Guideline for the Preparation of Road Structure Durability Plans

Queensland Government

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Executive Summary

The Bridge Asset Management Group has developed a series of documents to provide guidance and uniformity of approach to the operation of structures. This document forms part of this programme and sets out the preferred approach for the development and implementation of road structure durability plans for new construction projects, to enable effective management and maintenance of the structure and achieve better asset performance.

It is recognised that decisions made at design stage have a profound impact on in-service performance, maintenance intensity and whole of life cost. By requiring designers to consider the materials, design detailing, construction methods and operational aspects of structures the objective of longer lasting, low maintenance structures, that represent a good investment for the State Government of Queensland, can be achieved.

This guideline sets out the requirements for the content of durability plan reports to be prepared by the designer and provides the format of summary tables to enable the standardised collation of information. The durability plan requirements include:

- the identification of deterioration mechanisms;
- materials selection;
- the development of mitigation measures to ensure that the design intent is met;
- the identification of durability critical construction activities; and
- verification of constructed components to confirm compliance with durability requirements.

To facilitate uniformity in the durability planning process between projects, and assist with the standardised collation of information the durability planning process will require the completion of a series of summary tables. These tables must be completed in addition to the preparation of the durability plan report, and will provide a summary of the:

- durability intent;
- protective measures implemented in the project;
- inspection and maintenance provisions;
- record on going activities during the service life pertaining to durability.

Glossary

Term	Definition
BAM	Bridge Asset Management Group.
Bridge and Culvert Servicing	Refer to Bridge & Culvert Servicing Manual. Works to prevent damage or deterioration of a structure that would otherwise be more costly to restore if left to progress to structural damage. This includes routine, preventative and programmed maintenance.
Bridge Rehabilitation	This is the restoration of a structure to its original functional performance. The strengthening of bridges to provide a load capacity greater than its original design is excluded as this should be considered as part of the capital enhancement programme of works. Typical rehabilitation activities include deck replacement, pile splicing, timber component replacement, concrete repairs and joint and bearing replacement.
Condition State	The condition of component assessed by an accredited inspector in accordance with the Bridge Inspection Manual. (Condition deteriorating 1 to 4)
Design Life	The expected operational life of a structure.
DPR	Durability Plan Report.
DTMR	Department of Transport and Main Roads.
Durability	The ability of materials or structures to resist environmental loads while maintaining desired performance parameters.
Periodic Maintenance	Planned maintenance over and above those activities undertaken during routine servicing.
Preventative Maintenance	These are maintenance activities designed to maximise the performance and longevity of the bridge and its components. Typical activities include the application of pest and fungal treatments, bearings lubrication, spot/patch repairs to coatings and crack sealing.
Programmed Maintenance	This is cyclical maintenance that generally does not apply to structures with the exception of painting steelwork. However, given the poor state of many of our steel bridges where a regular painting regime has not been implemented, this activity will be initiated within the bridge rehabilitation programme when steel bridges are restored to their original condition. (It is acknowledged that this definition is not universally attributed to programmed maintenance across the department at this time.)
Routine Maintenance	This includes activities that maintain the serviceability of a structure but do not change the condition of the bridge or its components. Typical activities include clearing of drainage, localised repairs of deck surface, cleaning and adjusting of deck joints, vegetation control and debris removal.
Service Life	Expected operational life of the component before failure during which time it will be able to carry all normal traffic loads, maintain user safety and comfort, and an acceptable appearance.
Servicing	Periodic routine work required to preserve the structure so that it can perform as intended. Activities undertaken are defined in Main Roads Bridge / Culvert Servicing Manual.





Part One
Introduction

Queensland Government

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List of Figures and Tables

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1 Introduction

1.1 General

Queensland Department of Transport and Main Roads (DTMR) Bridge Asset Management Section is responsible for the development and implementation of robust and reliable mechanisms for the inspection, maintenance and operation of the Government of Queensland's bridge and road structure asset portfolio.

To improve the performance of bridge assets and ensure value for money for the Government of Queensland, the Bridge Asset Management Group (BAM) has developed a series of documents to provide guidance and uniformity in the approach to the operation of structures. This document forms part of this programme and sets out the preferred approach for the development and implementation of road structure durability plans, to enable effective management and maintenance of the structure and achieve better asset performance.

Key to an effective asset management programme is the detail and reliability of information available on which to plan maintenance and investment programmes. The intention of DTMR's durability plan process is intended to:

1. Integrate structure serviceability performance parameters into the design process, such that materials performance, specifications, construction practices, servicing and maintenance requirements are considered from the outset. Through this process it is intended that a design philosophy for structures will be developed that will emphasise lowest whole of life costs, long life and low maintenance.
2. Provide a mechanism for the transfer of key maintenance and servicing information between the asset creation stage and the asset operator. The design philosophy for durability greatly influences the overall strategy for operation and maintenance of the structure. It is therefore important to handover pertinent information on the durability approach produced at design phase to the operator as part of the operation and maintenance manual. The Durability plan provides the mechanism to facilitate this.
3. Ensure that consideration is given to hazards associated with inspection and maintenance activities, and where practicable, the hazards are eliminated or mitigated through design and detailing.

1.2 Asset Performance

Key factors governing how a structure will perform throughout its service life and the total cost of ownership is greatly influenced by decisions made during the design and construction stages. The approach to ensure durable performance adopted by the designer will influence how a structure will be maintained, the intensity of maintenance and the total cost of asset ownership.

> **Law of Fives**
>
> “$1 spent getting the structure designed and built correctly, is as effective as $5 spent in subsequent preventative maintenance in the pre-corrosion phase while carbonation and chlorides are penetrating inwards towards the steel reinforcement. In addition, this $1 is as effective as $25 spent in repair and maintenance when local active corrosion is taking place, and as effective as $125 spent where generalised corrosion is taking place, and where major repairs are necessary and possibly including replacement of complete members.”
>
> *W R De Sitter, Costs for Service Life Optimization: The Law of Fives, Durability of Concrete Structures (1984)*

The objective of the durability guideline is to set out a systematic process to clearly identify how the service life will be achieved by thorough consideration of materials, exposure environment, design detailing and maintenance activities. This will in turn lead to better in service performance of structures that are long lasting and low maintenance. The benefits for long lasting structures will include reduced cost of ownership, reduced disruption to users, increased availability, and improved safety to users and maintenance personnel.

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1.3 Guideline Structure

The retention of relevant information from the asset creation phase through to asset operation and management is an underpinning concept of good asset management. The intent of the durability planning process is to gather and record the information generated through the asset creation process, and collate it in a standard format to facilitate consistency of approach to durability planning. Figure 1 shows the production of documents relevant to durability planning during the design and construction process and how these documents are inter-related.

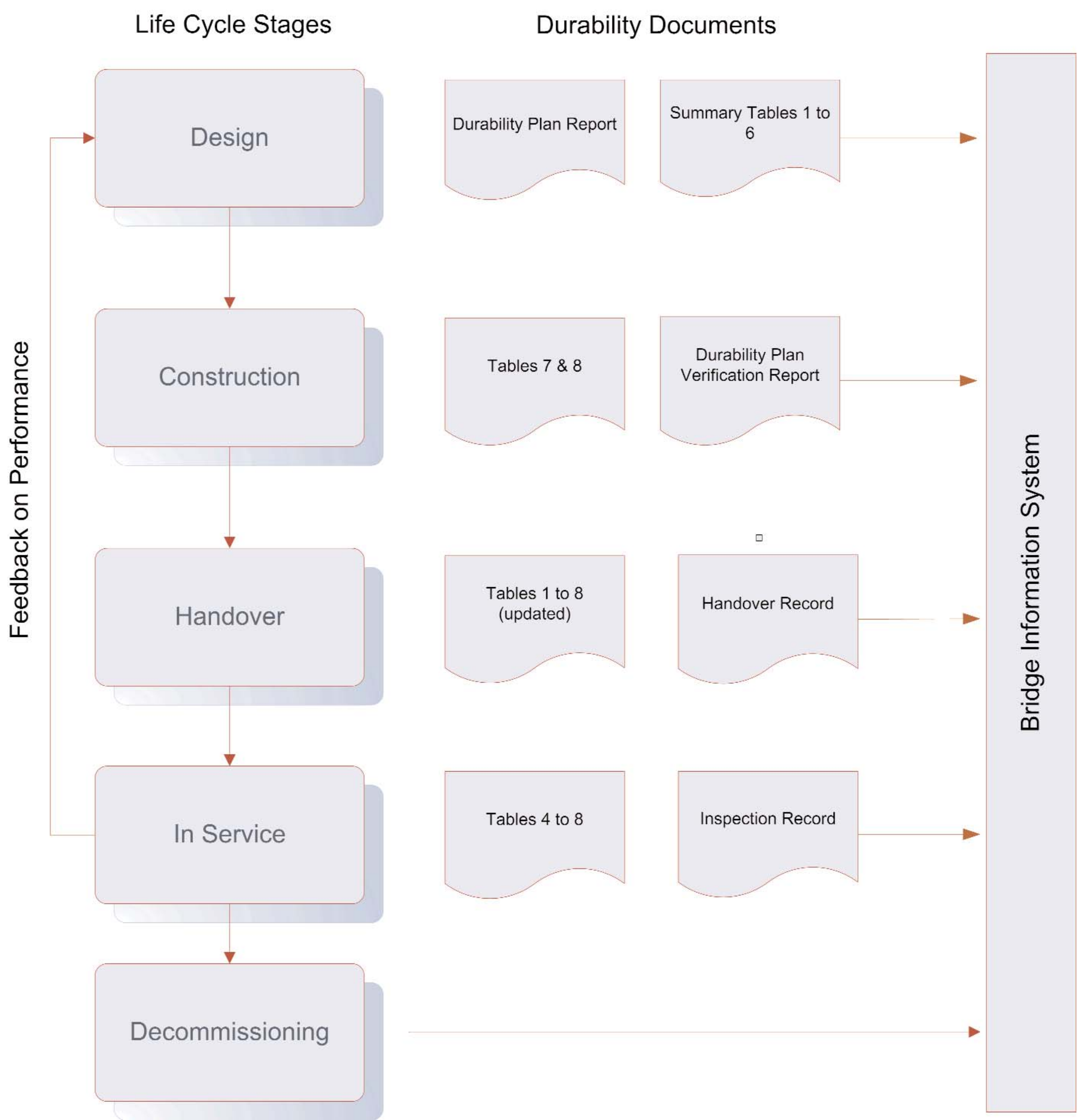

Figure 1: Durability Planning Documentation

The guideline is not intended to provide prescriptive solutions, but provide a framework for the designer and constructor to develop appropriate solutions for the specific structure. A list of reference documentation is provided to assist with development of durability plans, including DTMR specifications for durability planning. This is not an exhaustive list, but provides a prompt for design details that have been adopted successfully elsewhere.

A key intent of the durability planning process is the evaluation of standard details and standard specifications for the situation under consideration. Standard details which work effectively in one application may not be as effect in another. The evaluation process in selecting design details must be thorough and recorded.

To simplify and standardise the presentation of information, tables are provided in the document, which must be completed at the appropriate project phase, and submitted as part of the durability plan for review by DTMR. The information in these summary tables must be sufficiently detailed to convey the approach to achieving the required durability, as this summary information will be uploaded into DTMR's Bridge Information System.

The submission of documents, comprising the DPR and summary tables at key milestones in the design process will enable DTMR to engage in the durability design process. These tables do not negate the need for a durability plan report, but are intended as a template to summarise information and act as a prompt for issues to be considered in the durability planning exercise. The report should as a minimum provide the commentary and explanation of the summary provided in the tables.

The systematic approach to durability planning will provide the starting point for preparation of a structure specific maintenance manual.

Examples of completed summary tables and example durability plan reports are provided in the Appendices.

Section 2 of this report provides guidance on the content required in the DPR and provides guidance on the issues to be addressed.

Section 3 provides guidance on the content of the Post Construction Durability Plan Verification Report.

Section 4 provides guidance on the completion of the summary tables. These tables are intended to be "live" documents, and as such maintenance activities carried out during the operational life of the structure must also be recorded. The design and implementation of repair works are recorded in the same way as for new build works, to ensure that the intended durability is achieved. The implementation of this framework is intended to ensure that cost effective and durable repairs and upgrades are designed and executed.

1.4 Scope

This guideline covers the preparation of durability plans for the following types of road structures:

- Bridges;
- Culverts; and
- Sign gantries.

The durability planning process outline in this document can equally be applied to other road assets. The preparation of durability plans for other road assets, if required, will be stipulated in the contract specifications.

1.5 Submission of Durability Plans

The consideration of durability issues and development of a strategy to achieve the required service life is intended to be integral with the structural design process. The durability plan report must be prepared by a designer (the Durability Consultant) experienced in durability design of similar projects. The durability consultant must be named in the concept report and must remain throughout the project.

Durability Plan Reports must be produced at concept and detailed design stages and will form part of the design report submissions for review by DTMR. On completion of construction a Durability Plan Verification Report must be prepared, containing records of product compliance with durability requirements, and noting

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construction stage departures and any repairs implemented. These durability plans, and associated tables will also be maintained throughout the service life of the structure and be updated with maintenance and upgrade activities. The DPR's and associated tables will act as a log of all durability related works undertaken on the structure.

It is intended that the durability plan will be a live document, and will be updated throughout the service life and include information on repair and upgrade programmes.

The submission of durability reports for review is envisaged as shown in Figure 2.

Deliverable	Scope	By Whom	Reviewed By
Durability Plan Report: Concept Design.	Considers the concept design and environmental information and highlights potential durability issues that will require consideration during engineering and detailed design process. It includes the development of specific durability requirements for incorporation into material supply specifications. Durability Summary Tables 1, 2 and 3.	Durability Consultant	DTMR Structures Division
Durability Plan Report: Engineering Design.	Considers durability issues of the Engineering design. Where appropriate it includes comments on compliance of design and supply specifications with requirements for durability. Highlights durability issues that will require Contractor's consideration and assessment during engineering design process. The report will, where appropriate, incorporate comments on design, specifications, construction method statements and process review procedures. Durability Summary Tables 1, 2, 3, 4, 5, 6 and 8.	Durability Consultant	DTMR Structures Division
Durability Plan Verification Report: Construction Stage.	Verification of Quality and Inspection Records in relation to compliance with requirements of Durability Plan. Verification of the finished product in relation to the requirements of Durability Plan. Durability Summary Table 7.	Durability Consultant	DTMR Structures Division
Durability Plan Report: Repair or Upgrade during service life.	Considers the durability issues related to the maintenance, repair or upgrade of the structure. The report will, where appropriate incorporate comments on design specifications and construction method statements. Durability Summary Tables 1, 2, 3, 4, 5, 6 and 8.	Durability Consultant	DTMR Structures Division
Durability Plan Verification Report: Repair or Upgrade during service life.	Verification of Quality and Inspection Records in relation to compliance with requirements of Durability Plan. Verification of the finished product in relation to the requirements of Durability Plan. Durability Summary Table 7.	Durability Consultant	DTMR Structures Division

Figure 2: Submission of Durability Plans

Part Two

Content of Durability Plan Report

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List of Figures and Tables

2 Content of Durability Plan Report

2.1 General

The following sections describe the required contents of the Durability Plan Report (DPR). A proposed structure of the report with suggested headings and the level of detail and issues to be addressed within the narrative of the report are presented. The exact content and level of detail in the DPR will be dictated by the project and type of structure under consideration.

The DPR must summarise the approach to achieving the specified design life. The report will provide a commentary on the environmental exposure conditions and how associated deterioration processes are expected to degrade the structure. Analysis of the rate of deterioration and comparison against the required design life must be provided and what mitigation measures will be used to resist or slow this deterioration. Factors influencing the degradation such as material properties, design details to manage water shedding and buildability must be considered and documented. For uniformity of presentation of information, and to ensure a similar level of detail between projects, standard summary tables have been produced. These tables must be completed as part of the durability planning process, and hard copies must be included as an appendix to the DPR. In addition a soft copy of the summary tables must able be submitted.

A typical content of a DPR would comprise sections as follows:

- Introduction
- Service Life Criteria
- Exposure Conditions
- Deterioration Mechanisms
- Assessment of Design Solutions
- Inspection and Maintenance Access Requirements
- Replacement of Components
- Construction Verification Plan
- Service life management of Durability

In addition to the narrative given in the body of the report, the pertinent points of the durability design must be summarised in standard tables. These Summary Tables act as prompts from the durability design and must be completed and included as an appendix to the DPR. Blank tables are provided in Appendix E. The tables must contain sufficient detail to summarise the selected durability design as described in detail in the text of the DPR. Completion of the summary tables does not negate the need for the commentary in a durability plan report.

The following sections are intended as guidance on the content of a durability plan report. Particular project requirements or conditions may necessitate additional sections as required to be incorporated in the durability plan report.

Photo 1: Structural Failure due to degradation of timber piers

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2.2 Overview of Durability Design

The approach to durability design and the provisions to ensure durability set out in current Australian Design Codes such as AS 5100 Bridge Design and AS 3600 Concrete Design, may not be adequate to achieve the durability performance that DTMR require. The classification of environments, the effects on a structure and therefore the overall durability are generalised, and may not in any case relate to the design life specified. Consequently the deemed-to-satisfy approach described in standards and codes, where by a particular concrete strength and cover are prescribed for a given service life in broadly defined exposure environment may not be sufficient to meet the particular performance requirements of the structure. The durability design process for DTMR projects must be site and structure specific. The range of deterioration mechanisms that the structure will be exposed to must be assessed at component level, and appropriate protective measures designed. In some instances, where a component may be subject to multiple exposure zones, then assessment at a sub component level may be necessary.

Similarly standard details, including DTMR details, must be assessed on a project specific basis for appropriateness, and may need to be modified. This assessment process must be recorded in the DPR, to confirm that the detail will achieve the intended performance. The slavish application of standard details without a thorough evaluation of the implications on durability is to be avoided.

The approach described below for durability design, sets out a process that is similar to that adopted for structural design. The load cases for an element are defined, in this case environmental loads, and the element designed to resist those loads taking into account the potential for progressive degradation. The durability design process is schematically shown in Figure 3.

This design process is akin to serviceability limit state design approach to durability. Upper bound performance limit states are defined for different components, and this is the level of performance to be achieved during the service life.

The durability design process must accurately assess the local environments, identify the relevant deterioration mechanisms and appropriate mitigation measures. Factors to be included in the assessment include:

- materials,
- design detailing,
- construction method,
- construction quality and
- degree of maintenance.

The durability philosophy adopted must take into consideration all these requirements and be an integral part of the design process to ensure that opportunities in the design process to adopt a long life low maintenance solution are realised. Review of a completed design for durability and inclusion of protective measures, is unlikely to produce an efficient and durable solution.

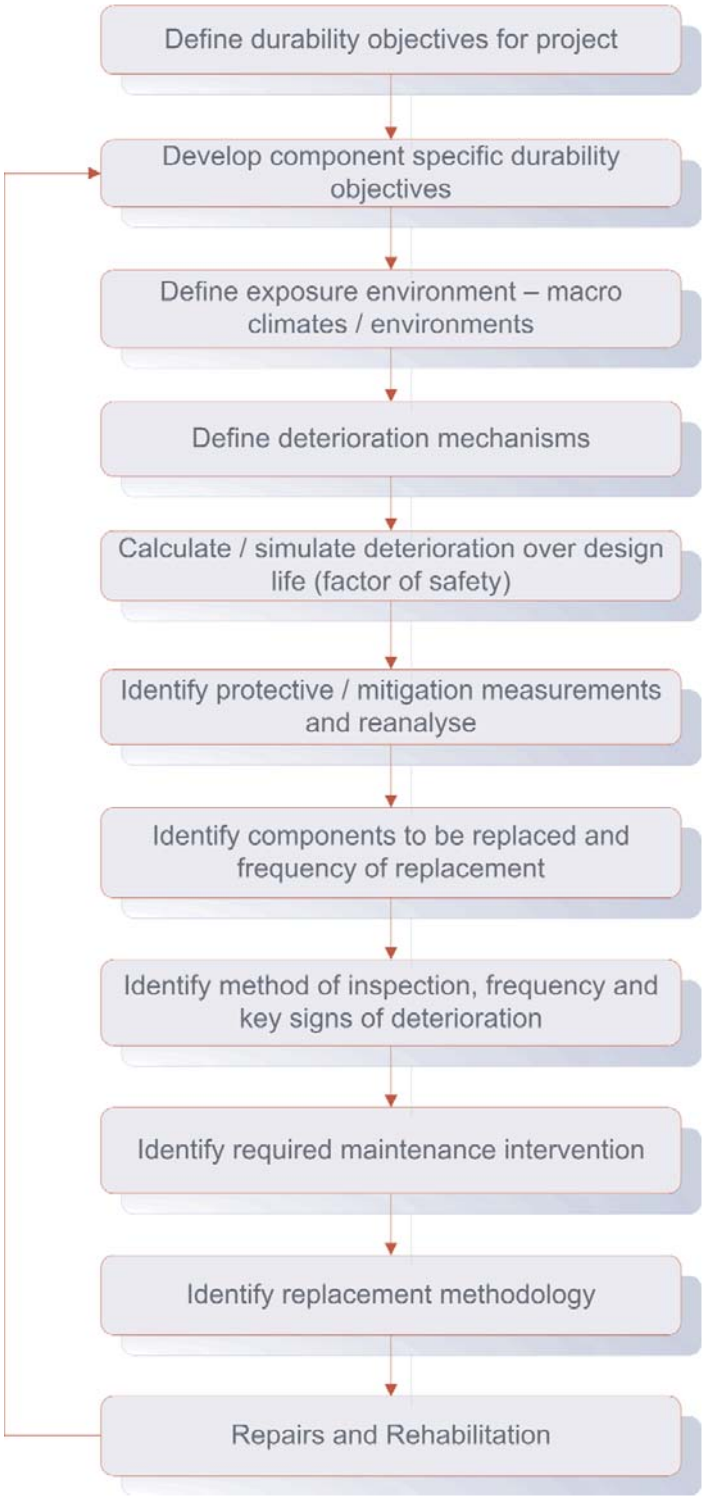

Figure 3: Durability Design Process Flow Chart

Figure 4 shows the production of durability planning documents at each stage of the asset creation process. The durability related documents are shown in bold.

The durability plan process outlined in this guideline document will collate information in a systematic and structured manner to ensure consistency in approach between projects.

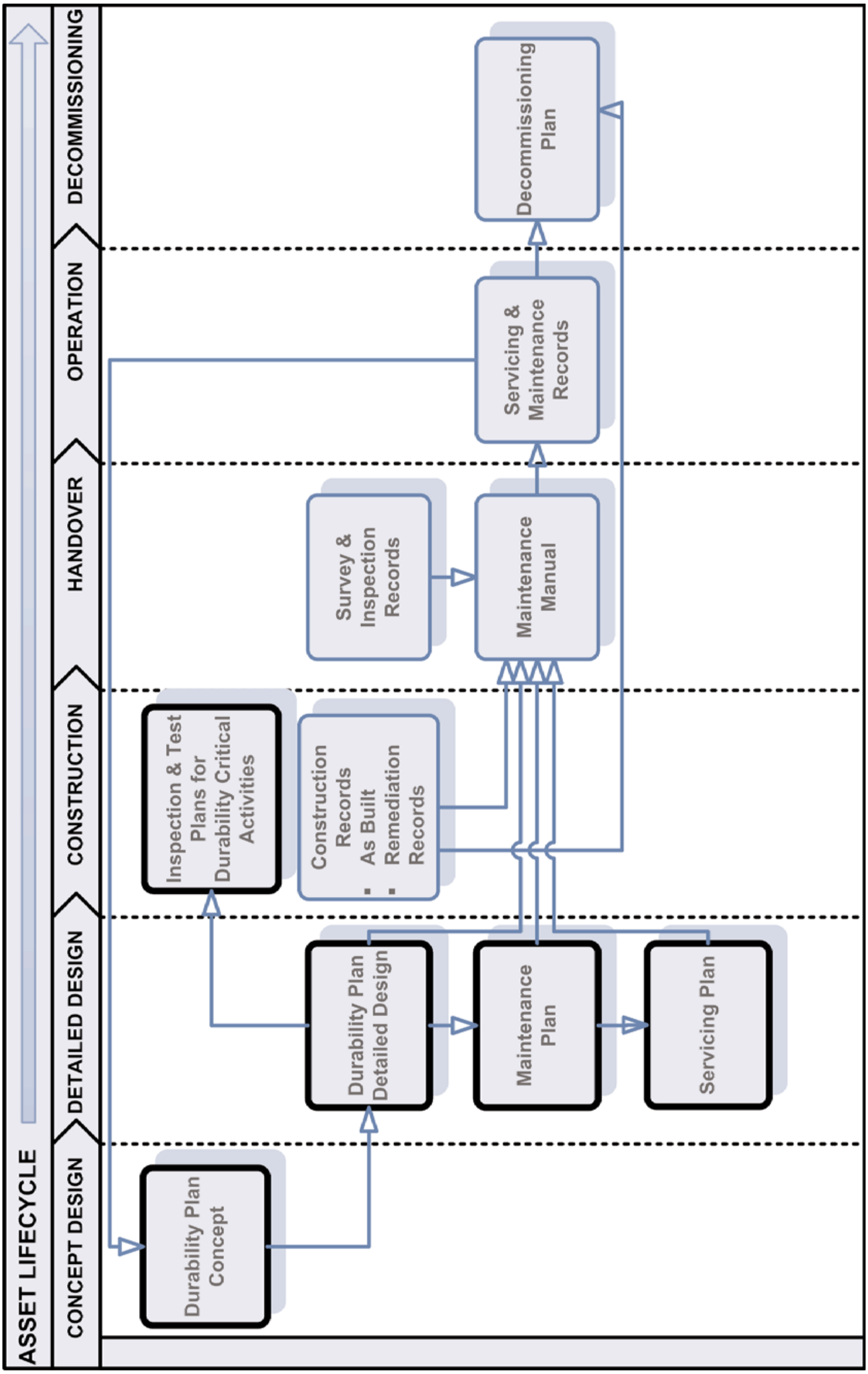

Figure 4: Production of Durability Plan Documents and flow of information through the asset lifecycle

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2.3 Durability Plan Report Introduction

The Introduction section of the durability plan report should include a brief description of the project and the structures covered. The durability performance requirements as set out in the scope of work or project brief must be itemised so that it is clear to all stakeholders that the durability performance of the structure is a defined project outcome. These requirements would typically include broad description of the project design life, any warranties to be provided and maintenance requirements. The designer will have to interpret what the design life would entail for specific components of the structure.

