposed to, drawings and previous inspection reports.

A template for a Level 3 Inspection brief is provided at Appendix A.

5.2 Planning for a Level 3 inspection

5.2.1 General

Prior to commencing site inspections, the Inspector must ensure that all the relevant documentation (e.g. reference manuals, inspection reports, drawings) is collected and the inspection and safety equipment is appropriately calibrated/certified and tested as applicable. Planning for Level 3 Inspections is carried out to allow an on-site investigation to be undertaken for the specific bridge in an efficient and safe manner. The activities of the planning include the following processes:

a) Previous detailed visual inspections (Level 2) and Level 3 Inspections should be reviewed prior to the site visit.

b) Identify the location of each bridge and determine parking, access and traffic management requirements. Determine what approvals if any are required for access.

c) Plan operational safety to ensure Occupational Safety & Health and Environmental Regulations and any other Regulations are met

d) Determine what access equipment is required

e) Estimate the time and materials required for the bridge inspection.

f) Determine surface cleaning requirements, if any (grime, coating removal) for suitable testing

5.2.2 Component Identification

Component identification and bridge orientation shall be as defined in Section 5.3 of the Detailed Visual Bridge Inspection Guidelines for Concrete and Steel Bridges (Level 2 Inspections) Document No: 6706-02-2233.

5.2.3 Operational Safety

All inspection procedures and operations must comply with the relevant rules and regulations of the Occupational Safety and Health Act 1984 and appropriate MRWA operational safety guidelines and documents.

If inspection from water is required, any vessel used for this purpose and its operation must satisfy the legal obligations of the Western Australian Marine Act 1982, other relevant Acts, and associated regulations.

Where inspections are to be carried out on bridges located over or under the assets of other Authorities, the relevant regulations and Codes of Practice relating to work on or close to their assets must be adhered to, and where necessary, referred to in the procedures developed for the inspection. This is particularly important when inspecting bridges over electrified railways.

The Inspector must also ensure that the appropriate arrangements are in place with the relevant road, railway or other authorities for temporary access as required to carry out the inspection. For structures over railways, the Inspector must hold a relevant permit for access to electrified rail (currently in the Perth Metropolitan Area), or a relevant permit for access to non-electrified rail networks in both the Perth Metropolitan and regional areas.

5.2.3.1 Parking and Access

At some bridge sites it may be difficult to find a safe parking location especially at bridge sites on major roads and highways where the traffic volumes and speeds are high or where there is insufficient room within the roadside. It is important that the position of the Inspector's parked vehicle does not block the road and road sight distances to motorists in both directions.

Access to bridge components for inspection via bridge abutment embankments can potentially pose a safety risk to the Inspector due to steep embankments and loose surface material. The Inspector should take note of conditions prior to arriving on site and make suitable arrangements for safe access as required.

Allowance should be made for removal of cover plates where required to access some bridge components.

The safe use of necessary access systems such as an underbridge inspection unit or scaffolding should also be considered.

5.2.3.2 Traffic Management

Traffic management such as lane closures, shoulder closures or speed reductions may be absolutely necessary to access certain bridge components. The Inspector will be responsible for adequate traffic management in accordance with the Main Roads Traffic Management for Works on Roads Code of Practice.

5.2.3.3 Confined Spaces

The governing regulations for confined spaces are the 'Occupational Safety and Health Regulations 1996', Regulation 3.82 which provides a definition for how a 'confined space' is identified and these definitions are reproduced below:

"Confined space means an enclosed or partially enclosed space which -

(a) is not intended or designed primarily as a workplace, and

(b) is at atmospheric pressure during occupancy, and

(c) has restricted means for entry and exit,

and which either-

(d) has an atmosphere containing or likely to contain potentially harmful levels of contaminant; or

(e) has or likely to have an unsafe oxygen level; or

(f) is of a nature or is likely to be of a nature that could contribute to a person in the space being overwhelmed by an unsafe atmosphere or a contaminant."

In addition, these Regulations make reference to Australian Standard AS 2865 'Confined Spaces' with respect to work being done in a confined space. Note that AS 2865 also contains definitions of a confined space, however where there is a difference the Regulations will take precedent.

A number of the larger bridge structures within Western Australia have an enclosed space that would be covered by the definitions (a), (b) and (c), however Structures Engineering with its knowledge of the bridge infrastructure is unaware of a bridge space which meets the definitions (d), (e) or (f) of the Regulation. This means that there are no known bridges with a 'confined space' as defined in the Regulations.

If the Inspector feels the air quality in an enclosed or partially enclosed space is compromised and will impact the safety of inspection, the Inspector shall seek guidance from MRWA.

5.3 On-site Investigations

Visual Inspection

A visual inspection is the first step taken in a L3 Investigation and shall be completed to determine the general condition of the structures, identify areas of distress, likely sources of problems and identify the visible defects. This will confirm that the test plan is suitable and critical areas have been included. All visible defects such as concrete cracking, apparent delamination, concrete spalling, exposed corroding reinforcement etc. need to be recorded with standardised notation. Reinforcement size, type, orientation, corrosion state and loss of cross section area is to be recorded at all exposed reinforcement locations. Photographs of visible defects of Condition State 3 and worse shall be taken to document the in situ condition. This inspection is similar to a Level 1 Inspection but limited to the extent of the Level 3 Inspection.

In Situ Testing

Surface tests – such as rebound hammer, half-cell potentials, resistivity, ultrasonic pulse velocity, corrosion rate measurements etc.

Material sampling – methods for taking samples of materials from the structure to determine composition and properties of the material or the presence of deleterious substances, such as chlorides in concrete. Some sampling may be taken from inside core holes.

Intrusive testing – such as drilling holes or concrete breakout to determine the condition inside the structure that is not revealed by visual inspection, such as the corrosion state of steel reinforcement, the condition of post-tensioning tendons or the interior of box girder sections.

5.4 Laboratory Investigation

Laboratory investigation becomes essential where the available information is insufficient to complete a non-destructive evaluation with confidence. In conjunction with the onsite investigation program, laboratory investigation may be required to understand potential causes of deterioration. For example chemical analysis can provide a wealth of information on mix constituents, contaminants and possible causes of deterioration.

Samples from structures for laboratory investigations are in various forms such as concrete fragments, cores or powder obtained from drilling. The ideal method of sampling concrete is by diamond core drilling (refer Figure 2). Large fragment samples may also be of use but damage during sampling can limit investigation. Sampling locations should be chosen to represent the variation in the condition of the materials on site. In many cases it is useful to examine samples of undamaged as well as damaged materials in order to establish the original quality of the material.

Powder samples can be collected rapidly and inexpensively with readily available hand held equipment (refer Figure 3). Drilled powder samples can be used for simple analyses such as chloride content, but are not recommended for more complex determinations such as cement content.

Figure 2 – Example of taking a core

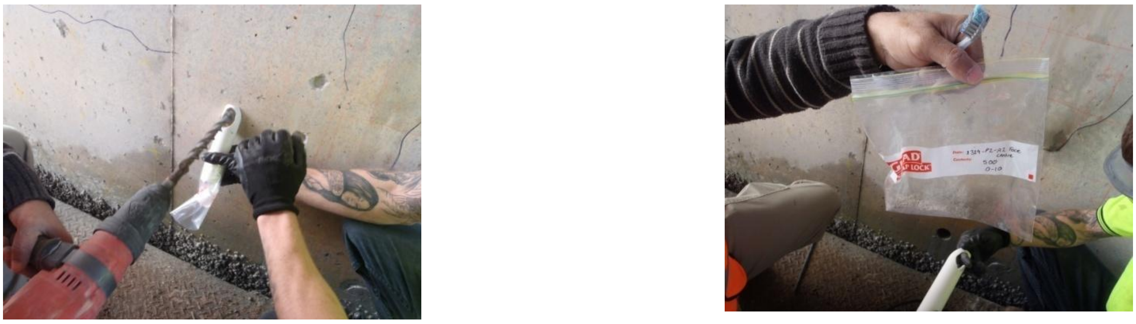


Figure 3 - Example of collecting powder sample for laboratory analysis

A photographic record at sample locations must be taken. The samples obtained should be labelled, their orientation clearly marked and wrapped in cling film or stored in airtight sample bags as soon as practicable after sampling.

Once samples of concrete have been obtained, whether by coring, drilling, or other means, they should be examined in a qualified laboratory. In general, the examination will include one or more of the following examinations:

a) Petrographic examination (cores or fragments only)

b) Chemical analysis (chloride content, cement content, original water cement ratio, sulfatecontent etc.)

c) Physical analysis (compressive strength, density etc.)

5.5 Reporting

The report on the investigation, testing, conclusions and recommendations must be consistent for quality, content format. A template for a Level 3 Investigation Report is provided in Appendix B. The Level 3 Inspection Report should consist of the following general sections:

- Executive Summary
- Introduction,
- Detailed investigation results.
- discussion of investigation results,
- Summary of current condition
- Conclusions,
- Recommended remedial actions for defects identified
- Recommendations on further testing and schedule, if required.

It is expected that any conclusions and recommendations be made by an engineer suitably experienced in bridge engineering.

5.5.1 Introduction

The introduction should provide the background of any issues with the bridge and the detailed scope of works. The introduction may include some or all of the following depending on their relevance:

- **Background –** Summarises the reason why the inspection is being undertaken. It includes bridge number, MRWA region the bridge is in, site investigation dates.

- **Scope of works –**The scope of work performed and the test schedule should be detailed. Section 8 of this Manual provides assistance in what to include in a test schedule. Any bridge specific scope should be also detailed, for example if inspection is only limited to some components (at the request of MRWA), or if sections of the bridge were not accessible, and therefore not inspected.
- **Bridge details –** Include bridge general arrangement drawing, or if not available, the construction date (or an estimate of the construction date), brief dimensional details, main bridge components and possibly a photo of the general arrangement.
- **Bridge Location and Exposure Environments –** Includes a map of bridge location, distance from the coast line or the nearest corrosive environment (wasteland, acid sulfate soils etc), general comment about the immediate bridge surrounds (saline water, corrosive soil, airborne chloride/carbon dioxide levels) and type of area the bridge is in (urban, industrial etc). It also includes the exposure classifications of the bridge for each zone (e.g. buried or submerged zone, atmospheric zone etc), in the form of a table and/or figure (refer Appendix F).
- **Durability Information –** States information regarding the durability of the bridge obtained from available documents that may include the original design, *Asset Management Plans* and repair documents. Relevant information includes original design life, minimum concrete/reinforcement strength(s) for each component, minimum design cover, current exposure classifications and whether or not the original design complies with the current design standards.
- **Project Inputs –** a summary of existing bridge drawings and previous reports (such as a Level 2 Inspection Report). This section should also summarise previous inspection findings and recommendations, past maintenance /repair history and establish original projected service life.

5.5.2 Detailed Investigation Results

As a minimum, this section comprises the following:

- The key findings from the visual and delamination survey, including representative photos, and any changes to previous Inspection comments, particularly if the comments were from a Level 2 Inspection carried out three or more years before the Level 3 Inspection.
- Reassignment of the condition states of the bridge components based upon MRWA assessment guidelines, only if the condition differs from the most recent Level 2 Inspection.
- The key results from the detailed investigations, in table format where possible. Where available, the results should include summaries of the rebound hammer tests, concrete breakouts, covermeter surveys, half-cell potential surveys, resistivity measurements, carbonation depth measurements and predicted/modelling results, chloride content and predicted/modelling results, sulfate content, water or soil analysis (if any) and any other tests carried out. Steel reinforcement corrosion condition observed from concrete breakout tests shall be classified in accordance with Appendix E.
- All investigation results should be summarised in a summary table and provided in detail within the appendices. All drawings should be to scale and content should be in accordance with the survey legend provided in Appendix C. Example template sheets are provided in Appendix D.

5.5.3 *Discussion of Investigation Results*

As a minimum, this section should comprise the following:

- Discussion of the different results and observations for each component in one sub-section of the Report, and preferably showing the relationship between the results and the observed deterioration,
- Discussion of contradictory results and observations (if any), this may include the limitations of the test methods,
- Diagnosis of the cause of defects and prognosis to identify future deterioration. A number of possible causes may be provided however in this case further testing should be recommended in the report.

If part of the scope, this section should also comprise the best estimates of the extent and rate of deterioration. Information collected and assumptions used to predict the remaining service life (e.g. the measured or estimated corrosion rate) are to be included. The predicted remaining life for each component should be provided in the conclusion, however assumptions regarding this prediction based on theory, engineering judgement and test results should be provided and justified in this section.

5.5.4 *Summary of Current Condition*

As a minimum, this section should summarise the findings of:

- Visible defects and possible non-visible defects
- Current rate of deterioration, and
- Impact of deterioration on the long term durability and /or service life.

5.5.5 *Conclusion*

With the current materials condition (based on visual and test results) and information obtained from the service life determination (including assumptions and deterioration rate test/modelling results) the predicted remaining service life of the bridge structural components should be determined and summarised. An example of a summary of the predicted service life for a bridge is presented in Table 5.1.

Component Number	**Structural Component**	**Approximate Remaining Service Life** (Years from 2011)	**Works Required in Next 10-20 Years**
1	Abutments	>100	No
2	Pier cap beams	>100	No
3a	Pier Columns in tidal and splash zones	<10	Concrete Patch Repair
3b	Pier Columns in atmospheric zone	<65	Protective Coating
4	Deck Beams	>100	No
5	Deck Slab	~50	No
6	Pier Pile Caps	~20 (based on chloride modelling only, needs further investigation)	Yes

Table 5.1 - Example of Summary of Service Life Prediction

5.5.6 Recommendations

The recommendations should include further testing required (if any) and the test schedule, as well as recommendations for remediation and maintenance works and the appropriate options available.

Recommendations for remediation and maintenance works, their extent and the approximate timing of the works shall also be included.

Where appropriate, include comment on further testing/assessments required (if any) and the scope of any such work.

6.0 PERFORMING LEVEL 3 INSPECTIONS

6.1 Selection of Test Methods

There is a wide range of test methods available for Level 3 Inspection of steel and concrete based structures that provide information of the condition of the materials and identify the presence of defects within the structure. Collectively these techniques enable almost any irregularity of practical significance to a structure to be detected. Whether this can be achieved at an acceptable cost and speed of inspection is more problematic. Often no single method of testing is capable of detecting all the defects within a structure, or checking the condition of the materials within the structure.

The investigator should not only understand the inherent capabilities and limitations of the chosen methods, but should also have an understanding of construction materials, structural behaviour, and deterioration mechanisms. Knowledge of construction materials is helpful in anticipating the most likely locations of internal anomaly. Knowledge of structural behaviour is valuable in selecting those portions of the structure that are most vulnerable to the presence of defects. Knowledge of deterioration mechanisms is important in deciding what needs to be measured.

It is essential that information from several individual techniques be used collectively for a thorough investigation. Based on reliability, simplicity, and cost, some method or techniques are preferable over others.

PIARC 2011 report presented various types of non-destructive testing techniques utilised to determine key characteristics for different bridge materials and the condition assessment of road bridges in different countries. Table 6.1 summarises commonly used test methods extracted from PIARC that could be performed during Level 3 Inspections of road bridges.

6.2 Recommended Test Approaches

Table 6.1 presents the test methods available and guidance in when the tests might be used. The recommended approach has been made based on the advantages and limitations of each test method presented in Table 6.2. which provides a more detailed description of each test method.

Properties	Available Test Methods	When to use
Integrity and Structural Performance	Visual Inspection Delamination Survey Impact-echo Ground Penetrating Radar (GPR)	When likely delamination or spalling is suspected, or when areas of delamination or spalling have been identified. When voids are suspected.
Concrete Properties Affecting Durability and Deterioration	Cement content and type Chloride and Sulfate Contents Ultrasonic Transmission Velocity (Ultrasonic Pulse Velocity) Alkali-Aggregate Reaction (AAR, ASR) Petrographic examination Apparent Volume of Permeable Voids (AVPV)	Dependant on purpose – to determine internal factors and origin of cracking (such as in the case of AAR or DEF), to assist in determining concrete strength and durability, to identify and assess extent of chemical deterioration mechanisms such as sulphate, acid sulphate or soft water (leaching) attack.
Location of Reinforcement	Ground Penetrating Radar Covermeter Survey	When cover issues exist and where knowledge of cover is required as input for other assessments e.g. comparison to depth of carbonation
Corrosion of Embedded Steel	Carbonation depth Chloride Profile Corrosion Potential (electrochemical, half-cell) Concrete Resistivity Corrosion Rate Measurements/Linear Polarisation Concrete Breakout/Reinforcement Inspection	When condition of steel reinforcement needs to be known. Visual inspections may have identified significant cracking, rust staining, and moisture ingress issues.
Concrete Strength	Pullout Compressivestrengh Tensile Strength Rebound Hammer	To determine concrete strength or an indication of concrete strength
Steel Structures Deterioration	Visual Inspection Dye Penetrant Testing Magnetic Particle Testing Ultrasonic Testing Radiographic Testing Eddy Current Tensile Testing Hardness/Rebound Testing Microstructure Testing	When corrosion is identified on welds or weld defects are suspected. When thickness or properties of steel needs to be determined. When “work hardening” or brittleness is suspected. When general corrosion or environment-assisted cracking is suspected.

Table 6.1 - Summary of Test Methods for use in Level 3 Inspections

TEST METHODS				
Name of Tests	**Standard Test Methods / Techniques**	**Principle**	**Application**	**Advantages/ Limitations**
Integrity and Structural Performance				
Visual Inspection	MRWA Level 2 Inspections Guideline	Observe, classify and document the appearance of distress and defects on exposed surfaces of the structure. Map distress and defects.	Surface defects such as cracking, spalling, leaching, erosion or construction defects.	Simplest and least expensive; extensive information can be gathered from visual inspection to give a preliminary indication of the condition of the structure and allow formulation of a subsequent testing programme. Does not cover areas not visible to the eye.
Delamination Survey	ASTM D4580	Tap the concrete surface using a light hammer to identify delaminated concrete through a “hollow” impact sound.	Assessment and location and extent of discontinuity in the cover concrete which is substantially separated, but not completely detached, from the concrete.	Low cost; quick; no instrumentation needed; easy; portable; can measure large areas; can identify a variety of additional information (hardness, voids, peeling etc.) Indicative only; need access to surface; depths of around100mm; non-specific; inconsistent results; requires further tests to confirm results; not good for thin components; cannot define deep voids.
Impact Echo	ASTM C1383	Receiver adjacent to impact point monitors arrival of stress waves as they undergo multiple reflections between surface and opposite side of plate-like member or from internal defects. Frequency analysis permits determination of distance to reflector if wave speed is known.	Locate a variety of defects within concrete components such as delaminations, voids, honeycombing, or measure component thickness.	Relatively low cost; easy; portable; direct results; surface roughness does not affect results. Experience for testing and interpretation; requires surface preparation; background noise. Reinforcement clouds results.
Ground Penetrating	ASTM D6432	Radio frequency waves from radar transmitter are directed	It is capable of detecting a number of parameters within	Good identification of reinforcing bars, prestressing strands, cable ducts, zones of

Radar (GPR)		into the material. The waves propagate through the material until a boundary of different electrical characteristic is encountered. Then part of the incident energy is reflected and the remainder travels across the boundary at a new velocity. The reflected (echo) wave is picked up by a receiver. The transducer is drawn over a surface and forms a continuous profile of the material condition below. The equipment consists of a radar console, a graphic scanning recorder and a combined transmitting and receiving transducer.	concrete structures such as the location of reinforcement, the depth of cover, the location of voids, location of cracks, in situ density and moisture content variations. Can also detect the location of reinforcement and the depth of cover.	varying moisture content and thickness of slabs, and a fair assessment of delaminations and large voids in concrete. Quick; non-disruptive; no need to open up structure; good coverage. Expensive; cannot see through areas with heavily congested steel; does not differentiate between defect types: reliant upon operator judgement; complicated equipment setup; specialist experience in data interpretation. Other tests needed to confirm results.
Concrete Properties Affecting Durability and Deterioration				
Cement content and type	BS 1881: Part124: 1988	Cement content by calcium oxide - analysis for hardened concrete.	Assess concrete quality	Quick; low cost; reliable Reliability is affected by knowledge of cement chemistry and aggregates related to the particular structure. Experience and correlation with other test data is needed for interpretation.
Chloride and Sulfate Content	AS1012.20-1992, BS 1881: Part 124	The sample is dissolved in hot nitric acid to provide a solution from which aliquots may be tested for chloride or sulfate content.	Assess susceptibility of concrete to sulfate attack. provide input data for chloride induced corrosion service life modelling.	Low cost; quick; direct results, reliable; accurate. Interpretation requires experience. Need to drill holes or collect core samples and repair; core holes may cause damage to the member from which the core is taken.
Ultrasonic Transmission	ASTM C597-	This technique measures the transit time (in microseconds)	Determination of the variability and quality of concrete by	Easy; portable; relatively quick; relatively low cost; measures from one side only; excellent

Velocity (Ultrasonic Pulse Velocity)	02	of ultrasonic waves passing from an emitter transducer through a concrete sample to a receiver transducer.	measuring pulse velocity. Using transmission method, the extent of such defects such as voids, honeycombing, cracks and segregation may be determined. This technique is also useful when examining fire damaged concrete.	for determining the quality and uniformity of concrete; path lengths of 10m to 15m can be inspected with suitable equipment. Data can only be usefully interpreted where the distance between the transducers is accurately known (generally better than ±2%). Indirect surface testing; difficult to use on rough surface (PUNDIT or AU2000); variations with concrete quality; no information about defect depth; affected by many factors including type of concrete, aggregate, temperature, humidity, roughness, high density of reinforcement etc; calibration to estimate concrete strength is required; expertise is needed to interpret the results; very time consuming as it takes only point readings.
Identification of presence of ASR	MRWA Test Method WA 621.1	Application of acidic uranyl acetate solution forming a complex with components of ASR gel that fluoresces under UV-C light.	Identification of likely presence of ASR.	Quick and relatively inexpensive. Requires experience in interpretation, especially in structures exposed to marine or other high salinity conditions.
Petrographic Examination	ASTM C856 (hardened concrete) or C295 (aggregate)	Microscopic visual examination of polished thin sections under polarised and unpolarised light.	Identification of the presence of ASR susceptible aggregates and diagnosis of the presence of ASR.	Accurate determination of cause(s) of distress; degree of damage; quality of concrete when originally cast and current. Expensive, needs to be performed by a person experienced with concrete and specifically with diagnosis of ASR. Additionally can provide information on: Cement content and type; aggregate type; cement replacement materials; water cement ratio; air void content including entrained air and entrapped air; deterioration mechanisms

				such as sulfate, acid sulfate, leaching, fire damage, DEF; aggregate or cement paste shrinkage; carbonation; detection of unsound contaminants.
Apparent Volume of Permeable Voids (AVPV)	AS 1012.21; ASTM C642-06	The volume of interconnected void space of a concrete specimen which is emptied during the specified oven-drying and filled with water during the subsequent immersion and boiling as a result of capillary suction, expressed as a percentage.	Determine the water porosity or permeability of the concrete microstructure.	General indication of quality of the concrete Can be used on a regular basis as a diagnostic tool as a part of condition surveys of existing concrete structures. As it requires the extraction of concrete cores, it nevertheless provides a very cheap and a non specialised way of establishing the quality and potential long-term performance of concrete, although is not intended as an absolute measure of durability.
Location of Reinforcement				
Ground Penetrating Radar (GPR)	ASTM D6432	As above	GPR is capable of detecting the location of reinforcing bars, prestressing strands and cable ducts in concrete as well as their depth of cover. Can also detect a number of parameters in reinforced concrete structures including the location of voids, location of cracks, in situ density and moisture content variations.	As above
Covermeter	MRWA Test Method WA 623.1 or BS1881-204:1988	A low frequency magnetic field is applied on the surface of the structure; the presence of embedded reinforcement alters this field, and a measurement of this change provides information on the	Locate embedded reinforcement, measure depth of cover, and estimate approximate diameter of reinforcement.	Cost effective; quick; portable; easy; direct results; can measure large areas; reasonably accurate; readily available. Difficult to identify separate bars in heavily reinforced areas; need access to the surface; depth range 30-180; can only detect the first layer of reinforcement;; accuracy depends on

		reinforcement.		correct calibration; high voltage cables can disturb readings; presence of perpendicular reinforcement and iron inclusions in the concrete can alter readings; cannot detect stainless steel; must make numerous readings.
Corrosion of Embedded Steel				
Carbonation Depth	MRWA Test Method WA 620.1	Apply phenolphthalein solution uniformly to freshly exposed concrete surface and measure the depth of carbonation (absence of colour).	Assess corrosion protection value of concrete with depth and susceptibility of steel reinforcement to corrosion due to carbonation. Carbonation depth is used to assess whether the reinforcement is likely to have depassivated leading to corrosion. The results can be used to model concrete carbonation rates to estimate remaining service life.	Low cost; quick; direct results, easy; evaluation; reliable; accurate. Need trained people; need to drill holes or collect core samples and repair; only freshly exposed surface can be tested; powder contamination can affect the results; one location only; not valid for concrete with large aggregate unless a representative area of surface can be tested.
Chloride Profile	AS1012.20, BS 1881: Part 124	Core samples or powder samples are collected for laboratory analysis. The sample is dissolved in hot nitric acid to provide a solution from which aliquots may be tested for chloride or sulfate content.	Assess risk of steel reinforcement to corrosion due to chloride ingress. Chloride profile will show if chloride has reached the corrosion activation threshold concentration at the steel reinforcement. The chloride profile also can be used to model future deterioration and remaining service life.	Direct results; easy; rapid results for estimate of chloride content; low cost compare to other methods; accurate, variation only 4-5%; quick; strong indicator of corrosion potential; determine areas to be rehabilitated; helps to determine type of repair needed. Need trained people; need access to surface; repair of drill or core holes required; specialist evaluation, can be time consuming; need several samples to draw reliable conclusions; core holes may cause damage to the member from which the core is taken. Interpretation of concentration profiles and

				service life modelling requires experience.
Corrosion Potential (electrochemical, half-cell)	ASTM C876	Measure the potential difference (voltage) between the steel reinforcement and a standard reference electrode; the measured voltage provides an indication of the likelihood that corrosion is occurring in the reinforcement.	Identify region or regions in a reinforced concrete structures where there is a high probability that corrosion is occurring at the time of the measurement.	Quick; easy; objective data; relatively low cost; large areas can be measured; strong indicator of corrosion potential; can result in timely intervention; helps to determine type of repair needed; allows quantification of area likely to be corroding. Need trained people; specialist interpretation required; requires complementary testing to verify results; calibration is required before using the results; can only measure potentials on first layer of reinforcement; needs connection to reinforcement, with subsequent repairs required; influenced by humidity in the concrete; cannot be used when it is cold, <5°C ; coatings may need to be removed; readings overhead could be misleading; requires numerous readings; results can be inconsistent; experience in corrosion required for assessment.
Concrete Resistivity	MRWA Test Method WA 622.1, ASTM G57-06	Resistivity is measured by inserting 4 electrodes into small holes on the surface and passing a current between the outer electrodes and measuring the voltage between the inner contacts. This potential difference and measured current provide an estimate of resistance which can be converted into resistivity.	It is used for measuring the ability of the concrete to conduct the corrosion current. It gives an indication of the rate of corrosion which may occur if corrosion of the reinforcement commences.	Low cost; objective data; quick; safe to use; indication of corrosion rate; large areas can be measured; low operative expertise is sufficient; very useful when used in conjunction with other methods of testing, e.g. half-cell potential. Requires complementary testing (e.g. use of covermeter) to obtain best results; ; coatings may need to be removed locally; concrete must have some moisture content; precise results are not usually obtained; interpretation by experienced personnel required.
Corrosion Rate Measurement/	Linear polarisation (SHRP-S-	Measure the current required to change by a fixed amount the potential difference	Determine the instantaneous corrosion rate of the reinforcement located below	Objective data; possible to estimate residual load capacity of the bridge; monitoring

Linear Polarisation	324 and S-330)	between the reinforcement and a standard reference electrode; the measured current and voltage allow determination of the polarization resistance, which is related to the rate of corrosion.	that test point.	corrosion over time. Very expensive and time consuming to perform on more than a limited number of representative locations; special equipment; need trained people and effective teamwork; specialised interpretation required; need complimentary testing for verification; measures localised corrosion at a single point in time that cannot be extrapolated over seasonal variations; difficult to obtain reliable data in solutions of high linear resistance; need to expose reinforcement. To provide an overall assessment of a structure, repetitive measurements at many locations are required over a representative annual cycle.
Concrete Breakout	Follow process in AS 1012.14	To remove (by saw cutting or coring) small, isolated areas of concrete covering the reinforcement to enable inspection and measurement of the exposed reinforcement.	Determine reinforcement details (e.g. bar size, type, orientation and taped cover) and its condition (e.g. corrosion state and loss of cross sectional area).	Direct results; low cost compared to other methods; accurate, quick; strong indicator of corrosion; provide connection for half-cell potential measurements and corrosion rate measurements; opportunity to collect the breakout sample for laboratory testing. Need access to surface; repair of breakouts required; may need several breakouts to draw reliable conclusions; coring may cause damage to the member from which the core is taken.
Concrete Strength				
Pullout	ASTM C900 ASTM E488	Measurement of the force required to pull an embedded metal insert and the attached concrete fragment from a concrete test specimen or structure.	It provides an estimation of the compressive and tensile strengths of hardened concrete; comparison of strength in different locations.	User expertise is low and can be used in the field; in-place strength of concrete can be measured quickly and appears to give good prediction of concrete strength. Small areas of concrete are removed, necessitating minor repairs; only tests a

		With the help of calibration charts the maximum force gives an indication of the strength of concrete. The insert can be either cast into fresh concrete or installed in hardened concrete.		limited depth of material.
Compressive Strength	AS 1012.9 AS 1012.14	Cores are extracted from hardened concrete by using a core drill (AS012.14), then trimmed, capped and tested for compressive strength (AS012.9).	Strength of in-place concrete; comparison of strength in different locations.	Fairly direct results and accurate; easy; rapid results for overall strength assessment; indirect assessment of concrete durability; can be utilised for structural analysis. Destructive testing; repair of core holes required; care should be taken as core holes may cause damage to the member from which the core is taken.
Tensile Strength (Indirect)	AS 1012.10 AS1012.14.	Cores are extracted from hardened concrete by using a core drill (AS1012.14), then prepared, laid in attest jig and tested (AS1012.10) by compression forces to determine the indirect tensile strength.	Estimation of tensile strength of in-place concrete; comparison of strength in different locations.	Fairly direct results and accurate; easy; rapid results for overall strength assessment; can be utilised for structural analysis. Destructive testing; repair of sample core holes required; care should be taken as core holes may cause damage to the member from which the core is taken.
Rebound Hammer	ASTM C805-08	The test is based on the principle of the elastic rebound of a spring-driven mass running down a central guide bar onto a plunger pressed firmly against the surface of the concrete.	Provides a measure of the local surface “hardness” of the concrete and under laboratory conditions the resulting rebound number has been empirically related to compressive strength of concrete.	Low cost; quick; easy; portable; direct results; gives accurate assessment of the strength of the surface layer of material; the entire structure can be tested in its 'as-built' condition; a good comparative test. It is an imprecise test and does not provide a reliable prediction of the strength of concrete; indicative only (+/- 25%), other tests needed to confirm results; can be affected by smoothness of the concrete surface, moisture content of the concrete, type, size and

				location of coarse aggregate, shape, and rigidity of the component and carbonation of the concrete surface; affected by direction of application.
Steel Structures Deterioration				
Visual Inspection	AS 3978 MRWA Level 2 Inspections Guideline	Observe, classify and document the appearance of distress and defects on exposed surfaces of the component, including welds.	Identifies defects on the surface only.	Simplest and least expensive method. Detects only surface defects; need well trained inspector; need full access to surface.
Weld Inspection - Dye Penetrant Testing	AS 2062 BS EN 571 Non-destructive testing - Penetrant Testing, 1997	A developer is applied to test surface to reveal locations where the fluorescent or visible dye has penetrated.	Identify the location and extent of weld discontinuities open to the surface eg: cracks, porosity, seams and surface defects such as fatigue cracks.	Portable, low cost; high accuracy; expedient results; very small surface cracks with a minimum depth of 3 times surface roughness can be detected, if the surface preparation is diligent; personnel are easy to train. Results are easy to interpret. Surface films such as coatings, scale, smeared metal may hide defects; surface has to be cleaned and protected after evaluation; surface roughness can give rise to spurious indications.
Weld Inspection - Magnetic Particle	AS 1171	A magnetic field is induced in a ferromagnetic material and then the surface is dusted with iron particles (either dry or suspended in liquid). Surface and near-surface imperfections distort the magnetic field and concentrate iron particles near imperfections, previewing a visual indication of the flaw	Surface and near surface (up to 2 mm below the surface) cracks in ferromagnetic materials.	Low cost; expedient; personnel are easy to train; exact results are obtained for locally limited area. No limit to the size or shape which can be tested; very small surface cracks on accessible surfaces up to a width of 0.2 µm and length of 0.5 – 2 mm with use of reference samples (EN ISO 9934-2, Non-destructive testing – Magnetic particle test); convenient for inspection of target oriented small areas. Can inspect through thin coatings. Can only detect surface or near surface defects; should not be applied during direct

				sun exposure; not usable for non-ferromagnetic material; photographic documentation is to be made without flash. surface has to be protected after testing; structures painted with aluminium paint can provide poor results.
Ultrasonic Testing	AS 1710	Very high frequency sound waves are passed through the metal structure under test. The waves are reflected back by either a defect or from the far surface of the member.	Ultrasonic flaw detection methods can detect voids and defects within a metal section and are best viewed as being complementary methods. Identifies most weld discontinuities including cracks, slag, lack of fusion; accurate metal thickness measurements possible.	Most sensitive to planar type defects; immediate results; portable; provides relatively rapid, and cost effective, defect detection; detect defects that are too small, or incorrectly oriented, for detection by radiography; requires access from one surface only; when correctly calibrated and employed, permits the detection of extremely small defects. Irregular, rough, non-homogeneous, very small or thin components are difficult to test; surface condition should be suitable for coupling of transducer; requires highly skilled operators to use the equipment and to interpret the results.
Radiographic testing (Gamma)	AS 3507.2 Non-destructive testing – Radiographic determination of quality of ferrous castings EN 1435, Non-destructive testing of welds –	Penetration of electromagnetic radiation through the body under the test to produce a shadow image of any defects within the bulk.	Detects voids and defects within a metal section and are best viewed as being complementary methods. Identifies most weld discontinuities including cracks, slag, lack of fusion; incomplete penetration, slag as well as corrosion and fit-up conditions	Can detect subsurface damage; good delectability of cracks in hidden members of typical built-up sections; can be evaluated on films or digital foils; removal of paint and corrosion protection is not necessary; permanent record. **Radiation is a safety hazard – requires control of nearby facility or area including lane/road closure and/or special monitoring of exposure levels and dosage to personnel.** Relatively slow and expensive techniques; requires skilled operator and interpretation.