2.4 Service Life Criteria (Table 1)

The expected life of the structure is typically specified in the project brief as a design life. This needs to be developed in the service life criteria section of the DPR into the expected service life performance of individual components. The components of a structure can have a service life less than the required design life, necessitating maintenance and replacement. Where practicable, design solutions must be sought that provide for long component life and low maintenance.

The durability design process requires that the structure be considered on a sub component basis, as degradation processes may differ for different parts of a structure. That is the structure must be divided into components and evaluated individually for degradation, in much the same way that elements of the structure are designed. For example the foundations of a bridge may be exposed to aggressive ground water below ground while the deck may be in a relatively dry and benign environment. Both components, while fabricated from concrete, may require different mixes, cover and fabrication methods to meet the overall design life and so must be considered separately.

2.4.1 Structure Components

Guidance for the sub-division of the structure into components can be obtained from the DTMR Bridge Inspection Manual. In selecting the degree of sub division of the structures into components consideration should be given to service life, construction materials, method of construction and exposure environment. Further sub-division may be required as the detailed design progresses.

Consideration must be given to the "replaceability" of the component. E.g. piles will need to last the life of the structure as it would not be economically feasible to replace them. Conversely hand rails are unlikely to reach the typical 100 years expected of a bridge, without resorting to costly corrosion protection, but can be readily maintained or replaced and an acceptable maintenance regime and replacement frequency should be considered. The requirements of the DTMR servicing manual for bridges and culverts must be included in this assessment, as this describes DTMR routine servicing activities. Servicing and maintenance requirements over and above routine activities must be identified. Specifications for these maintenance activities must be developed if required.

Components that are expected to have a service life less than the expected design life of the structure, and therefore will need to be maintained or replaced one or more times during the design life, must be identified. Protective measures should be evaluated to achieve the required design life with the level of maintenance typically undertaken by DTMR as detailed in DTMR Bridge servicing manual.

2.4.2 Durability Limit States

The designer must elaborate on the general project design life and performance criteria by defining service lives of individual components.

The definition of the serviceability limit state given in AS 5100 may need to be expanded upon to provide sufficient clarity to component serviceability limit state, and will need to take into account the potential deterioration mechanisms and the likely dominant mechanism.

AS 5100.1 Bridge Design Clause 6.3.3 which defines serviceability limit states in terms of:

a) Deformation of foundation to give limitation on use or is of public concern.

b) Permanent damage due to corrosion, cracking or fatigue, which significantly reduces the structural strength or useful service life of the structure.

c) Vibration leading to structural damage or justifiable public concern.

d) Flooding damage.

It is critical at this stage that the designer understands and agrees with the client:

- The durability requirements, i.e. service life and what constitutes end of service life
- The expected maintenance and component replacement covering the life of the structure.

Bullet point b) of clause 6.3.3 AS 5100, identifies permanent damage due to corrosion which significantly reduces structural capacity as a limit state. This definition is subjective. A more robust definition for design purposes would be that the structure must resist deterioration and expected wear throughout the service life without the need for undue and unplanned maintenance.

In considering the expected service life of a component the governing deterioration mechanism must be identified and the impact this will have on the component during its service life. The expected visual signs of deterioration will drive the criteria for defining the end of life of that component, and determine the serviceability limit state.

The level of acceptable performance required, or more precisely what level of performance is considered unacceptable, must also be defined in terms that can readily be measured. For example, no cracking due to reinforcement corrosion; no loss of section for structural steel elements; no cracking of elastomeric bearings; etc.

The Client requirements for design life of the structure, and the expected serviceability during that life must be identified for each major component of the structure. The designer must specifically define the end of life criteria i.e. the condition that indicates the end of the service life. In most cases this will align with the condition state 3 rating defined in the DTMR Bridge Inspection Manual. This in effect is the limit state condition for the subsequent durability design. These trigger levels, when identified on site will instigate maintenance activities as detailed in the DPR.

Tabulation of the durability requirements on an element by element basis recognises that the structure has different components that are exposed to different environments and hence will have different design lives. In addition it recognises that some elements will require planned replacement to achieve the overall life of the structure.

Photo 2: Scour of abutment

2.5 Exposure Conditions (Table 2)

2.5.1 Classification of Environment Conditions in Design Standards

The classification of exposure conditions and the associated durability provisions in the design standards must be thoroughly evaluated for applicability as part of this process. AS 5100 provides a global classification for the site location relative to the coast line and recommends combinations of concrete strength and cover. Similarly, concrete in contact with aggressive elements in the ground are treated the same way, with no evaluation of the concentrations and hence severity of the exposure environment. This broad bush evaluation omits opportunities for efficient design.

The approach to durability design in design codes is suitable for many instances, particularly for simple structures or benign environments, but has a number of shortfalls. No provision is made for the influence of total cement content, water cement ratio or supplementary cementitious materials such as fly ash, silica fume, metakaolin and blast furnace slag to name a few. The incorporation of these and other technologies can lead to more efficient designs. The global classification of exposure environment, based on proximity to the coast does not allow for the multiple degradation process that may affect a structure, or may lead to over specification of protection of components. In either case it will result in a costly and inefficient design.

The classification of exposure conditions recognises that environmental loads are site specific and different components of a structure will also have different performance levels depending on the materials used in their fabrication. That is, different components of a structure will be subject to different macro environments.

The environmental exposure classifications for each component must be defined in terms of identifying the presence of aggressive agents and their concentrations, such that severity of exposure can be assessed. E.g. the severity of exposure to a tidal environment can vary depending on chloride concentration, ambient temperatures, presence of microbial bacteria etc. As a consequence stainless steel in a saline environment where the temperature is typically 20°C, will resist corrosion. The same grade of stainless steel in a saline environment where the temperature is 35C or more will corrode. Similarly elements in contact with groundwater will have different environmental severity depending on the aggressiveness of the ground water.

> It is envisaged that the initial classification of potential deterioration mechanisms at concept stage will drive the site investigation to confirm the presence and concentrations of potentially deleterious agents, enabling the durability design to be based on site specific data.

The components exposed to the macro environments should be identified. If the environment is aggressive a description of the potential deterioration mechanisms and how the component will degrade must be given. Factors affecting the rate of degradation should also be considered.

The environmental exposure classifications should be summarised in Table 2.

Photo 3: Steel Roller bearings and poor drainage provision on headstock

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2.5.2 Macro Environments

Exposure to aggressive agents, such as chlorides, sulphates, acid sulphate soils and the presence of water will act to degrade a structure. A variety of factors will influence how fast these aggressive agents will degrade a structure, including concentrations, length of exposure period, material properties, temperature and quality of construction to name a few. Components on a structure, depending on location, will be exposed to a number of different macro environments. Consequently the exposure conditions need to be assessed at a component level. For example an abutment founded on a river bank may be exposed to chlorides below the water table, acid sulphate soils below ground but above water table, splash zone, spray zone etc. By way of example the following table identifies the classification of environmental loads on components. This list is not exhaustive and the designer is required to assess the site-specific environment and develop project specific details.

Environment	Description	Factors that influence degradation	Example
Below ground, permanently submerged.	Components below ground and constantly submerged	Aggressive compounds in ground water. Mobility of ground water.	Foundations, tunnel structures.
Below ground, above water table	Components above the permanent water table. May be subject to periodic exposure to water due to seasonal variations in water table.	As above but also wetting and drying effects. Acid Sulphate soils	Foundations, Tunnel structures
Permanently submerged below water	Component is located below the lowest expect water mark.	Aggressivity of water. Scour and mechanical damage to components	Piles, Foundations, tunnel structures
Intertidal zone	Area of the component between the MHHW and MLLW marks.	Salinity. Fresh water flushing. Flooding.	Piles, Piers, Abutments
Splash Zone	Area above MHHW that is subject to wetting due to splashing from boat wash or wave action.	Effect of cyclic wetting and drying on surface concentration of aggressive species. Salinity. Flooding.	Piles, Piers, Pile Caps, Abutments Deck
Spray Zone	Area above the splash zone, subject to deposition of spray.	Prevailing wind direction.	Abutments, Piers, deck, Signage
Atmospheric (exposed)	Areas exposed to atmospheric weathering. Direct rain or sunlight.	Site specific weather patterns including, rainfall and temperature. Proximity to coastline / Industrial site. Drainage details and water management on and around structure.	Abutments, Piers Deck Parapets Bearings Signage.
Atmospheric (Sheltered)	Areas exposed to the atmosphere but sheltered.	Leakage of water, which could increase aggressivity of environment.	Inner areas of deck soffit. Bearings.

Table 2: Example Environmental Classification

The exposure conditions comprise the environmental load case in the durability design process. As such the loading must be defined as clearly as possible, and assessed with a factor of safety to allow for variability as would be the case in structural design. The approach to durability design is not intended to lead to the specification of overly conservative solutions, but to make sure that components at risk of deterioration are identified and appropriate protective measures are incorporated. Conversely the durability planning process is intended to eliminate the specification of overly protective measures where they are not required.

Photo 4: Chloride induced reinforcement corrosion of columns

2.5.3 Define Deterioration Mechanisms

Combining the list of components and their design lives with the exposure assessment will highlight a series of potential deterioration mechanisms. All should be listed but typically one mechanism is likely to be governing load case and will dictate the durability design requirements.

A detailed assessment is unlikely to be required for all components, particularly in a benign environment. Conversely, for a critical component or a significant structure, diffusion modelling of chloride ingress, or carbonation depth calculations may be required to provide confidence that the durability measures adopted will achieve the design life.

A durability assessment would typically include but not be limited to:

- Chloride ingress;
- Depth of Carbonation;
- Corrosion of steel components;
- Assessment of concrete mix for resistance to sulphate attack;
- Evaluation of reactivity of aggregates for alkali silica reaction;
- Evaluation of acid sulphate soils risk; and
- Early age thermal modelling and crack risk assessment of critical elements.

There is a wealth of information available on the common degradation processes, and some publications are listed in the reference section of this guideline. The parameters influencing the rate of degradation from sulphate attack, alkali aggregate reaction, carbonation and chloride induced reinforcement corrosion and general steel corrosion are widely known. Specialist advice may be required for degradation due to less common processes, such as microbial induced corrosion.

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2.6 Assessment of Design Solutions (Table 3)

2.6.1 Identify Protective or Mitigation Measures

The assessment of the performance and in service degradation of structure components must consider solutions that achieve a low maintenance and long life solution. Where code provisions are assessed as being insufficient, additional protective measures must be considered.

The full range of solutions should be considered. Improvements to the resistance to attack of the concrete matrix, such as the inclusion of cement replacement materials, low water cement ratios, high range water reducing agents etc. Activities that are critical to the achievement of the required performance must be identified. For example the application of early age curing of concrete would be critical to achieving the diffusion characteristics and low permeability envisage by the design. This is particularly critical for mixes containing fly ash. Poor early age curing can result in a significant reduction in expected diffusion and permeability characteristics than would be expected if well cured. The required curing regime must be detailed.

The management of water is an important factor in determining the rate of deterioration of a structure. The longer the contact time of water with the structure the greater the risk of corrosion. Protective measures should therefore be considered to drain or shed water, or where appropriate seal surfaces with waterproofing. Damage to expansion joints can result in water penetrating the joint and pooling around bearings. Continued exposure to water can lead to corrosion of bearings. Consideration should be given to drainage on the bearing shelf to remove any water leakage. If the structure is in a saline environment, provision of a waterproof membrane should be considered to prevent reinforcement corrosion of the bearing shelf, and thereby mitigate the need for concrete repair works.

For critical elements, where future inspection, repair or replacement may not be possible consideration should be given to additional protective measures and or measures that will facilitate the later implementation of protective measures. E.g. ensuring steel reinforcement continuity during construction to facilitate the installation of cathodic protection in future. Primary and secondary protective measures should be considered for components that cannot be readily accessed for inspection, maintenance or repair.

> Protective measures that are not covered in current design standards and could be considered as part of a corrosion protection strategy include:
>
> - Corrosion Inhibitors
> - Controlled permeability formwork
> - Cathodic prevention
> - Cement replacements such as fly ash, super fine fly ash, blast furnace slag, metakaolin, Micro Silica.
> - Stainless steel reinforcement
> - Fibre reinforcement
> - Surface sealers
> - Enhanced servicing and maintenance regime.
>
> This list is not intended to be exhausted, but indicates a range of measures available to the designer that are not prescribed in design codes..

Where a specific maintenance regime is required to achieve the service life, and this regime differs from that typically adopted by DTMR as described in the Bridge Servicing Manual, this must be described in the durability Plan report and summarised in Table 4.

2.6.2 Durability Design in Standards

The provisions for protective measured given in Australian design Standards should be thoroughly evaluated for the expected exposure environment. The designer should document this evaluation process and determine whether the suggested combination of concrete grade and cover is sufficient to achieve the requisite design life. A "slavish" application of the code requirements may not necessarily achieve the required durability objective.

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2.6.3 Durability Modelling

In most cases it is not envisaged that detailed durability modelling using chloride diffusion modelling, as described for example in "Enhancing Reinforced Concrete Durability", (Technical Report 61, Concrete Society 2004) will be required. The level of detail required will depend on a number of factors including the complexity of the structure, the classification of road carried, and its location. Detailed modelling would be required for landmark, very long life structures or structures in extremely aggressive environments.

The durability design process requires that an assessment of deterioration rate of a component be made, and if the degradation results in a service life less than the required design life then protective measures must be adopted. Where a sufficiently long service life cannot be achieved, even through the implementation of protective measures, then the design must allow for replacement of the component.

2.6.4 Component Inspection & Replacement

The designer must consider all the necessary activities in the replacement of a component, and incorporate features into the design to facilitate, safe component replacement. Replacement activities should minimise the disruption to the operation and use of the structure. In assessing suitability of design solutions, the designer must take into account the nature and location of the road, and the implications on congestion of lane closures.

It may be prudent to include primary and secondary protective measures to ensure the durability of a component. This would be the case for components that cannot readily be accessed for inspection and or replacement, or where such activities would result in significant disruption to road users and congestion.

Similarly where access to components for routine condition inspection is restricted or unavailable the inclusion of embedded monitoring probes, such as corrosion ladders or half cells must be considered.

2.6.5 Detailing

Good detailing benefits the structure in terms of appearance and durability. Through good detailing of drainage and water shedding unsightly staining and degradation can be reduced or eliminated at minimal extra cost. The avoidance of crevices or discontinuities can avoid the ponding of water and build up of detritus that can impair structure movement or act as sites for corrosion. Good detailing can also facilitate safe and thorough inspection and maintenance.

Where DTMR have issued design standards, the designer must evaluate the suitability of the detail for the particular project conditions, and summarise this evaluation in Table 2.

In developing construction details consideration should be given to the construction process, operation, inspection, maintenance and demolition. While not an exhaustive list the following issues should be considered:

- Buildability;
- Drainage and water shedding;
- Access to the bridge for inspection and maintenance;
- Isolation of dissimilar metals to avoid bi-metallic corrosion;
- Reinforcement detailing.

Photo 5: Corrosion resulting in necking of holding down bolt

i. Water Shedding and Drainage

The presence of water is critical in many deterioration processes. Without water the degradation reaction can be stopped or its rate severely reduced. Therefore the design should include detailing to ensure that water is channelled and discharged away from the structure so that it does not lead to pooling, scour or undermining of components. A structure that is detailed well to prevent pooling of water will typically dry rapidly after wetting preventing the formation of corrosion cells. Poor water management can promote other associated degradation mechanisms, such as accumulation of debris, that may seize joints and lead to restraint induced cracking. The prolonged accumulation of debris can result in plant growth which can cause structural damage to components.

Consideration should be given to the water shedding and drainage. Guidance on details that have worked well is given in Application Guide 33, "Water Management for Durable Bridges", published by TRL. In most cases the measures likely to be implemented do not require design changes or involve cost increases, but do require a global assessment of water collation, flow and discharge around and on a structure.

The detailing of structural steel elements must avoid details where water and debris can collect. In addition the design details must facilitate the application of protective coatings. For instance, components to be protected by hot dipped galvanising must prevent the accumulation of large zinc deposits at gusset plates that can in future break off.

Photo 6: Poor drainage detailing leading to corrosion of outer girder and staining on pier.

ii. Access for Inspection and Maintenance

The designer must ensure that the completed structure allows safe access to the structure for inspection and maintenance activities. Typically this would entail providing for access to the face of abutments, and providing a flat level working platform in front of the abutment for inspection, cleaning and bearing replacement. Sufficient clearance must be provided between the deck soffit and abutment bearing shelf to allow inspection, cleaning and replacement of the bearings.

On critical structures provision of a hard shoulder wide enough to provide working space for inspection and maintenance activities without the need to close a traffic lane.

Where components cannot readily be accessed for visual inspection then alternative inspection provisions must be made. The external reinforcement on a tunnel is at risk of corrosion. The external face cannot readily be inspected, therefore corrosion monitoring probes should be embedded in the external face and monitored as part of the structure specific inspection and maintenance plan.

iii. Reinforcement Detailing

In detailing the reinforcement the designer should consider the diameter and spacing of bars, in terms of constructability. How will the bars be fixed, and once fixed is the reinforcement cage so congested that placing concrete may cause segregation. Can the detailing be amended, for example through the use of couplers to eliminate lapped bars. Similarly modifications to construction method such as modifications to the concrete mix to facilitate placement, through the use of self compacting concrete or use of smaller maximum aggregate size must be considered.

iv. Shrinkage

The designer must consider the implications of long term drying shrinkage and the potential for cracking of components and the impact on durability. Calculations of expected shrinkage must use the material properties of local materials employed on site and not generic shrinkage factors given in codes, which may not be appropriate.

Assumptions on the expected rate of shrinkage of precast components must be clearly stated in the DPR and the implications for the timing of construction detailed. The design of joints and bearing details must allow for expected shrinkage of the girders. The premature installation of girders can result in greater shrinkage movements than envisaged in the design resulting in the build up of stresses in the girder and or the movement of bearings.

v. Bi-Metallic Corrosion

The electrical contact of dissimilar metals that can result in corrosion must be avoided. Where dissimilar metals are in contact, the surfaces must be insulated to prevent corrosion. This is particularly relevant on balustrades, lamp posts and sign posts.

2.6.6 DTMR Standards and Standard Details

The durability provisions in design codes and DTMR standards must be assessed as to whether they will achieve the project specific durability requirements for the site specific exposure regime. If the measures in the code are deemed adequate these measures can be adopted without modification. However, if they are insufficient then additional measures need to be considered. This evaluation process should be recorded in Table 2, and confirms that the designer has considered the applicability of the code requirements for the particular conditions on the project and verified that they are applicable or otherwise.

Where standard provisions or details are considered inadequate, additional or alternative measures should be proposed as part of the engineering design process for review by DTMR. In justifying the departure from standards a review of the inadequacies must be documented and a commentary provided on the alternative detail provided.

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Photo 7: Inadequate fixity around elastomeric bearing

2.6.7 Construction Issues

The effectiveness of protective measures is greatly influenced by construction quality, which in turn is influenced by method of construction. Therefore a key part of the durability planning process is identifying the key factors that dictate durable performance and ensuring that appropriate attention is paid during construction to compliance with specification requirements. Any relaxation or amendment to the specification on site must first be reviewed and approved by the designer to assess the implications on durability.

Components must therefore be checked, following construction, to verify compliance with the durability objective. For example if a steel component is reliant for corrosion protection on a coating, the applied coating should be verified to confirm that it was applied with the required dry film thickness, without holidays and with the required adhesion. As part of the durability planning process the designer must specify what measures will be used to verify compliance with the design intent and incorporate these into the project drawings or specifications as appropriate. For example the designer must specify the nominal cover to be provided to a component, with an allowance for construction tolerance where:

Nominal Cover = Minimum Cover + Construction Tolerance

The compliance criteria would be attainment of the minimum cover, verified by a cover meter survey or similar.

Where, owing to the nature of the construction process, good control of the finished product cannot be ensured and verified and therefore the effectiveness of the primary protective measure cannot be confirmed the durability design must take this into account and if necessary secondary protective measures implemented. Where on site product verification identifies a deficiency, the extent of any departure from the specified requirements would need to be assessed by the designer for its impact on the overall durability. Necessary supplementary durability measures or repairs must be implemented to restore the original durability intent.

The fabrication and installation processes should be considered at the design phase. For instance the effectiveness of placing concrete in bored piles will be influenced by the consistency of the concrete mix, which in turn will be dictated by the mix design. A homogenous, well compacted concrete will provide better durability performance than one that is segregated. Therefore the specification of the concrete mix must also ensure that consistence and cohesion and other factors influencing buildability must be adequately addressed in the specifications. These factors must be described in the durability plan report.

The critical requirements of the durability design solution, such as concrete grade, cover to reinforcement must be summarised in Table 3.

2.6.8 Control of Early Age Thermal Cracking in Concrete

The control of early age thermal cracking in elements influences durability as in many instances durability is predicated on the protection provided by the cover concrete. Cracking can provide a path for deleterious agents to by-pass protection offered by the cover and penetrate the body of the concrete. The provision of reinforcement to control cracking is based on assumptions on potential temperature gradients and the associated stresses, made at design stage. These assumptions can be different to actual conditions on site. Consideration should be given to the combination of reinforcement, concrete mix properties, ambient temperatures, construction method, formwork etc and the impact on the likelihood and extent of cracking.

Thermal control of pours to eliminate early age thermal cracking should be considered, particularly for aggressive environments, where cracking of the cover concrete will reduce the durability performance of a component. There are a number of methods for modelling expected thermal stresses, which are far more rigorous than the approach in AS 5100. CIRIA C660, Early Age Thermal Crack Control in Concrete, describes one approach. For large or critical pours, consideration should be given to trial mixes and collation of adiabatic temperature rise data for the preferred concrete mixes. This will enable site specific data to be used in the assessment of the risk of thermal cracking.

Photo 8: Concrete placement in a pier

2.6.9 Safety Review of Inspection & Replacement Activities

During the development of the design a safety review should be undertaken that covers the construction process and the operational phase. Inspection, maintenance and component replacement activities undertaken on the structure mean that the site is subject to workplace health and safety legislation. As such the design review should address features of the design that will reduce the hazards to inspection and maintenance personnel. This might include provision of safe access from deck level to the abutment to facilitate activities associated with bearing inspection, cleaning and replacement.

It is envisaged that outline method statements for inspection and maintenance activities will be prepared as part of the maintenance manual for the structure. The method statement must describe the equipment to be used, and how the activities will be undertaken. Traffic management requirements and access equipment must be detailed. Where hazards are identified that are not eliminated by the design these must be clearly identified so that the owner is aware of the residual safety risk.

2

2.7 Maintenance & Servicing Assumptions (Table 4)

To achieve the required service life and appropriate performance level during the service life the structure must be appropriately maintained.

The maintenance requirements must be identified in terms of a description of the activities to be undertaken and the frequency of those activities. This will enable a review of the appropriateness of the maintenance activities and frequency of those activities. The frequency of any maintenance activity will to a large degree be an assumption, and so the actual performance of a component in service may be better or worse. Therefore a description of the intervention level that would necessitate maintenance should be provided.

> Steel girders forming the support structure to a bridge deck are protected from corrosion by a protective coating. The life of this coating would be expected to last 20 years before reapplication, assuming cleaning and patch re-painting as part of the servicing regime. In practice the coating might perform better or worse than expected. Therefore the trigger level for any intervention must be identified, such that if the deterioration described was noted during routine inspection then recoating could be applied earlier than predicted, of conversely could be delayed. The trigger level for intervention would typically be localised blistering or peeling of paint or slight rust streaking.

The design must be cognisant of the maintenance activities and make appropriate provision for access to enable the works to be carried out safely. Furthermore the access to the structure component must be detailed in the durability plan and summarised in Table 4. Any traffic management required must also be identified.

2.8 Routine Inspection (Table 5)

2.8.1 Access

DTMR undertake regular inspections of their bridge structures. Details of the scope and frequency of level 1, 2 and 3 inspections are provided in DTMR Bridge Inspection Manual. Inspection requirements identified in the design process that are over and above the requirements given in the DTMR bridge inspection manual must be detailed in the Durability Plan.

During the life of the structure it will be inspected in accordance with the DTMR inspection manual. The requirements for access for the various levels of inspection shall be identified and allowed for in the design. Consideration must include, but not be limited to:

- Applicable workplace health and safety requirements;
- Location of access equipment; and
- Traffic control measures.

The inspection regime must consider the nature of the road and the impact of road closures. If inspection of the structures is likely to be limited to night work, then consideration must given to how this will impact inspection activities and the safety of maintenance personnel.

Routine inspections require visual inspection and in some cases non-destructive testing and sampling. As such access to within touching distance of all components of the structure must be provided. While it may not be practicable to provide permanent access facilities the DPR must identify how inspections will be performed and the associated hazards. Where safety hazards are not mitigated by the design the hazards must be described. A summary of the inspection requirements, access provision and safety hazards must be summarised in Table 5 in appendix E. It is recommended that a hazard analysis is undertaken during the design process, and the outcomes recorded in Table 5 (Appendix E).

Where elements cannot be inspected directly alternative means of confirming condition must be provided. Where a concrete element is concealed or inaccessible, then provision for assessment of condition may be provided by the inclusion of permanently embedded half cells (or corrosion ladders to measure the corrosion potential of reinforcement) connected to accessible test points. Where architectural cladding is provided, the cladding must be detailed to allow access or removal.

2

2.8.2 Condition State Guidelines

The timing of maintenance or replacement activities will be dependent on in situ performance. Key signs that trigger the need for maintenance or replacement shall be identified. E.g. appearance of rust spots triggers the need for spot recoating. This provides the inspection team with a briefing on the expected signs of deterioration, which components are likely to be affected and where to look for deterioration.

2.9 Replacement of Components (Table 6)

Components that need to be replaced during the service life of the structure must be identified in Table 6. For each component an expected service life and hence frequency of replacement shall be identified.

The servicing, maintenance and replacement intervention levels must be clearly identified, and summarised in Table 6. Designers should endeavour to avoid short-life components requiring intensive maintenance or frequent replacement. Detailing should be such as to promote long service life, with low maintenance, for example through providing sloped surfaces for self draining and avoidance of dirt and detritus accumulation, and ease of replacement.

Included in the durability report the designer must provide a method statement for the replacement of the component and the provisions in the design.

A hazard assessment must be carried out for the replacement activity and as far as is reasonably practical the design should be amended to mitigate or eliminate the hazard. Where a residual hazard remains this should be identified clearly so that it can be managed by the relevant organisation undertaking the replacement works.

2.9.1 Bearings

A method statement and drawings for replacement of bearings must be provided. As a minimum the following must be taken into account and documented as part of the durability plan:

- The location of jacking points and size of jack must be detailed on a drawing;
- Temporary loads on the headstock and member being jacked when a bearing is removed for replacement;
- Provision of sufficient clearance for jacks; and
- Health and safety requirements.

Consideration should also be given to the practicality of restrictions to lane loading during replacement activities. Lane closures on high traffic structures or structures in urban areas may result in significant traffic congestion and disruption and therefore should be minimised. Conversely lane or weight restrictions may be acceptable for structures located in rural areas with low traffic volumes.

Photo 9: Detailing of bearing shelf, showing bearing plinths and inspection platform

2

2.10 Construction Phase Durability Plan Compliance (Table 8)

2.10.1 General

The overall durability of a structure is influenced by the ability to build it to the required quality and ease of conducting maintenance activities.

The in service performance of an element will be greatly influenced by the quality of fabrication, which in turn will depend on the ease with which the element can be constructed, the materials used and workmanship.

Design and construction planning must take into account the problems associated with construction and the consequences for durability. It is important during construction to ensure that particular attention is paid to durability critical components and inspection and verification procedures focused to ensure the design intent is achieved on site.

The durability plan is intended to provide a documented record of the assumptions on the construction methodology for durability critical components made at design stage. Where the method of construction changes from that envisaged in the design there may or may not be implications for durability. In either case the impact on durability of the construction method should be evaluated by the designer, and if necessary appropriate steps taken to ensure durability objective is maintained.

For example, the use of a concrete mix with a high fly ash content may be used to enhance the durability to chloride ingress. In order to achieve the expected performance concrete mixes containing fly ash must be water cured, and are particularly sensitive to the application of early age curing. Poor curing can result in the concrete mix having a lower than expected performance. The implications on durability and service life must be evaluated by the designer and mitigation measures implemented to ensure that the intended performance is achieved.

2.10.2 Durability Critical Processes (Table 8)

Critical construction processes identified by the designer must be reviewed by the contractor and appropriate methods of work adopted. As part of this review the contractor must put in place appropriate quality control process to ensure that the assumptions made in design are achieved. This may require amendments to standard construction practices and standard specification and drawing details. These issues must be highlighted to DTMR prior to appropriate amendments being made to specifications. Similarly validation of the components, to confirm that the durability design intent has been met, must also be identified. These verification processes, which may include non-destructive testing, must be clear and unambiguous methods of verifying that the component has achieved the expected performance criteria. These validation measures are intended as a mechanism to communicate clearly to the site supervision team the importance of certain parameters in achieving durability.

This information must be recorded in the durability plan and included in specifications or drawings.

2.10.3 Verification Procedures and Tests

Quality Assurance procedures are typically focused on surveillance of the methods and processes in the production of an element, whereas for durability critical components end product compliance is the objective. Accordingly for durability critical activities this process surveillance must be supplemented with a detailed inspection and test plan (ITP) and product compliance testing.

It is essential to undertake product verification checks to ensure that what has been constructed complies with the design intent. Testing must as a minimum verify that critical durability parameters have been met. This might include a cover survey of a representative area of a component to confirm minimum cover requirements have been achieved. Coating verification would include, wet and dry film thickness measurements, adhesion testing and holiday testing.

Any construction defects such as honey combing, cracking in excess of specified allowable crack widths must be identified and rectified. It is essential that a durability assessment be conducted, by the designer, in the determination of the appropriate repair or remediation measure. The location and details of the repair must be recorded on as built drawings for compilation in the maintenance manual.

The inspection and product verification must be undertaken as soon as practical after construction.

Product Verification

The implementation of site quality assurance systems may not adequately address the requirements of the durability design. The designer must therefore identify the key elements of the durability design measures and describe the verification measures to confirm that these design measures have been achieved on site. These verification measures must be incorporated into the contractor's inspection and test plans for the respective component.

Eg. The durability design measure for a pile cap is the provision of a concrete with a maximum w/c ratio, minimum cement content and cover to reinforcement. The effectiveness of this protective measure is reliant on the concrete being placed and compacted to achieve an homogenous mix, free of voids, defect and cracks. The placement and compaction of concrete will be influenced by: the concrete mix design; the placement method; the compaction method; the consistence of the concrete; thermal control of the pour; seasonal climatic conditions and curing. In addition the rigidity of the cage and accuracy of fixing, and any movement during concrete pouring will influence the final cover achieved.

The provision of the correct mix could be verified by inspection of mix dockets, and cylinder strength results. Significant variation in compressive strength either up or down would indicate variation from the approved mix.

To confirm that adequate cover had been provided, a cover meter survey of the component would be undertaken. The pile cap must also be inspected to ensure there are no construction defects such as honeycombing or cracking. If the curing is suspected of being poorly done, then water absorption testing such as initial surface absorption could be instigated.

2.10.4 Defects and Treatment Records

The nature of the construction activities means that defects or deviations from the planned and design components will occur. Where this has an implication on durability the remediation work must address any loss in durability.

Non-Conformance records (NCR) must record the rectification treatment and a comment from the designer on the overall effect on durability.

Defects that occur during construction must be recorded and passed to the asset operator as part of the record of construction.

Identifying when a defect occurred and how well the repair performs during the service life is important for maintenance planning.

Where a non-conformance or construction departure from the durability requirements is identified, the designer must evaluate the implications on performance and whether any further protective measures are required to restore the intended durability.

Where there is a reduction in durability, additional protective measures or repairs are required to be undertaken and must be designed or specified by the designer to restore the intended durability. This process must be recorded, and include the location of any repairs, the materials used and verification records that the repair was carried out to the required standard.





Part Three

Durability Plan Verification Report

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3 Durability Plan Verification Report

3.1 Submission Requirements

The provision of relevant information to the owner and operator of the structure is an important part of the asset management process. This information can be referred to as the birth certificate of the structure and documents design issues, construction and condition of the structure at the time of hand over.

The items described below are to be included with all other as-built documentation, and only relate to durability design information to be handed over.

On completion of construction, the designer in conjunction with the contractor must compile the durability plan verification report as part of the as built records and hand over documentation.

The durability plan verification report would include an update of the durability plan report incorporating any changes necessitated during construction records of materials used, construction departures and mitigation measures adopted. The updated durability plan summary tables must also be updated to reflect the as built structure. If the preparation of a maintenance manual for the structure is a project requirement the summary tables must be included as appendices in the manual. The tables will serve as a concise summary of the durability intent, how this was achieved and verified, and the required maintenance and inspection activities. These documents will be stored in the DTMR Bridge Information System.

3.2 As Built Records

3.2.1 Amendments to the design

Where site conditions require an amendment to the design, the original designer must be consulted. Any changes must be recorded on as built records and implications on durability recorded in the durability plan. Where necessary, additional protective measures must be adopted to reinstate the intended durability objective.

3.2.2 As Built Drawings

The requirements listed below supplement those specified in the contract documentation.

A full set of as built drawings must be included in the maintenance manual. The information must include but not limited to the following:

- Location of all services;
- Drainage system details;
- Reduced Level of pile toe; and
- Deviation from tolerances of piles, headstocks and bearing pedestals.

The drawings must be provided in both sets of the following formats:

- AutoCAD; and
- Scanned copy in JPG or TIF format.

3.2.3 Construction Records

The requirements listed below supplement those specified in the contract documentation. Construction records to be provided in the maintenance manual include:

- Summary of concrete test results;
- Post tensioning records;
- Weld tests;
- Shop drawings; and
- Precast records.

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3.2.4 Durability Verification Report (Table 8)

The durability verification report must be updated to reflect changes to the design and construction departures as outlined in section 3.1, 3.2 and 3.3.

Table 8 must be updated to include construction stage departures such that details of the structure actually constructed must be recorded.

3.2.5 Level 2 Inspection Report

As part of the handover process for the asset a Level 2 inspection, to DTMR standards must be undertaken by an accredited Main Roads Level 2 Inspector. The completed report must be included as part of the asset documentation.

3.2.6 Photographic Records

A full photographic survey of the structure must be undertaken. The photographs should record general views of the main structural components, with close up photographs of repaired areas. The photographic record provides a snap shot in time of the original condition of the structure. Future condition inspections can then be compared against the original records to assess the progression of deterioration.

3.3 Materials and Proprietary Product Details (Table 7)

Details of proprietary products or components must be provided. This not only provides a record of construction, but should components need to be investigated or replaced the information pertaining to the replacement component can readily be obtained. Data sheets pertaining to all proprietary products should be provided as described in 3.3.2.

3.3.1 Precast concrete components

The details of precast concrete components, including but not limited to bridge beams, crash barriers, parapets must be provided. These details must include:

- Records of concrete mixes;
- Casting and production records;
- Test certificates for aggregates and cementitious materials;
- Compressive strength results;
- Details of any repairs undertaken;
- Mill certificates for reinforcement of Pre-stressing tendons; and
- Contact details of manufacturer.

These requirements supplement those specified in the contract.

3.3.2 Data sheets

The data sheets of all materials used must be provided, including:

- Coatings;
- Concrete repair mortar;
- Sealants;
- Movement joints;
- Bearings; and
- Parapet barriers, handrails and balustrades.

In addition for bearings and movement joints, manufacturer's shop drawings must be provided.

The name, address and contact details of the supplier must be provided.

3.3.3 Warranties

Details of any warranties must be provided, along with an original copy of the warranty. The duration of the warranty and any servicing required to maintain the validity of warranty must be detailed in the servicing requirements.

Part Four

Durability Design Summary Tables

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4 Durability Design Summary Tables

4.1 General

The following sections describe the process for completion of the durability design summary tables. These tables are intended to summarise, in a succinct manner, the key outcomes and durability design described in detail in the Durability plan report narrative. A commentary is given on the information that is required to be completed in the durability plan summary tables and a process flow chart provided in Appendix A. Examples of the completed tables are presented in Appendix B for reference, and blank tables are provided in Appendix E.

Soft copies of the summary tables are to be provided in MS Excel format to facilitate uploading of pertinent durability and maintenance information into DTMR's bridge management system (BMS). Soft copies of the tables are provided on the disk provided in Appendix F. Consequently additional rows can be added to the spread sheets but the headings are locked. This has been developed to assist in the compilation of durability plans through table linkage. Additionally the standard formatting will facilitate the uniform collation of information for uploading into the Bridge Information System.

4.2 Structure Durability Outline: Table 1

The expected durability and service life performance of the structure components is summarised in this table, as derived in accordance with section 2.4.

The table must be completed as part of the concept design, and the contents agreed with DTMR prior to progressing with detailed design.

4.3 Component Exposure Assessment: Table 2

For each component the environmental load case(s) that apply, as derived in accordance with section 2.5 must be summarised in Table 2. In many instances a component will be exposed to several different exposure mechanisms and these must all be identified.

4.4 Durability Provisions for Critical Components: Table 3

This table summarises the durability design mitigation measures derived in accordance with section 2.6.

The contents of the first 3 columns of this table, which summarise the exposure and degradation mechanisms on a component basis, are populated as part of Table 2.

4.5 Maintenance Intervention Assumptions: Table 4

This table summarises the expected maintenance activities at a component level, derived in accordance with section 2.7.

The contents of the first 3 columns of this table, which identify the component and exposure conditions, are transferred from Table 2.

This table must be completed as part of the detailed design process.

4.6 Inspection & Access Provisions: Table 5

The contents of this table summarise the access provisions to required to undertaken the maintenance activities identified in Table 4, and the access requirements for inspection. These entries are derived in accordance with sections 2.7 and 2.8 respectively.

4.7 Replacement of Components: Table 6

This table summarises the provision for the replacement of components derived in accordance with section 2.9.

The first 6 columns of this table collate information from Tables 1 to 5.

Information on the components and exposure conditions are transferred from Table 2.

The replacement frequency is transferred from Table 1.

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The condition state guideline, that describes the trigger point for intervention is transferred from Table 5. The design mitigation measures are transferred from Table 3.

4.8 Proprietary Products Record: Table 7

Table 7 summarises the requirements for maintaining proprietary product records in accordance with section 3.3.

Where proprietary products are specified in the design, the details must be listed. The first 4 columns of this table must be completed as part of the detailed design. At construction stage the proprietary product may be changed, and this change must be approved by the designer to ensure that functionality or durability performance is not compromised.

Following construction the warranty details and installation details must be completed. The updated table will be included in the Construction Verification Report.

4.9 Construction Phase Departures: Table 8

Table 8 summarises the validation of the original durability requirements or approved variations in accordance with section 2.10. The first 6 columns of this table are populated with data from Tables 2 and 3. The intervention trigger levels must be completed. The remaining columns in the table must be completed during the construction phase.

4.10 Summary of Submission Requirements

4.10.1 Concept Design

Before commencing detailed design the durability performance criteria and environmental load cases must be agreed with DTMR. A concept durability plan report comprising a narrative of the durability design philosophy, expected degradation mechanisms must be submitted as part of the concept design report. The following summary tables must also be completed.

- Table 1 Structure Durability Outline
- Table 2 Component Exposure Assessment

4.10.2 Detailed Design

The detailed design durability plan report will comprise the narrative with the following tables completed.

- Table 1 Structure Durability Outline (Updated from concept design)
- Table 2 Component Exposure Assessment Updated from concept design
- Table 3 Durability Provisions for Critical Components
- Table 4 Maintenance Intervention Assumptions
- Table 5 Inspection and Access Provisions
- Table 6 Replacement Components
- Table 7 Proprietary Products Records
- Table 8 Durability Provisions for Critical Components

4.10.3 During Construction

The results of product verification inspection and testing must be recorded in Table 8 during construction. Where construction departures from the intended design occur the designer must be made aware of the departure and evaluate and record the impact on durability performance. Where durability performance has been compromised remedial measures must be implemented to restore the component durability objective.

4.10.4 Construction Verification Report

On completion of construction the designer must compile a summary of the construction departures and the subsequent repairs undertaken to restore durability. The defects, repairs and verification records must be summarised in Table 8.

Marked up drawings showing the location of defects and repairs described in Table 8 must be provided.

Photographic record of all components at hand over must be provided.

- Table 7 Proprietary Products Records (Updated)
- Table 8 Durability Provisions for Critical Components (Updated)

Part Five

Reference Documents

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5 Reference Documents

Application of the durability provisions given in design codes such as AS 3600 and AS 5100 may not adequately meet the performance requirements for long life low maintenance road structures. As such guidance on appropriate durability measures should be sought from other sources.

The following list of reference documents are provided as sources of guidance. The list is not exhaustive, nor is all the guidance provided in these documents relevant to the range of exposure environments found in Australia.

- Concrete in Aggressive Ground, BRE Special Digest 1, 2005
- Early Age Thermal Crack Control in Concrete, Report C660, CIRIA, 2007
- Enhancing Reinforced Concrete Durability, Technical Report 61, Concrete Society 2004.
- Durable Concrete Structures, Recommended Practice, Concrete Institute of Australia, 2001.
- Austroads Standards
- Queensland Department of Transport and Main Roads, Bridge / Culvert Servicing Manual
- Queensland Department of Transport and Main Roads, Bridge Inspection Manual
- Bridge Detailing Guide, Report C543, CIRIA, 2001.
- Bridges - Design for Improved Buildability, Report 155, CIRIA, 1996
- Water management for durable bridges, TRL Application Guide 33, 1998.

Appendices

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Appendix A: Process Flow Charts

Flowchart - Serviceability limit state design	Section Reference in Guideline Document	Reference Table
Define durability objectives for project Develop component specific durability objectives Define exposure environment – macro climates / environments	2.2 Structure Durability Outline 2.4.1 Structure Components 2.4.2 Durability Limit States	Table 1
Define deterioration mechanisms Calculate / simulate deterioration over design life (factor of safety)	2.5.2 Macro Environments 2.5.3 Deterioration Mechanisms	Table 2
Identify protective / mitigation measurements and reanalyse	2.6 Durability Provisions for Critical Component	Table 3
Identify required maintenance intervention	2.7 Maintenance Assumptions	Table 4
Identify components to be replaced and frequency of replacement	2.8 Routine Inspection	Table 5
Identify method of inspection, frequency and key signs of deterioration	2.9 Replacement Components	Table 6
Identify replacement methodology Repair and Rehabilitation	3.3 Materials and Proprietary Product Details	Table 7

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Flowchart - Provisions for Durability Critical Construction Activities	Section Reference in Guideline Document	Reference Table
Assess impact of construction method on durability performance ↓		
Develop mitigation measures and/or quality control procedures ↓	2.6.7 Durability Critical Processes	
Identify process and product verification measures ↓		
Identify on site defects or amendments to design ↓	2.10.3 Verification Procedures and Tests	Table 8
Evaluate impact of change on durability of affected components ↓		
Develop repair measures to restore durability performance ↓	2.10.4 Defects and Treatment Records	
Identify supplementary inspection and maintenance requirements associated with repair		

Appendix B: Summary Tables

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Table 1: Structure Durability

Critical Durability Components	Service Life					
	Duration (years)	Contract Reference	Potential Mode of Failure	Deterioration Mechanism	Factors Controlling Rate of Deterioration	End of Life Criteria (Expected governing failure mechanism)
Pile caps	100 years	Project Specification requires 100 year design life	Reinforcement Corrosion induced cracking, delamination and spalling of reinforced concrete	Chloride induced	Environmental characteristics Concrete mix Crack control Reinforcement cover As placed concrete quality Joint preparation	Chloirde induced reinforcement corrosion
				Carbonation induced		
				Macrocell corrosion		
				Localised corrosion at cracks and joints		
Piles	100 years	Project Specification requires 100 year design life	Sulfate attack of concrete.	Sulfate attack of concrete	Mobility of ground water	Softening of the concrete surface due to sulfate attack
			Weakening and erosion of concrete from acid sulfate soils	Degradation of concrete and steel from acid	Degree of oxidation of PASS and extent of ASS	
Elastomeric Bearings	40 years	Project Specification requires 100 year design life	Cracking and/or bulging of bearing. Hardening of elastomer.	Oxidation and exposure to UV light.	Environmental characterisitics and composition of elastomer	Excessive bulging and/or cracking or splitting
Expansion Joints	20 years	Project Specification requires 100 year design life	Loss of water tightness	Oxidation and cracking of elastomer	Environmental characterisitics and composition of elastomer	Puncture of sealant
				Puncture of elastomer		
Girders	100 years	Project Specification requires 100 year design life	General atmospheric corrosion leading to loss of section.	Atmospheric corrosion	Environmental Characteristics	Corrosion leading to a loss of section greater than 10%
Deck	100 years	Project Specification requires 100 year design life	Reinforcement Corrosion induced cracking, delamination and spalling of concrete	Chloride induced	Environmental characterisitics, Concrete mix, Crack control, reinforcement cover, joint preparation, quality of concrete placement.	Carbonation induced reinforcement corrosion
				Carbonation induced		
				Localised corrosion at cracks and joints		

Table Legend

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Table 2: Component Exposure Assessment (All Components)

Environment No.	Environment Zone	Environmental Details	Affected Structural Components	Potential Deterioration Mechanisms	Code Requirements	Assess if Code Sufficient	Identified Additional
1	Below River Mud line	Chloride Concentration 1000mg/l	steel piles	Soil unlikely to contribute to deterioration process. Anaerobic conditions, possible microbial induced corrosion at mud line.			
		Sulphate Concentration 250mg/l	concrete piles	Sulphate attack of concrete	50 MPa concrete	Cement content, water cement ratio and quantity of cement replacement inadequate to achieve 100 year life.	Concrete mix designed in accordance with BRE special digest 1.
		pH 6.5					
2	Within River below water level	Salinity of the river at this point has been based on BCC monitoring at Indooroopilly 10 year average of 10 ppt. Salinity at site has been taken as 10-15 ppt. Possible sulphate or low pH attack.	steel piles	Steel pile casing and/or concrete reinforcement corrosion potential due to saline water and oxygen supply.			
			concrete piles				
			Pile cap				
3	Below Water table		steel piles	No Potential Acid Sulphate Soils (PASS) have been identified by investigations to date but would require confirmation. Ground water likely to be saline to some extent.			
			concrete piles				
			Pile cap				
			footings				
			wingwalls				

Environment No.	Environment Zone	Environmental Details	Affected Structural Components	Potential Deterioration Mechanisms	Code Requirements	Assess if Code Sufficient	Identified Additional
4	In ground above water table or retaining ground		steel piles	No Potential Acid Sulphate Soils (PASS) have been identified by investigations to date but would require confirmation, notably on the West Bank flood plain.			
			concrete piles				
			Pile cap				
			footings				
			wingwalls				
5	Intertidal and Splash Zone	Chloride Concentration 1000mg/l	Cast in situ Concrete Pile Caps	Chloride Induced corrosion	AS3600 50MPa concrete, minimum cover 50mm	No.	Blended mix. Maximum water binder ratio of 0.4. Cover 60 mm
		Sulphate Concentration 250mg/l	concrete piles				
			Pile cap				
			footings				
			wingwalls				
5a	Intertidal and Splash Zone behind pile cap skirts		steel piles	Affecting pile caps of bridge towers but not subject to constant wetting and drying. Salinity of the river as noted at Environment 2 above.			
			concrete piles				
			Pile cap				
			footings				
			wingwalls				
6	Spray Zone		steel piles	Minimal chloride spray above the pile cap established by Merivale Bridge Testing.			
			concrete piles				
		Pile cap					
		Bearings					
		wingwalls					

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Environment		Environmental Details	Affected Structural Components	Potential Deterioration Mechanisms	Code Requirements	Assess if Code Sufficient	Identified Additional
No.	Zone						
7	High Spray Zone	steel piles	In view of Merivale testing this area may now be taken as combined with Environment 8, Atmospheric within 50 km of the coastline. Considered as deck level and above.				
		concrete piles					
		Pile cap					
		footings					
		wingwalls					
8	Atmospheric Exposure	steel piles	Near Coastal < 50 km from coastline as defined by ABDC.				
		concrete piles					
		Pile cap					
		footings					
		wingwalls					

Table Legend

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Table 3: Durability Provisions for Critical Components - Design

Environment Classification No.	Environment Classification Zone	Affected Structural Components	Identified Deterioration Mechanisms	Design Mitigation Measure	Construction Contract Provision	Validation Measure/Test
2	Within river below water level.	Cast in situ bored piles	Chloride induced corrosion of reinforcement only above water level.	Sacrificial permanent 16 mm steel casing		QC measurement of casing thickness.
1	Below River mud line	Cast in situ bored piles	Sulphate or low pH attack.	40 MPa, Max w/c 0.4. Minimum 30% FA and 8% SF. Minimim 400kg/m3 cementitous content.	High workability mix suitable for placement by tremie tube.	Concrete mix dockets and cylinder results.
5	Intertidal and splash zone.	Pile caps	Chloride induced corrosion of reinforcement.	Concrete to be up to 60% FA replacement with appropriate mix design and monitoring to reduce temperature rise. Top surfaces to have 60 mm in situ cover. Curing membrane to be applied. Elastomeric barrier coating to the outer surfaces of the skirt units if diffusion results are higher than expected. Silane application only to the top surface Electrical continuity of reinforcement to facilitate impressed current Cathodic Protection to extend life if required.	Large mass of concrete, heavy reinforcement and deep section may lead to placement related difficulties. Risk of early age thermal induced cracking. Requirement for concrete curing. Engineer's approval required for early age thermal modelling of contractors proposed pour sequence and methodology. Hot weather concreting measures in specification. Self compacting concrete in deep sections to avoid need to man access for placement and compaction. Specification requirements for frequency of electrical continuity testing and resistivity criteria.	Electrical continuity testing during concrete placement. Thermo couples installed in pour to confirm modelling. Visual inspection of pile cap 48 hours after casting.

Table Legend

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Table 4: Maintenance Intervention Assumptions (to achieve service life)

Element			Servicing				Cyclic		
Affected Structural Component	**Environment No.**	**Zone**	**Activity**	**Frequency**	**Access Provision**	**Intervention Level**	**Activity**	**Frequency**	**Access Provision**
Steel Bearings	8	Atmospheric Exposure	Cleaning bearings	2 years	Paved footway in front of abutment.	Rust staining on bearings	Recoat bearings	10 years	Paved footway in front of abutment.
Steel girders	8	Atmospheric Exposure	Wash girders	2 years	UBIU parked on hard shoulder of bridge. Traffic management required.	Greater than 10% of the surface area affected by bubbling, peeling or rust staining.	Repainting	20 years	UBIU parked on hard shoulder of bridge. Traffic management required to close near side lane to allow sufficient working space.
			patch painting	5 years	UBIU				

Table Legend

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Table 5: Inspection & Access Provisions

Element			Condition State Guidelines	Inspection Requirement	Supplementary Requirements to BIM	Access	Safety Hazards	
Affected Structural Component	Environment No.	Zone					Description	Mitigation
Steel Bearings	8	Atmospheric Exposure	Corrosion staining to metal parts, blistering and peeling of coatings.	Annual - QDMR Level 1 – visual inspection. At 5 years - QDMR Level 2 –Visual inspection plus photos to within 3m or telephoto equivalent	None.	Foot access from road level to front of abutment.	Slip and fall hazard ascending and descending embankment.	Paved stairway with hand rail up to abutment. Paved footway in front of abutment.
Precast Concrete Bridge Beams	8	Atmospheric Exposure	Cracking and corrosion staining, delamination and spalling	At 10 years QDMR Level 3 inspection	Chloride and carbonation sampling and testing Delamination survey	UBIU Parked on hard shoulder. Traffic Management required to close near side lane.	Reduced headroom to traffic under bridge when UBIU in use.	Close road while inspection in progress.

Table Legend

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Table 6: Replacement of Components

Element			Replacement Frequency (years)	Condition State Guidelines	Design Provisions	Maintenance Plan & Drawing References	Access	Safety Hazards	
Affected Structural Component	Environment No.	Zone						Description	Mitigation
Steel Bearings	8	Atmospheric Exposure	20	Corrosion of holding down bolts. Extrusion of PTFE.	Provision of bearing plinths and jacking points for lifting deck and replacement of bearings.	Method statement for bearing replacement and drawings provided. Drawing ref 1234/D/567	Paved walkway in front of abutment can be used as working platform.		Bearing plinths
Crash Barriers	8	Atmospheric Exposure	40	Corrosion of crash barriers.	Galvanised steel holding down bolts.	Method statement for parapet crash barrier repalcement provided.	Hard shoulder and near side lane to be closed to allow safe working space.	Passing traffic	Traffic management to include safe working area.