	Radiographic testing of welded joints			
Eddy Current Testing	AS 4544 AS 2331.1.4	Inducing electromagnetic fields within a test piece and sensing the resulting electrical currents (eddy) so induced with a suitable probe or detector. A localised change in induced current flow indicates the presence of a discontinuity in the test object. The size of the discontinuity is indicated.	Detect discontinuities near the surface (i.e. cracks, inclusions, porosity). Capable of finding small discontinuities of <100 µm in highly conductive materials. Detect heat treatment variation, plating or coating thickness.	Low cost; requires minimum surface preparation. Reliable inspections can be performed through a nonconductive coating up to thickness of ~0.4 mm. Inspection can be performed very rapidly with instantaneous results. Inspection equipment is considered portable Requires a skilled operator to calibrate and interpret indications, limited to conductive materials, some indication may be masked by part geometry due to sensitivity variations.
Tensile Testing	AS 1391	Test piece of metal strained in uni-axial tension	Measure the mechanical properties of the steel	Accurate measurements of mechanical properties of steel including tensile strength, yield strength and ultimate strength. May cause considerable damage to the member from which the coupon is taken, extreme care must be exercised.
Hardness/ Rebound Testing	AS 1816.1 for Brinell test	An indenter (hardened metal ball) is pressed into the surface of a test piece by an accurately controlled test force.	Samples can be re-used for additional testing.	UWA uses Vickers test using a pointed indenter for use on smaller or harder samples. UWA also uses the Krautkramer Dynapocket rebound hardness test.
Microstructure Testing	ASTM E407 ASTM A247	Etch a metal sample to reveal or inspect its microstructure. Cross section an iron casting to reveal its graphite microstructure.	Evaluates the microstructure of the sample to identify defects and phases	Requires significant preparation of surface. Experienced metallurgist is required to interpret results. Only intended for the microstructure of graphite.

Table 6.2 – test methods

7.0 CONCRETE DEFECTS AND DETERIORATION MECHANISMS

7.1 General

This section should be read in conjunction with Detailed Visual Bridge Inspection Guidelines for Concrete and Steel Bridges (Level 2 Inspections) Document No: 6706-02-2233 for common defects that occur in concrete.

This section examines the causes of concrete deterioration and how various deterioration mechanisms affect the structural performance of reinforced concrete members. The initiation and propagation of reinforcement corrosion in concrete structures can be influenced by both internal and external factors. These sources of deterioration depend on concrete properties and exposure conditions and, to a large extent, govern structural performance and remediation practices.

7.2 Internal Factors

The constituents of concrete may be key contributors to its internal degradation.

Early age thermal restraint and shrinkage of concrete can cause cracking of the concrete, with a subsequent impact on the durability of the component and a potential for a reduction in service life. These mechanisms should be fully addressed during design.

The presence of undesired impurities in the concrete can also be a severe cause of deterioration, primarily due to chemical reaction of the constituents. Several common undesired impurities are discussed below.

7.2.1 Sulfate Content

The presence of excess sulfate from contaminated aggregate in freshly made concrete can cause severe degradation due to sulfate attack. The percentage by mass of acid-soluble SO_3 to cement must not exceed 5.0%. Heat accelerated cured concrete, or concrete that has reached temperatures above 65-70°C during early cure, can also suffer from a form of internal sulfate attack called “delayed ettringite formation” (DEF).

7.2.2 “Delayed Ettringite Formation” (DEF)

DEF is a potential degradation mechanism that may occur in steam cured, concrete components. A reaction between sulfates and calcium hydroxide (lime) to produce calcium sulfate (or gypsum) may occur in concrete with a high concentration of sulfate. This consumption of lime lowers pH, allowing sulfate to react with destabilised aluminate minerals in the current paste to form an expansive mineral, ettringite, which results in the breakdown of the cement paste. This mechanism is more likely to occur at elevated concrete temperatures.

It is generally accepted that to effectively prevent concerns relating to DEF, the temperature of the concrete during steam curing has to be monitored (rather than the steam temperature) and that for concrete temperatures of 70°C or less for cement type GP and 80°C or less for cement type LH, the formation of DEF is not likely with normally available cements.

7.2.3 Chloride Content

Similarly, aggregate contaminated with chlorides, or chlorides dissolved in chemical admixtures or mixing waters, can induce steel reinforcement corrosion. To avoid this, the mass of acid-soluble chloride ion per unit volume of concrete as placed shall not exceed 0.4 kg/m3.

7.2.4 Alkali Aggregate Reaction

Susceptible aggregates may be subject to chemical reactions that can lead to concrete expansion, cracking or loss of strength. One reaction of concern is alkali-aggregate reaction

Concrete can be damaged by an expansive, chemical reaction between active constituents of the aggregates and the alkalis (sodium and potassium as soluble hydroxides) in the cement; this process in known as alkali-aggregate reaction (AAR). There are three main forms of AAR; the most common alkali-silica reaction (ASR), alkali-silicate reaction and alkali-carbonate reaction. The product of these reactions is a sodium-rich silicate gel which absorbs water, with a consequent increase in volume. The expanding gel is either accommodated in the pores of concrete, or exerts internal pressures which eventually lead to expansion, cracking and disruption of the concrete. The visible signs of AAR damage are characterised by a network of cracks known as map cracking (refer Figure 4).

Gel may be present in cracks and voids both within aggregate particles and the cement paste. The best technique for the identification of ASR is the examination of concrete in thin section, using a petrographic microscope. Alternatively, polished sections of concrete can be examined by optical microscopy and/or scanning electron microscopy (SEM). This has the advantage that the gel can be analysed using X-ray spectrometry in order to confirm the identification beyond any doubt. A simple test using fluorescence of concrete samples treated with acidic uranyl acetate and exposed to UV-C light can often identify the presence of the gel product.

Figure 4 - Cracking due to AAR and related dampness

7.3 External Factors

Concrete deterioration from external sources can occur in a variety of ways. The most important environmental causes of deterioration are the attack of sulfate, carbonation, chlorides, and the effects of stress, temperature and moisture.

7.3.1 Environmental Loadings

7.3.1.1 Carbonation

Carbon dioxide in the atmosphere diffuses through the empty pore of concrete and reacts with the hydration products, which is known as a carbonation process. The reduction in alkalinity will provide an environment conducive to the corrosion of the reinforcing steel should this carbonated layer reach the steel, and oxygen and moisture are present.

In chloride-free concrete, corrosion will not take place unless the pH drops below 11. Atmospheric carbon dioxide can penetrate concrete and react with calcium hydroxide in the cement paste to form calcium carbonate and this reaction reduces the pH of the concrete to around 9. Carbonation of concrete in the atmosphere is represented by the following simplified equation.

$$Ca(OH)_2 + CO_2 \rightarrow CaCO3 + H_2O$$

This process is usually most pronounced in dry concrete. When the carbonation front reaches the carbon steel reinforcement, the passive oxide on the steel surface becomes unstable, and corrosion of the steel commences. Corrosion rates are aggravated by wet/dry cycling.

7.3.1.2 Chloride attack

The transportation of chloride ions into concrete is a complicated process which involves diffusion, capillary suction, permeation and convective flow through the system and micro-cracking network, accompanied by physical adsorption and chemical binding (Kropp and Hilsdorf 1995).

Chlorides penetrate the hardened concrete and break down the protective iron oxide layer on the steel reinforcement to initiate corrosion and the subsequent expansive disruption of the concrete matrix. The diffusion of chloride ions into concrete from external sources is dependent on concrete quality, cement type, cover, and exposure conditions. Periodic wet and dry exposure conditions accelerate corrosion rates severely.

7.3.1.3 Sulfate attack

The deterioration of concrete exposed to sulfate is the result of the penetration of aggressive agents into the concrete and their chemical reaction with the calcium hydroxide in the cement matrix. The main reactions involved are ettringite formation, gypsum formation and weakening of the calcium silicate hydrates in the binder. These chemical reactions can lead to expansion and cracking of concrete, and/or the loss of strength and elastic properties of concrete. In extreme cases, subsequent cracking and spalling of the concrete affords easy access to the reinforcing steel for the very aggressive sulfate ion which causes pitting corrosion of the steel and accelerates the degradation process.

7.3.1.4 Chemical attack

Dissolution and disintegration of the concrete matrix due to the effect of harmful substances such as acids, soft water, grease and oils, which subsequently assist in the corrosion mechanism by reducing the depth and quality of the concrete cover to the reinforcing or prestressing steel.

Portland cement is generally not very resistant to attack by acids, although very dilute or weak acids can be tolerated. The products of combustion of many fuels contain sulfurous gases which combine with moisture to form sulfuric acid. Other possible sources for acid formation are sewage, some peat soils, acid sulfate soils and some mountain water streams. Visual examination will show disintegration of the concrete leading to the loss of cement paste and aggregate from the matrix. If reinforcing steel is reached by the acid, rust staining, cracking, and spalling may be seen. If the nature of the solution in which the deteriorated concrete is located is unknown, laboratory analysis can be used to identify the specific acid involved.

7.3.2 Mechanical damage

Mechanical damage may be caused by abrasion (e.g. tyre wear on deck planks), erosion (e.g. flow of river water around piers), vapours or gases, or impact damage from over height or overwidth vehicles, derailed vehicles etc or from floating debris.

7.3.3 Fire damage

Fire damage can cause weakening of steelwork and can cause spalling of cover concrete. If the heat from a fire penetrates sufficiently deep into a reinforced concrete component, it can cause weakening of conventional reinforcement and even prestressing tendons.

7.3.4 Leaching

Loss of $Ca(OH)_2$ from hardened cement paste, increasing the porosity, reduces the alkalinity of the concrete and therefore may initiate the corrosion mechanism._Where $Ca(OH)_2$ is completely leached, the calcium silicate hydrates are destabilised by the resultant reduction in pH, and consequently the cement matrix becomes weak and friable.

7.3.5 Restrained movement

Cracking in concrete may occur due to internal stresses caused by restrained shrinkage, thermal contraction and expansion or other causes. This type of cracking may be identified by its location (e.g. along centreline of flat deck soffits, transverse cracks in kerbs) and the likelihood it is through the full thickness of the component.

7.3.6 Structural overloading

Structural: Cracking or failure of members due to change of use, one off loadings or severe impact damage (i.e. traffic accidents, natural forces, explosions etc).

Accidental: These loadings are generally short-duration, one-time events such as vehicular impact or very heavy loads. These loading can generate stresses higher than the strength of the concrete resulting in localised or general failure. This type of damage is typically indicated by spalling or cracking of the concrete**.**

Structural overloading may occur due to change of use, one off loadings or severe impact damage (e.g. traffic load increases over time, heavy loads, traffic accidents, natural forces, explosions etc). These loadings can generate stresses higher than the strength of the concrete resulting in localised or general failure. This type of damage is typically indicated by cracking or spalling of the concrete.

7.3.7 Foundation movement

Foundation movements may cause serious cracking in structures.

7.3.8 Flood, scour

Flood water and the debris in it can impart large lateral forces on a bridge. Scouring around and beneath the footings of pier walls and abutments can result in settlement and/or displacement of the component. This settlement/displacement can induce forces into the components that may exceed their capacity, leading to cracking and/or failure of components.

7.4 Design and Construction Factors

In addition to external factors, consideration should be given to the way the structure was initially designed and constructed which may contribute to its current condition.

7.4.1 Poor Design

Inadequate structural design or lack of attention to relatively minor design details can lead to in-built deficiencies in structures. Typical errors are discussed below.

7.4.1.1 Inadequate structural design

This may result in cracking and/or spalling in areas which are subject to the highest stresses. To confirm inadequate design as a cause of damage, the capacity of the locations of damage should be compared to the types of stresses that should be present in the concrete. A detailed structural analysis may be required.

Inadequate structural design can include use of an incorrect mix design to suit the exposure conditions.

7.4.1.2 Insufficient reinforcement cover

The concrete cover provides a physical barrier against the ingress of aggressive agents such as chlorides, carbon dioxide, oxygen and moisture. Under certain conditions, lack of cover will impair the ability of the concrete to provide protection from both physical and chemical deterioration, thus leading to corrosion of the steel reinforcement with subsequent cracking and spalling of the concrete.

7.4.1.3 Insufficient drainage

Ponded water can lead to accelerated corrosion, staining and debris collection.

7.4.1.4 Poor detailing

Poor detailing can result in the formation of defects and/or reduced durability. Examples of defects from poor detailing include corroding reinforcement from inadequate cover at drip grooves, honeycombed concrete at congested reinforcement, staining and corroding reinforcement from inadequate scupper projection below soffit, cracking from widely spaced reinforcement.

7.4.2 Poor Construction Practice

Failure to follow specified procedures and good practice, or outright carelessness during construction may lead to a number of adverse conditions. Poor workmanship that creates porous or permeable concrete, placement of concrete in high temperatures, plastic and restrained shrinkage and settlement of concrete may all lead to defects in the concrete and a reduction in durability.

Typically, most of poor practices do not lead directly to failure or deterioration of concrete. Instead, they enhance the adverse impacts of other mechanisms. The following sections describe some of the most common poor practices.

7.4.2.1 Improper Curing

Symptoms of improperly cured concrete can include various types of cracking and surface disintegration. In extreme cases where poor curing leads to failure to achieve anticipated concrete strengths, structural cracking may occur.

7.4.2.2 Improper Concrete Consolidation

Unsatisfactory compaction of concrete may result in a variety of defects, the most common being “bug-holes”, honeycombing and cold joints. These defects can make it much easier for deterioration mechanisms to enter the concrete and initiate deterioration.

7.4.2.3 Improper Casting Techniques

Cold joints and construction joints may separate/crack due to differential thermal movement or shrinkage between the two parts.

Inaccurately placed reinforcement and/or inaccurately constructed formwork may result in cover to reinforcement being less than required, usually resulting in reduced durability of the component.

7.4.2.4 Improper Construction Sequence

If consideration is not given to the construction sequence, construction loads and movements associated with construction, excessive deflection and cracking may result.

7.5 Formation and Types of Cracks

The cause of concrete cracking significantly influences the type, width, orientation and extent of cracking in a structural member. In practice, possible remedial measures may be considered only when the causes of the cracks have been properly identified. Why cracks occur, and their relevance to steel reinforcement corrosion and service life of structures, is briefly examined in this section.

Cracks of a greater width allow a more rapid penetration into the concrete of aggressive agents such as chloride, sulfates and carbon dioxide, thus creating a more rapid rate of deterioration than could have been anticipated from un-cracked concrete.

Figure 6 and Figure 7 below, re-created from HB 84-2006 Guide to Concrete Repair & Protection, present a generalised diagram of types of cracks and a classification of cracks in structures respectively. The more common types of cracks are discussed subsequently in some detail.