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Table 7: Proprietary Product Records

Component Description	Manufacturer Details	Product reference	Servicing Requirements	Warranty	Installation Details
Deck Expansion Joint	J. Bloggs & Sons 1 Nowhere Street, Back of Beyond, QLD	Expansion joint XYZ	Inspect and remove debris every 12 months. Replace or repair damaged gaskets and seals.	10 year warranty from date of installation. Warranty documentation attached.	Ambient Temperature: Dimensions / separation
Bearing					
Expansion Joints					
Waterproofing					
Barriers					
Precast Concrete Products					
Steel Fabrication					

Table Legend

Table 1	**Table 5**
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Table 8: Durability Provisions for Critical Components - Construction & Service Phases

Environment		Affected Structural Components	Identified Deterioration Mechanisms	Design Mitigation Measure	Construction Contract Provision	Validation Measure/Tests (departures to be added)	Intervention Level	Construction or Repair Method	Validation Records		Defect Records	
No	Zone								Location	Description	Location	Description
8	Atmospheric Exposure	North abutment	Chloride induced corrosion of reinforcement due to wind borne salts.	50 Mpa concrete with 50 mm cover		Cover meter survey				Cover meter survey to face of abutment.	Western Side of Abutment	Cover as low as 40 mm
								Application of Silane coating to Abutment face	North Abutment	Reagent applied to split face of finger core to confirm depth of penetration of silane.		
			Sulphate or low pH attack.	DMR concrete to B2, 40 MPa								
				70 mm cover								
5	Intertidal and splash zone.	Pile Caps – River	Chloride induced corrosion of reinforcement Large mass of concrete, heavy reinforcement and deep section may lead to placement related difficulties. Early age thermal induced cracking. Requirement for concrete curing.	Concrete to be up to 60% FA replacement with appropriate mix design and monitoring to reduce temperature rise. Top surfaces to have 60 mm in situ cover. Curing membrane to be applied. Elastomeric barrier coating to the outer surfaces of the skirt units if diffusion results are higher than expected. Silane application only to the top surface Electrical continuity of reinforcement to facilitate impressed current Cathodic Protection to extend life if required.	Large mass of concrete, heavy reinforcement and deep section may lead to placement related difficulties.Requirement for concrete curing. Risk of early age thermal induced cracking.Engineer's approval required for early age thermal modelling of contractors proposed pour sequence and methodology. Hot weather concreting measures in specification. Self compacting concrete in deep sections to avoid need to man access for placement and compaction. Specification requirements for frequency of electrical continuity testing and resistivity criteria.	Electrical continuity testing during concrete placement. Cast hot block prior to construciton to obtain heat of hydration data for thermal modelling. Thermo couples installed in pour to confirm modelling. Visual inspection of pile cap 48 hours after casting.			Pile cap 1	Post pour inspection records.		Honeycombing and segregation in localised areas and cracking in excess of 0.3 mm wide.
								Honeycombed areas to be broken back behind reinforcement and patch repaired with proprietary polymer modified repair mortar. Cracks to be repaired by V- notch and epoxy resin.	Pile cap 1- refer to repair records drawing 23455/R/34 34 for exact locations.	Pull off test on repair area.		

Table Legend	
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Appendix C: Example - Steel Structure

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The information contained in this sample report is for indicative purposes only, and should not be construed as a Main Roads Standard or Directive. Project specific design criteria will take precedence at all times.

Queensland Transport and Main Roads

Sample Durability Plan Report 2

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SAMPLE 2

1. Introduction

1.1 Background

GHD has been commissioned to undertaken the design of a twin tower cable stay bridge linking Dutton Park with the University of Queensland in Brisbane. The bridge will carry bus, pedestrian and bicycle traffic with the capability of a light rail installation at a later date.

As part of the commission the consortium designing and constructing the project is required to prepare a Durability Plan for the Bridge demonstrating how the design life of 100 years will be achieved.

1.2 Components of a Durability Plan

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A Durability Plan is a procedure by which the quality and suitability of all processes and materials involved in the production of the final end product are continuously assessed. It involves the preparation of a number of documents (Durability Assessment Reports) that outline the requirements for Durability and the assessment of compliance with these requirements of the final design, construction process and end product.

The key deliverables that are to be prepared as part of the Durability Plan are outlined in Table 1.

Table 1 Key Deliverables of the Durability Plan

Process or Deliverable	Scope
Durability Assessment Report – Concept Design	Considers the concept design and environmental information and highlights potential durability issues that will require consideration during detailed design. It includes the development of specific durability requirements for incorporation into material supply specifications
Durability Assessment Report – Detailed Design	Reissue of initial DAR with modification to reflect the detailed design. Where appropriate it includes statement of compliance of design and supply specifications with requirements for durability It shall also incorporate comment on final design, specifications, method statements and process control procedures established by others.

This report addresses the Detail Design component of the Durability Assessment Report for key components required to be completed early in the construction process.

1.3 Scope of this Report

This report addresses the detailed design durability issues excluding the pavements.

The contents of this report are provided as an example format for preparation of a Durability Plan Report. The project specific criteria referenced in this report and the project specific design solutions adopted do not constitute a Main Roads Standard or Directive.

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2. Approach to Durability

2.1 Contract Requirements

The bridge works are required to be designed to the Austroads Bridge Design Code (ABDC) amended in certain areas by the contract document.

An overall design life of 100 years is required for bridge and drainage structures with 20 years for pavements (Annexure 3 Part 1 Design Cl 1.4).

Specific durability amendments by clause are summarised in Table 5.

Within those requirements this report will deal only with the major components of the bridge structure and/or components where replacement is not possible. Minor components and fittings likely to be replaced within the life of the bridge will be dealt with under the Asset Management Report unless specifically referred to in the Contract Documents.

2.2 Design Life Definition

For the purpose of this Durability Plan, Design Life is defined in terms of Ultimate and Serviceability Limit States. These limits being defined in Austroads Bridge Design Code Clause 1.1.8 in terms of the following four parameters:

- Deformation to foundation materials.
- Reduction in structural strength due to durability issues.
- Vibration.
- Flooding.

Accordingly, with respect to durability, the end of service life is defined to occur when either:

- Deterioration progresses to a level that makes the structure unsafe or unserviceable; or
- The level of maintenance required to retain the serviceability of the structure becomes uneconomical; or
- The level of maintenance necessary to maintain the functionality of the structure becomes uneconomical.

The above definition recognises the need for ongoing maintenance to ensure that the service life is achieved.

2.3 Environmental Conditions

The microclimate in which each component of the bridge is situated is critical to its long-term performance. The environments have been categorised in Table 6.

This has been based on the following:

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The site is approximately 18 km from the coast in a straight line and considerably more following the meanders of the river. Chloride testing carried out on concrete from the Merivale Bridge and reported in detail in Report No 14313-R-004 Rev 1 indicated that chloride penetration was significant just above high tide but reduced to be negligible in terms of corrosion of reinforcing steel by 1.4 m above mean high tide level.

The site investigation, undertaken by others, makes no mention of Potentially Acid Sulphate Soils (PASS). No reference is made in any other portion of the documents provided to PASS. Bore holes midstream did not encounter Potentially Acid Sulphate Soils. At this distance upstream PASS are unlikely to be an issue. No provision has therefore been made for durability issues arising from PASS attack.

2.4 Potential Durability Issues

The general mechanisms of deterioration of relevant structural materials in aggressive environments are summarised in Table 7. Mechanisms requiring consideration during the design life of the Green Bridge include:

- Concrete degradation due to alkali aggregate reactivity (AAR) or delayed ettringite formation (DEF) or environmental exposure particularly to contaminated groundwater or soil.
- Reinforcement corrosion related spalling to all concrete elements.
- Degradation to bridge bearings and movement joints
- Degradation of coating systems applied to steel structures.
- Corrosion of the cable stay system

SAMPLE 2

3. Application by Design Component - Concrete

3.1 General

The contract specifications require that the ABDC recommendations be met with modifications as detailed in Table 5. It is intended that this will be carried out in all cases:

Table 8 summarises the key components of the structure, the environment to which they are exposed, the durability issues likely to arise and the approach taken to achieve the design life for that component. Further explanation is included here.

3.2 Global Concrete Durability Issues

3.2.1 Mechanisms of Concrete Degradation

There are a number of deterioration mechanisms that have a general effect on the performance of concrete components of the bridge. These are discussed in this section of the report.

Corrosion of Reinforcement

The primary deterioration mechanism that governs the overall durability of the lower parts of the bridge structure is the chloride-induced corrosion of reinforcement. This deterioration mechanism defines minimum requirements for concrete cover and concrete quality. It in turn is controlled by the rate of chloride diffusion through the concrete to the reinforcement. Modelling of chloride ingress into concrete is described below.

Alkali Aggregate Reaction

The results of AAR testing have not yet been provided.

However, it is noted that the use of fly ash will act to minimise any risk posed by reactive aggregates (if they were present).

Sulphate Attack

As the concrete is not exposed to ground water, there is little or no risk of sulphate attack occurring within the design life of the bridge.

Acid Attack

As the concrete is not exposed to acid sulphate soils, there is little or no risk of sulphate attack occurring within the design life of the bridge.

Aggressive Carbon Dioxide

As the concrete is not exposed to ground water, there is little or no risk of aggressive carbon dioxide attack occurring within the design life of the bridge

SAMPLE 2

Microbiologically Induced Attack

As the concrete is not exposed to ground water, there is little or no risk of microbial induced attack of the concrete occurring within the design life of the bridge

Magnesium Attack

As the concrete is not exposed to ground water, there is little or no risk of magnesium attack occurring within the design life of the bridge

3.2.2 Concrete Mix Designs

Three major premix concrete suppliers have been approached for information on their proposed mixes for High Performance Concrete (HPC). Detailed test data for concrete with high resistance to chloride penetration was not available. In particular, chloride diffusion testing to the preferred standard (NordTest NTBuild 443) was not available for mixes currently in use in Brisbane. This information is widely available for materials in Perth and Melbourne but is usually based on the use of Blast Furnace Slag. Testing to AASHTO T277-83 was available. GHD are of the opinion that this test is not a direct measure of the diffusivity of the concrete and can be affected by a number of other factors.

Discussions with the suppliers indicate that the preferred pozzolanic material in the Brisbane area is Fly Ash (FA). It is therefore proposed that a mix based on the FA/Silica Fume mix specified at tender time be used for HPC as follows:

- Minimum 440 Kg cementitious material
- 30% Fly Ash
- 8% Silica Fume
- w/c ratio <= 0.35

Long term testing of shrinkage, chloride diffusion and adiabatic temperature will be undertaken of the finalised mixes and all results will be presented as they become available.

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3.2.3 Chloride Modelling Performed at Detailed Design Stage

A probabilistic approach was taken to diffusion modelling to verify the adequacy of the design in terms of cover values and concrete quality to achieve the project durability requirements. The analysis has been undertaken for all key elements of the pile/pile cap/tower assembly in the tidal /splash zones. The model determines the probability that a Serviceability Limit State (SLS), based on cracking of the cover concrete due to reinforcement corrosion, will be exceeded during the required service life. The model draws on Bamforth's DETR work.

A minimum Reliability Index of 1.65 is considered appropriate by GHD, from consideration of the Project Service Life requirements, i.e. structure to operate in safe and serviceable manner, yet be due for replacement by a new bridge or require repairs and increased maintenance at 100 years.

The probabilistic approach makes no distinction between precast and in situ concrete, other than where a higher coefficient of variation may be applied to a variable, e.g. cover and diffusion coefficient.

Surface Chloride Levels (Cs)

The assumed surface chloride levels significantly influence durability modelling of concrete structures in chloride environments. Surface chloride concentrations are primarily related to exposure conditions.

The site testing of the existing Merivale Bridge has provided valuable information upon which to base a review of surface chloride levels for use in probabilistic durability modelling.

Results obtained for surface chloride concentrations were:

- Average of 0.43% by weight of concrete at 0-0.2 m above high tide mark
- Average of 0.1% by weight of concrete at 1.4 m above high tide mark

Proposed mean surface chloride values for use in probabilistic modelling are therefore:

- 0.43% by weight of concrete for piles, skirt units and boat units (Tidal and Splash Zone)
- 0.1% by weight of concrete for the towers and top surface of the pile caps (Spray Zone)

These are the average values for just above high tide and 1.4 m above high tide respectively determined from the Merivale Bridge Testing. There is some variation in this testing from side to side of the tower tested, the side facing the middle of the river being more exposed. The use of the average here is justified as the Green Bridge will be a further 6 km upstream and therefore 25% further from the sea along the length of the river than the tested Merivale Bridge.

The probabilistic modelling assumes that Cs does not vary with time. The Coefficient of Variation (CoV) of 20% has been adopted for the chloride concentrations at high tide level. This exceeds the standard deviation of the results obtained. A CoV for the chloride concentrations 1.4 m above high tide level of 50% has been adopted to match the variability of the results obtained.

Diffusion Coefficient (Dc)

Testing of proposed concrete has not yet been possible. In view of the programme restrictions it is unlikely that testing will be complete before precast segments are required to be placed in the river. In addition, curing of the segments will need to take place, particularly as the concrete will have a significant fly ash component, before immersion. It is therefore proposed that work proceeds on casting the precast segments before testing of the concrete to determine resistance to chloride diffusion is complete. For this reason a conservative approach has been taken to the selection of the Diffusion Coefficient.

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Testing of the concrete on another bridge project carried out by the same team determined Dc_{28} of 1.8 (CoV 0.21 for 24 samples) for high performance concrete and 8.5 $x10^{-12}$ m^2/s for a general purpose concrete (trial mix).

A conservative value of Dc_{28} of 2.5 $x10^{-12}$ m^2/s for High Performance Concrete has been used to allow for possible variations and unforeseen factors in view of the use of the mix before final testing is complete. In the model described here, Dc reduces from Dc_{28} (2.5 $x10^{-12}$ m^2/s) to Dc_{100yrs} (0.25 $x10^{-12}$ m^2/s). CoV of 20% has been assumed in all cases.

It should be noted that the results of NordTest NTBuild 443 can vary widely at early age depending on the raw materials used. Although the value assumed in the modelling is relatively conservative, a contingency measure of coating the upper part of the external face of the skirt units in the event that the results are not as expected has been allowed. This would be applied, if necessary, after preliminary results are obtained at 28 days curing and preferably before installation of the skirt units in the river.

A value of 8.5 $x10^{-12}$ m^2/s, reducing to Dc_{120yrs} 0.85 $x10^{-12}$ m^2/s has been used for the General Purpose concrete. This is also considered conservative as the concrete to be used in the main body of the towers will be 40 MPa exposure category B2 with a minimum fly ash content of 20%. (A Dc 28 day in range 3-5 x10-12 for the fly ash replacement concrete is considered likely. Diffusion modelling of concrete mix designs predicts a lower value, Dc 20 years of 0.3x10-12 for a similar mix assuming 360 kg binder).

The CoV for diffusion coefficients of site concrete will be assessed using CoV for compressive strength tests. If CoV for compressive strength tests exceeds 20%, cores will be taken for chloride diffusion testing to verify insitu values (this value is considered conservative, Sorell Bridge achieved 8% CoV for strength).

Required Concrete Cover to Reinforcement

Design cover values used in the probabilistic calculations for various elements are summarised in Table 2. A CoV of 20% has been adopted for covers, based on stringent site control. This has been reduced to 10% for precast elements. The proposed CoV's provide for cover variability greater than the added allowances for construction tolerances.

Site verification of covers will be achieved by testing of 5% of constructed elements using a covermeter.

Activation

C_{act} has been taken as 0.06% by weight of concrete for mixes This is based on the widely recognised value 0.4% chloride by weight cement, adjusted for the binder type and content, and the average site temperature (20°C). A CoV of 10% has been assumed.

Cracking

The GHD corrosion model used for this assessment assumes pitting corrosion. The algorithm in the model has been used to determine the amount of corrosion that will yield cracking. This assumes splitting tensile strength of 2 MPa, which should be

SAMPLE 2

conservative for the high strength concrete proposed. A bar size of 20 mm has been adopted for the model. CoV of 20% has been assumed.

It should be noted that the result indicates a 5% probability that cracking will occur at the time calculated.

Results

The results of the above combinations of parameters are summarised in Table 2.

The results indicate that the combination of cover and concrete type presented in Table 2 is acceptable in terms of durability.

Table 2 Summary of Analysis Results

Parameter/Result	**Precast Skirt**	**Cast In Situ Infill and Tower**
Zone	Airborne sea water and cyclic drying	Airborne sea water and cyclic drying
Diffusion coefficient	2.5×10^{-12}	8.5×10^{-12}
Coefficient of variability	0.2	0.2
Surface chloride (% by wt concrete)	0.43	0.1
Coefficient of variability	0.2	0.5
Cover (mm)	60	60
Coefficient of variability	0.1	0.2
Predicted cracking (years)	95	> 100

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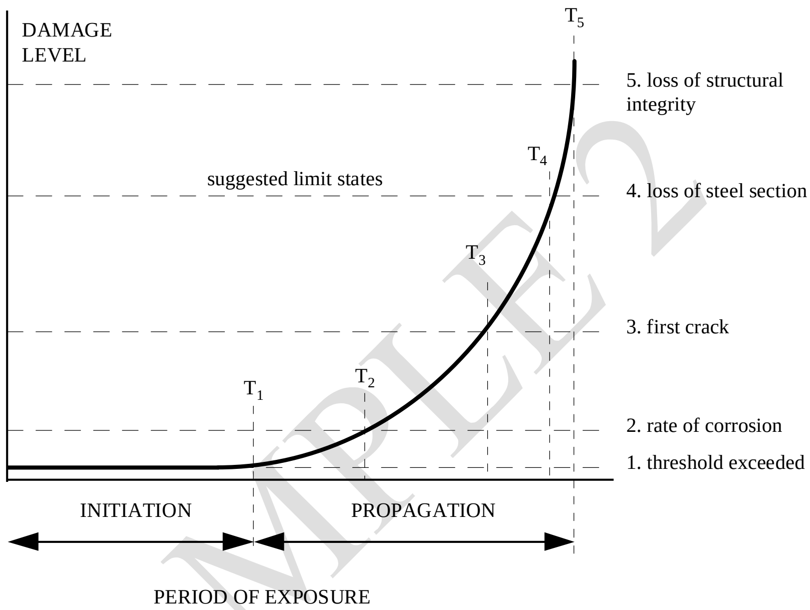
It should be noted that the model for the precast skirts calculates a 5% probability of cracking at 95 years. This is cracking of the cover zone only of the precast skirts, which are replaceable. Thereafter, cracking will increase and cover will spall off. The integrity of the skirt will, however, remain unaffected for some years. This process is illustrated graphically in Figure 2, cracking occurs at T3.

Figure 1 Graph Representing Corrosion Process



SAMPLE 2

The other components will remain unaffected for a matter of decades thereafter.

3.2.4 Site Practices

Site practices including curing and monitoring of cover pre and post placement of concrete will be addressed in the specification for the concrete.

3.3 Durability of Steel Piles

3.3.1 Pile Design

All piles are manufactured from 16 mm thick CHS grade 350 steel to AS 1163. Tower piles are 1500 mm in diameter. They are filled with load bearing reinforced concrete to a depth of 3.0 to 6.0 m below the bedrock. The concrete cover between the steel reinforcement and the inside face of the steel pile is 75 mm for tower piles. The steel piles are uncoated, and are designed as sacrificial over the concrete plug length (i.e. from the top of the piles down to approximately 4.0m below bed level).

3.3.2 Environment

The soffit of the pile cap is placed approximately 1m below the H.A.T. water level; hence the steel piles pass through the following microenvironments:

- below the mud line (buried zone);
- within the river (submerged zone); and
- intertidal zone.

The environment in the submerged and intertidal zones is estuarine river water with salinity varying from 0 to 15,000 ppm.

Likely corrosion rates in these environments were estimated from Australian Standard AS 2159-1995.

3.3.3 Likely Deterioration Mechanisms

It is important to be clear about the nature of the corrosion risks in the specific instance of the concrete filled steel piles. These are different in the various exposure zones of the piles. Where the steel liners are sacrificial, they are considered to protect the reinforced concrete from exposure until the liners are perforated due to corrosion of the steel.

Buried Zone

Corrosion of the steel liners below the mud line is expected to be extremely slow, given the lack of available oxygen for reaction. Perforation of the steel is not expected to occur within the 100 year design lifetime.

The piles are concrete filled and hence the steel liners are considered fully sacrificial. Even if perforation of the steel pile does occur, at this depth the integrity of the reinforced concrete within will not be adversely affected, due to low oxygen levels limiting possible corrosion of the reinforcement.

The groundwater would not be expected to have a significant effect on a fly-ash modified concrete.

Submerged Zone

In the submerged zone, slightly higher corrosion rates are expected, but the steel liners in this zone are still not expected to corrode through within the 100 year design lifetime. As with the buried zone, even if the steel is perforated, oxygen starvation will result in extremely small reinforcement corrosion rates in submerged conditions, even if reinforcement becomes active. Hence corrosion related deterioration of the concrete in this zone can be discounted and the durability is considered satisfactory.

Intertidal Zone

It is therefore only in the intertidal zone that deterioration due to reinforcement corrosion need be considered. The relevant mechanisms here are outlined in Table 3.

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Table 3 Deterioration Mechanisms – Intertidal Zone

Mechanism	Description	Impact on Structural Adequacy
A	General corrosion leading to complete loss of section in steel pile; followed by	Nil
B	Exposure of concrete to chloride ingress leading to corrosion activation of reinforcement; followed by	Nil
C	Build-up of corrosion product sufficient to cause cracking of cover concrete, and	Negligible
D	Loss of section of reinforcement	Potentially significant

3.3.4 Durability Assessment – Intertidal Zone

Durability Assessment Mechanism A

The scenario set out for the intertidal zone of the piles requires sufficient corrosion of the steel pile to initiate chloride penetration of the concrete. In the GBL environment, AS 2159-1995 identifies corrosion rates for design purposes ranging between 0.04 and 0.30 mm/year. Although it would be expected from experience elsewhere that a rate of 0.1 mm/year is most likely applicable (leading to nil perforation within 100 years), the extremely conservative corrosion rate of 0.3 mm/year has been adopted in this assessment. This corrosion rate results in a service life of approximately 53 years before exposure of the underlying concrete.

Durability Assessment Mechanism B

The concrete would achieve significant maturation before being exposed to the water at corrosion sites. Typically with fly ash modified concrete diffusion coefficients can be taken to reduce by a factor of 10 over 100-120 years. At 55 years, under ideal curing conditions, the diffusion coefficient will be low.

The sacrificial steel liner of the piles will still remain largely intact and so continue to limit oxygen availability.

The piles terminate at the underside of the pile cap at RL 0.0m. Reference to tide tables for the area of Tennyson (Long Pocket) on the Brisbane River indicates Mean High Water Neap Tides to have to be RL 0.77m. The piles will therefore be completely inundated twice a day even during the lowest high tides. Under these conditions the concrete remains saturated. This is particularly the case in this design where the piles when exposed at low tide are shaded and protected from drying winds by the skirt elements. Although chlorides may penetrate to the steel during the design life of the structure, the saturated concrete will not allow oxygen to reach the steel and corrosion rates will therefore remain insignificant.

SAMPLE 2

Even once exposed to the river water, as the permanent liner perforates the reinforcement in the piles will hence not corrode.

Intertidal Zone – Durability Summary

The likely conservative time frames involved in the above mechanisms are summarised in Table 4

Table 4 Summary of Durability of Steel Piles

	Mechanism	Timeframe for Piles (75 mm cover)
A	Loss of steel pile section	53 years
B	Reinforcement corrosion activation leading to cracking of cover concrete	> 50 years after A

3.3.5 Overall Pile Durability

In summary, provided the steel pile is considered sacrificial between the soffit of the pile cap and 6 m below the river bed, the unprotected concrete filled steel piles provide adequate durability to achieve the specified design life for the GBL of 100 years.

3.4 Durability of Pile Caps and Towers

3.4.1 General Configuration

Pile caps are to be constructed of pre cast permanent former with cast in situ infill. The former comes in two parts, boat shaped units that sit on top of the piles and form the base of the in situ infill first pour of approximately 0.5 m depth and skirt units that form the sides of the second pour and shield the piles exposed at low tide.

3.4.2 Corrosion Modeling

Constantly Saturated Concrete

As for the piles, concrete below the highest neap tide (RL 0.7m) will remain saturated and insufficient oxygen will be present to allow significant corrosion. Elements in this area will therefore not be considered for diffusion modelling.

Spray and Tidal Concrete

Concrete in the environment above RL 0.7 m, is considered to be in a tidal splash zone up to 1.4 m above mean high tide level and spray above that using both in situ and precast concrete elements. Probabilistic diffusion modelling has shown that covers exceeding 60 mm for precast and 60 mm for in situ concrete will provide adequate durability provision over the 100-year design life in this environment using HPC and mild steel in the precast units and low heat or GP concrete in the in situ, pile and tower concrete (Refer Table 2).

SAMPLE 2

Effect of Silane Coating

It should be noted that these predictions are based on protection from the concrete alone. The specified application of silane has been ignored. It is considered likely that the penetration of the silanes into the high quality concrete surface of the precast panels will be insufficient for the silane to perform as intended. Typically manufacturers in these circumstances require the surface of the concrete to be treated with some form of abrasive surface preparation to open the concrete pores and allow the silane to penetrate. This defeats the object of the manufacture of concrete elements with a dense low porosity surface.

Silane application in areas that will be immersed on a regular basis will not be effective. The silane acts as a repellent rather than a barrier.

The area where the silane is likely to bring most benefit is on the top of the infill concrete where it will be an exposed concrete finish, not off form precast.

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3.4.3 Precast Pile Cap Boat Units

Description

Environment	External Soffit	Tidal wet rarely dry
	External sides	Protected by skirt units
	Internal	Protected
Proposed Concrete Mix	HPC	
Proposed Covers (-5/+10mm)	Internal faces	45 mm
	Soffit	60 mm
	External side	60 mm
	Top lip and rebate	60 mm
Reinforcement type	Welded mesh with Class N bars	

Adequacy of Cover

The general cover of 60 mm for the soffit and 60 mm for the external sides will be sufficient. The units are below the lowest neap tide, will remain saturated and insufficient oxygen will be available for significant corrosion rates.