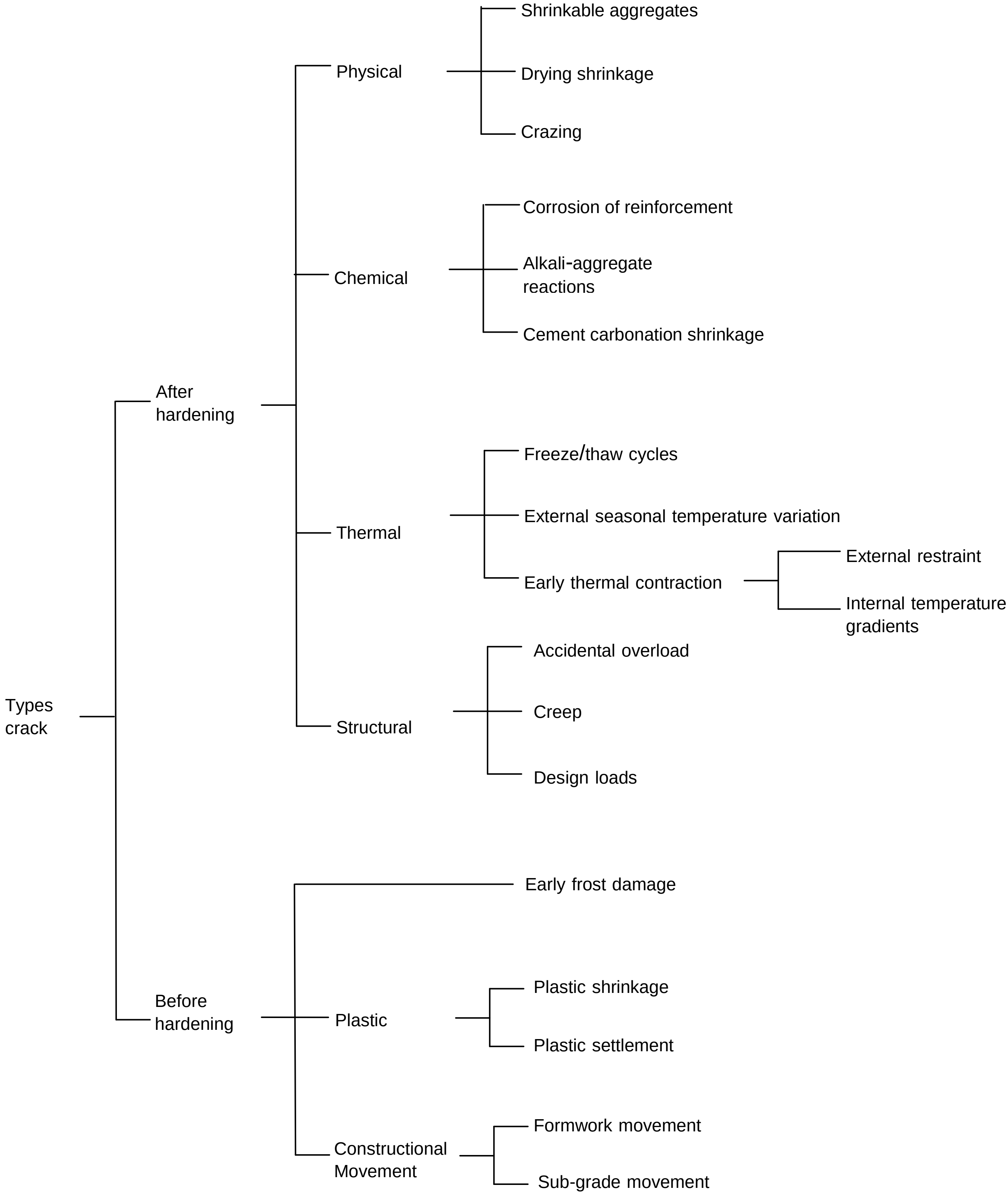
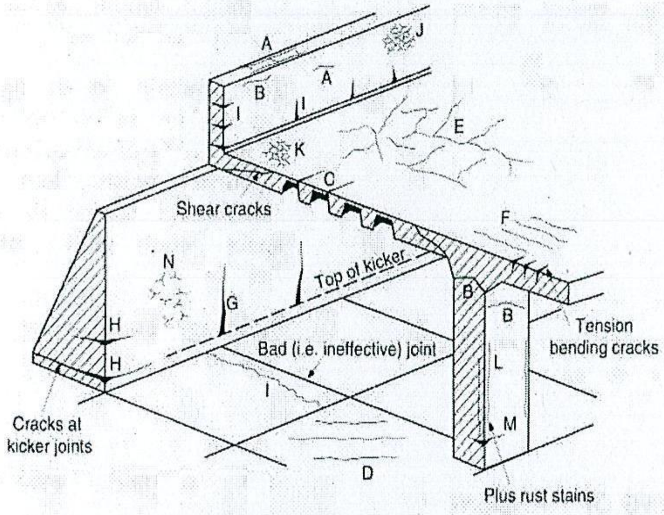


Figure 5 - Classification of cracks (Source HB 84-2006)



Type of cracking	Position on diagram	Subdivision	Most common location	Primary cause (excluding restraint)	Secondary causes/factors	Remedy (assuming basic redesign is impossible); in all cases reduce restraint	Time of appearance
Plastic settlement	A	Over reinforcement	Deep sections	Excees bleeding	Rapid early drying conditions	Reduce bleeding (air entrainment) or revibrate	10 minutes to 3 hours
	B	Arching	Top of columns				
	C	Change of depth	Trough and waffle slabs				
Plastic shrinkage	D	Diagonal	Roads and slabs	Rapid early drying	Low rate of bleeding	Improve early curing	30 minutes to 6 hours
	E	Random	Reinforced concrete slabs				
	F	Over reinforcement	Reinforced concrete slabs	Rapid early drying, steel near surface			
Early thermal contraction	G	External restraint	Thick walls	Excess heat generation	Rapid cooling	Reduce heat and/or insulate	1 day to 2–3 weeks
	H	Internal restraint	Thick slabs	Excess temperature gradients			
Long-term drying shrinkage	I		Thin slabs (and walls)	Inefficient joints	Excess shrinkage, inefficient curing	Reduce water content, improve curing	Several weeks or months
Crazing	J	Against formwork	'Fair-faced' concrete	Impermeable formwork	Rich mixes	Improve curing and finishing	1–7 days, sometimes much later
	K	Floated concrete	Slabs	Over-trowelling	Poor curing		
Corrosion of reinforcement	L	Nature	Columns and beams	Lack of cover	Poor quality concrete	Eliminate causes listed	More than 2 years
	M	Calcium chloride	Precast concrete	Excess calcium chloride			
Alkali–aggregate reaction	N		(Damp locations)	Reactive aggregate plus high-alkali cement		Eliminate causes listed	More than 5 years

Figure 6 - Schematic presentation of the various types of cracking and classification of cracks (Source: CONCRETE SOCIETY, Non-structural cracks in concrete, Technical Report, No.22, p. 38)

7.5.1 Crack Width and Durability

There is divergence of opinion in the effects of cracks on reinforcement corrosion. The subject is a complex area and the number of different variables including crack size, alignment and orientation with the reinforcing steel, cement/binder type, aggregate type and size, reinforcement spacing and type, moisture source and wetting time, etc makes it very difficult to define even basic rules regarding cracks in concrete structures that can be applied with confidence.

Not all cracks in concrete will necessarily allow the penetration of water/moisture. A number of cracks that appear wide at the surface may not penetrate the full depth of the section. Others may follow an irregular path that limits the flow of moisture, others may be blocked by leachates or contaminants.

Where cracks form at an early age some level of sealing may occur due to a process known as autogenous healing. The extent of such self healing is difficult to predict being dependant on a number of factors including the width of the crack, binder type, rate of flow of moisture through the crack, composition of the moisture etc. However, autogenous healing can be confirmed by core sampling and examination of the retrieved concrete core.

International research has shown that cracks of width less than 0.3mm generally do not pose a durability concern. Excluding water retaining structures and possibly some marine components, repair works to cracks are generally not carried out to cracks with surface widths less than 0.3mm.

For crack widths greater than 0.3mm width where a durability concern is identified, cracks should be repaired as good management practice to meet durability requirements.

7.6 Mechanism of Corrosion Initiation of Steel Reinforcement in Concrete

Corrosion of steel reinforcement is one of the predominant causes of deterioration or distress of reinforced concrete and accounts for most of the problems faced concerning reinforced concrete. Steel reinforcement inside concrete is normally protected against corrosion by the existence of a passive oxide film surrounded by $Ca(OH)_2$ due to the highly alkaline environment (p^H greater than 12.5) generated from hydration of cement.

The key environmental factors that reduce the passivation of steel are carbonation and reactions with other acidic gases, and chloride. Other factors which may influence either the initiation or rate of reinforcement corrosion include cracks in concrete, temperature, moisture, oxygen and inadequate concrete quality and cover.

The condition of passivation is altered when aggressive agents either lower the pH or disrupt the protective oxide film. Steel corrosion then proceeds with the formation of electrochemical cells comprising separate anodic and cathodic regions at the steel surface.

At the anode $Fe \rightarrow Fe^{2+} + 2e^-$ (1)

At the cathode $0.5O_2 + H_2O + 2e^- \rightarrow 2OH^-$ (2)

These two reactions must be balanced, i.e. the rate of generation of electrons at anodes must equal the rate of consumption at the cathode. Therefore, the rate of the overall loss of metal at the anode is dependent on the rate at which both half cell reactions can occur. This is important in assessment of corrosion risk in concrete components. It should be noted from the above that oxygen is required at cathodic sites in order for the reactions to proceed.

The products of the two reactions combine, after some intermediate stages, to form rust. In atmospheric conditions where oxygen is plentiful, the normal reaction will be generation of a hydrated iron (iii) oxide:

$$2Fe(OH)_3 \rightarrow Fe_2O_3.\ (xH_2O) \qquad (3)$$

These hydrated oxides occupy a greater volume than the iron dissolved in their production by a factor between 2 and 4, hence creating expansive forces in the surrounding concrete.

When these forces exceed the tensile capacity of the cover concrete, cracking and/or spalling occurs.

The corrosion of reinforcement may be considered negligible in conditions where concrete is either dry or in a water-saturated environment, as this restricts oxygen supply. Typically the highest corrosion rate will occur in concrete subjected to periodic wet and dry cycles, as in the splash zones of marine structures. In some instances areas below tidal level can experience high corrosion rates even though such areas are well saturated with water.

8.0 COMMON TYPES OF DEFECTS IN STEEL MEMBERS AND DETERIORATION MECHANISMS

This section should be read in conjunction with Detailed Visual Bridge Inspection Guidelines for Concrete and Steel Bridges (Level 2 Inspections) Document No: 6706-02-2233 for common defects that occur in steel.

Main forms of corrosion of metals and the underlying mechanisms by which they occur:.

Crevice Corrosion

Intense localised corrosion frequently occurs within crevices and other shielded areas on metal surfaces exposed to corrosive environments. This type of attack is usually associated with small volumes of stagnant solutions caused by the holes, gasket surfaces, lap joints, surface deposits, and crevices under bolts and rivet heads. As a result this type of corrosion is known as ‘crevice’ or ‘gasket’ corrosion. To function as a corrosion site, a crevice must be wide enough to permit liquid entry, but sufficiently narrow to maintain a stagnant zone. For this reason, crevice corrosion usually occurs at openings a few tenths of millimetre in width and rarely occurs in grooves or slots greater than 3 mm wide.

Crevice corrosion results from the difference in oxygen concentration between the interior of the crevice and its surroundings causing a potential difference to exist that is sufficient to cause preferential corrosion within the crevice.

Pitting

Pitting is a form of extremely localised attack that results in holes in the metal. Pits can occur in isolation or in groups (when they appear as deep surface roughness) and are characterised by having a surface diameter equal to, or less than their depth. Pitting is one of the most destructive and insidious forms of corrosion because of its localised and intense nature. It causes complete perforation of a structure with only a small weight loss and failure often occurs without warning. This is made worse by the fact that pits can be very small and difficult to detect by visual inspection since they are often covered with corrosion product.

Pitting is unique type of anodic reaction and the corrosion processes within a pit produce conditions which both catalyse and promote the continued activity of the pit. Rapid dissolution of the metal occurs within the pit, while oxygen reduction takes place on adjacent surfaces. The high concentration of metal ions in the pit results in the migration of chloride ions into the pit solution to maintain electro-neutrality. Oxidising metal ions with chlorides are the most aggressive pitting mechanism or anodic reaction.

The method used for combating crevice corrosion can also be used to reduce the chances of pitting although proper material selection is more important since materials that are known to be prone to pitting should be avoided in environment likely to contain chloride. Stainless steel alloys are more susceptible to pitting than any other group of alloys or metals (indeed ordinary steel is more pitting resistant).

Stress Corrosion Cracking

Stress corrosion cracking (SCC) refers to cracking caused by the simultaneous presence of tensile stress and corrosive environment and susceptible materials. Removal of, or changes in, any one of these three factors will often eliminate or reduce susceptibility to SCC and therefore are obvious ways of controlling SCC in practice. During stress corrosion cracking metal is not attacked over most of its surface, but the cracks progress through the metal perpendicular to the tensile stress. This cracking can have serious consequences for steel structures since it can occur at tensile stresses well below the yield strength of the metal. The failures often take the form of fine cracks that penetrate deeply into the metal with little or no evidence of corrosion on the nearby surface. Therefore, during casual inspection no macroscopic evidence of impending failure is visible.

The criteria for the stress corrosion is simply that it is tensile and of sufficient magnitude. Stresses may arise from a number of sources such as applied, residual or thermal, or may result from manufacturing or actual service application. The principle sources of high local stresses in manufacturer include thermal processing, surface finishing, fabrication and assembly. Sources of stress in-service are frequently introduced by damage from accidental mechanical impact or local electrical arcing, any form of localised corrosion, wear and environmental factors such as high and low temperature. Indeed, welded steels can contain sufficient residual stresses to initiate stress corrosion without any external stress being applied.

Several theories have been advanced to explain in detail the mechanisms of SCC. Two major theories are the electrochemical and stress-sorption theories. It is known that corrosion plays an important role in initiating the cracks, in fact stress corrosion cracks are often start at the base of corrosion pits. Once a crack has started there is considerable stress concentration at this point and the crack can be expected to grow in the presence of sufficient stress.

The site of initiation of a SCC may be sub-microscopic and determined by local differences in metal composition, thickness of protective film, concentration of the corrodent, and stress concentration. A pre-exiting mechanical crack or other surface discontinuities, or a pit or trench produced by chemical attack on the metal surface, may act as a stress raiser and thus serve as a site for initiation of SCC.

SCC can be produced in most metals under some conditions - for example, sensitised austenitic stainless steel cracks at room temperature in water containing about 10 ppm of chloride or 2 ppm of fluoride.

SCC is surface connected and may be detected visually in some cases. Dye penetration test, Ultrasonic Test or phase analysis Eddy Current techniques are the preferred methods for monitoring and inspection of SCC.

Corrosion Fatigue

Corrosion fatigue is special form of stress corrosion. Fatigue itself is defined as the tendency of a metal to fracture under repeated cyclic stressing and occurs at stress levels below the yield point. The process of fatigue can be divided into three parts:

i) Initiation of a crack,

ii) Subsequent growth of the crack through the section of the metal at right-angles to the stress and

iii) Sudden brittle failure of the component when the cross-sectional area of the remaining metal is reduced to the point where its ultimate strength is exceeded.

Corrosion fatigue is defined as the reduction in fatigue resistance due to the presence of a corrosive medium and is most pronounced at low stress frequencies since this allows greater contact time between the metal and corrodent.

The fatigue life of steel and other ferrous metals usually becomes independent of stress below a certain stress level called the fatigue limit. If the metal is stressed below the fatigue limit it will endure an infinite number of cycles without failure.

The reduction in fatigue resistance in the presence of a corrosive environment is thought to result from the formation of corrosion pits which acts as stress raisers and initiate cracks. Corrosion fatigue failures are usually transgranular (i.e. the cut through the metal grains rather than follow the grain boundaries) and do not show the branching which is characteristic of many stress corrosion cracks.

9.0 DETAILED INVESTIGATION TECHNIQUES

This section presents each investigation method, the evaluation criteria for interpretation of results and the equipment required for each investigation method of a Level 3 Inspection.

9.1 Visual Inspection

This inspection is similar to a Level 1 Inspection but limited to the extent of the Level 3 Inspection.

Visible defects should be recorded in standard proformas in accordance with the survey legend provided in Appendix C. The proformas are usually devised from bridge drawings of each component or are common proformas that may suit many components. Photographs of the visible defects shall be taken to document the in situ condition, with photo locations clearly noted.

The visual survey should also be used to confirm the diagnostic testing locations so the investigations can confirm the cause of deterioration and properly represent the condition of the bridge.

Equipment: observant eye, camera, measuring tape, crack width gauge, binoculars, survey proformas, pens, marker/crayon, required access equipment (e.g. scaffold, ladder) etc.

9.2 Delamination Survey

A delamination survey involves striking a concrete surface with a small hammer (mass of 1 - 2 kg) and listening to the noise produced. Sections of concrete that have delaminated from the bulk of the component will sound hollow.

Delaminated areas identified shall be marked on the structure and their location and extent recorded on survey proformas. Photographs should be taken of the majority of the delaminations.

Equipment: Light hammer, camera, measuring tape, pens, marker/crayon, specific access equipment (e.g. scaffold, ladder).

9.3 Concrete Breakout

A concrete breakout is a small, isolated area of concrete removed to expose reinforcement to enable inspection and measurement of the exposed reinforcement.

Concrete breakouts shall be completed on representative locations, their location dependent on the visible condition of the concrete and environments at the bridge. Where visible deterioration observed, it is recommended to carry out breakout on the deteriorated surface as well as on sound concrete nearby.