The inner faces of the pile cap shells will be exposed to salt water for a short period during construction, before the joints between shells can be made watertight. Provided the concrete is water saturated, ingress of chlorides to the inner surfaces will be minimal during this period. There is no opportunity for saline waters to penetrate to the inner face of the precast unit during service. As a consequence, the use of reduced covers to 45 mm for internal surfaces is considered acceptable.

Programme Requirements

The precast boat units will need to be installed before fully cured. In order to minimise the amount of chlorides absorbed while the concrete is immature, the units should be saturated with fresh water before being placed and rinsed daily with fresh water when exposed at low tide or protected from saturation with river water by a removable curing membrane applied to the sides and inside surface (e.g. Parchem Concure A90) before installation of the skirt units.

Critical Construction Issues

The following issues must be addressed during construction:

- Ensure that the precast unit can be placed over the pre driven piles and that grout seepage does not occur between the joint during placement of the in situ concrete
- Ensure all concrete surfaces are saturated with potable water prior to contact with river water and ensure that all concrete surfaces are rinsed thoroughly with potable water daily prior to installation of the concrete skirts and Pour 1 of the infill concrete.

SAMPLE 2

- Alternatively to rinsing, all surfaces should have a removable curing membrane applied before placing in the river
- Ensure watertight fit at the joint between precast panels and piles prior to placing in situ concrete
- Wash out with potable water and pump dry the inside of the panel prior to placement of in situ infill concrete
- Ensure the top surfaces have been water blasted to provide a mechanical key
- If crack widths greater than 0.2 mm in width develop, these are to be assessed for impact on durability and appropriate remedial measures taken.

3.4.4 Precast Skirt Units

Description

Environment	External	Tidal Splash,
	Internal	Protected
Proposed Concrete Mix	HPC	
Proposed Covers(-5/+10mm)	Internal faces	60 mm
	External side	60 mm
	Top lip and rebate	60 mm
Reinforcement type	Welded mesh with Class N bars	

Adequacy of Cover

The general cover of 60 mm for the external sides satisfies the requirements of the corrosion modelling. The modelling indicates cracking at 95 years. Structural damage will not occur for some time after cracking and not within the remaining 5 years. Refer Figure 1.

Some shrinkage between the precast skirt and the infill concrete may occur. The top of the skirt is above the area considered at risk from splash. However, the internal face of the skirt and the opposing face of the concrete infill have been considered as exposed faces with cover of 60 mm.

The corrosion modelling is based on a Dc_{28days} of 2.5×10^{-12}. The skirts should ideally be cast in advance to allow 56 days of curing before immersion. Testing of the proposed mix to confirm this value before casting commences is therefore not possible. It is intended that in the event that the diffusion coefficient measured is greater than that required, an elastomeric barrier coating will be applied to the area of the skirts likely to be affected by chloride induced corrosion. This is a narrow band approximately 1.5 m high from RL 0.7m. Testing has confirmed that above this the surface chlorides are too low to cause corrosion. The coating will be extended up to the top of the skirts for aesthetic reasons.

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The coating will not need to be maintained. Its function is to keep chlorides out of the concrete for the first 5-10 years. After this time the concrete will have attained a sufficiently low chloride diffusion coefficient to keep out the chlorides itself.

Critical Construction issues

The following issues must be addressed during construction:

- If crack widths greater than 0.2 mm in width develop, these are to be assessed for impact on durability and appropriate remedial measures taken
- In the event that the testing of the proposed mix indicates a higher than expected diffusion coefficient, apply an elastomeric coating from RL 0.7m to the top of the skirts
- Ensure that exposed faces of precast units are defect free, e.g. bug-holes, honeycombing
- Use concrete spacers of similar durability as the design concrete.

3.4.5 Cast Insitu Infill Concrete to Pile Cap Panels

Description

Environment	Top Face	Spray
	Other faces	Protected
Proposed Concrete Mix	Low Heat	
Proposed Covers (-5/+10mm)	Internal faces typical	45 mm
	Internal face top pile	75 mm
	Top Steel and facing skirt unit	60 mm
Reinforcement type	All	Class N bars

Adequacy of Cover

The top surface and area facing the top of the precast skirt are the only parts of the infill concrete to be exposed to the external environment. Cover to the main reinforcement of 60 mm complies with the requirements of probabilistic modelling.

Need for Crack Control Reinforcement

The pile caps are 2.5 m deep and approximately 10 m by 20m and contain significant reinforcement. The piles will impose significant restraint on movement. Thermal modelling will be undertaken to ensure minimised risk of cracking through measures such as insulation of the top of the pour to minimise temperature differential, restricting the maximum delivery temperature of the concrete and appropriate top surface reinforcement to control cracking.

Critical Construction issues

The following issues must be addressed during construction:

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- Ensure that adequate bond develops between the precast and the infill concrete. This can best be achieved by ensuring that the inner surface is roughened by water blasting
- Ensure that the reinforcement is washed down to remove surface chlorides, and protected from splash and spray prior to concrete placement
- Ensure that the pour is protected from splash and spray following concrete placement
- The top surface is insulated to control thermal cracking
- Finished surfaces are inspected to identify any thermal cracks
- If crack widths greater than 0.2 mm in width develop, these are to be assessed for impact on durability and appropriate remedial measures taken.

3.4.6 Towers – Below RL 10 m

Description

Environment	Spray	
Proposed Concrete Mix	DMR B2 40 MPa 80/20 GP/FA	
Proposed Covers (+5/-10mm)	External side	60 mm
Reinforcement type	Conventional N Grade bars	

Adequacy of Cover

Cover to the external faces of the towers complies with the minimum requirements as determined by probabilistic modelling.

Critical Construction issues

The following issues must be addressed during construction:

- Ensure that reinforcement is correctly placed to achieve the required cover
- Ensure that the pour is protected from splash and spray during curing.

3.4.7 Concrete in the Remaining Components

Concrete and cover in remaining components may be as follows with no cracking or spalling due to corrosion within the 100 year design life.

Element	**ABDC Exposure**	**Precast/cast in situ**	**Concrete**	**Cover**
Towers, Piers 6 & 7	B2 below Bridge Beam Soffit	Cast in situ	DMR B2, 40 MPa	60 mm
	B1 above Bridge Beam Soffit	Cast in situ	DMR B1 40 MPa	40 mm

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Remaining piers piles and columns	B1	Cast in situ	DMR B1 40 MPa	60 mm
Remaining piers all other elements	B1	Cast in situ	DMR B1, 40 MPa	40 mm
Headstocks not in contact with ground	B1	Cast in situ	DMR B1	40 mm
Bridge deck precast slabs	B1	Precast	DMR B1, 40 MPa	40 mm
Super Tee beams	B1	Precast	DMR B1, 50 MPa	30 mm, 20mm on upper face of top flange
Bridge Deck infill stitch concrete	B1	Cast in situ	DMR B1, 40 MPa + shrinkage reducing admixture	40 mm
Barriers	B1	Cast in situ	DMR B1, 40 MPa	50 mm
Tower top pre-cast panels	B1	Precast	DMR B1, 40 MPa	40 mm
Precast Kerb Units	B1	Precast	DMR B1, 40 MPa	30mm

This is based on the testing of the concrete from the Merivale Bridge, requirements of the ABDC and prediction of carbonation carried out using GHD's in house model.

Provision has been made in the pile cap and foundations of piers 6 and 7 to facilitate future installation of Cathodic Protection (CP). If applied, CP would arrest corrosion of the reinforcement and provide the capacity to extend the life indefinitely with appropriate maintenance.

Acid sulfate soils were only encountered between 5-9m below ground level at the location of Piers 3 and 4. Bored cast in situ piles will intersect these horizons. However, the disturbance to the in situ acid sulfate soil will be minimal and the soil will not dry out. No damage to the structural concrete of the pile is therefore anticipated within the 100-year design life.

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4. Application by Design Component - Other

4.1 Deck Joints

In the busway deck areas deck joints will be either an elastomeric gland or a modular joint type, depending on movement requirements. Industry common usage and suppliers experience indicates that the glands are the component with the shortest life but are expected to last at least 25 years. Three replacement cycles are expected to achieve the 100-year design life.

On the pedestrian and cycle paths the joints will be sliding plate systems. Although having no components that are likely to fail, surface wear and tear may require replacement of the plates at the same intervals as the modular joints.

The deck joint assemblies will be designed to facilitate future replacement.

The deck joint life will be extended by keeping the joints clean of dirt and debris which collects between the gland surfaces. Regular cleaning will extend their life. This will be addressed in the asset management programme.

4.2 Bearings

A summary of the bearing type, location and likely life are as follows

Location	Type	Typical life * (Years)	Expected shortest life component	Maintenance
Abutment A, Piers 1,2,3 & 4 east	Elastomeric	50+	Elastomeric shell	Clean and repaint at same intervals as steel
Pier 4 west	Guided pot	40	Mechanism	Ditto
Piers 5 & 8	Pot	40	Mechanism	Ditto
Abutment B	Guided pot	40	Mechanism	Clean and repaint at same intervals as steel

* Typical life has been estimated based on feedback from industry suppliers and GHD's experience in bridge refurbishment projects.

Sketches showing typical arrangements for access, jack placing and replacement of the bearings will be provided.

Pot bearings are expected to last 40 years and 2 replacement cycles are expected to achieve the 100-year design life. Industry common usage and suppliers experience

SAMPLE 2

indicates that the limiting life factor is the mechanism inside the bearings rather than corrosion of the casings. The life expectancy is influenced by the amount of live rotation and shear induced back into the bearing by the movement of traffic. If the mechanism fails the only solution is to remove and replace / recondition the bearing. This is the worst-case scenario for a Pot Bearing. Current experience amongst key users of Pot Bearings has been that 25 years is the limit for the expected life. However, design of current pot bearings has been improved compared with bearings of 25 years ago and they are likely to last significantly longer and at least the stated 40 years.

The use of bearings with stainless steel casing is not therefore justified as the life is dictated by the mechanism rather than corrosion.

The bearings will be inspected and maintained by grinding and spot coating the casing, where corrosion appears, at the same time as the planned coating maintenance to the structural steel of the deck.

Industry common usage and suppliers experience indicates that elastomeric bearings are expected to last 50+ years and 1 replacement cycle is expected to achieve the 100 year design life.

4.3 Bridge Deck Steel Structure

A high standard of surface preparation and coating application is easier to attain in a paint shop than on in situ steel work. For this reason a high performance coating system has been selected to maximise the life from that initial coating application. The paint system is in accordance with ISO12944-2: Global Corrosion Standard, and exceeds the requirements for high durability, >15 years, in a C3 corrosivity environment.

Within AS/NZS 2312-2002: Guide to the protection of structural steel against atmospheric corrosion by use of protective coatings, the system would be equivalent to a PUR5 designation, satisfying the requirements for 25+ years durability until first maintenance within a medium corrosivity category C.

The paint system used for all painted structural steel is a 3-coat Micaceous Iron Oxide Epoxy system applied to steel prepared by blast cleaning to Sa 2.5.

	Binder	**Standard**	**DFT microns**
1	Two-component high solids metallic zinc-rich epoxy primer	AS/NZS 3750.9 Type 2 APAS 2973	75
2	Two component, high build micaceous iron oxide epoxy	APAS 2973	125
3	Two component, high build micaceous iron oxide epoxy	APAS 2973	125
	Total film thickness		**325 microns**

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At the bolted junctions the matching faces of the steel shall be prepared to a similar high standard and coated with the primer only. The perimeter of the bolted areas shall be sealed with joint sealant and the exposed surfaces and bolts coated with the top coat Detailing will be provided on the drawings.

In view of the likely longevity of the proposed system it is proposed that the coating be monitored for any rust spots during the visual inspection programme that forms a key part of the QDMR Bridge Maintenance Manual. Detailed visual inspection is carried out every 5 years. At this time any evidence of rust spots or staining would be recorded and appropriate spot repairs carried out. As the projected life of the coating approaches and/or the quantity of spot repairs increases the frequency of inspection may need to be increased. A decision to over coat completely would be taken on the basis of increasing small failures.

It is anticipated that the coating will last at least 20 years and possibly up to 30 years in this environment with minor maintenance. Coating systems of a lesser performance/older technology have been applied to structural steelwork on bridges over the Brisbane River and have not had to be completely over coated in 25 years.

The coating condition will be monitored at the 15, 20 and 25 year inspections. A decision will be made to overcoat based on the condition of the coating. At that time all rust spots would be ground and painted with the complete system in accordance with the manufacturer's instructions. The remaining areas would have an additional topcoat of the same material applied.

It is anticipated that a full application of a new top coat would be required 3 to 4 times in the 100 year life of the bridge.

From a durability stand point a high performance coating with a likely time to next application of 25-30 years has been selected. This is more appropriate to long term durability than a shorter life coating to be over coated at 10 years and then 10 year intervals. The over coating at handover is therefore not recommended.

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4.4 Hand Rail

The hand rail will be stainless steel as specified. The support structure will be hot dip galvanised to MRS 11.78 including AS 4690 at 600 g/m^2.

The materials would be isolated electrically using appropriate insulating fittings.

Reference to various industry publications indicate that at 600 g/m^2 the expected life of the galvanising would be 30-50 years depending on the exposure of the individual components. This is well in excess of the 20 years required for replaceable components such as this in accordance with Annexure 3, Part 1, Section 1.4.

These elements would also be part of the inspection programme and would be spot repaired by grinding and application of proprietary inorganic zinc silicate coating systems.

As the maintenance commitment increases, likely to be around 30 years, a decision would need to be made to coat all the galvanised support structure. A coating system would be selected at that time based on the latest technology, and is likely to last 10-30 years between applications depending on coating selected and quality of application.

Alternatives would be to grit blast the galvanized components and apply flame sprayed zinc metal coating or replace with new hot dip galvanised components, both of which are considered unlikely to be acceptable.

4.5 Walkway Canopy

The walkway canopy structure will be in line galvanised sheet, Galvabond or similar. AS 2512, Table 5.6 indicates that for an atmospheric corrosivity category most likely B, possibly C this type of coating will give a life of 25 years. The Canopy would be inspected every 5 years. Maintenance coating in the form of grinding and spot coating with proprietary inorganic zinc silicate coating systems will extend the life of the canopy, to 30-40 years. At this time replacement of the sheet is likely to be more economical than re coating.

4.6 Cable Stay System

The cable stay system will be the subject of a detailed submission by the selected specialist. Much of the detailed assessment of the durability and maintenance requirement is dependent on the proprietary system selected. This submission to be incorporated in the Asset Management Plan will include:

- design life(durability)
- protection system
- how the anchorages can be inspected, maintained, replaced etc

The design accommodates the sudden failure of 1 cable and the replacement of one cable at a time as specified in Annexure 3, Attachment 2, Section 2.17.5

All components will be subject to comprehensive monitoring programme, the nature of which will be dependent on the selected system. Replacement of deteriorated components as and when required will be carried out.

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5. Accessibility

As part of the approach to durability the specification requires accessibility to maintain / monitor / inspect / replace all components of the bridge.

The two areas of the bridge that will require installed access are the underside of the bridge deck and the surfaces and cable anchorages of the towers. The intended access arrangements for these are:

5.1 Piers 6 & 7

From the deck level up inserts will be cast into the concrete to facilitate the fixing of temporary towers for mast climbing work platforms or scaffolding if required in the future.

Access for visual inspection and routine maintenance including replacement of aviation light, if required, will be from a truck mounted cherry picker.

5.2 Underside of the Deck

A gantry, including a movable "work platform" arrangement will enable access to the underside of the deck in all areas above the gantry. The gantry shall be detachable so that, if required, it can be disconnected from the bridge and loaded onto a truck parked on Sir William McGregor Drive beneath the deck at the UQ-end of the bridge and taken away for storage between occasions when it is needed.

Access for visual inspection and routine maintenance will be carried out at the deck between abutment A and pier 5 from a truck mounted cherry picker.

5.3 Deck Areas

All components on or fixed to the deck will be accessed from the deck surface A light cherry picker or crane type access may be used if required.

5.4 Piers Beneath the Deck

Concrete surfaces have no planned rehabilitation activities other than inspection and therefore require no access. In the event that remedial activities are required scaffolding would be erected from the ground level or off the pile caps in the river.

Bearings at the top of some of the land piers may require replacement. This will be carried out from scaffolding erected from ground level. There are no bearings at Piers 6 & 7.

5.5 Requirements by Component

Table 9 summarises the activities and access requirements broken down by component.

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6. Remedial Works to Extend Bridge Life to 150 Years

6.1 Introduction

The Design Life is 100 years. The Contract requests consideration of remedial works that may be required to extend the bridge life to 150 years and beyond.

For the period beyond 100 years, the majority of the components will continue to be maintained and/or replaced in accordance with the asset management programme as follows:

- Structural steel deck Continue to inspect and coat at 25-30 year intervals
- Bearings Continue to replace at 40-50 year intervals
- Deck Joints Continue to replace at 25-30 year intervals
- Canopy Replace or continue to coat as appropriate.
- Hand Rails Replace or continue to coat as appropriate.
- Cable Stay System Continue to inspect and replace components as required.

6.2 Concrete

6.2.1 Pile Caps below RL 0.7m

The areas below RL 0.7 are expected to remain unaffected. Saturation of the concrete will continue to stifle corrosion due to lack of oxygen.

6.2.2 Pile Caps - Skirts

In the original 100-year life design the skirt units are predicted to crack and spall at approximately 100 years. They would have been monitored for chloride diffusion at 10 yearly intervals as required by the QDMR inspection procedures. From this information it would be possible to predict time to corrosion and cracking fairly accurately any time after 30 years.

Three options were available to extend the life of the skirts:

a. ***Application of a barrier coating***. This would need to be installed before chlorides reach the bar, likely to be 50-60 years. The barrier coating would then need to be maintained. Current coatings would require reapplication under these conditions every 10-15 years. Cost of each application in 2005 dollars is approximately $20,000 for both pile caps.

b. ***Application of CP***. This can be delayed until appearance of first cracking which is predicted to be at 95 years, but could be longer. Cost of CP installation for both pile caps at 2005 dollars is approximately $350,000. Annual maintenance would be $3-5,000 per year. A schematic drawing of a typical CP system will be provided.

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c. ***Replace Skirts*****.** The skirts are replaceable in the event of a serious collision.

The original design intent of 100 year design life has been modified by the client to extend the life beyond 100 years by measures built in at construction time.

The revised design uses stainless steel in the outer layer of the external face of the skirts at a cover of 40 mm with plain high yield reinforcing behind at a cover of 60 mm. Modelling of chloride penetration to the “black” steel layer indicates that cracking and spalling will not occur until 100+ years.

Methods to extend this remain as for the original design.

6.2.3 Pile Caps – Top Surface

In the consortium 100-year life design, modelling of chloride penetration in this area indicated that cracking would not take place until well over 100 years. The model did not take into account the effectiveness of the silane application in this area and the rate of chloride absorption/diffusion is likely to be less as a result of the silane. Maintenance of the silane application has been scheduled every 10 years. With this initial application and provided maintenance is continued, it was considered unlikely that corrosion would occur within 150 years.

Chloride penetration could have been monitored through 10 yearly testing and more accurate prediction of the time to corrosion activation and cracking would be possible after 30 years. In the event that chlorides would reach the steel within the 150 year life, cathodic protection to the top steel in the pile cap could be initiated.

The original design intent of 100 year design life has been modified by the client to extend the life beyond 100 years by measures built in at construction time.

The revised design uses stainless steel in the outer layer of the top face of the pile cap at a cover of 60 mm with plain high yield reinforcing behind with a cover of 100 mm. Modelling of chloride penetration to the plain high yield steel layer indicates that cracking and spalling will not occur within a time foreseeable by the model.

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6.2.4 Towers – Piers 6 and 7 Below Deck

Modelling of chloride penetration in this area indicates that cracking would not take place until well over 100 years.

The concrete will again be monitored for chloride penetration and more accurate prediction of the time to corrosion activation and cracking will be possible after 30 years. Application and maintenance of a silane impregnation before corrosion activation will extend the time to cracking possibly indefinitely.

Cathodic protection may be applied at any time after 100 years.

6.2.5 Remaining concrete.

The deterioration of remaining concrete will likely be due to carbonation induced reinforcement corrosion alone. Monitoring of carbonation rates will take place. This will facilitate prediction of time to corrosion initiation. Based on modelling of likely carbonation rates, this is well beyond 100 years for all components.

Cover of 60 mm will be provided in all piers below the deck up to the soffit of the headstocks and continue 40 mm cover in the Piers 6 and 7 to the top. With these covers and the intended concrete, carbonation is unlikely to reach the reinforcing steel until 150 years and probably significantly beyond.

Cover in the deck concrete and in the pier headstocks where there is a lower risk of chloride exposure will be 40 mm. Modelling of this indicates that carbonation will not reach the steel within 100 years. The model does not, however, take account of the improved compaction and surface finish likely in pre-cast segments. It could be that the rate of carbonation will be significantly lower with extension of this time to beyond 150 years. This will be monitored as part of the QDMR inspection programme and in the event that it becomes an issue within the 150 year life an anti-carbonation coating can be applied to the soffit and sides and some form of topping to the upper surfaces. These would arrest carbonation. The coatings would need to be maintained and could be re-applied at the time of recoating the steel and/or resurfacing the deck.

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Table 5 Contract References to Durability (Maintenance Requirements Considered in Separate Report)

Section	Subsection	Component	Requirement
Annexure 3 Part 1	Section 1.4	General	Bridge design life 100 years, pavements 20 years
Annexure 3 Attachment 2	Section 1.2	General	Materials to deliver lowest whole of life cycle cost
Ditto	Ditto	Corrosion Protection System	To provide details of coating, methods and frequency of coating, maintenance & susceptibility to vandalism
Ditto	Ditto	General	Accessibility to maintain / monitor / inspect / replace
Ditto	Ditto	General	Material Selection to meet the design life
Ditto	Ditto	General	Member selection and connection details to facilitate longevity, inspection, testing and maintenance
Ditto	Ditto	General	Shut down and disruption to traffic on and under the bridge
Ditto	Ditto	General	Remedial works required to extend bridge life to 150 years and beyond
Annexure 3 Attachment 2	Section 2.17.5	Amendments to ABDC - Cable Stay	Ability to withstand failure of 1 cable and to replace 1 cable at a time.
Annexure 3 Attachment 2	Section 4.2	Amendments to ABDC - Bearings	Steel in bearings to be carbon steel. Elastomeric bearings to AS 1523. Bearings to be accessible for maintenance and designed to minimize ingress of dirt, moisture etc.
Annexure 3 Attachment 2	Section 4.3	Amendments to ABDC – Deck Joints	Joint cover plates for the pedestrian walkway, cycle path and joints in kerbs and barriers shall be stainless steel. Refer to BCC's S600 Stainless Steel Specification.
Annexure 3 Attachment 2	Section 5.2	Amendments to ABDC – Reinforcing steel	Reinforcing in tidal and splash zones to be carbon steel.

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Section	Subsection	Component	Requirement
Annexure 3 Attachment 2	Section 6.1	Amendments to ABDC – Handrails and cover plates	Handrails and cover plates for walkway and cycle path to be stainless to BCC S600.
Annexure 3 Attachment 2	Section 6.3	Amendments to ABDC – Bridge Bearings	Exposed components of bridge bearings to be carbon steel.
Annexure 4 Part 1	Section 7.2.1	Amendments to MRS Spec – tidal and splash zone concrete	Concrete in saline water or in the splash zone shall have minimum 20% FA cement replacement.
Annexure 4 Part 1	Section 7.2.2	Amendments to MRS Spec – tidal and splash zone concrete	Two coat silane application to splash zone and tidal concrete.
Annexure 4 Part 1	Section 7.5	Amendments to MRS Spec – Bridge Bearings	Exposed components to be carbon steel.

SAMPLE 2

Table 6 Environmental Analyses

Environment	Classification	Description
Environment 1	Below River Mud line	Soil investigations have not shown any properties that may contribute to deterioration process. Anaerobic conditions.
Environment 2	Within River	Steel pile casing and/or concrete reinforcement corrosion potential due to saline water and oxygen supply. Possible sulphate or low pH attack.
Environment 3	Below Ground Below Water table	Potential Acid Sulphate Soils (PASS) have been identified by investigations in piers 3 & 4 only.. Ground water likely to be saline to some extent.
Environment 4	In ground above water table or retaining ground	Potential Acid Sulphate Soils (PASS) have been identified by investigations in piers 3 & 4 only.
Environment 5	Intertidal and Splash Zone	Affecting pile caps of bridge towers to a maximum of 1.4m above mean high tide. Salinity of the river as noted at Environment 2 above.
Environment 5a	Intertidal and Splash Zone behind pile cap skirts	Affecting pile caps of bridge towers but not subject to constant wetting and drying. Salinity of the river as noted at Environment 2 above.
Environment 6	Spray Zone	Minimal chloride spray above the pile cap established by Merivale Bridge Testing. Taken as piers to underside of tower tie beam soffit.
Environment 7	High Spray Zone	In view of Merivale testing this area may now be taken as combined with Environment 8, Atmospheric within 50 km of the coastline. Considered as deck level and above.
Environment 8	Atmospheric Exposure	Near Coastal < 50 km from coastline as defined by ABDC.