A breakout shall generally be carried out by saw-cutting an approximately 100mm x 100mm panel or by using a hand-held percussion core bit or a diamond core bit to remove an approximately 80mm diameter core). Repairing of breakouts shall be completed using a MRWA approved polymer modified repair mortar in accordance with the manufacturer's recommendations.

Equipment: Covermeter (to locate the reinforcement), tools to carry out breakout (saw, hammer drill with percussion core bit and chisel, or coring machine with diamond core bit and water), water spray bottle to clear concrete slurry and powder, measuring tape, camera, stable access equipment (e.g. scaffold - a ladder is not adequately stable), proforma, note taking equipment, repair mortar.

Figure 7 - Carrying out a breakout with a percussion core bit, with covermeter results lightly marked in red crayon

	**NOTES:** 9.3.1 **Location:** Bridge ####, Pier #, Column # 9.3.2 **Direction:** Horizontal **Type:** Plain round **Taped Cover:** 40mm 9.3.3 **Bar Diameter:** 12mm **Notes:** 50% of rebar surface is covered with brown and black corrosion products emanating from severe localised attack, and the rust scale at this point was up to ~2mm thick. The corroded area had suffered significant section loss. **Estimated Condition Rate:** 4L

Figure 8 - BREAKOUT PHOTO WITH EXAMPLE NOTES

9.4 Covermeter Survey

The covermeter survey shall be performed in accordance with MRWA Test Method WA 623.1. The covermeter survey shall be conducted over the concrete surface and the position of the outermost reinforcement detected should be marked in crayon on the surface. The typical depth of the reinforcement in the other direction should also be noted. The position and value of the concrete cover measurements should be temporarily marked on the surface of the component (refer Figure 9) and recorded. The results should be accurately drawn with locations of all other tests marked in reference to the cover survey. The distances between bars in both directions should be measured.

Should a breakout exist in that component (or similar/nearby component), and a taped cover is obtained, the covermeter results should be calibrated with the taped cover.

The extent of the covermeter survey at each area investigated should be:

1. Abutments – 2 or 3 elevations (e.g. near ground, approximately 1.5 m above ground, near top of wall if accessed by scaffold); minimum 20 readings per elevation; measurements at delaminations
2. Wing Walls – 2 or 3 elevations (e.g. near ground, approximately 1.5 m above ground, near top of wall if accessed by scaffold); minimum 20 readings per elevation; measurements at delaminations
3. Pier Walls – 2 or 3 elevations (e.g. near ground, approximately 1.5 m above ground, near top of wall if accessed by scaffold); minimum 20 readings per elevation; measurements at delaminations
4. Columns/Piles – All sides x reachable height from ground (approximately 2.5 m high); near top of component if accessed by scaffold
5. Capbeams – 25% of the length of the capbeam side accessed
6. Pilecaps – minimum 20 readings per side
7. Beams – 25 to 50% of the beam length; both sides and soffit
8. Deck Sides – minimum 20 readings per side accessed
9. Deck Soffits – minimum 20 readings at area accessed

Equipment: Electromagnetic covermeter, marker/crayon, measuring tape, camera, access equipment (e.g. scaffold, ladder-if safe to use).

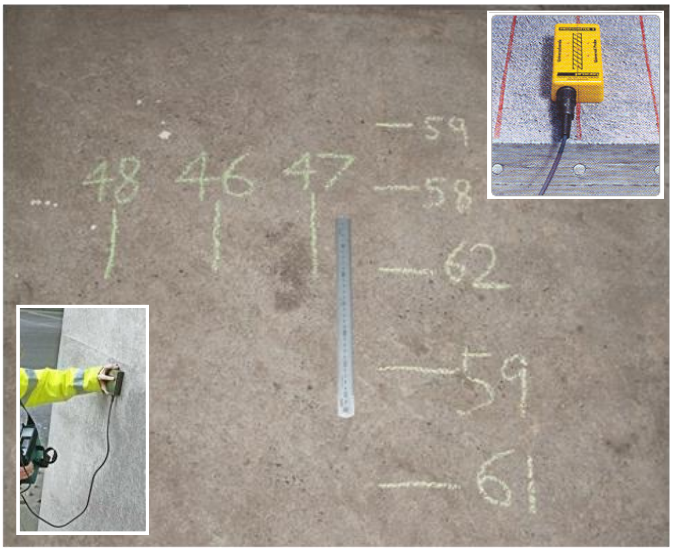


Figure 9 - TYPICAL COVER SURVEY

9.5 Rebound Hammer Survey

Rebound hammers should be used in accordance with the manufacturer's written instructions. In practice, the results from this test are very dependent upon the surface condition and moisture content on the concrete as well as the ratio of aggregate cement paste. It is recommended, should a suitably sized core be taken for other tests, to test the side of the core to obtain an estimate of concrete hardness through the depth of the concrete. Prior to testing the concrete surface, a grinding stone should be used to create a smooth test surface, and then a rag used to remove all dust from the test area. The equally spaced test sites should be marked out on the surface of the test area and then the tests should be performed at those test sites to ensure the tip is directly on the cleaned surface and in a different position each test. Typically, 9, 16 or 25 test sites per component are used. Two or three test areas should be tested on larger components such as abutments and pier walls.

Manufactures documentation should include calibration charts that allow conversion of the in situ measurements to an indicative concrete compressive strength. Table 9.1 provides indicative guidelines of concrete quality based on rebound hammer test results.

Average Rebound (Q Value)	Quality of Concrete
>40	Very Good
30 – 40	Good
20 – 30	Fair
<20	Poor and/or Delaminated

Table 9.1 – APPARENT Quality of concrete from rebound values

Equipment: Rebound Hammer, grinding stone, cleaning rag, marker/crayon, proforma, note taking equipment.

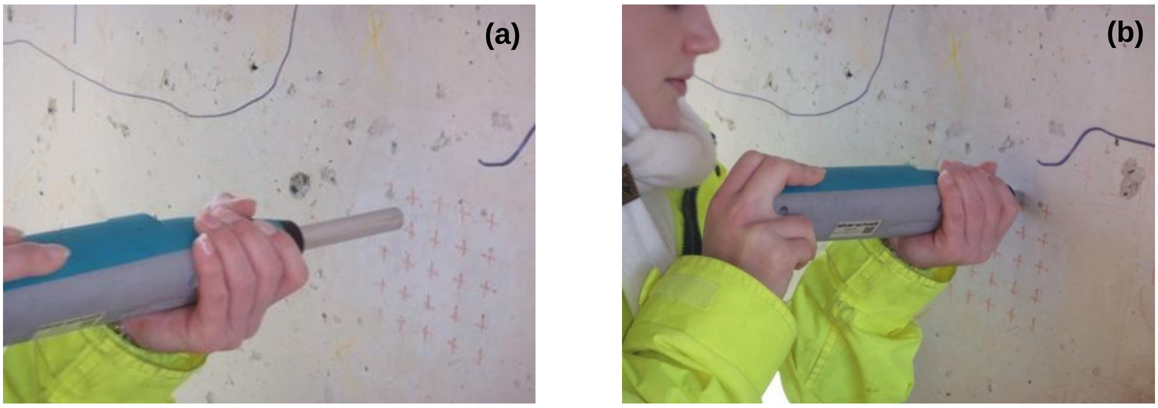


Figure 10 – REBOUND HAMMER TEST TAKING 25 TEST POINTS BEFORE COMPRESSION (a) AND AT FULL COMPRESSION (b)

9.6 Depth of Carbonation

Depth of carbonation testing shall be performed in general accordance with MRWA Test Method WA 620.1.

It is recommended that carbonation testing is carried out in itu where the concrete has been drilled and cored for other testing (e.g. chloride testing, breakout). Cores taken for testing other than carbonation should tested on their cut face. Cores taken for carbonation testing should be broken in half (on or off site), and a carbonation test carried out on the fresh concrete. Cores provide a clearer depth of carbonation compared with most in situ tests due to better lighting and access. Drill/cored holes should be thoroughly washed with potable water and allowed to become reasonably dry prior to spraying the phenolphthalein.

Where possible from access provided to elevated areas, the depth of carbonation near the top of components investigated should also be determined.

An example of a carbonation test is shown in Figure 11.

Equipment: Freshly exposed concrete (from coring, drilling, breakouts or a core freshly broken), phenolphthalein (acid/base indicator), ruler/measuring tape, proforma, note taking equipment, camera.

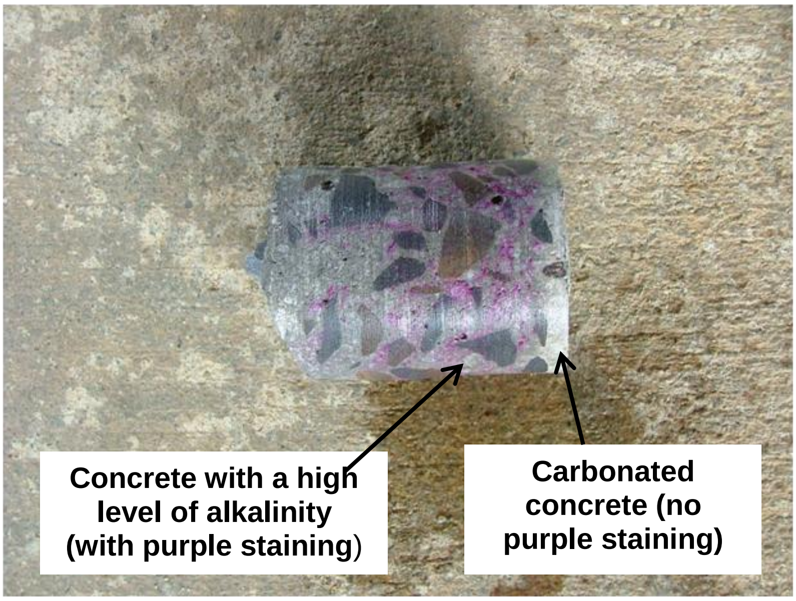


Figure 11 - TYPICAL CARBONATION TEST

9.7 Half-Cell Potential Survey

The half-cell potential survey should be undertaken generally in accordance with ASTM C876 and in conjunction with resistivity measurements and, where possible, corrosion rate measurements to obtain the overall electrochemical condition of the reinforced concrete and also to enable a clearer interpretation of test results.

A representative area of each component should be selected for testing. As a minimum, this area should be:

1. Abutments – full width x reachable height from ground (approximately 2.5 m high)
2. Wing Walls – full length x reachable height from ground (approximately 2.5 m high)
3. Pier Walls – full width plus Left Hand Side or Right Hand Side x reachable height from ground (approximately 2.5 m high)
4. Columns/Piles – All sides x reachable height from ground (approximately 2.5 m high)
5. Capbeams – 25% of the sides of a capbeam
6. Pilecaps – full width of all accessible sides

7. Beams – 25 to 50% of the beam length; both sides and soffit
8. Deck Sides – 10 to 15% of the total area of both sides
9. Deck Soffits – 5 to 10% of the total area of all soffits

The surface of each representative area should be marked with a grid in crayon to identify the measurement locations. Where possible, the spacing shall typically be 500 mm on large components (e.g. abutment walls, wing walls, pier walls, capbeams, deck soffits), with the grid spacing reduced for smaller components such as deck sides, piles and beams. The size of the grid should be revised downwards where there are large differences between adjacent reading values. The surface of the concrete shall be wetted prior to taking readings such that the readings made are fairly stable. If necessary, the surface may need to be pre-soaked (for at least for 20 minutes if concrete is dry) or to suit the half-cell manufacturer's recommendations. Readings should be recorded on the proformas or within the equipment itself if it has this capability. Figure 13 shows a typical half-cell potential survey arrangement.

The potential results should be used to develop a potential map for the area tested.

An interpretation guideline for half-cell potential results is presented in Table 9.2.

Potentials (mV) (Relative to $Cu/CuSO_4$)	**Risk of Active Reinforcement Corrosion at Time of Measurement**
> -200	5%
-200 to -350	50%
< -350	95%

Table 9.2 - Half-Cell Potential Corrosion Criteria

Equipment: All half-cell potential equipment (includes an electrode, appropriate cables, voltmeter/recorder and a connection with reinforcement), concrete breakout equipment and covermeter if breakout does not already exist, detailed testing proformas, water spray bottle (to saturate surface before testing), measuring tape and marker/crayon (to mark out test grid).

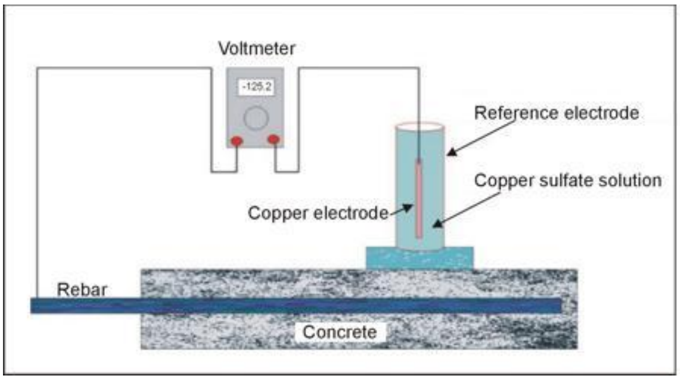


Figure 12 - Typical Connection of Half Cell Measurements

Figure 13 - Typical Half-cell potential test showing electrode (centre), digital display box (bottom right) and connection with the reinforcement (top left)

9.8 Concrete Resistivity

Concrete resistivity testing shall be performed in general accordance with MRWA Test Method WA 622.1.

As resistivity is highly influenced by the moisture content of the concrete, pre-soaking the test location for at least for 20 minutes (if the concrete surface is dry) or as per manufacturer's recommendations may be necessary.

Placement of the resistivity meter pins should avoid being directly over and as away as possible from the reinforcement to minimise the error of the reading which occurs due to interference of the reinforcement (refer Figure 14). A measurement diagonal to vertical and horizontal reinforcement is often necessary. A covermeter survey should be carried out prior to testing to identify the position of the reinforcement.

Concrete resistivity testing at each component investigated should be carried out at elevations likely to have of differing moisture content e.g. near ground level and approximately 1.5 m above ground level on one of a streambed pile.

The interpretation guideline for resistivity measurements is presented in Table 9.3.

Resistivity (ohm.cm)	Likelihood of Corrosion
< 5,000	Very high
5,000 – 10,000	High
10,000 – 20,000	Low
>20,000	Negligible

Table 9.3 - Guidelines on interpretation of Resistivity Measurements

Equipment: Specific resistivity equipment (this depends on apparatus used, if it is not a ResiPod or similar tool then a 4 pin Wenner probe, voltmeter and appropriate cords are required), water spray bottle to saturate the surface.

Figure 14 - TYPICAL CONCRETE RESISTIVITY TEST

9.9 Cement Content and Type (Aggregate / Cement Ratio)

Concrete core samples for cement content testing can be extracted at representative locations, with a covermeter survey performed to avoid reinforcement when coring. The cement content can be used to indicate concrete characteristic strength and, therefore, concrete quality.

On site Equipment: Diamond core bit and drill, fixed stand for drill (fixed to concrete surface or stable ground), fresh water supply, covermeter, proforma, note taking equipment. It is assumed that the testing laboratory has all equipment required to perform the test on the core.

9.10 Concrete Compressive Strength

Concrete core samples for compressive strength shall be extracted at representative locations, following a covermeter survey to avoid the reinforcement. Laboratory testing in accordance with AS1012.9 and AS1012.14 to determine the compressive strength of the core samples shall be completed in a NATA registered laboratory.

The compressive strength can be used to obtain the strength of in-place concrete; a comparison of strength in different locations and for structural assessment. The compressive strength can also be used as an indirect measurement of concrete durability i.e. resistance to the penetration of chlorides, sulfate and carbon dioxide.

On site Equipment: Diamond core bit and drill, fixed stand for drill (fixed to concrete surface or stable ground), fresh water supply, covermeter, proforma, note taking equipment. It is assumed that the testing laboratory has all equipment required to perform the test on the core.

9.11 Apparent Volume of Permeable Voids (AVPV)

Concrete core samples for Apparent Volume of Permeable Voids (AVPV) testing shall be extracted at representative locations, following a covermeter survey to avoid the reinforcement when coring. Laboratory testing to determine the VPV of the core samples is to be completed in NATA accredited laboratory in accordance with ASTM C642-06 or AS1012.21.

Durability classifications based on AVPV values as developed by Vic Roads are given in Table 9.4.

Durability Classification Indicator	**Vibrated cylinders (AVPV%)**	**Rodded cylinders (AVPV%)**	**Cores (AVPV%)**
Excellent	<11	<12	<14
Good	11-13	12-14	14-16
Fair	13-14	14-15	16-17
Marginal	14-16	15-17	17-19
Bad	>16	>17	>19

Table 9.4 - VicRoads classification for concrete durability based on the AVPV limits

On site Equipment: Diamond core bit and drill, fixed stand for drill (fixed to concrete surface or stable ground), fresh water supply, covermeter, note taking equipment. It is assumed that the NATA accredited laboratory has all equipment required to perform the test on the core.

9.12 Chloride Ion Penetration

Concrete core (sample to have minimum diameter of 50 mm) or powder samples for chloride content testing shall be extracted at representative locations. Laboratory testing to determine the chloride content of the samples is to be completed in a NATA accredited laboratory in accordance with AS1012.20.

To take drilled powder samples, the drill bit used should be larger than the likely aggregate size (i.e. should be >20 mm diameter) and the minimum total area of the surface drilled should be equal to the end area of a 50 mm core i.e. 1,960 mm^2. This equates to 4 x 25 mm diameter sample holes. All drillings shall be in the desired increments to suit bridge age, exposure conditions and reinforcement cover. Commonly, these increments are in 10mm increments up to 50-70mm. A minimum of 20g of powder sample should be collected for each increment. The chloride content test results shall be analysed to predict the approximate time when the chloride “threshold” is attained at the depth of reinforcement. For core or powder samples the reinforcement should be located to avoid when sampling.

Concrete core sample results are more accurate in obtaining the chloride profile, however powder samples can be collected inexpensively (especially in overhead applications e.g.

soffits), with less damage to the structure, and the results obtained from properly collected samples are acceptably reliable.

On site Equipment for Core Samples: Diamond core bit and drill, fixed stand for drill (fixed to concrete surface or stable ground), fresh water supply, covermeter, note taking equipment. It is assumed that the NATA accredited laboratory has all equipment required to perform the test on the core.

On site Equipment for Powder Samples: Hammer drill with a 15-25mm drill bit, collection tube (usually a PVC pipe with an angled top and a hole for the drill), collection bags (any bag with an air tight seal), marker to mark depth increments and test location on the bags, ruler to measure depth increments in drill holes, covermeter, note taking equipment. A small bottle brush or toothbrush may be used to clear concrete powder from the hole before collecting the next depth increment. It is assumed that the NATA accredited laboratory has all equipment required to perform the test on the core.

9.13 Sulfate Content

Concrete core samples for sulfate content testing shall be extracted at representative locations. Laboratory testing to determine the sulfate content of the core samples shall be completed in NATA accredited laboratory in accordance with AS1012.20-1992. The reinforcement should be located to avoid it when drilling.

Maximum sulfate content in concrete should not exceed 50 g/kg by weight of cement in accordance with AS1379.

On site Equipment for Core Samples: Diamond core bit and drill, fixed stand for drill (fixed to concrete surface or stable ground), fresh water supply, covermeter, note taking equipment. It is assumed that the NATA accredited laboratory has all equipment required to perform the test on the core.

9.14 Corrosion Rate Measurement

The easiest method to perform this test is using Linear Polarisation Resistance (LPR) measurements to determine the instantaneous corrosion rate of the reinforcement located below the test point. Corrosion rates vary during the life of the structure due to age, exposure environment, and time of year, depending on the variations in concrete moisture content, wet/dry cycles, carbonation, chloride concentration profile and temperature. These measurements should generally be carried out at the minimum measurement obtained through the half-cell potential tests, as this is expected to have the highest corrosion activity and can help differentiate low potential values due to corrosion activity from low potential values due to oxygen starvation conditions.. Measurements at different time intervals should be carried out. The following broad criteria have been given in Table 9.5.

Corrosion Current Density (i_{corr})	Corrosion Rate
<0.2 ($\mu A/cm^2$)	Passive condition
0.2 to 0.5 ($\mu A/cm^2$)	Low corrosion rate
0.5 to 1.0 ($\mu A/cm^2$)	Moderate corrosion rate
>1 ($\mu A/cm^2$)	High corrosion rate

Table 9.5 - Broad criteria for corrosion

Equipment: LPR specific equipment (includes a lap top, appropriate software, potentiostat or galvanostat, specific measuring probe, connection with the reinforcement), concrete breakout equipment and covermeter if breakout does not already exist, and water spray bottle (to saturate surface before testing).

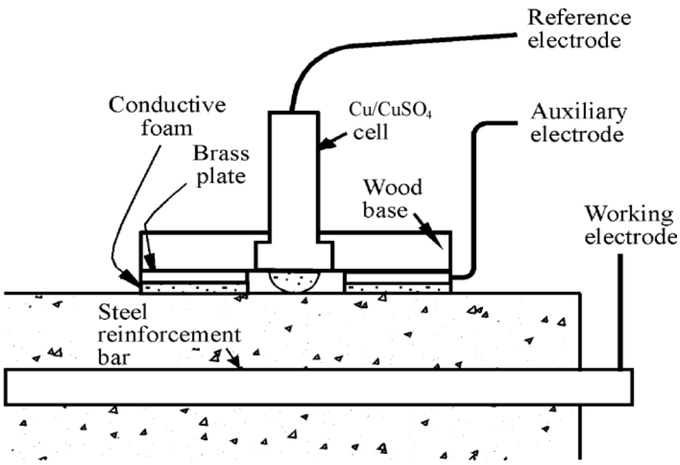


Figure 15 - Typical Three Electrode LPR Measurement

Figure 16 - Example of LPR Corrosion Rate test on Pier Leg

9.15 Petrographic Analysis

A petrographic test can be used to determine chemical and physical irregularities in concrete. This test should be performed by an appropriately experienced laboratory on cores taken from the bridge. The reinforcement should be located to avoid it when coring and the core should be taken from sound concrete.

On site Equipment for Core Samples: Diamond core bit and drill, fixed stand for drill (fixed to concrete surface or stable ground), fresh water supply, covermeter, proforma, note taking equipment. It is assumed that the testing laboratory has all equipment required to perform the test on the cores.

9.16 Ultrasonic Pulse Velocity

This technique measures the transit time (in microseconds) of ultra-sound waves passing from an emitter transducer through a concrete sample to a receiver transducer. The faster the transmission time, the more dense the concrete.

Table 9.6 provides the classification of concrete on the basis of pulse velocity.

Longitudinal Pulse Velocity	Quality of Concrete
>4.5 km/s	Excellent quality
3.5 – 4.5 km/s	Good quality
3.0 – 3.5 km/s	Doubtful
2.0 – 3.0 km/s	Poor quality
<2.0 km/s	Very poor

Table 9.6 - Classification of the quality of concrete on the basis of pulse velocity (Neville, 1995)

APPENDIX A
LEVEL 3 INSPECTION BRIEF TEMPLATE

LEVEL 2 INSPECTION BRIEF TEMPLATE

The work undertaken in the brief and the output report shall be undertaken in accordance with this Guideline document.

The brief is prepared to identify the objective and scope of the investigation. The brief will most likely be based upon a Level 2 Inspection and the same consultant may be used for both Level 2 and Level 3 Inspections. The consultant may assist in preparing the scope of the Level 3 Inspection. The Level 3 Inspection brief should include the items discussed in the following sections. One brief may cover more than one bridge.

1.0 Background

The background should summarise any past inspection findings, the history of any issues with the structure and identify the prime reason, or reasons, for undertaking the Level 3 Inspection.

2.0 Objective

The objective should clearly state what outcome is expected of the Level 3 Inspection report, for example a baseline report that documents current condition parameters or a scoping report that requires appropriate recommendations for remediation.

3.0 Scope of Works

The scope will include in tabular and possibly diagrammatic form what tests and inspections are required at what locality at each component.

The scope shall include particular requirements of the report. (refer to Appendix B for report template).

If specific design checks are required to confirm/clarify if material problems could be design originated, this may be included in the scope.

The scope shall make clear the Operational and Safety requirements of the inspectors.

4.0 Bridge Location

This section is to include a clear map of each bridge location. Any access particulars should also be included.

5.0 Component Identification

This section is to include the standard phrase " Component identification and bridge orientation shall be as defined in section 5.3 of the Detailed Visual Bridge Inspection Guidelines for Concrete and Steel Bridges (level 2 inspections) Document no: 6706-02-2233"

APPENDIX B
REPORT TEMPLATE

REPORT TEMPLATE

Executive Summary

Provide an executive summary that captures only the essential findings and recommendations. Include the inspection's most pertinent facts in a clear and concise manner.

1.0 Introduction

The introduction should provide background information about the bridge, highlight any issues with the bridge and present a detailed scope of works. The introduction may include some or all of the following:

1.1 Purpose of the inspection

1.2 Background

1.2.1 Bridge Details

1.2.2 Bridge Location and Exposure Environment

1.2.3 Summary of Design for Durability

1.2.5 Summary of Review of Previous Inspection Findings

1.2.4 Project Inputs (including required service life)

1.3 Scope of Works

1.3.1 Visual and Delamination Survey

1.3.2 Testing Schedule

1.3.3 Other (such as specific design check associated with testing)

2.0 Detailed Investigation Results

This section is to present investigation results for each test undertaken. Full inspection data should be included in Appendices. This report section should complement, and refer to, these guidelines and only describe test procedures to the extent necessary for the understanding of the report, or where not covered by these guidelines. This section may include some or all of the following:

2.1 Visual and Delamination Survey Results

2.2 Rebound Hammer

2.3 Concrete Breakout

2.4 Covermeter Survey

2.5 Half-cell Potential Survey

2.6 Resistivity Measurements

2.7 Carbonation Depth and Modelling Results

2.8 Chloride Content and Modelling Results

2.9 Sulfate Content

2.10 Water Analysis

3.0 Summary of Current Condition

This section shall be clearly set out by bridge component

4.0 Discussion

This section should discuss and recommend the various options for maintenance or remediation.

Discussion of investigation results shall be clearly set out by bridge component

5.0 Conclusions and Recommendations

This section should provide a prediction of remaining service life of bridge, or of individual components if relevant.

4.1 Predicted Remaining Service Life of Components and Bridge

4.2 Durability Design Requirements.

4.3 Summary of Recommended Maintenance and Remedial Options

4.4 Conclusion

Appendices

Appendices are to include any relevant workings, predictions, certificates etc.

Typical appendices may include:

Appendix A: Bridge Details

Appendix B: Photographs

Appendix C: Inspection and Testing Schedule

Appendix D: On-site Investigation Results

Appendix E: Depth of carbonation Predictions/other predictions

Appendix F: Laboratory Test Certificates

APPENDIX C
SURVEY LEGEND

CONDITION SURVEY LEGEND

Symbol	Description
	DESIGNATE BOUNDARY OF TEST AREA TO IDENTIFY POTENTIAL TIES OR DOWEL CONNECTIONS IN BRICKWORK
	DELAMINATION
	LIGHT ETCHING
	HEAVY ETCHING
	SPALLING
	SPALLING EXPOSING STEEL REINFORCEMENT
	POPOUT
	HONEYCOMBING
	CRAZING
	RUST STAINING
	LEACHING
	PATCH REPAIR INCLUDING APPARENT BRICK REPLACEMENT OR NOT LEVEL IN PLANE
0.1	CRACK LOCATION AND WIDTH (mm)
	CRACK WITH CRAZING
	CRACK WITH LEACHING DEPOSIT
	CRACK WITH RUST STAINING
'CJ'-----	CONSTRUCTION JOINT
	COLD JOINT
TC =	TAPED COVER (mm)
D of C =	DEPTH OF CARBONATION (mm)
-150	ELECTROCHEMICAL POTENTIAL RESULT (mV)
50	REINFORCEMENT LOCATION AND COVER MEASUREMENT (mm)
	BREAKOUT TO STEEL REINFORCEMENT
	CORE SAMPLE LOCATION
R	RESISTIVITY TEST LOCATION
S	REBOUND HAMMER TEST LOCATION
	DRILLING SAMPLE LOCATION
M	MORTER MISSING BETWEEN BRICKS
	WEEP HOLE IN BRICK POINTING
	BRICK AREA WITH MORTER MISSING OR APPARENTLY FRIABLE
	BRICK AREA WITH SOME VERTICAL CRACKS IN BRICKS
X	APPARENT TIES IN BRICKWORK AND MOST LIKELY 'U' SHAPED TO INNER LEAF BRICKWORK

APPENDIX D
INSPECTION RESULTS TEMPLATE

INSPECTION RESULTS TEMPLATE

Item	Description
Table D1	Summary of Visual Inspection and Delamination Survey Results
Table D2	Summary of OnSite Testing Results – Reinforced Concrete
Table D3	Summary of Onsite Testing - Covermeter Survey Results
Table D4	Summary of Onsite Testing Results - Steel Components
Table D5	Summary of Laboratory Investigation and Modelling Results

These tables are to be used in report, but may be modified to suit testing. The summary of visual inspection and delamination survey results may use photographic or diagrammatic record in addition or instead of table if necessary.

TABLE D1 - Summary of Visual and Delamination Survey Results

Structure	Component Type	Crack Details			Delaminations/ Spalling/ Rust Stain/ Patch Repair / Other Defects			MRWA Condition State (refer MRWA L2 Guideline)
		No. of cracks	Type	Max width	Type	Area (Approximate)	% of total area	
				mm		m^2	%	%

LEGEND

HL Hair Line

NOTE Defects not available in the MRWA Level 2 Guidelines Condition State criteria are assessed based on engineering judgement

TABLE D2- Summary of Onsite Testing Results - reinforced concrete components

Component	Location	Crack Details				Delaminations/ Spalling/Rust Stain			Rebound Hammer		Concrete Breakout			Depth of Carbonation	Half-cell Potentials		Resistivity	
		No. of cracks	Type	Rust stain	Max width	Type	area	% of total area	Ave	SD	Rebar size/type	Taped Cover	Rebar Corrosion Classification	Average Depth	Range	Risk of Active Corrosion	Ave	Probability of Corrosion
				Yes/No			m2		MPa	MPa	mm	mm	(refer App G)	mm	mV		Ohm.cm	

LEGEND

T Transverse
H Horizontal
L Longitudinal
V Vertical
D Diagonal

TABLE D3 - Summary of Onsite Testing - Covermeter Survey Results

Component	Location	Design Cover	Taped Cover	Concrete Cover by Covermeter		
				min	mean	max
		mm	mm	mm	mm	mm

LEGEND

T Transverse
H Horizontal
L Longitudinal
V Vertical
D Diagonal

TABLE D4- Summary of Onsite Testing Results - Steel Components

Component	Location	Dye Penetration	Magnetic Particle	Radiographic

<u>LEGEND</u>

T	Transverse
H	Horizontal
L	Longitudinal
V	Vertical
D	Diagonal

TABLE D5 - Summary of Laboratory Investigation and Modelling Results

Component	**Location**	**Sulphate Content by wt of Concrete**	**Depth of Carbonation**	**Modelling Carbonation Activation**			**Cover used for Modelling (specify min, mean, max)**	**Chloride Content (% by wt of Concrete)**		**Modelling Chloride Activation at 0.06% or 0.03% wt Concrete**		
			Average Depth	Diffusion Coefficient	Years from construction	Years from now (year)		Surface	At cover	Diffusion Coefficient	Years from construction	Years from 2011 (now)
	(Refer Appendix C)	%	mm	mm/√yr	yrs	yrs	(mm)	%	%	m^2/s	yrs	yrs

LEGEND

C Mean cover as determined by covermeter

T Taped cover

D Design Cover

APPENDIX E
REINFORCEMENT CORROSION CLASSIFICATION

Reinforcement Corrosion Classification

Classification 1

This bar appears passive. There is no observable corrosion, but the bar may have mill scale on the surface. Surface “lime” deposits from the surrounding concrete may also be seen on the bar. There is no loss of section of the bar, nor are there any signs of pitting attack on the profile of the ribs. (refer Figure E1 below)

Classification 2

The bar is largely passive. There is a slight surface corrosion in the form of orange and brown corrosion products appearing in discrete “blotches”, particularly at the intersection points of the ribs. The corrosion products are easily removed by scraping.

Classification 3G

This type of bar has a thin corrosion scale on its surface, mainly red or red/brown in colour. There is no noticeable loss in bar section, although slight damage to the profile of the ribs may have taken place. There will be no cracking of the concrete associated with the formation of these corrosion products.

Classification 3L

This bar has a form of localised attack, which should be distinguished from the general type of attack. It leaves the bar with large areas (up to 75%) which could be in Classification 1 or 2, but the remainder of the bar having undergone a moderately severe attack. There may also have been cracking of the surrounding concrete, due to the formation of expansive corrosion products on the surface of the bar.

Classification 4G

This classification of bar has the majority of its surface area covered with a heavy red or dark brown corrosion scale, up to a thickness of 1mm. The scale is very difficult to remove by simply scraping. There is a loss of bar section associated with this level of corrosion, as much as 10% in places. The rib patterns will have been seriously damaged and, in places, totally removed. The concrete surrounding the bar may have been cracked by the formation of expansive corrosion products. (refer Figure E2 below)

Classification 4L

This classification of bar has approximately 50% of its surface area covered with thick, dark red, brown and black corrosion products emanating from severe localised attack. The rust scale at these points may be up to 1.5mm thick. The bar at other areas may fall into the general Corrosion Classifications 1 to 3. The corroded areas may also have suffered severe loss in section, up to 25% in some cases. This will be associated with a very noticeable loss of rib patterns at the corroded points. The corrosion product formation at these points will almost certainly result in cracking on the concrete and may, in some cases, result in spalling.