SAMPLE 2

Table 7 Deterioration Mechanisms

Potential Mode of Failure	Deterioration Mechanism	Factors Controlling Rate of Deterioration
Reinforcement Corrosion induced cracking, delamination and spalling of reinforced concrete	Chloride induced Carbonation induced Macrocell corrosion Localised corrosion at cracks and joints	Environmental characteristics Concrete quality Crack control Reinforcement cover As placed concrete quality Joint preparation
Concrete Degradation	Alkali-aggregate reactivity Sulphate attack Acid attack from contaminated groundwater and soil	Environmental characteristics Aggregate properties Crack control Binder type Concrete quality Surface preparation
Exposed Steel and Other Metal	Corrosion Coating/protection failure	Dissimilar metals Detailing Electrical insulation Coating maintenance
Movement Joints	Excessive movement leading to poor running surface Concrete cracking, delamination and spalling Wear	Capacity for replacement Design detail Workmanship Prevention of water penetration Reinforcement detailing Traffic load and volume
Bridge Bearings	Construction loading Environmental degradation Wear	Capacity for replacement Positioning Protection Design Traffic load and volume
Ground Movement	Deflection & settlement Poor running surface Joint failure - Cracking	Foundation design Environmental characteristics

SAMPLE 2

Table 8 Durability Issues by Design Element

Design Element	Design Life (Years)	Zone	Issues Which May Affect Durability	Design Approach to the Durability Issues
Cast in situ bored piles – River Bed	100	Below Mud line and within river.	Chloride induced corrosion of reinforcement only above water level. Sulphate or low pH attack.	Sacrificial permanent 16 mm steel casing ABDC B2 40 MPa concrete with 20% FA replacment 70 mm cover Electrical continuity of reinforcement to facilitate impressed current Cathodic Protection to extend life if required.
Cast in situ bored piles – Flood Plain	100	Below Ground Above and Below Water Table	Chloride contamination from ground water only above water level.	Pile buried, inundation events infrequent, oxygen minimal. ABDC Class B2 adequate with 20 % FA cement replacement.
Abutments Retaining Walls	100	Atmospheric Exposed & Below Ground Above Water Table	Atmospheric pollutants Requirement for concrete curing. Early age thermal induced cracking in retaining walls	Design as ABDC exposure classification B2 and include 20 % FA replacement. Curing membrane applied immediately following removal of formwork. Review casting sequence for thermal issues.
Driven pre-cast concrete piles – Flood Plain	100	Below Ground Above and Below Water Table	Chloride contamination from ground water only above water level.	Pile buried, inundation events infrequent, oxygen minimal. Precast concrete to ABDC Class B2 adequate.

SAMPLE 2

Design Element	Design Life (Years)	Zone	Issues Which May Affect Durability	Design Approach to the Durability Issues
Pile Caps – River	100	Intertidal and splash zone.	Chloride induced corrosion of reinforcement Large mass of concrete, heavy reinforcement and deep section may lead to placement related difficulties. Early age thermal induced cracking. Requirement for concrete curing.	Permanent precast concrete panel formwork with FA cement replacement to 30%, Silica Fume to 8% and cover of 60 mm.. Infill concrete to be up to 40% FA replacement with appropriate mix design and monitoring to reduce temperature rise. Top surfaces to have 60 mm in situ cover. Curing membrane to be applied to all surfaces except top of pile cap, latter to be wet cured.. Silane application to top of pile cap only. Stainless steel top layer of reinforcing in top of pile cap. Electrical continuity of reinforcement to facilitate impressed current Cathodic Protection to extend life if required.
Pile Caps – Flood Plain	100	Below Ground Above and Below Water Table	Chloride contamination from ground water and/or flooding.	Pile cap buried, inundation events infrequent, oxygen minimal. ABDC Class B2 adequate with 20 % FA cement replacement.
Piers 6 & 7,– (river to underside of tie beam soffit)	100	Spray Zone	Chloride induced corrosion of reinforcement Large mass of concrete, restrained by pile cap at the base. Early age thermal induced cracking. Requirement to strip and raise formwork within 24 hours Requirement for concrete curing.	Testing at Merivale Bridge indicated minimal chlorides. ABDC Class B2 adequate with 20 % FA cement replacement, 60 mm cover Curing membrane applied immediately following removal of formwork.

SAMPLE 2

Design Element	Design Life (Years)	Zone	Issues Which May Affect Durability	Design Approach to the Durability Issues
Piers 6 & 7 - Remainder	100	Atmospheric Exposed	Atmospheric pollutants. Requirement for concrete curing Requirement to strip and raise formwork within 24 hours. Early age thermal induced cracking	Classify as ABDC B1 but 60 mm cover continued from below the deck. Incorporate minimum 20% FA cement replacement Curing membrane applied immediately following removal of formwork.
Remaining Piers & headstocks	100	Spray Zone	Chloride induced corrosion of reinforcement Requirement for concrete curing.	Classify as ABDC B2, 20% FA replacement Curing membrane applied immediately following removal of formwork.
Deck Joints	25	Atmospheric – Protected	Applied dynamic loads, static loads, movements. Construction loads. Atmospheric pollutants. Service life.	Select joint materials RTA B316. Elastomeric glands are the shortest life component. 25 years is the minimum expected life, possibly longer. Designed to facilitate replacement at 25 years if required.
Pot Bearings	20	Atmospheric – Protected	Applied dynamic loads, static loads, movements. Construction loads. Atmospheric pollutants. Service life.	The bearing mechanism is expected to have a life of 40-50 years. Corrosion resistance can be achieved by maintaining coating on the bearing housing. Designed to facilitate replacement at 40 years as required and to allow coating at same time as structural steel.

SAMPLE 2

Design Element	Design Life (Years)	Zone	Issues Which May Affect Durability	Design Approach to the Durability Issues
Elastomeric Bearings	20	Atmospheric – Protected	Applied dynamic loads, static loads, movements. Construction loads. Atmospheric pollutants. Service life.	The bearing is expected to have a life of 40-50 years. Designed to facilitate replacement at 40 years as required.
Bridge Steel Deck Structure	100	Atmospheric Exposed	Atmospheric Pollutants Deterioration of Applied Coatings Corrosion Dissimilar metals	Shop based coating of components will be undertaken with required surface preparation under controlled conditions. A quality coating system will be applied under controlled conditions to maximise interval to maintenance. Make provision for access to grind corrosion spots and reapply top-coat at 20 year or greater intervals based on coating performance. All members to be “open” to allow inspection and maintenance.
Bridge Precast Deck	100	Atmospheric Exposed	Potential for DEF and AAR. Early age thermal induced cracking. Shrinkage at stitch joints leading to open joint Atmospheric pollutants. Rainfall and drainage. Applied dynamic loads. Construction loads. Curing	Maximise curing time before placement of units and incorporate shrinkage reducing admixture in stitch joints Design as ABDC exposure classification B1 and include 20 % FA replacement. Consider temporary case loading for handling. Prepare handling procedures to minimise damage. Ensure drainage is designed and managed to minimise standing water.

SAMPLE 2

Design Element	Design Life (Years)	Zone	Issues Which May Affect Durability	Design Approach to the Durability Issues
Hand Rail	20	Atmospheric Exposed	Atmospheric pollutants. Galvanising. S/S handrail.	Hand rails themselves stainless steel Stainless steel components will be electrically insulated with appropriate washers. Support structure galvanised at 600g/m2 with an estimated life of 30 years. Make provision for access to paint after 30 years
Cable Stay System NB: generic requirements only. To be confirmed on appointment of specialist contractor	100	Atmospheric Exposed	Atmospheric Pollutants Deterioration of Applied Coatings Corrosion Dissimilar metals	Cable system will be the subject of a detailed durability submission by BBR. All components will be subject to comprehensive monitoring and maintenance in accordance with BBR nominated procedures; see maintenance programme. Replacement of deteriorated components as and when required will be carried out.
Walkway Canopy	30	Atmospheric Exposed	Atmospheric Pollutants Deterioration of Applied galvanising Corrosion Dissimilar metals	Inspect at 5-year intervals and touch up. Planned replacement at 30 years from deck and gantry. In line galvanised sheet 42 micron thickness AS 2312 indicates exposure category B/C border with projected life in the region of 25 years. With maintenance coating this will extend to 30 years. Replace or paint at this time.

SAMPLE 2

Design Element	Design Life (Years)	Zone	Issues Which May Affect Durability	Design Approach to the Durability Issues
Concrete Barriers	100	Atmospheric Exposed	Early age thermal induced cracking. Shrinkage cracking Atmospheric pollutants. Rainfall and drainage. Curing	Incorporate movement joints at appropriate intervals Inspect and renew joint sealant at expansion joints as required, projected life 20-30 years depending on exposure location. Design as ABDC exposure classification B1 and include 20 % FA replacement. Curing membrane applied immediately following removal of formwork.
Drainage	100	Atmospheric Exposed	Atmospheric Pollutants Deterioration of Applied galvanising on grate and frame and pipe hangers Corrosion Dissimilar metals UV degradation of PVC/GRP pipe	Hanger galvanising beneath bridge has an estimated life of 30 years. Make provision for access to paint or progressively replace after 30 years. Galvanising in grate and frame will have reduced life with build up of debris holding moisture. Clean monthly as part of maintenance programme. Inspect and maintenance coat with zinc rich coating as required. PVC/GRP in shaded environment likely to reach 100 years. Inspect and replace/maintain as necessary.

SAMPLE 2

Design Element	Design Life (Years)	Zone	Issues Which May Affect Durability	Design Approach to the Durability Issues
Precast Girders.	100	Atmospheric Exposed	Potential for DEF and AAR. Early age thermal induced cracking. Atmospheric pollutants. Applied dynamic loads. Construction loads. Curing	Maximise curing time before placement of units Design as ABDC exposure classification B1 and include 20 % FA replacement. Consider temporary case loading for handling. Prepare handling procedures to minimise damage.
Tower tip – steel frame	100	Sheltered	Moisture penetration at joints	Support structure galvanised with an estimated life in excess of 30 years in sheltered dry environment. Joints designed with weather seal. Maintain seal, life 20-30 years depending on exposure. Drainage provision to allow any water to escape. Inspect and maintain galvanising beyond 30 yrs if required.
Tower tip – precast panels	100	Atmospheric Exposed	Potential for DEF and AAR. Early age thermal induced cracking. Atmospheric pollutants. Rainfall and drainage. Applied dynamic loads. Construction loads. Curing Thermal cracking	Maximise curing time before placement of units and incorporate shrinkage reducing admixture in stitch joints Design as ABDC exposure classification B1 and include 20 % FA replacement. Consider temporary case loading for handling. Prepare handling procedures to minimise damage. Design fixing system to accommodate thermal movement.

SAMPLE 2

Table 9 Access Arrangements

Main Component	Sub Component	Activity	Access
Pier	Pile cap	QDMR Level 1 – Visual inspection	Boat/foot
		QDMR Level 2 – Visual inspection plus photos to within 3 m or telephoto equivalent	Boat/foot plus telephoto
		QDMR under water	Boat/diver/under water camera
		QDMR Level 3 – Chloride and Carbonation sampling and testing, selected locations Delamination survey, selected location	Boat/foot plus telephoto
	Pier & Tower concrete beneath deck	QDMR Level 1 – Visual inspection	Boat/foot and shore based
		QDMR Level 2 – Visual inspection plus photos to within 3 m or telephoto equivalent	Boat/foot and shore based with binoculars
		QDMR Level 3 – Chloride and Carbonation sampling and testing, selected locations Delamination survey, selected location	Boat/foot Carbonation testing at base and above deck representative of remainder.
	Tower concrete above deck including precast tops & Cable anchorages	QDMR Level 1 – Visual inspection	Deck based
		QDMR Level 2 – Visual inspection plus photos to within 3 m or telephoto equivalent	Deck based with telephoto Plus check covered cable anchorages from cherry picker or mast climbing work platform

SAMPLE 2

Main Component	Sub Component	Activity	Access
		QDMR Level 3 – Chloride and Carbonation sampling and testing, selected locations Delamination survey, selected location	Cherry Picker or mast climbing work platform.
	Cable System NB Activities to be confirmed by specialist contractor	Load check	Cherry picker or mast climbing work platform or deck depending on final design
	Cable System	Visual inspection for corrosion, deflection, cracking, leakage of wax, impact or vandalism damage.	
		Progressive inspection,. Replacement of deteriorated components.	Cherry picker, mast climbing work platform or scaffold at top, from bridge deck at bottom.
		Measure tension on 4-8 cables as required	Cherry picker, mast climbing work platform or scaffold at top, from bridge deck at bottom.
	Steel support frame of tower top	QDMR level 3 – Close visual inspection plus testing of connections	Cherry picker, mast climbing work platform or scaffold at top, from bridge deck at bottom to access hatch.
Underside of deck	Steel structure	QDMR Level 1 – Visual inspection	Boat/foot and shore based
		QDMR Level 2 – Visual inspection plus photos to within 3 m or telephoto equivalent Spot coating repairs if any arising	Boat/foot and shore based with telephoto.
		QDMR Level 3 – Close visual inspection plus testing of connections	Gantry
		Grind rust spots, patch prime and top coat Monitor at QDMR 2 inspection	Gantry

SAMPLE 2

Main Component	Sub Component	Activity	Access
	Concrete deck soffit	QDMR Level 1 – Visual inspection	Boat/foot and shore based
		QDMR Level 2 – Visual inspection plus photos to within 3 m or telephoto equivalent	Boat/foot and shore based with telephoto.
		QDMR Level 3 – Close visual inspection plus testing for carbonation/chlorides	Gantry
	Services	Visual inspection for leakage	Boat/foot and shore based with binoculars
		Close up inspection	Gantry
		Replacement	Scaffolding/Gantry
	Bearings and joints	QDMR Level 1 – Visual inspection	Ground level/cherry picker
		QDMR Level 2 – Visual inspection plus photos to within 3 m or telephoto equivalent	Gantry
		QDMR Level 3 –	Gantry
		Replacement of joints	Scaffolding from ground level
		Coating of bearings	Scaffolding from ground level
		Replacement of bearings	Scaffolding from ground level

NB QDMR Level 2 Inspection interval extended to 5 years to align with other inspection and maintenance activities.

SAMPLE 2

Appendix A

Durability Design Summary Tables

SAMPLE 2

Table 1: Structure Durability Outline

Critical Durability Components	Service Life					
	Duration (years)	Contract Reference	Potential Mode of Failure	Deterioration Mechanism	Factors Controlling Rate of Deterioration	End of Life Criteria (Expected governing failure mechanism)
Pile caps in river - Pre cast units	100 years	Project Specification requires 100 year design life	Reinforcement Corrosion induced cracking, delamination and spalling of reinforced concrete	Chloride induced	Environmental characteristics Concrete mix Crack control Reinforcement cover As placed concrete quality Joint preparation	chloirde induced reinforcement corrosion - first crack
				Carbonation induced		
				Macrocell corrosion		
				Localised corrosion at cracks and joints		
Pile caps in river - In situ in fill concrete	100 years	Project Specification requires 100 year design life	Reinforcement Corrosion induced cracking, delamination and spalling of reinforced concrete	Chloride induced	Environmental characteristics Concrete mix Crack control Reinforcement cover As placed concrete quality Joint preparation	chloirde induced reinforcement corrosion - first crack
				Carbonation induced		
				Cracking due to heat of hydration		
				Localised corrosion at cracks and joints		
Pile caps	100 years	Project Specification requires 100 year design life	Sulfate attack of concrete.	Sulfate attack of concrete	Mobility of ground water	Softening of the concrete surface due to sulfate attack
			Weakening and erosion of concrete from acid sulfate soils.	Degradation of concrete and steel from acid	Degree of oxidation of PASS and extent of ASS.	
Piles in river	100 years	Project Specification requires 100 year design life	Sulfate attack of concrete.	Sulfate attack of concrete	Mobility of ground water	Softening of the concrete surface due to sulfate attack
			Weakening and erosion of concrete from acid sulfate soils.	Degradation of concrete and steel from acid	Degree of oxidation of PASS and extent of ASS.	
			Reinforcement Corrosion induced cracking, delamination and spalling of concrete	Chloride induced	Environmental characteristcis, concrete mix, crack control, reinforcment cover, as placed concrete quality	

Critical Durability Components	Service Life					
	Duration (years)	**Contract Reference**	**Potential Mode of Failure**	**Deterioration Mechanism**	**Factors Controlling Rate of Deterioration**	**End of Life Criteria (Expected governing failure mechanism)**
Piles	100 years	Project Specification requires 100 year design life	Sulfate attack of concrete.	Sulfate attack of concrete	Mobility of ground water	Softening of the concrete surface due to sulfate attack
			Weakening and erosion of concrete from acid sulfate soils.	Degradation of concrete and steel from acid	Degree of oxidation of PASS and extent of ASS.	
Main Piers in River	100 years	Project Specification requires 100 year design life	Reinforcment corrosion induced cracking and spalling of reinforced concrete	Chloride induced	Environmental characteristics Concrete mix Crack control Reinforcement cover As placed concrete quality Joint preparation	chloirde induced reinforcement corrosion - first crack
				Carbonation induced		
				Cracking due to heat of hydration		
				Localised corrosion at cracks and joints		
Other piers	100 years	Project Specification requires 100 year design life	Reinforcment corrosion induced cracking and spalling of reinforced concrete	Chloride induced	Environmental characteristics Concrete mix Crack control Reinforcement cover As placed concrete quality Joint preparation	carbonation induced reinforcement corrosion - first crack
				Carbonation induced		
				Cracking due to heat of hydration		
				Localised corrosion at cracks and joints		
Steel Deck Girders	100 years	Project Specification requires 100 year design life	General atmospheric corrosion leading to loss of section.	Atmospheric corrosion	Environmental Characteristics	Corrosion leading to a loss of section greater than 10%

Critical Durability Components	Service Life					
	Duration (years)	**Contract Reference**	**Potential Mode of Failure**	**Deterioration Mechanism**	**Factors Controlling Rate of Deterioration**	**End of Life Criteria (Expected governing failure mechanism)**
Precast Concrete Deck, T Roffs and Pre cast Tower Units	100 years	Project Specification requires 100 year design life	Reinforcement Corrosion induced cracking, delamination and spalling of concrete	Chloride induced	Environmental characterisitics, Concrete mix, Crack control, reinforcement cover, joint preparation, quality of concrete placement.	Carbonation induced reinforcement corrosion
				Carbonation induced		
				Localised corrosion at cracks and joints		
				Potential for DEF		
Tower Tip Steel Frame	100 years	Project Specification requires 100 year design life	General atmospheric corrosion leading to loss of section.	Atmospheric corrosion	Environmental Characteristics	Corrosion leading to a loss of section greater than 10%
Cable Stay System	100 years	Project Specification requires 100 year design life	Corrosion leading to loss of section	Atmospheric Corrosion	Environmental Characteristics	Corrosion leading to a loss of section greater than 10%
				Deterioration of Applied Coatings		
				Dissimilar metals		
Elastomeric Bearings	20 years	Project Specification requires 100 year design life	Cracking and/or bulging of bearing. Hardening of elastomer.	Oxidation and exposure to UV light.	Environmental characterisitics and composition of elastomer	Excessive bulging and/or cracking or splitting
Pot Bearings	20 years	Project Specification requires 100 year design life	Mechanical wear	Applied dynamic loads, static loads, movements.	Mecahnism life	Loss of movment capabiltiy
				Construction loads.	Design	
				Corrosion	Environamnatl characteristics, coating	
Expansion Joints	20 years	Project Specification requires 100 year	Loss of water tightness	Oxidation and cracking of elastomer	Environmental characterisitics and composition of elastomer	Puncture of elastomer
				Puncture of elastomer		
Hand Rails	20 years	Project Specification requires 100 year design life	General atmospheric corrosion leading to loss of section.	Atmospheric corrosion	Environmental Characteristics	Corroded appearnace, sharp havard to pedestrians

Critical Durability Components	Service Life					
	Duration (years)	Contract Reference	Potential Mode of Failure	Deterioration Mechanism	Factors Controlling Rate of Deterioration	End of Life Criteria (Expected governing failure mechanism)
Walkway Canopy	30 Years	Project Specification requires 100 year design life	General atmospheric corrosion leading to unacceptable appearance loss of section.	Atmospheric corrosion	Environmental Characteristics	Corroded appearnace
Concrete Barriers	100 years	Project Specification requires 100 year design life	Reinforcement Corrosion induced cracking, delamination and spalling of concrete	Chloride induced	Environmental characterisitics, Concrete mix, Crack control, reinforcement cover, joint preparation, quality of concrete placement.	Carbonation induced reinforcement corrosion
				Carbonation induced		
				Localised corrosion at cracks and joints		
Drainage System	100 years	Project Specification requires 100 year design life	Corrosion of hanging system	Atmospheric corrosion	Environmental Characteristics	Corrosion leading to a loss of section greater than 10%
			Corrosion of gratings and surrounds	Atmospheric corrosion/standing water	Environmental Characteristics	Corrosion leading to a loss of section greater than 10%
			UV embrittlement of PVC pipes	UV exposure	Environmental Characterisitcs	Cracking of pipe

Table Legend

Table 1 **Table 2** **Table 3** **Table 4** **Table 5** **Table 6** **Table 7** **Table 8**

Table 2: Component Exposure Assessment (All Components)

Environment No.	Environment Zone	Environmental Details	Affected Structural Components	Potential Deterioration Mechanisms	Code Requirements	Assess if Code Sufficient	Identified Additional Requirements
1	Below River Mud Line	Sulphate Concentration 250mg/l	concrete piles	Sulphate attack of concrete	50 MPa concrete	Cement content, water cement ratio and quantity of cement replacement inadequate to achieve 100 year life.	Concrete mix designed in accordance with BRE special digest 1.
		pH 6.5					
2	Within River below water level	Salinity of the river at this point has been based on BCC monitoring at Indooroopilly 10 year average of 10 ppt. Salinity at site has been taken as 10-15 ppt. Possible sulphate or low pH attack.	concrete piles, permananent steel casing used for constructuion. Remainds in palxce as sacrificial steel	Steel pile casing and/or concrete reinforcement corrosion potential due to saline water and oxygen supply.	50 MPa concrete	No.	steel casing estimated minimum time to perforation ~ 50 years
			pile caps		as above		
3	Below Water table	Fully submerged below water table. Oxygen restricted. Benign conditions in relation to chloride and sulphate concentrations in the soil.	concrete piles	No Potential Acid Sulphate Soils (PASS) have been identified by investigations to date but would require confirmation. Ground water likely to be saline to some extent.	50 MPa concrete	yes	none required.
			Pile cap				
4	In ground above water table or retaining ground	Benign conditions in relation of sulphate and chlorides in the soil. Periodic wettign and drying from groundwater movements / rainwater.	concrete piles	No Potential Acid Sulphate Soils (PASS) have been identified by investigations to date but would require confirmation, notably on the West Bank flood plain.	50MPa concrete	code requirements adequate	none required.
			Pile cap		as above		
5	Intertidal and Splash Zone	Chloride Concentration 1000mg/l	Pre cast skirt and "boat" units of the pile cap	Chloride Induced corrosion	AS5100.5 - 50MPa concrete, minimum cover 50mm	No.	Blended mix, 20% FA, 8% Silica fume. Maximum water binder ratio of 0.35 cover 70 mm
		Sulphate Concentration 250mg/l	steel cased concrete piles		as above	no.	steel casing estimated minimum time to perforation ~ 50 years
5a	Intertidal and Splash Zone behind pile cap skirts	Chloride Concentration 1000mg/l	Pre cast skirt and "boat" units of the pile cap	Affecting pile caps of bridge towers but not subject to constant wetting and drying. Salinity of the river as noted at Environment 2 above.	as above	no.	Options include: Silane coating; cathodic protection; demountable units.
		Sulphate Concentration 250mg/l	steel cased concrete piles	Steel casing will prevent deterioration of the concrete	not specified		
6	Spray Zone	Chloride Concentration 1000mg/l in river water. Potential for higher concentrations due to wetting and drying effects.	Pile caps in situ concrete	Minimal chloride spray above the pile cap established by Merivale Bridge Testing.	40 Mpa	No.	Large concrete pours for both, full thermal crack assesment to be carried out
		Sulphate Concentration 250mg/l	In river piers 2m above pile cap	Sulphate attack of concrete	none specified		Concrete mix designed in accordance with BRE special digest 1.

Environment		Environmental Details	Affected Structural Components	Potential Deterioration Mechanisms	Code Requirements	Assess if Code Sufficient	Identified Additional Requirements
No.	Zone						
7	High Spray Zone	Atmospheric exposure conditions similar to near coastal <50km from coastline as defined by ABDC.	Pier concrtete >2m above pile cap	In view of Merivale testing this area may now be taken as combined with Environment 8, Atmospheric within 50 km of the coastline. Considered as deck level and above.	40 MPa		
			Other piers	Carbonation induced reinforcement corrosion	none specified	No.	Cement content, water cement ratio and quantity of cement replacement inadequate to achieve 100 year life.
8	Atmospheric Exposure	Near Coastal < 50 km from coastline as defined by ABDC.	Steel deck girders	General atmospheric corrosion	none specified	No.	high quality coating applied in the workshop prior to installtion on site.
			Concrete Deck	Carbonation induced reinforcement corrosion	none specified	No.	Cement content, water cement ratio and quantity of cement replacement inadequate to achieve 100 year life.
			Parapets	Carbonation induced reinforcement corrosion	as above	as above	as above
			Handrails	General atmospheric corrosion	none specified		Stainless steel handrails
			Pot Bearings	General atmospheric corrosion of metallic parts	none specified		Provision to be made to allow replacement.

Table Legend

Table 1	**Table 2**	**Table 3**	**Table 4**	**Table 5**	**Table 6**	**Table 7**	**Table 8**

Table 3: Durability Provisions for Critical Components - Design Phase

Environment Classification No.	Environment Classification Zone	Affected Structural Components	Identified Deterioration Mechanisms	Design Mitigation Measure	Construction Contract Provision	Validation Measure/Test
2	Within river below water level.	Cast in situ bored piles	Chloride induced corrosion of reinforcement only above water level.	Sacrificial permanent 16 mm steel casing		QC measurement of casing thickness.
1	Below River mud line	Cast in situ bored piles	Sulphate or low pH attack.	40 MPa, Max w/c 0.4. Minimum 30% FA and 8% SF. Minimim 400kg/m3 cementitous content.	High workability mix suitable for placement by tremie tube.	Concrete mix dockets and cylinder results.
5	Intertidal and splash zone.	Pile cap pre cast skirts and boat units	Chloride induced corrosion of reinforcement.	Concrete to be 20% FA 8% silica replacement with appropriate mix design.Elastomeric barrier coating to the outer surfaces of the skirt units if diffusion results are higher than expected.	Appropriate curing before immersion	Mix dockets, cover meter and visual inspection
5	Intertidal and splash zone.	Pile caps	Chloride induced corrosion of reinforcement.	Concrete to be 40% FA replacement with appropriate mix design and monitoring to reduce temperature rise. Top surfaces to have 60 mm in situ cover. Curing membrane to be applied. Silane application only to the top surface Electrical continuity of reinforcement to facilitate impressed current Cathodic Protection to extend life if required.	Large mass of concrete, heavy reinforcement and deep section may lead to placement related difficulties. Risk of early age thermal induced cracking. Requirement for concrete curing. Engineer's approval required for early age thermal modelling of contractors proposed pour sequence and methodology. Hot weather concreting measures in specification. Self compacting concrete in deep sections to avoid need to man access for placement and compaction. Specification requirements for frequency of electrical continuity testing and resistivity criteria.	Electrical continuity testing during concrete placement. Thermo couples installed in pour to confirm modelling. Visual inspection of pile cap 48 hours after casting.
6	Spray Zone	Main in river piers	Chloride induced corrosion of reinforcement.	Testing at adjacent bridge indicates minimal chlorides in this area, considered Class B2		
8	Atmospheric exposure	Steel Deck Girders	Atmospheric corrosion	Maximise surface prpararation, shop blasting to Sa 2.5. Application of maximum quality/thickness coating system to maximise time to first maintenance.	Nominate surface preap and coating system	Engage NACE coating inspector to confirm surface preparation, dft and no "holidays" before erecton.