Classification 5G

This level of deterioration is the most severe general attack. The major characteristic of this type of attack is the scale thickness and colour. The scale will be orange and dark brown; the thickness may exceed 2.5mm. Unlike most of the other levels of corrosion, this scale flakes very easily and large pieces will come away from the bar, simply by tapping it. There will be a variable loss in bar sections, but it may reach 50%. There will be a severe loss of rib profile; in certain areas they will be completely removed. The concrete cover to reinforcement which has undergone this type of corrosion will be severely deteriorated, with spalling and delamination likely.

Classification 5L

This bar has suffered a severe localised attack, whereas the rest of the bar may have suffered general corrosion to a much lesser degree. This will produce heavy black corrosion products, which may, in certain cases, resemble soot. This type of attack will produce very deep pits, which may result in up to

50% loss in bar section and rib patterns at these points will be completely destroyed. The corrosion products produced may cause severe cracking, spalling or delamination of the cover concrete. This type of attack is the most severe and is likely to be the least common.

Measurement of Loss of Section

A small amount of loss of reinforcement section can produce copious rust product because of its expansive nature. Therefore extreme care should be taken to ensure that the loss of bar section is not estimated from the thickness of corrosion product. Estimates of loss or bar section should be made by cleaning of the bar and measuring its diameter.

Where:

G = General

L = Localised

Figure E1 – Classification1

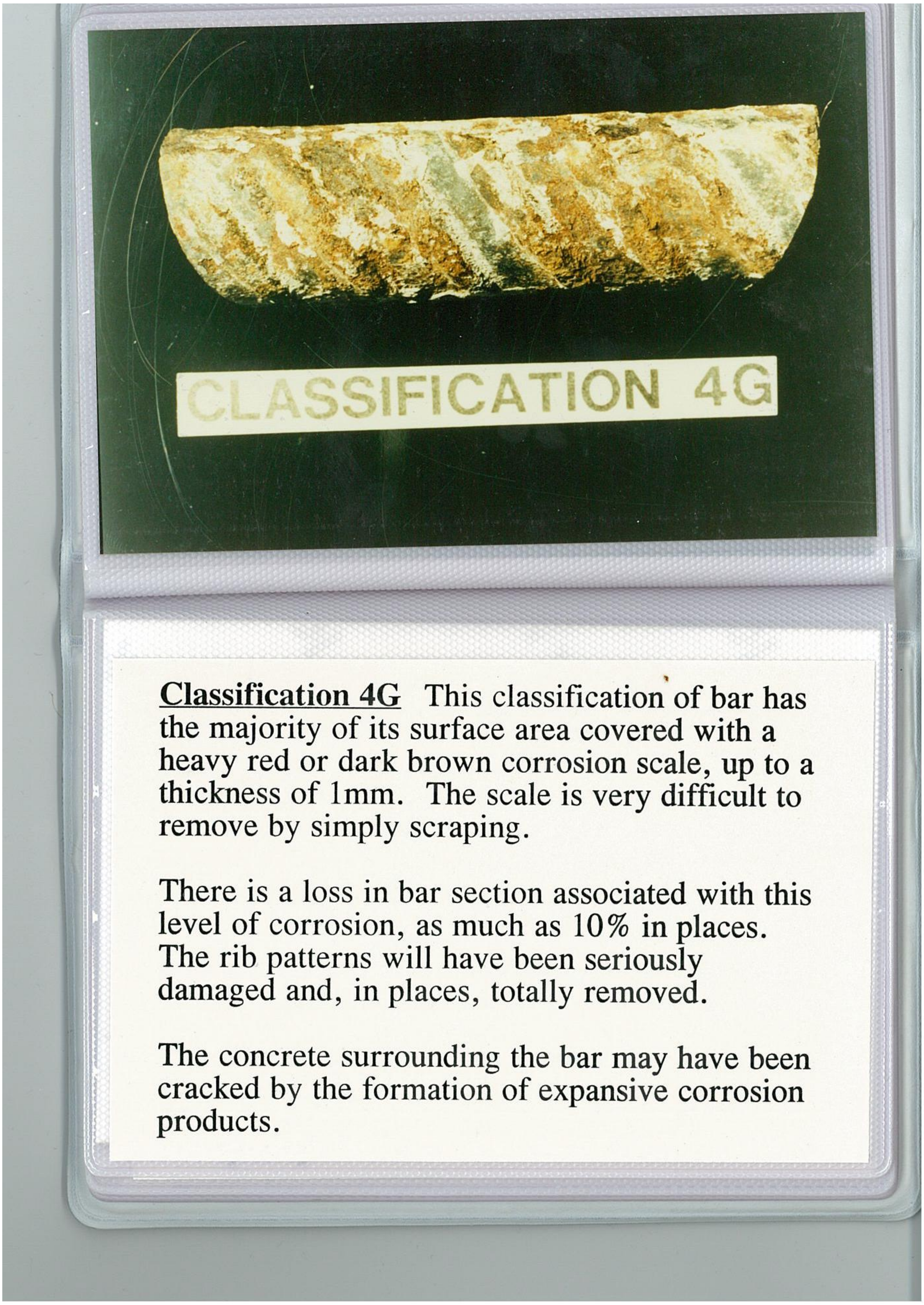


Figure E2 – Classification 4G

APPENDIX F

Exposure Environment for Bridge Structures

Exposure Environment for Bridge Structures

Bridge structures can be located in various exposure environments, characterised by various degrees of severity of exposure. The exposure environments of a bridge's structural components should be classified in accordance with AS5100.5 Section 4.

These exposure environments may include moderate environments such as inland or non-coastal locations where carbonation or ingress of moisture may be problematic depending on the quality of the concrete. Bridge members may be in aggressive soil such as acid sulfate soils, salt rich arid areas etc. The bridge could be in a coastal area but may not have a tidal or splash zone.

The exposure environments for structural components in sea water can be classified as submerged, tidal, splash and atmospheric. An example of the exposure classification for the bridge shown in Figure F1 below with components provided in Table F1 below.

Zone Description	Location of Zone	AS5100.5 Exposure Classification
Atmospheric Zone	Columns – Higher than (say) 0.5m above High Water Level Bridge Deck Beams and Cap Beams Abutments	B2
Splash Zone	Columns - Between High Water Level and 0.5m above High Water Level	C
Tidal Zone	Pile Caps and Columns Between Low Water Level and High Water Level	C
Submerged Zone	Pile Caps and Piles below Low Water Level	B2

Table F1 - Example of Bridge Exposure Classifications

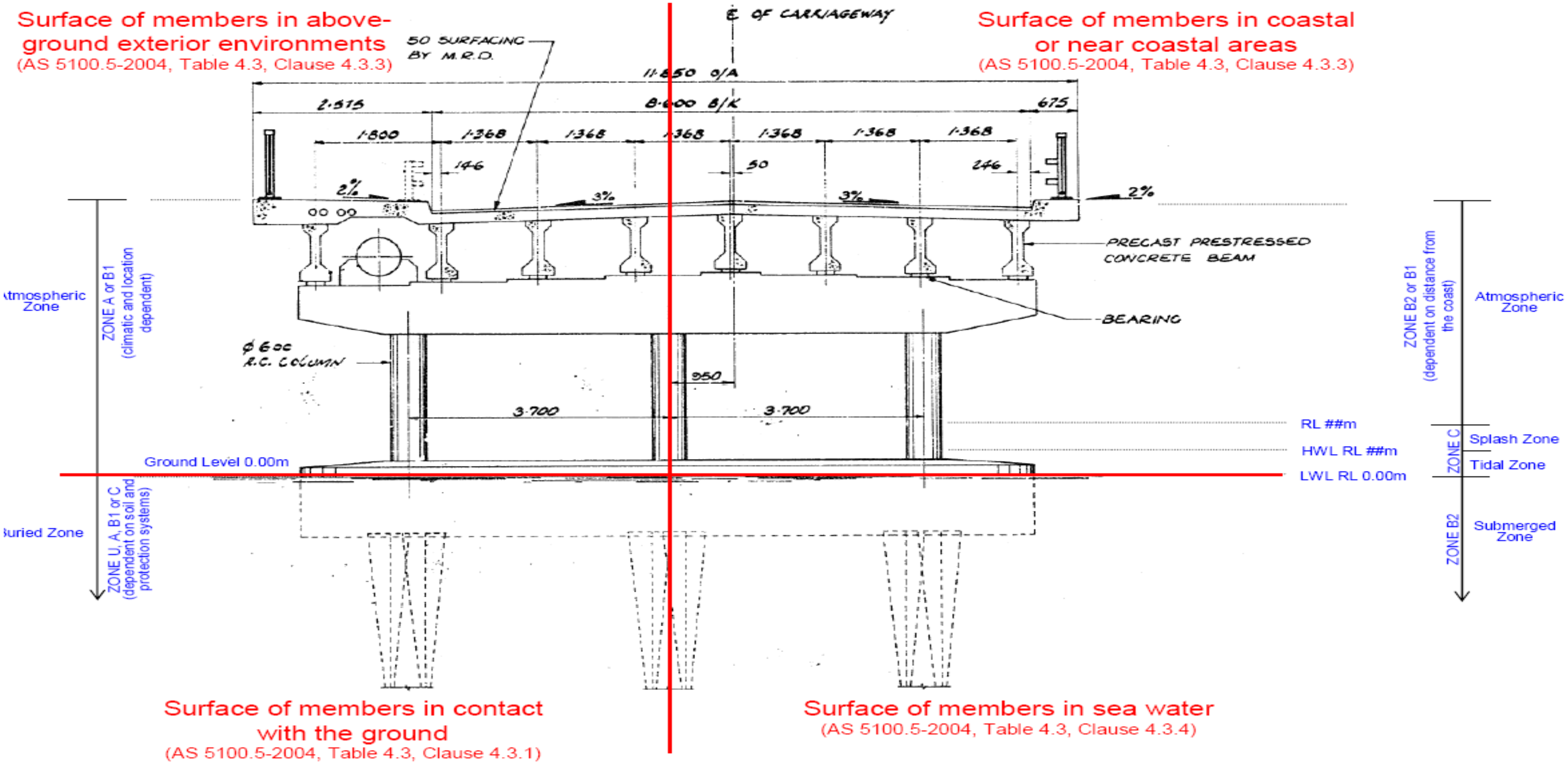


Figure F1 - Example Bridge Structural Components Exposure Classifications

Design for Durability

As a “benchmarking” survey it is important to determine whether or not the bridge originally met and is expected to meet, all durability design requirements. The design for durability requirements are:

a) **Original Durability Design Requirements** - to identify if the bridge has been designed for a specific intended service life.

b) **Current Durability Design Requirements** - to identify if the bridge’s structural components comply with the durability design requirements in accordance with AS5100.5-2004.

c) **MRWA Expected Durability Requirements** - if the original bridge design does not comply with the current code i.e. if it differs from the 100 year design life required by AS5100.5-2004 and MRWA will define an intended service life and subsequent management strategy will be developed to achieve the intended service life.

Represents a typical example of durability design requirements of bridge structural components against AS5100.5-2004

Structure Components	**Exposure Classification**	**Minimum Compressive Strength** (MPa)	**Design Cover** (mm)	**Compliance with AS5100.5-2004 for 100 years design life?**
Abutment Cap Beams and Diaphragms				
Deck Slab				
Beams				
Pier Cap Beams				
Pier Columns (atmospheric)				
Pier Columns (tidal and splash zones)				
Pier Pile Cap (tidal zone)				
Pier Pile Cap (submerged zone)				

Table F2 - Example summary of design information for durability

APPENDIX G
Estimation of the Time of Initiation of Corrosion

Estimation of the Time of Initiation of Reinforcement Corrosion

G1.1 Initiation of time of chloride ion induced corrosion

A reliable prediction model for the ingress of chloride ions into concrete should consider the complex combination of several transport mechanisms (Neville 1995; Kropp et al. 1995), such as diffusion, capillary sorption (i.e. absorption of water containing chlorides into an unsaturated concrete surface layer, typically down to 10-20 mm depth), and permeation (i.e. water flow through concrete due to a pressure gradient). Beyond the surface capillary sorption zone, the ionic diffusion process will dominate (Tuutti 1996).

Once the chlorides have penetrated the concrete cover and reached the reinforcement and their concentration has increased to the "threshold level" after a certain time period (initiation time), corrosion of the reinforcement can be initiated.

Chloride diffusion is the transfer of mass by random motion of the free chloride ions in the pore solution resulting from regions of higher concentration to regions of lower concentration (Crank 1975). Since the ingress of chloride ions into concrete involves inward movement of water containing chloride ions through its pore structure, the prediction of chloride ion penetration into concrete is usually obtained using Fick's second law of diffusion.

Theoretically, the initiation time (t) can be estimated by Fick's second law of diffusion, which has the form:

$$C(x,t) = C_i + (C_S - C_i)\,erfc\left[\frac{x}{2\sqrt{Dt}}\right]$$

where D is the chloride diffusion coefficient; C_i is the initial background chloride concentration of concrete and is usually negligible; C_S is the surface chloride content, x is depth in concrete, $C(x,t)$ is the chloride concentration at depth x after time t and *erfc* is the complement of the error function. Concrete's resistance to chloride penetration is characterised by D and C_S.

Despite its simplicity and extensive use, this model has some shortcomings, because the diffusion coefficient is not a constant but rather depends on time, temperature and depth because of the heterogeneous nature and aging of concrete (Cady and Weyers 1982; Neville 1995; Kropp et al. 1995).

In general, the values of the surface chloride concentration and diffusion coefficient can be estimated from the above equation by determining the best fit curve through data obtained by laboratory analysis of chloride ion content of concrete samples (refer Figure G1).

For an existing structure, once the surface chloride concentration and diffusion coefficient are known, taking account of the current age, the initiation time can be estimated (refer Figure G2). In some cases, the chloride threshold will already have been exceeded at the reinforcement depth at the time of testing, and no calculation of initiation time is required.

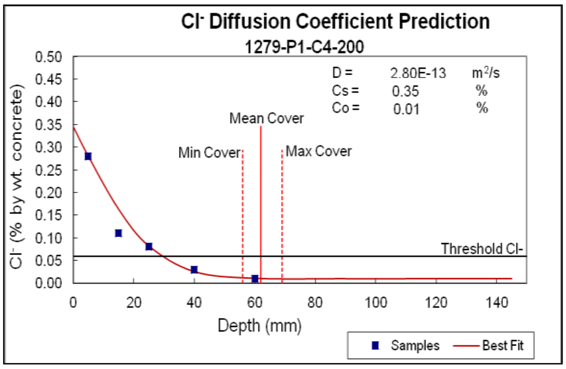


Figure G1 – PREDICTION OF DIFFUSION COEFFICIENT & SURFACE CONCENTRATION

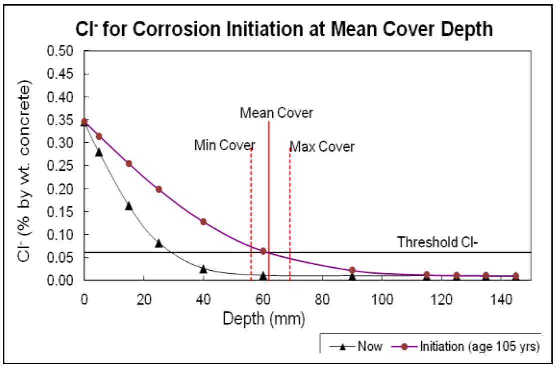


Figure G2 – ESTIMATION OF CORROSION INITIATION TIME FROM KNOWN DIFFUSION COEFFICIENT AND SURFACE CONCENTRATION

The critical chloride threshold concentration, above which the risk of reinforcement corrosion initiation becomes significant, depends on factors such as cement content, water content, cement chemistry, pH, oxygen concentration, type of reinforcement and exposure conditions.

Historically, the critical chloride threshold for plain (black) steel adopted in modelling was 0.4% by weight cement, based on Building Research Establishment (BRE) publications. For a typical 32 MPa concrete with GP cement, this equated to a value of 0.06% by weight of concrete, and this was a value widely adopted for assessment of corrosion risk in the absence of detailed knowledge of the concrete properties (Bertolini et al., 2004; Lay et al., 2003). Given the heterogenous nature of concrete, a risk based approach as set out in BRE Digest 444.2 is now often adopted.

The chloride threshold value for prestressed steel in concrete is less well defined and is assumed to be lower than the value for reinforced concrete. Use of a lower threshold is based on a number of factors including the consequences of corrosion (catastrophic failure of the prestressing strands) and susceptibility to stress corrosion cracking as discussed in ACI 222R-01. Also, lower threshold values are based on research findings by Stark who determined critical chloride concentrations around 0.17% by weight of cement (Stark, 1984). Assuming a minimum cementitious content of 450 kg/m^3 and concrete density of 2,350 kg/m^3 for S50 precast prestressed piles, this equates to a threshold of 0.032% by weight of concrete.

Hence, critical chloride threshold for prestressed steel in concrete is often taken as 0.03% by weight of concrete. Stainless steel has significantly greater resistance to corrosion than mild steel, and a critical chloride threshold of 0.45% by weight of concrete has been proposed by some researchers for modelling purposes (Concrete Society TR61).