Environment Classification		Affected Structural Components	Identified Deterioration Mechanisms	Design Mitigation Measure	Construction Contract Provision	Validation Measure/Test
No.	Zone					
8	Atmospheric exposure	Cable stay system	Atmospheric corrosion	Require specialist supplier to provide details of inspection procedure at ends and along cable length combined with capability to remove and repalce one cable at a time without affecting the bridge	Nominate procedure	Demonstrate procedure and cable replacement as part of the contract

Table Legend

Table 1	Table 2	Table 3	Table 4	Table 5	Table 6	Table 7	Table 8

Table 4: Maintenance Intervention Assumptions (to achieve service life)

Element			Servicing			Cyclic			
Affected Structural Component	**Environment No.**	**Zone**	**Activity**	**Frequency**	**Access Provision**	**Intervention Level**	**Activity**	**Frequency**	**Access Provision**
Steel Bearings	8	Atmospheric Exposure	Cleaning bearings	2 years	Cherry picker from road adjacent	Rust staining on bearings	Recoat bearings	10 years	Cherry picker from road adjacent
Steel girders	8	Atmospheric Exposure	Wash girders,	2 years	Gantry provided by contractor and stored at client yard	Greater than 10% of the surface area affected by bubbling, peeling or rust staining or already spot coated	Repainting	20 years	Gantry provided by contractor and stored at client yard
			patch painting	5 years	Gantry				
Expansion joints	8	Atmospheric Exposure	clean joints	1 year	Deck				
Cable stays	8	Atmospheric Exposure	Load check	3 years	Mast climbing work platform and deck				
Hand rail stanchion	8	Atmospheric Exposure	Wash	2 years	Deck	Greater than 10% of the surface area affected by bubbling, peeling or rust staining.	Recoat	30 years	Deck
Canopy	8	Atmospheric Exposure	Wash	2 years	Deck	Greater than 10% of the surface area affected by bubbling, peeling or rust staining.	Recoat	30 years	Deck

Table Legend

Table 1	Table 2	Table 3	Table 4	Table 5	Table 6	Table 7	Table 8

Table 5: Inspection & Access Provisions

Element			Condition State Guidelines	Inspection Requirement	Supplementary Requirements to BIM	Access	Safety Hazards	
Affected Structural Component	**Environment No.**	**Zone**					**Description**	**Mitigation**
River Piers	5,6	Tidal, splash and spray	Reinforcement Corrosion induced cracking, delamination and spalling of reinforced concrete	Annual - QDMR Level 1 – visual inspection. At 5 years - QDMR Level 2 –Visual inspection plus photos to within 3m or telephoto equivalent. At 10 years QDMR Level 3 inspection		Boat, foot, telephoto lens		
Remaining Piers below deck	8	Atmospheric	Reinforcement Corrosion induced cracking, delamination and spalling of reinforced concrete	Annual - QDMR Level 1 – visual inspection. At 5 years - QDMR Level 2 –Visual inspection plus photos to within 3m or telephoto equivalent. At 10 years QDMR Level 3 inspection		Foot, telephoto lens, sampling at base only		
Tower tip precast	8	Atmospheric	Reinforcement Corrosion induced cracking, delamination and spalling of reinforced concrete	Annual - QDMR Level 1 – visual inspection. At 5 years - QDMR Level 2 –Visual inspection plus photos to within 3m or telephoto equivalent. At 10 years QDMR Level 3 inspection		Foot, telephoto lens, cherry picker.		Close road while inspection in progress.
Cable Stay System	8	Atmospheric	Corrosion	Annual - QDMR Level 1 – visual inspection. At 5 years - QDMR Level 2 –Visual inspection plus photos to within 3m or telephoto equivalent.		Foot, telephoto lens, mast climbing work platform.		Close road while inspection in progress.
Steel deck girders	8	Atmospheric	Corrosion, spot rust	Annual - QDMR Level 1 – visual inspection. At 5 years - QDMR Level 2 –Visual inspection plus photos to within 3m or telephoto equivalent.	Raise any spot rust for action in next maintenance interval, likely after 15 years. Assess % affected as recoating trigger.	Foot/telephoto for Level 1 Gantry provided by contractor and stored at client yard for others		Close road while gantry hoisted.
Underside of precast deck units and precast bridge beams	8	Atmospheric	Reinforcement Corrosion induced cracking, delamination and spalling of reinforced concrete	Annual - QDMR Level 1 – visual inspection. At 5 years - QDMR Level 2 –Visual inspection plus photos to within 3m or telephoto equivalent. At 10 years QDMR Level 3 inspection		Foot/telephoto for Level 1 Gantry provided by contractor and stored at client yard for others		Close road while gantry hoisted.

Element			Condition State Guidelines	Inspection Requirement	Supplementary Requirements to BIM	Access	Safety Hazards	
Affected Structural Component	Environment No.	Zone					Description	Mitigation
Services	8	Atmospheric	Cracking and leakage or corrosion of supports	Annual - QDMR Level 1 – visual inspection. At 5 years - QDMR Level 2 –Visual inspection plus photos to within 3m or telephoto equivalent.		Foot/telephoto for Level 1 Gantry provided by contractor and stored at client yard for others		

Table Legend

Table 1	Table 2	Table 3	Table 4	Table 5	Table 6	Table 7	Table 8

Table 6: Replacement of Components

Element			Replacement Frequency (years)	Condition State Guidelines	Design Provisions	Maintenance Plan & Drawing References	Access	Safety Hazards	
Affected Structural Component	Environment No.	Zone						Description	Mitigation
Steel Bearings	8	Atmospheric Exposure	20	Corrosion of holding down bolts. Extrusion of PTFE.	Provision of bearing plinths and jacking points for lifting deck and replacement of bearings.	Method statement for bearing replacement and drawings provided. Drawing ref 1234/D/567	Paved walkway in front of abutment can be used as working platform.	Slip and trip hazards	Defined access provision and paved walkway
Pot Bearings	8	Atmospheric Exposure	30	Mechanism seizing	Provision of bearing plinths and jacking points for lifting deck and replacement of bearings.	Method statement for bearing replacement and drawings provided. Appendix A, report 320359	Travelling gantry below bridge.	falls from height	Safety harnesses to be connected to the gantry.
Exapsion Joints	8	Atmospheric Exposure	20	Elastomer split	Bolted connections	Method statement and drawings provided. Drawing ref 9876/a	Travelling gantry below bridge.		Traffic management to include safe working area.
Hand Rails	8	Atmospheric Exposure	20	Any sharp hazard due to corrosion, aesthetics at the time	Bolted connections	Coating as needed to stanchions, uisnpection at QDMR Level 2	Footpath on deck	Falls from height, and being hit by passing traffic.	Traffic management to include safe working area.
Canopy	8	Atmospheric Exposure	30	Perforatioon or aesthetic appearance	Bolted connections	Coating as needed, inspection at QDMR Level 2	From deck	Hit by passing traffic	Traffic management to include safe working area.

Table Legend

Table 1	**Table 2**	**Table 3**	**Table 4**	**Table 5**	**Table 6**	**Table 7**	**Table 8**

Table 7: Proprietary Product Records

Component Description	Manufacturer Details	Product reference	Servicing Requirements	Warranty	Installation Details
Deck Expansion Joint	To be completed on finalisation of detailed design				
Bearing	as above				
Expansion Joints	as above				
Waterproofing	as above				
Barriers	as above				
Precast Concrete Products	as above				
Steel Fabrication	as above				

Table Legend

Table 1	Table 2	Table 3	Table 4	Table 5	Table 6	Table 7	Table 8

Table 8: Durability Provisions for Critical Components - Construction & Service Phases Records and Departures

Environment		Affected Structural Components	Identified Deterioration Mechanisms	Design Mitigation Measure	Construction Contract Provision	Validation Measure/Tests (departures to be added)	Intervention Level	Construction or Repair Method	Validation Records		Defect Records	
No.	Zone								Location	Description	Location	Description
											Panle 3, North Pier Pile Cap	Shrinkage crack at box out due to panel left in mould over weekend

Table Legend

Table 1	Table 2	Table 3	Table 4	Table 5	Table 6	Table 7	Table 8

GHD Pty Ltd ABN 39 008 488 373

201 Charlotte Street Brisbane Qld 4000
GPO Box 668 Brisbane Qld 4001 Australia
T: (07) 3316 3000 F: (07) 3316 3333 E: bnemail@ghd.com.au

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Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date

Appendix D: Example Concrete Structure

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The information contained in this sample report is for indicative purposes only, and should not be construed as a Main Roads Standard or Directive. Project specific design criteria will take precedence at all times.

Department of Main Roads

Report for Durability Guideline Document

Sample Durability Plan

INFRASTRUCTURE | MINING & INDUSTRY | DEFENCE | PROPERTY & BUILDINGS | ENVIRONMENT

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1. Introduction

1.1 General

The contents of this report are provided as an example format for preparation of a Durability Plan Report. The project specific criteria referenced in this report and the project specific design solutions adopted do not constitute a Main Roads Standard or Directive.

1.2 Project description

The proposed bridge spans approximately 75 m across Wallace creek, adjacent to the mouth of the Burnett River, Bundaberg. The location of nearby open saline water (Hervey Bay) is approximately 500 m to the East of the Wallace Creek Bridge. The form of construction comprises precast concrete piles, in situ concrete abutments and headstocks and a prestressed concrete beam deck. The bridge comprises 5 spans, each approximately 15m long.

1.3 Scope of this Durability Plan Report

This Durability Plan Report (DPR) has been prepared in accordance with the requirements of Scope of Works and Technical Criteria (SWTC) Clause 3.4(c) Durability Plan. This report satisfies the requirements for the provision of a Durability Plan for the achievement of the specified design lives, based on the technical details included and stage of works covered by this report.

This report presents an assessment of the environment that the structures will be exposed to and establishes minimum performance requirements to comply with the design life requirements. This report incorporates a number of specific requirements of the SWTC Clause 3.4 that are intended to enhance the overall durability of the completed project.

1.4 Durability Performance Requirements

The durability performance key requirements are given at Clause 3.4(b) of the Scope of work and Terms and Conditions (SWTC) and are restated below.

1.5 Durability Related Excerpt from the SWTC

As a minimum, durability standards and guidelines for the various materials and components used in all permanent structures must be in accordance with the Bridge Code, with the following additional requirements:

a. Dense, durable high strength concrete must be used in structural elements constructed of concrete. The minimum strength concrete to be used in the Project Works must be Special Class 40MPa, except for blinding and mass unreinforced concrete which must have a minimum concrete strength of 20MPa. In areas of severe exposure (Bridge Code classification B2 and above), supplementary cementitious materials, such as blast furnace slag, fly ash or silica fume or both, must be used if feasible and if it assists in achieving the required durability.

b. Concrete mixes must be properly designed, especially to safeguard against the possibility of alkali silicate reaction.

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c. High performance coatings must be used on exposed steelworks. Hollow steel or composite steel members must include drain holes and all interior surfaces must also receive protective coatings.

d. Attention must be given to deterioration of elements, which cannot be easily accessed for maintenance or repair. The design must ensure that the durability of any such element and the minimum design life applicable to the whole structure are attained without maintenance.

e. Special measures must be taken to minimise the possible deleterious effects of heat of hydration stresses in thick concrete sections, e.g. by the use of selected supplementary cementitious materials or special curing/insulating regimes or both.

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2. Design Life

2.1 Component Design Life

SWTC Clause 3.2 component design life requirements are given in Table 3.1 and are restated in Table 1 for those items covered by this durability plan report. Components not included in the SWTC but considered by this DPR are given in Table 2.

Table 1 Design Life of Components Stated in SWTC

Component	SWTC Minimum design life (years)
Bridge works	100
Protective coatings to steelwork	25
Lighting	40
Deck expansion joints	40 years for metallic components 20 years for rubber components
Bearings	40 years where replacement is possible

Table 2 Design Lives of Components Not Stated in SWTC

Component	Service life (years)	End of life criteria
Piles	100	Cracking due to reinforcement corrosion. Loss of cover concrete due to sulphate attack.
Headstocks and Abutments	100	Cracking due to reinforcement corrosion.
Deck Girders	100	Cracking due to reinforcement corrosion.
Traffic barriers	50	Surface corrosion
Balustrade and Safety Rail	50	Surface corrosion
Parapets	100	Cracking due to reinforcement corrosion

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3. Environmental Exposure Classifications

3.1 Environment Designation and Description

Various types of environments are identified and described in Table 3, along with the structure component that is affected.

Table 3 Environment Descriptions

Environment	Designation	Description	Component Affected
Environment 1	Soils below permanent water	Soils immediately below the creek.	Piles
Environment 2	Within permanent water	Elements of the structure permanently submerged below the water line and above the creek bed.	Piles
Environment 3	Natural ground below water table	The local soils below the peak of the water table. A region of these soils will be subject to season wetting and drying with the rise and fall of the water table.	Piles
Environment 4	Natural ground above water table	The local soils above the peak of the water table. However, still subject to contamination and wetting due to flood waters, irrigation and precipitation. Chloride content within soil assumed in the absence of groundwater and soil chemical tests.	Pile and abutments
Environment 5	Specified fill above water table	Fill with no PASS or significant residual salts (e.g. no chlorides, no sulphates) and neutral pH.	Abutments
Environment 6	Tidal and Splash zone	The region surrounding the mean water level subject to changes in the water level. Includes seasonal variations, frequent flooding and splashing from water traffic.	Piles
Environment 7	Spray zone	Region immediately above the intertidal and splash zone. Only considered in locations where wave action generates spray.	Abutments and headstocks
Environment 8	Atmospheric exposed	Encompasses the atmospherically exposed areas that are subject directly or indirectly to rainfall.	Deck, bearings and expansion joints

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3.2 Reinforced Concrete Exposure Classifications

a) The exposure classifications of the various environments have been assessed in accordance with AS5100.5-2004 Table 4.3.

b) Alternative criteria are given in AS3600 (Reinforced Concrete) and AS2159 (Piling). However, these standards only give guidance for achieving a design life of 40-60 years, while AS5100 accounts for a design life of 100 years.

c) The exposure classification determined for the bridge are summarised below in table 4.

Table 4 Environment Descriptions

Component	Classification as given in AS 5100 or AS2159	Durability Provisions
Piles	B2	50 MPa Concrete
Abutments	B2	50 MPa Concrete
Deck	B1	40 MPa Concrete

3.3 Groundwater and Creek Corrosion Aggressivity

The water quality data from bore holes and the creek is provided in the tender geotechnical survey report. The chloride content of the creek is 1800mg/l, and sulphate content is 200mg/l.

The assumed surface chloride levels significantly influence durability modelling of concrete structures in chloride environments. Surface chloride concentrations are primarily related to exposure conditions, assumed surface chloride levels are listed in Table 5 based on previous project experience on an existing bridge with similar proximity to the open saline water.

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Table 5 Assumed Surface Chloride Levels

Exposure environment	Bridge Element Area	Surface Chloride Level (% wt conc.)
Atmospheric	Deck soffit and above	0.5 %
Splash zone	Piers above mean high water level Headstock soffits & Abutment base at HAT	2.0 %
Tidal Zone	Pier surface from mean low water level to mean high water level	1.0 %
Saturated / Submerged	Piers below mean water level	1.0 %

3.4 Creek and Ground Water Levels

Creek and ground water levels given in the tender geotechnical report are used for durability assessment in this report.

- Highest astronomical tide level is 0.3 m below the underside of headstock and abutment level.
- Mean high water spring level is 1.024m below the underside of the headstock and abutment level.
- Mean low water spring level will expose the full length of the piles above the creek bed.
- The ground water table in the vicinity of the bridge is expected to mirror the tide height.

3.5 Location of Open Saline Water

The location of nearby open saline water is approximately 250 m to the East of the Wallace Creek Bridge.

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4. Deterioration Mechanisms

Deterioration mechanisms for reinforced concrete and steel materials are summarised in Table 6.

More detailed general overview durability issues included in this report as follows:

- Reinforced concrete durability at Appendix B.
- These deterioration mechanisms were used as the basis for identifying the project specific potential durability issues to be addressed in this report.

Table 6 Deterioration Mechanisms Summary

Construction Elements	**Deterioration Mechanism**	**Factors Controlling Rate of Deterioration**
Reinforced and Prestressed Concrete	Chloride induced Corrosion Carbonation induced corrosion Macrocell corrosion Localised corrosion at cracks and joints Thermal/Restraint and Shrinkage Cracking	Environmental characteristics Concrete mix quality Crack control Reinforcement cover As placed concrete quality, particularly cover, compaction and curing Joint preparation Coatings
Concrete	Acid Sulphate Soils Alkali-Aggregate Reactivity Sulphate attack Acid attack from contaminated groundwater and soil Soft Water Attack Delayed Ettringite Formation Thermal/Restraint and Shrinkage Cracking	Environmental characteristics Aggregate properties Crack control Binder type Concrete quality Surface preparation Construction methods
Exposed Steel and Other Metallic objects	Corrosion Coating/protection failure Cathodic protection inference Stray currents Fatigue Wear	Environmental characteristics Material selection Dissimilar metals Detailing Electrical insulation Coating selection/maintenance
Movement Joints	Excessive movement leading to poor running surface Concrete cracking, delamination and	Construction sequence (particularly approaches and prestress girder construction/erection lag time)

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	spalling Wear Corrosion	Capacity for replacement Design detail Workmanship Prevention of water penetration Reinforcement detailing Traffic load and volume
Bridge Bearings	Construction loading Environmental degradation Wear	Construction quality –line, level, planarity Capacity for replacement Positioning Protection Design Traffic load and volume
Coating	Vandalism deterioration Coating failure by cracking, flaking, peeling, spalling Reinforced concrete corrosion induced defects by lack of coating environmental barrier resistance (where appropriately required)	Design detail Workmanship Material quality and barrier resistance properties Process control – hydrogen porosity in galvanizing parent metal
Waterproof Membranes	Construction loading Environmental degradation	Design detail Construction process control to prevent damage to membrane such as during DDWS placement Workmanship Prevention of water penetration Material quality

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4.1 Reinforced Concrete Elements

4.1.1 General

The following sections identify the expected dominant durability degradation processes that are expected to govern the durability design.

4.1.2 Concrete Degradation

a) Acid Sulfate Soils (ASS)

i) Acid Sulfate Soils (ASS) are naturally occurring soils containing pyrites, or chemical precursors of pyrite, which have begun to oxidise through exposure to oxygen. When water passes through ASS, sulphuric acid is leached out. Potential Acid Sulfate Soils (PASS) are similar to ASS in nature but are in an unoxidised state. Engineering operations on ASS and PASS, such as excavation, dredging and draining accelerate the exposure of pyritic material to air and speeds up the production of acidic waters.

ii) Critical elements subject to ASS are precast concrete piles and cast in-situ abutments. Precast concrete piles in particular are prone to ASS attack, and therefore may require isolation using a membrane or coating. Imported clean fill will be used to back fill around abutments, therefore eliminating the risk of degradation from ASS.

iii) Table 7 summaries the findings from the preliminary PASS Assessment from tender stage. Further testing of pH is recommended to confirm site conditions and the need for protective coatings to piles.

Table 7 Acid Sulfate Soils Assessment Summary

Reference Location	Assessment Summary	ASS Likelihood	PASS Likelihood	Isolation Required
BH 01	pH generally > 5.5 pH_{fox} generally 3.0 to 6.0 Sulphate content up to 0.05%	Possible	Possible	No

b.) Sulphate Attack

The expected levels of sulphate in the soil and ground water are not considered to be a significant cause of degradation for the buried structures. In addition the chlorides in the groundwater will assist in mitigating the risk of sulphate attack. The cement contents in excess of 400 kg/m^3 combined with the use of fly ash will further mitigate sulphate attack.

c.) Alkali Aggregate Reaction (AAR)

In order for AAR to occur, there must be sufficient alkali in the concrete and reactive silica in the aggregate together with moisture to cause the expansion. The approach to mitigate AAR risk is to eliminate one or more of the factors. This is achieved by limiting the alkali content in the concrete mix to 2.5 kg/m^3 and the inclusion of a minimum 20% fly ash in the concrete. These requirements have been incorporated into the Concrete Specification.

d.) Backfill & Soil Chemical Induced Deterioration

The back fill materials may contain unacceptable contaminants such as chlorides and sulphates that induce reinforcement corrosion or directly attack the concrete. These materials must therefore be tested for compliance to the specification. As a minimum the tests relevant for reinforcement concrete shall include pH, sulphate and chloride content.

e.) Delayed Ettringite Formation (DEF)

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To mitigate the concerns relating to DEF, the peak concrete hydration temperature must be limited. It is well established that DEF is unlikely to occur if the concrete temperature is kept below 65^{o}C for GP cement and 75^{o}C for concrete with secondary cementitious materials such as fly ash. Particular risk exists for precast concrete piles and precast deck units due to the steam curing process. Therefore, it is required to limit concrete temperature of 65^{o}C for precast elements.

4.1.3 Reinforcement Corrosion

a.) Carbonation Induced Mechanism

The combination of cement content, use of fly ash, low water cement ratio and cover will ensure that carbonation induced corrosion of the steel reinforcement will occur well beyond the 100 year service life for concrete elements.

b.) Chloride Induced Mechanism

The primary risk of reinforcement corrosion would be chloride induced corrosion. Concrete cover and permeability would be the prime factors determining the service life of the concrete. Chloride diffusion coefficient has been specified for some critical elements at risk and this has been checked against design concrete cover with respect to their exposure environment using MTG in-house chloride modelling software enabling evaluation of the concrete mixes required to achieved to the 100 year design life.Appendix B lists the theoretical background behind the chloride modelling. Table 8 list the assumptions and input parameters for the modelling. Table 10 lists the concrete mix parameters for the project.

The results of the analysis are summarised below. The expected life is the time to cracking (from construction) of the cover concrete

c.) Acceptably Compacted Concrete Cover

Carbonation and chloride induced reinforcement corrosion both require cover concrete to be well compacted with no unacceptable voids, cracks, honeycombing or other defects which would be confirmed by visual inspection and quality control testing.

All cast in situ elements are to be water cured following formwork removal for a minimum 7 days. Precast elements shall be steam cured followed by application of a 90% efficiency curing compound.

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Table 8 Durability Modelling Parameters

Parameter	Design Value	
Surface Chloride Concentration (% by weight concrete)	**Atmospheric**	**0.5 %**
	Splash zone	2.0
	Tidal Zone	1.0%
	Submerged	1.0%
Average Temperature	28^{o}C	
Grade of Concrete	S50 for insitu and precast concrete. S80 for piles	
Corrosion Activation Threshold* (% by weight concrete)	0.06% for reinforcement 0.04% for prestressed reinforcement	
Cement content (kg/m3)	Minimum 400 kg/m^3	
Cover	50 (35 mm deck units)	
Bar diameter	Element specific	

Table 9 Chloride Modelling Results

Exposure environment	Bridge Element Area	Time Corrosion Initiation (years)	Time to Cracking (years)
Atmospheric	Deck soffit and above	100	140
Splash zone	Piers above mean high water level Headstock soffits & Abutment base at HAT	85	110
Tidal Zone	Pier surface from mean low water level to mean high water level	90	120
Saturated / Submerged	Piers below mean water level	90	140

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Table 10 Concrete Mixes Summary

Concrete Class	S50a	S50b	S80
In-situ / Precast	Insitu	Precast	Precast
Specific applicable structural element(s)	Abutments and Headstocks	PSC Beams	Precast Concrete Pile
Minimum Cover (mm) (Note 4)	50	35	50
Nominated strength (f'_c MPa)	50	50	80
Binder type (Note 1)	GB	GB	GB/SF
Min cement content (kg/m^3)	400	400	400
Max cement content (kg/m^3)	450	480	550
Max water/binder ratio	0.40	0.38	0.28
Max aggregate size (mm)	20	20	20
Chloride diffusion coefficient (x10^{-12} m/s^2) (Note 2)	2.0	2.0	1.5
Max shrinkage ($\mu\varepsilon$)	600	600	600
Nominal Slump (mm) (Note 3)	80	80	180
Expected Service Life (years)	100	100	100

1) Cement type designations are in accordance with AS 3972. Type GB cement shall contain a minimum 20% fly ash. Pile concrete shall contain a minimum 20% FA and 5% silica fume.

2) Tested by NT Build 443, commencing at 28 days age for GP cement and at 56 days age for blended cements.

3) Tolerances per AS 1379 Table 6. It should be noted that higher slumps than those shown above may be specified for particular pours, for example where reinforcement is congested or for deep pours. Slump higher than Table 1 shall be achieved by use of appropriate admixtures and not by increasing water content and cement content.

4) Stipulated cover is minimum cover. Nominal cover will need to include a tolerance of ±10 mm for insitu concrete and ±5 mm for precast concrete.

5) Limit steam curing temp to 75^{o}C

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4.1.4 Early Age Concrete Thermal/Restraint Behaviour and Crack Risk Assessment

Preliminary assessment of the reinforcement provisions and expected construction method for early age thermal assessment has been undertaken in accordance with CIRIA C660 the results are listed in Table 11. The following conclusions from the analysis can be drawn:

- Early age thermal cracking is likely to occur.
- C660 predicts that the crack widths at early age will be between 0.13 and 0.18 mm wide. In the long term some of these cracks may open up to between 0.21 and 0.28 mm but most are likely to remain below 0.2 mm.
- The reinforcement provided for crack control exceeds the minimum required by AS 5100

A further assessment should be undertaken once the method of construction, timing and concrete mixes are finalised. A hot block shall be cast to obtain adiabatic temperature data for more accurate prediction of cracking risk.