G1.2 Initiation of time of carbonation induced corrosion

The carbonation process causes the pH value of the concrete pore solution to decrease from around pH13 to lower than pH10, which is due to the conversion of sodium, potassium and calcium hydroxides, calcium silicate hydrates (CSH) and other cement hydration products into carbonates. The equilibrium pH for $CaCO_3$ is about 9.5. The passive iron oxide layer on reinforcing steel will not be stable when pH is lower than about pH9 to 1011 (BRE Digest 444.1), and the steel reinforcement can then readily corrode in the presence of oxygen and moisture.

Carbonation occurs progressively from the surfaces of the concrete exposed to atmospheric CO_2, but does so at a decreasing rate because the CO_2 has to diffuse through the pore system, including the already carbonated surface zone of concrete.

The rate of carbonation is dependent on the permeability of the concrete to carbon dioxide, which is dependent on the total alkali content (which is a function of the cement content and type of cements), water/cement ratio, and available moisture in the hardened concrete (which is a function of the atmospheric relative humidity). At low relative humidity, there is insufficient water in the concrete for the carbonation reactions to progress. If the pores of the hydrated cement paste are filled with water, the diffusion of CO_2 is slowed. The rate of carbonation is highest in the relative humidity range RH 60 to 75%.

A generalised carbonation model involving a relationship between depth of carbonation x_1, time of exposure t_1, and carbonation coefficient D_{Carb} is used for carbonation penetration predictions. This relationship is as follows:

$$x_1 = D_{Carb} \sqrt{t_1}$$

The actual depth of the concrete cover that has carbonated is determined by phenolphthalein indicator. Using this depth and with the age of the concrete, the above equation can be used to determine the D_{Carb}. The above equation is then used again to determine an estimate of the length of time needed for the whole cover concrete to become carbonated and leave the reinforcement in an environment where corrosion can commence.

Figure G3 shows is a typical plot of the prediction of the depth of carbonation over time and shows the time at which the carbonation depth will attain a depth equivalent to the minimum reinforcement cover i.e. carbonation reaches a depth of 30 mm when the concrete is at an age of approximately 80 years.

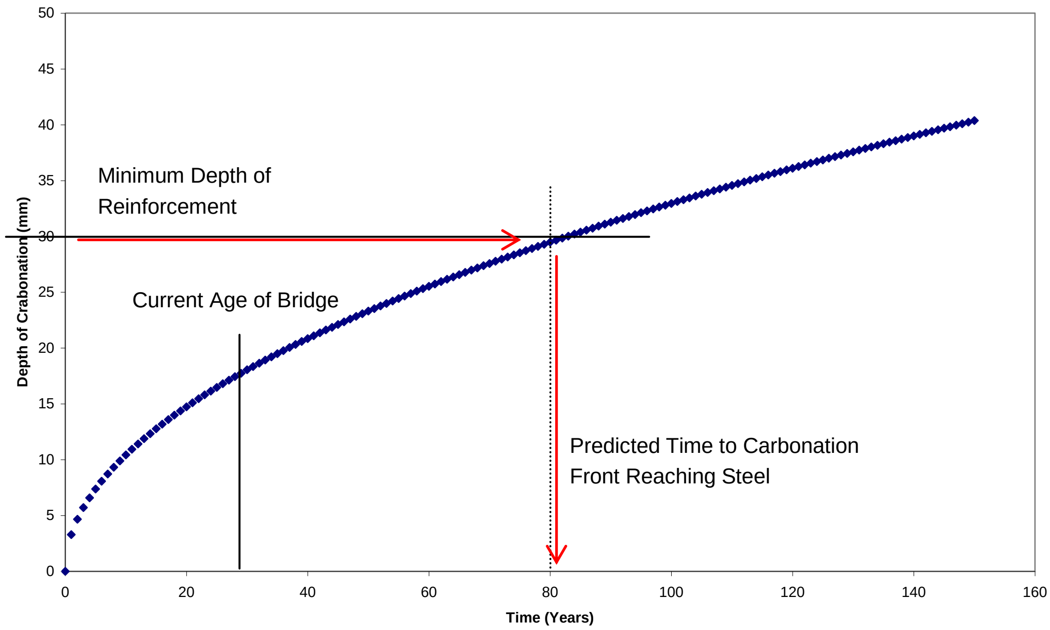


Figure G3 - Prediction of Depth of Carbonation vs Time

G1.3 Service Life Determination

An exact or definitive remaining service life is not possible to devise, however a reasonable prediction can be made based on the visible evidence, relevant laboratory results, modelling results and existing literature regarding deterioration of reinforced concrete.

G1.4 Condition Limit States

The definition of failure or service life of corrosion-damaged concrete structures is not a straight forward task. According to BRE Digest 434 service life can be defined as the time at which any of the following limit states are reached: onset of corrosion, cracking, delamination, spalling, or accumulated damage reaching some specified amount. An appropriate definition of failure, and consequently service life should consider the acceptable risk of failure, which depends on the risk of loss of life and injury, type of structure, mode of failure, etc.

The main failure mechanism affecting the remaining service life of the reinforced concrete components of bridges is corrosion of the reinforcement. The commonly adopted mechanism for corrosion damage states that the time to loss of structural integrity is made up of time to corrosion activation followed by time of corrosion propagation (Bamforth and Pocock, 2000), as seen in Figure G4.

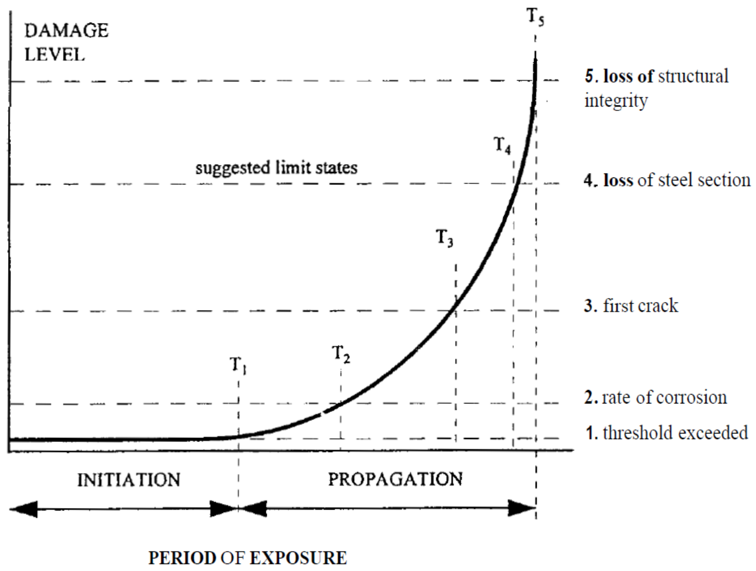


Figure G4 - Condition Limit states for deterioration caused by reinforcement corrosion (Bamforth and Pocock, 2000)

The determination of service life must take into account three contributing factors, which may be related to the Condition Limit States in Figure G4. These are:

1. Limit of acceptability – key indicators would be loss of aesthetics and safety for all users and the environment. This could be loosely related to T_3.
2. Limit of serviceability – meaning a reduction in load carrying capacity, related to T_4 as a loss of steel section may result in a loss of strength.
3. Structural Adequacy Compromised – the bridge is unsafe to go over or under, related to T_5, and may require major rehabilitation

In terms of the management of a group of common structures, an Asset Management decision to define the acceptable damage level can be made in one of two ways. These are:

- Generic definition – This identifies generic condition state limits for all bridges, and if the damage level is at or beyond certain condition state limits, then either intervention is required, or the service life has been compromised.
- “Bridge-by-bridge” definition – This defines the condition state limit for both end of service life and intervention based on bridge specific requirements.

The Consultant carrying out the Level 3 Inspection should consult with the MRWA Asset Manager Structures to determine an appropriate damage level for the bridge before it either requires intervention, or is deemed to have reached its service life.

For example, it may not be acceptable to allow a bridge over a busy highway to reach T_3 as cracking and spalling may present a safety risk to motorists below. Conversely, a bridge nearing the end of its functional life or scheduled for replacement may be considered to be able to reach T_4 or beyond with specific additional management activities such as frequent inspection and monitoring.

G1.5 Initiation Phase

The initiation of corrosion of the first layer of reinforcing steel (time to T_1) due to chloride ingress or carbonation may be estimated by chloride modelling and carbonation modelling, as discussed in G1.2.

G1.6 Corrosion Propagation Phase

Some time after $T_{1.}$ stresses induced by the expansion of corrosion products will lead to fracture of concrete (cracking, delamination, spalling), loss of ultimate strength, loss of bond between steel and concrete and ultimately loss of structural capacity. This deterioration is primarily dependant on the rate of corrosion, fracture properties of concrete, reinforcement area, size and spacing and cover depth. The time after T_1 depends on the corrosion propagation rate, which is often difficult to estimate. In situ corrosion rate tests such as Linear Polarisation Resistance (LPR) or galvanostatic pulse corrosion rate may be used to determine the rate of corrosion, however these tests have limitations and only provide an approximate estimate of corrosion rate at the time of the test. Measurement of actual corrosion loss can also provide an estimate of corrosion rate, although this may tend to over-estimate the general rate. Other site measurements that assist in estimating probable propagation time include concrete resistivity, half-cell potential maps and visual inspections of the reinforcement at breakouts.

From literature and based on industry experience, the period between activation and first significant crack (between T_1 and T_3 in Figure G4) is typically of the order 10 to 20 years at high corrosion rates up to 10 microns/year (BRE 434) and for crack widths up to 0.3 mm. The time to cracking is longer for small diameter bars due to a smaller volume of rust product being formed, and where cover is greater.

The significance of cracking depends on the structural component.

G1.7 Prediction of Remaining Service Life

The predicted service life, however, would be the addition of time to T_1 and time from T_1 to the designated condition limit state for end of service life (whether this is T_3, T_4 or T_5).

With current materials condition, MRWA will estimate the service life as follows:

Age at end of Service life = (Time to T_1) OR

(Time to T_1) + (Time from T_1 to T_3) OR

(Time to T_1) + (Time from T_1 to T_3) + (Time from T_3 to T_4)

The assessment of time from T1 to T3 or T4 requires a considerable amount of engineering judgment when the reinforcement arrangement and number of bars departs from that studied in the literature above.

Any engineering judgement on this time from T1 to T3 and T4 should be backed up as far as practicable with results from testing and observations of the exposed reinforcing bar and all assumptions/considerations clearly identified in the report for consideration by MRWA.

APPENDIX H
REFERENCES

REFERENCES

Australian Standard

AS 1012.9	Determination of the compressive strength of concrete specimens.
AS 1012.10	Methods of testing concrete - Determination of indirect tensile strength of concrete cylinders (Brasil or splitting test)
AS 1012.14	Method for securing and testing cores from hardened concrete for compressive strength.
AS 1012.20-1992	Determination of chloride and sulfate content in hardened concrete and concrete aggregates.
AS 1012.21	Determination of water absorption and apparent volume of permeable voids in hardened concrete.
AS 1171	Non-destructive testing - Magnetic particle testing of ferromagnetic products, components and structures.
AS 1379	Specification and supply of concrete
AS 1391 – 2007	Metallic materials – Tensile testing at ambient temperature.
AS 1710	Non-destructive testing - Ultrasonic testing of carbon and low alloy steel plate and universal sections - Test methods and quality classifications
AS 1816.1	Metallic materials - Brinell hardness test - Test method (ISO 6506-1:2005, MOD)
AS 2062	Non-destructive testing – Penetrant testing of products and components.
AS 3507.2	Non-destructive testing – Radiographic determination of quality of ferrous castings.
AS 3978	Non-destructive testing – Visual inspection of metal products and components.

ASTM Standard

ASTM C295 concrete.	Standard Guide for Petrographic Examination of Aggregates for
ASTM C597 – 09	Standard Test Method for Pulse velocity through concrete.
ASTM C642 – 06	Standard Test Method for density, absorption, and voids in hardened concrete.
ASTM C805-08	Standard Test Method for Rebound Number of Hardened Concrete.
ASTM C856-11	Standard Practice for Petrographic Examination of Hardened Concrete.
ASTM C876-09	Standard Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete.
ASTM C900	Standard Test Method for Pullout Strength of Hardened Concrete
ASTM C1383-04(2010)	Standard Test Method for Measuring the P-Wave Speed and the thickness of concrete plates using the Impact-Echo Method.

ASTM D4580 - 03(2007)	Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding
ASTM D6432 – 99 (2005)	Standard Guide for using the surface ground penetrating radar method for subsurface investigation.
ASTM E247 - 01(2010)	Standard Test Method for Determination of Silica in Manganese Ores, Iron Ores, and Related Materials by Gravimetry
ASTM E407 - 07e1	Standard Practice for Microetching Metals and Alloys
ASTM E488	Standard Test Methods for Strength of Anchors in Concrete Elements
ASTM G57 – 06	Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four Electrode Method.

British Standard

BS 1881 Part 124:1988 Methods for Testing Concrete Part 124: methods for Analysis of Hardened Concrete.

BS 1881 Part 204:1988	Testing concrete. Recommendations on the use of electromagnetic covermeters.
BS EN 444	Non-destructive testing. General principles for radiographic examination of metallic materials by X- and gamma-rays.
BS EN 571-1:1997	Non-destructive testing. Penetrant testing. General principle.
BS EN 1435:1997	Non-destructive examination of welds. Radiographic examination of welded joints

MRWA Test Methods

MRWA 620.1	Carbonation of Concrete
MRWA 621.1	Alkali-Silica Reaction (ASR)
MRWA 622.1	Resistivity of Concrete
MRWA 623.1	Concrete Cover to Reinforcement

Found at:

http://standards.mainroads.wa.gov.au/NR/mrwa/frames/standards/standards.asp?G={E582C897-FF5E-4C02-8B46-51E88C1E5DD8}

and go to Materials Engineering/Test Methods

Strategic Highway Research Program

SHRP-S-324	Condition evaluation of concrete bridges relative to reinforcement corrosion. Volume 2, DC 1993
SHRP-S-330	Condition evaluation of concrete bridges relative to reinforcement corrosion. Volume 8, DC 1993

Cited References

1. American Concrete Institute 222.2R-01 (2001) Protection of Metals in Concrete Against corrosion.
2. Australian Standards HB 84-2006 Handbook, Guide to Concrete Repair and Protection.
3. AS 5100.5 Bridge design - Concrete
4. Austroad 2002, Management of Concrete Bridge Structures to Extend their Service Life.
5. Bamforth, P.B. and Pocock, D.C. (2000) Design for durability of reinforced concrete exposed to chlorides, Workshop of structures with Service Life of 100 years-or more, Bahrain.
6. Bertolini, L., Elsener, B., Pedeferri, P. and Polder, R (2004) Corrosion of Steel in Concrete, Wiley-VCH, Weinheim.
7. BRE Digest 434, 444 parts 1 and 2
8. Broomfield, J.P. (1997) Corrosion of Steel in Concrete: Understanding, investigation and repair, E & FN SPON, London.
9. Cady, P.D., and Weyers, R.E. (1983) Chloride penetration and the deterioration of concrete bridge decks, Cement, Concrete and Aggregates, 5(2), 81-87.
10. Cement Concrete & Aggregate Australia (2009) Chloride Resistance of Concrete.
11. Concrete Society (1982) Non-structural cracks in concrete, Concrete Society, Technical Report, No. 22, p 38.
12. Aldred J.M., Grace W.R. and Trinder P.W. (2003) Fine tuning Service Life Prediction of Coastal Structures using Early Age Site Testing, Coasts & Ports Australasian Conference, Auckland.
13. Kirkby G., Collins F. and Grace W.R. (1994) A Quantitative Model for Deisgn Life for use by Designer as Well as Asset Manager, 1994, Austroads Bridge Conference, Australia.
14. Kropp, J. and Hilsdorf, H.K. (1995) Performance criteria for concrete durability, Rilem report 12, E&FN Spon, London.
15. Lay, S., Schießl, P. and Cairns, J. (2003) LIFECON Deliverable 3.2, Service Life Models, 2003
16. Lounis, Z. and Amleh, L. (2004), Reliability-based prediction chloride ingress and reinforcement corrosion of aging concrete bridge decks, ASCE 2004, pp. 113-122.
17. Neville, A. (1995) Chloride attack of reinforced concrete: an overview. Materials and Structures, 28, 63-70.
18. Neville, A.M. (1995) Properties of Concrete, 4th Edition, S & T Press, page 506. ISBN 0582230705.
19. PIARC (2011) Technical Committee D.3 Road bridges Working Group 2 – Summary of Used Non-Destructive Tests.
20. Stark, D.(1984), Determination of Permissible Chloride Levels in Prestressed Concrete, PCI Journal, July-August 1984, pp. 106-116.
21. VicRoads (2009) Guide to the assessment, maintenance and rehabilitation of concrete bridges.
22. Main Roads Western Australia (2008) Level 2 Guidelines for concrete & steel bridges.