4.2 Bridge Bearings

Bridge bearings are normally expected to be well protected from the elements by virtue of their position on the structure. Water ingress through movement joints will be prevented by the elastomeric gland. Moisture collection around the bearing is a possibility, particularly at the more exposed outer bearings and if the movement joint fails allowing water ingress through the joint. This may cause localised degradation, and overtime can lead to the build up of detritus that can promote plant growth. It is proposed to use elastomeric bearings, which can degrade due to UV and moisture exposure. The bearings will be installed on plinths to prevent direct contact with water on the bearing shelf, and facilitate inspection. In addition the bearing shelf shall be sloped to facilitate drainage and rapid drying.

4.3 Deck Movement Joints

a) Deck movement joints will be fabricated from extruded metal with an elastomeric gland. Industry common usage and suppliers experience indicates that the glands are the component with the shortest life but are expected to last at least 25 years. The deck joint assemblies are bolted to the deck and designed to facilitate future replacement.

b) The deck movement joint life would be extended by keeping the joints clean of dirt and debris, which collects between the gland surfaces, and can lead to puncture of the gland. Regular cleaning will extend their life.

4.4 Waterproof Membranes

a) There is a risk of corrosion to the top steel in the upper surface of the deck units. As this surface cannot readily be inspected or repaired without significant disruption to the use of the bridge a secondary protective measure of a waterproof coating, Bituthene or similar, will be applied.

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Table 11 Summary of Early Age Concrete Thermal/Restraint Crack Risk Assessment

Case	Case Description	Concrete Mix Design	Restraint	Peak Temperature (^{o}C)	Predicted Strain (με)	Crack Control Reinforcement (provided)	Area of steel per face (provided)	Reinforcement required (Early age cracking, long term cracking) mm2	Early age Thermal Cracking (predicted width)
1	Abutment Pour 1200 mm thick	50 MPa (25% FA)	0.56	71	192 105	N28 @ 150 EF	4105	2125,3553	0.13 @ 700 centres 0.21
2	Headstock Pour 950mm thick	50 MPa (25% FA)	0.72	71	260 146	N28 @ 150 EF	4105	2125, 3553	0.18 @ 700 centres 0.28 @ 700 centres

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4.5 Coating Systems

The performance of coating systems varies considerably depending on the materials selected. The following highlights the coating systems used for various elements:

4.5.1 Protective Membrane/Coating for Pile Caps and Precast Concrete Piles

The protective membrane or coating for precast concrete piles is to isolate the concrete from aggressive acid sulphate soil at selected environmental exposure. The location and extent of coating will be confirmed following confirmation of site conditions. The fundamental requirement for this coating is to resist a low pH, acid sulphate soil environment and attack to the concrete surface in the long term, since the coating is not maintainable. Recommended coating systems include Line-X XS-350 polyurea spray or equivalent system with extreme chemical and abrasion resistance.

4.5.2 Corrosion Protection Coating for Street Furniture

All exposed metal components for lighting and signage poles will be hot dipped galvanised or primed with inorganic zinc silicate. The coating will be touched when minor rust staining is observed during routine servicing exercises. Some minor corrosion would be expected to develop within 25 years service life of the coating. Hot dip galvanising in accordance with AS2312:2002 for a class D environment is recommended (i.e. HDG900)

4.6 Metal Roadway Elements

Items such as balustrades and safety rails requiring a 50 year design life without maintenance will be fabricated from aluminium. Galvanised steel is not expected to provide the required performance given the likelihood of salt spray exposure. Grade 316 stainless steel can be considered as an alternative to aluminium by the Client for these items as in addition to good corrosion resistance they will provide a high level of aesthetic performance.

SAMPLE

5. Durability Assessment by Component

5.1 Introduction

A durability assessment was undertaken of each component of the bridge structure. The dominant deterioration mechanism was identified and protective measures evaluated.

5.2 Piles

Concrete piles will be subject to potential acid sulphate soils below ground and saline ground water, saline water in the creek and salt spray above ground.

5.2.1 Within the Creek

The piles will be precast concrete and installed by driving into the creek bed. Splices between piles will be formed by galvanised steel dowels embedded in 400 mm long core holes. The dowels will be fixed using epoxy resin. The splice will be protected with a grade 250 hot dipped galvanised steel sleeve. The sleeve will be grouted into place with epoxy resin.

The soil below the creek bed and below the permanent water table will remain saturated with water and as such the risk of oxidation of the PASS and associated liberation of acid is considered negligible (environment 1 table 3).

The sections of the piles exposed to water within the creek will be exposed to saline conditions and as such there is a risk of reinforcement corrosion. However where the pile is permanently submerged the corrosion of the steel reinforcement will be restrained by the availability of oxygen (environment 2).

Above the water line in the intertidal and splash zone (environments 6 and 7) the piles will be exposed to periodic wetting and drying. This will be the most corrosive exposure environment due to the potential for build up of salts on the surface of the pile due to cyclic wetting and drying and the ready availability of oxygen. This environment will govern the durability design of the piles.

5.2.2 Landside

The piles driven for the abutments are subject to environment 3 and 4. Areas below the water table are restricted from deterioration for chloride induced reinforcement corrosion owing to the lack of oxygen. Similarly this will restrict oxidation of PASS. For areas above the permanent water table ASS and chloride induced corrosion are a possibility.

Degradation of the concrete due to sulphate attack is also a possibility.

5.3 Abutments and Headstocks

The cast insitu abutments and headstocks are located in environment 7 and as such are exposed to salt water spray and splash. To prevent the build up and poling of water on horizontal surfaces the bearing shelf will be sloped to facilitate water shedding and rapid drying.

The risk of ASS to the abutment is considered negligible as clean imported fill will be used to backfill behind the abutment. Drainage provision behind the abutment will prevent washout of material.

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5.4 Scour Protection to Abutments

Scour protection to both abutments will be provided in accordance with Main Roads detail MRD1117.

5.5 Bearings

Bearings will be elastomeric, set atop 200 mm high bearing plinths. These plinths will assist with keeping the bearing clear of water on the bearing shelf, facilitate inspection and allow sufficient space for jacking to enable bearing replacement. The bearings will be set into a recess in the plinth to prevent excessive lateral movement or “walking”. The horizontal and vertical loading relationship including management of the lag time between PSC girder manufacture and erection must be controlled. The expected service life of the bearing is 25 years, owing to hardening of the elastomer due to environmental exposure.

5.6 Precast Parapets and Deck Units

The deck units and parapets are precast prestressed concrete beams. The deck units are above the water level in the creek and as such are relatively sheltered (exposure environment 8). The top surface of the deck units which will potentially be exposed to moisture penetrating through the deck wearing surface will be additionally protected with a waterproof coating.

The cut ends of the stressing tendons will be recessed and protected with a greased end cap, and sealed with epoxy resin. This epoxy coating will be inspected on site following delivery and installation, and any damage repaired.

To maintain the sheltered exposure environment under the bridge drip checks will be provided to underside of parapet units on outer edges. This will prevent water running down the sides of the parapets and tracking onto the soffit.

5.7 Expansion Joints

The expansion joints will comprise an extruded metal plates and elastomeric gland. The gland is expected to have a service life of 25 years, and is designed to facilitate replacement. The service life of the gland can be reduced if debris, which can puncture the gland, is not removed during annual servicing.

5.8 Ballustrade

The balustrade will be fabricated from aluminium. To prevent alkaline induced corrosion of the aluminium the base plate will be insulated from direct contact with the concrete.

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6. Construction Stage Verification Plan

6.1 General

Achievement of the durability objectives are contingent on the insitu construction meeting particular protective measures.

Table 12 Verification Plan

Component	Durability Criteria	Method of Verification	Acceptance criteria
Abutments, Precast deck units, precast piles and headstocks.	Concrete grade	Cylinder Strengths	Compliance with specification
	Cover	Cover meter survey	Meet minimum cover requirements
	Cracking / Honeycombing	Visual inspection	No honeycombing. No cracks greater than 0.2 mm in width
Prestressed concrete deck units	Epoxy coating to cut ends of tendons	Visually inspected at precast yard, delivery to site and following installation.	Epoxy coating free from damage.
Waterproof deck coating	No defects, blisters, pin holes. Adequately adhered to concrete surface	Visual inspection. Adhesion testing.	Compliance with specification

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7. Inspection Requirements

7.1 General

The inspection requirements for the bridge are taken from QDMR Bridge Inspection manual.

7.2 Summary of Inspection Programme

7.2.1 Level 1 inspection

The level 1 inspection is a visual inspection from accessible locations. The deck, expansion joints, parapets, balustrade and crash barrier can be inspected from the deck and pedestrian footway. If access to the hard shoulder is required traffic management will be required to warn road users of working locations.

Access to the abutments and bearings from deck level is provided by stairs constructed into the scour protection. A flat a paved walkway is provided immediately in front of the abutment for safe access. The elastomeric bearings are located on bearing plinths to facilitate cleaning and inspection.

Inspection of headstocks and headstock bearings will be from the shore using binoculars or telephoto camera.

The underside of the deck will be visually inspected from the shoreline using binoculars or telephoto camera.

7.2.2 Level 2 & 3 Inspection

The level 2 and 3 inspections require access to be able to touch the structure.

Access to the deck will be from the deck surface. Traffic management to close one lane will be required. It is envisaged that “Stop” and “go” lollipop men positioned at each end of the bridge will be used to control traffic flow to one lane across the bridge. This will facilitate close inspection of the condition of expansion joints, balustrade fixings, balustrade and crash barrier condition.

Access to the abutments and bearing will be as for level 1 inspection.

Access to the headstocks will be via boat. The inspection will take place at high tide to facilitate access to the headstock and bearings. Inspection at low tide will facilitate inspection of the piles in the tidal zone.

Diver inspection of the submerged portions of the piles may be required to check for scour, depending on the timing of the inspection. However it is envisaged that inspection of the piles will be timed to coincide with mean spring low water when the water level will drop sufficiently to allow inspection of the creek bed.

The inspection and access requirements are summarised in the table 11 below.

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Table 13 Inspection Requirements

Component	Activity	Access
Piles	QDMR Level 1 – Visual inspection	Boat/foot
	QDMR Level 2 – Visual inspection plus photos to within 3 m or telephoto equivalent	Boat/foot plus telephoto
	QDMR under water	Boat/diver/under water camera
	QDMR Level 3 – Chloride and Carbonation sampling and testing, selected locations Delamination survey, selected location	Boat required to access pier in creek.
Abutments	QDMR Level 1 – Visual inspection	Foot via walkway in front of abutment
	QDMR Level 2 – Visual inspection plus photos to within 3 m or telephoto equivalent	Foot via walkway in front of abutment
	QDMR Level 3 – Chloride and Carbonation sampling and testing, selected locations Delamination survey, selected location	Foot via walkway in front of abutment
Concrete deck soffit	QDMR Level 1 – Visual inspection	Boat/foot and shore based
	QDMR Level 2 – Visual inspection plus photos to within 3 m or telephoto equivalent	Boat/foot and shore based with telephoto.
	QDMR Level 3 – Close visual inspection plus testing for carbonation/chlorides	Underside of deck can be viewed at high tide from a boat. Sampling from deck taken from shore at abutments or under bridge inspection unit.
Street Furniture	Visual inspection for corrosion or damage	Footway
Bearings and joints	QDMR Level 1 – Visual inspection	Foot via walkway in front of abutment or from pedestrian footpath
	QDMR Level 2 – Visual inspection	Foot via walkway in front of abutment

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	QDMR Level 3 –	Foot via walkway in front of abutment. Inspection of the movement joint will require traffic management.
Ballustrade	Visual inspection for corrosion and damage	Via pedestrian footpath and hard shoulder.

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8. Servicing & Maintenance Requirements

8.1 General

The expected maintenance and servicing requirements are in accordance with QDMR Bridge Servicing Manual.

8.2 Servicing and Maintenance

8.2.1 General

The following anticipated maintenance activities are identified and method of access described. In addition to the planned regular maintenance programme, additional maintenance may be required following flooding or accidents.

Following flooding events that result in the creek level rising to bearing level or higher the bridge should be checked for damage and any debris removed. Similarly the structure should be checked for damage following bridge strikes or road traffic accidents on the bridge and any damage rectified.

Table 14 Maintenance Requirements

Component	Activity	Frequency	Access
Piers and Headstocks	Washing	12 mths	Under bridge inspection unit. Single lane traffic required over bridge.
Abutments	Washing	12 mths	Via walkway in front of abutment
Drainage	Rodding, flushing and cleaning	12 mths	Drainage channels can be accessed from the deck. Abutment drainage can be accessed from under the deck via the walkway in front of the abutment.
Ballustrade	Washing	12 mths	Via pedestrian walkway.
Bearings and joints	Cleaning and removal of debris around bearing shelf. Removal of debris from movement joint.	12 mths	Via pedestrian walkway.

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9. Replacement Requirements

9.1 General

Based on the durability assessment described in this report the service life of the following components is expected to be less than the required 100 year design life of the structure, and therefore these elements will require replacement.

Table 15 Component Replacement Summary

Component	Expected Service Life	Replacement Trigger Levels
Elastomeric Bearings	40	Cracking, splitting of the elastomer, or excessive distortion.
Expansion Joints	40 for metallic components, 20 years for rubber components	Cracking or splitting of gasket. Corrosion of locating bolts.
Ballustrade	50	Pitting corrosion of balustrade. Corrosion of holding down bolts.

9.2 Bearings

The elastomeric bearings are predicted to have a useful life of 25 years.

9.2.1 Replacement Trigger levels

The degradation of the elastomer, due to UV exposure and weathering will result in hardening of the bearing. Cracking or splitting of the bearing may also occur.

When the hardness of the bearing, as tested by penetration hardness, exceeds the manufacturer's limits the bearing should be replaced.

9.2.2 Replacement Methodology

The bearing replacement method and equipment required is detailed on drawing 119563.

9.3 Movement Joint

The elastomeric gland of the movement joint is expected to have a service life of 25 years, provided it is regularly maintained.

9.3.1 Replacement Trigger levels

Degradation of the gland is expected to be caused by UV exposure or puncture from debris. Minor puncture damage can be repaired by application of a suitable sealant.

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9.3.2 Replacement Methodology

The movement joint is designed to facilitate removal and replacement at the end of its service life. The bridge will need to be closed to traffic during replacement activities.

The epoxy protection to holding down bolts shall be removed, the retaining bolts removed to facilitate removal of the joint. The threaded bolts are fabricated from 316 stainless steel, and it is expected that these bolts will not normally need replacement. The new movement joint can be installed using the existing holding down bolts. Damaged bolts can be replaced by over coring and fixing a replacement bolt in place with epoxy resin.

9.4 Balustrade

The balustrade is expected to have a service life of 50 years.

9.4.1 Condition State Guideline

Owing to the saline environment the balustrade is expected to deteriorate due to atmospheric corrosion, resulting in the pitting of the surface.

9.4.2 Replacement Methodology

The balustrade and safety rail can readily be replaced by removing the holding down bolts. At this time if corrosion to the bolts is noted they should be removed by over coring and replaced with new bolts fixed in place with resin epoxy.

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10. Durability Plan Summary Tables

The durability design described in this DPR is summarised in QDMR standard Summary tables in Appendix A

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11. Construction Phase DPR Compliance Assessment

Durability Compliance will be in accordance with the requirements of Tables 10. A durability verification report will be submitted on completion of construction.

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Appendix A

Durability Design Summary Tables

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Table 1: Structure Durability Outline

Critical Durability Components	Service Life					
	Duration (years)	**Contract Reference**	**Potential Mode of Failure**	**Deterioration Mechanism**	**Factors Controlling Rate of Deterioration**	**End of Life Criteria (Expected governing failure mechanism)**
Piles	100 years	Project Specification requires 100 year design life	Reinforcement Corrosion induced cracking, delamination and spalling of reinforced concrete	Chloride induced	Environmental characteristics Concrete mix Crack control Reinforcement cover As placed concrete quality Joint preparation	chloirde induced reinforcement corrosion
				Carbonation induced		
				Macrocell corrosion		
				Localised corrosion at cracks and joints		
Abutments and Headstocks	100 years	Project Specification requires 100 year design life	Reinforcement Corrosion induced cracking, delamination and spalling of reinforced concrete	Chloride induced	Environmental characteristics Concrete mix Crack control Reinforcement cover As placed concrete quality Joint preparation	chloirde induced reinforcement corrosion
				Carbonation induced		
				Macrocell corrosion		
				Localised corrosion at cracks and joints		
Balustrade	50 years	Project Specification requires 100 year design life	Surface corrosion	Pitting corrosion	Environmental characteristics	Through corrosion or deep pitting of surface.
Elastomeric Bearings	25 years	Project Specification requires 100 year design life	Cracking and/or bulging of bearing. Hardening of elastomer.	Oxidation and exposure to UV light.	Environmental characterisitics and composition of elastomer	Excessive bulging and/or cracking or splitting
Expansion Joints	25 years	Project Specification requires 100 year design life	Loss of water tightness	Oxidation and cracking of elastomer	Environmental characterisitics and composition of elastomer	Puncture of sealant
				Puncture of elastomer		
Deck	100 years	Project Specification requires 100 year design life	Reinforcement Corrosion induced cracking, delamination and spalling of concrete	Chloride induced	Environmental characterisitics, Concrete mix, Crack control, reinforcement cover, joint preparation, quality of concrete placement.	Carbonation induced reinforcement corrosion
				Carbonation induced		
				Localised corrosion at cracks and joints		

Table Legend

Table 1	**Table 2**	**Table 3**	**Table 4**	**Table 5**	**Table 6**	**Table 7**	**Table 8**

Table 2: Component Exposure Assessment (All Components)

Environment No.	Environment Zone	Environmental Details	Affected Structural Components	Potential Deterioration Mechanisms	Code Requirements	Assess if Code Sufficient	Identified Additional Requirements
1	Below River premanent water level	Chloride Concentration 1800mg/l, Sulphate Concentration 200 mg/l, pH 6.5	concrete piles	Chloride Induced corrosion, Sulphate attack of concrete	50 MPa concrete	Cement content, water cement ratio and quantity of cement replacement inadequate to achieve 100 year life.	Concrete mix to have low W/c ratio, minimum cement content, and Fly Ash.
2	Within Creek below water level	As above	concrete piles	Reinforcement corrosion due to saline water and oxygen supply.	50 MPa concrete	Cement content, water cement ratio and quantity of cement replacement inadequate to achieve 100 year life.	Concrete mix to have low W/c ratio, minimum cement content, and Fly Ash.
3	Below Water table	As above	concrete piles	Reinforcement corrosion due to saline water and oxygen supply.	50 MPa concrete	Cement content, water cement ratio and quantity of cement replacement inadequate to achieve 100 year life.	Concrete mix to have low W/c ratio, minimum cement content, and Fly Ash.
4	In ground above water table or retaining ground	PASS, ASS, chlorides and sulphates	Abutments and piles	No Potential Acid Sulphate Soils (PASS) have been identified by investigations to date but would require confirmation.	50 MPa concrete	yes, subject to confirmation of ASS and PASS conditions.	Concrete to contain Fly Ash
5	Specified Fill Above Water	No agressive agents expected. Clean fill to be used.	Abutments	None.	AS3600 50MPa concrete, minimum cover 50mm	Yes.	
6	Intertidal and Tidal and Splash Zone	Surface Chloride Concentration 4%	concrete piles, abutments and headstocks	Chloride Induced corrosion	50 MPa concrete	No	Concrete mix to have low W/c ratio, minimum cement content, and Fly Ash.
7	Spray Zone	Surface Chloride Concentration 6%	concrete piles, abutments and headstocks	Chloride Induced corrosion	50 MPa concrete	No	Concrete mix to have low W/c ratio, minimum cement content, and Fly Ash.
8	Atmospheric Exposure	Carbon dioxide	Deck Units	Chloride or carbonation induced corrosion	50 MPa concrete	Yes	None
		Wind borne chlorides	Bearings	Hardening and cracking of elastomeric components		n/a	Allow for replacement
			Balustrade	Chlroide induced pitting corrosion		n/a	Allow for replacement
		UV expsoure	Expansion Joints	Puncture of gland. Corrosion of metallic components		n/a	Allow for replacement

Table Legend

Table 1 **Table 2** **Table 3** **Table 4** **Table 5** **Table 6** **Table 7** **Table 8**

Table 3: Durability Provisions for Critical Components - Design Phase

Environment Classification No.	Environment Classification Zone	Affected Structural Components	Identified Deterioration Mechanisms	Design Mitigation Measure	Construction Contract Provision	Validation Measure/Test
1	Below River premanent water level	concrete piles	Chloride Induced corrosion, Sulphate attack of concrete	S80 concrete containing 20% FA and 5% SF. Minimum cover 50 mm. Steam cured. Maximum water cement ratio 0.28. Minimum cover of 50 mm.		Visual inspection for surface defects. Cover meter survey to confirm minimum cover requirements achieved.
2	Within Creek below water level	concrete piles	Reinforcement corrosion due to saline water and oxygen supply.	as above		
3	Below Water table	concrete piles	Reinforcement corrosion due to saline water and oxygen supply.	as above		
4	In ground above water table or retaining ground	Abutments and piles	No Potential Acid Sulphate Soils (PASS) have been identified by investigations to date but would require confirmation.	S50 concrete containing 20% FA. Minimum cement content 400 kg/m3, maximum water / cement ratio 0.4. Chloride diffusion coefficient of 5e-12 m/s/s. Maximum shrinkage 600 micro strain. Minimum cover of 50 mm.	Thermal modelling to be undertaken prior to construction on site to confirm design assumptions. 7 days wet curing.	Visual inspection for surface defects. Cover meter survey to confirm minimum cover requirements achieved.
5	Specified Fill Above Water Table	Abutments	None.			
6	Intertidal and Tidal and Splash Zone	concrete piles, abutments and headstocks	Chloride Induced corrosion	S50 concrete containing 20% FA. Minimum cement content 400 kg/m3, maximum water / cement ratio 0.4. Chloride diffusion coefficient of 5e-12 m/s/s. Maximum shrinkage 600 micro strain. Minimum cover of 50 mm.	Thermal modelling to be undertaken prior to construction on site to confirm design assumptions. 7 days wet curing.	Visual inspection for surface defects. Cover meter survey to confirm minimum cover requirements achieved.
7	Spray Zone	concrete piles, abutments and headstocks	Chloride Induced corrosion			
8	Atmospheric Exposure	Deck Units	Chloride or carbonation induced corrosion	S50 concrete containing 20% FA. Minimum cement content 400 kg/m3, maximum water / cement ratio 0.38. Chloride diffusion coefficient of 6e-12 m/s/s. Maximum shrinkage 600 micro strain. Minimum cover of 35 mm. Application of waterproof coating to top surface.	Steam curing followed by application of curing compound.	Visual inspection for surface defects. Cover meter survey to confirm minimum cover requirements achieved.

Table Legend

Table 1	**Table 2**	**Table 3**	**Table 4**	**Table 5**	**Table 6**	**Table 7**	**Table 8**

Table 4: Maintenance Intervention Assumptions (to achieve service life)

Element			Servicing			Cyclic			
Affected Structural Component	**Environment No.**	**Zone**	**Activity**	**Frequency**	**Access Provision**	**Intervention Level**	**Activity**	**Frequency**	**Access Provision**
Elastomeric Bearings	8	Atmospheric Exposure	Cleaning bearings. Visual Inspection.	12 months	Paved footway in front of abutment.	Cracking of elastomer. Hardening of elastomer.	Hardness testing of elastomer.	10 years for hardness testing.	Jacking points on bearing shelf and deck units have been provided.
Drainage	8	Atmospheric Exposure	Cleaning and flushing of drainage	12 months	From deck surface or walkway in front of abutment.				
Balustrade	8	Atmospheric Exposure	Cleaning	12 months	From predestrian footpath				
Movement joints	8	Atmospheric Exposure	Debris removal	12 months	From deck surface or walkway in front of abutment. Traffic management to close one lane required.				
Abutments and headstocks	7	Spray Zone	Washing	12 months	UBIU parked on hard shoulder of bridge. Traffic management required.				

Table Legend

Table 1	**Table 2**	**Table 3**	**Table 4**	**Table 5**	**Table 6**	**Table 7**	**Table 8**

Table 5: Inspection & Access Provisions

Element			Condition State Guidelines	Inspection Requirement	Supplementary Requirements to BIM	Access	Safety Hazards	
Affected Structural Component	**Environment No.**	**Zone**					**Description**	**Mitigation**
Elastomeric Bearings	8	Atmospheric Exposure	Hardening of elastomer. Spliting, tearing of shearing os bearing.	Annual - QDMR Level 1 – visual inspection. At 5 years - QDMR Level 2 –Visual inspection plus photos to within 3m or telephoto equivalent. At 10 years QDMR Level 3 - hardness testing of elastmoeric bearing	Hardness testing of elastomeric bearing	Foot access from road level to front of abutment.	Slip and fall hazard ascending and descending embankment.	Paved stairway with hand rail up to abutment. Paved footway in front of abutment.
Piles	6	Intertidal and Tidal and Splash Zone	Cracking and corrosion staining, delamination and spalling	QDMR level 1, 2 and 3 inspections at the frequency stated above.	QDMR Level 3 inspection at 10 years - Chloride profile testing of piles in splash zone to confirm expected durability performance.	Visual inspection from shore. Sampling from UBIU or boat access.	Working over a water way.	
Abutments	7	Spray Zone	Cracking and corrosion staining, delamination and spalling	QDMR level 1, 2 and 3 inspections at the frequency stated above.	QDMR Level 3 inspection at 10 years - Chloride profile testing of piles in splash zone to confirm expected durability performance.	Foot access from road level to front of abutment.	Slip and fall hazard ascending and descending embankment.	Paved stairway with hand rail up to abutment. Paved footway in front of abutment.
Headstocks	7	Spray Zone	Cracking and corrosion staining, delamination and spalling	QDMR level 1, 2 and 3 inspections at the frequency stated above.	QDMR Level 3 inspection at 10 years - Chloride profile testing of piles in splash zone to confirm expected durability performance.	Visual inspection from shore. Sampling from UBIU or boat access.	Working over a water way.	
Balustrade	8	Atmospheric Exposure	Deep pitting corrosion	QDMR level 1, 2 and 3 inspections at the frequency stated above.	None	Access from pedestrian footpath		
Street furniture	8	Atmospheric Exposure	Rust staining on surface	QDMR level 1 and 2 inspections at the frequency stated above.	None	Access from pedestrian footpath		
Movement joints	8	Atmospheric Exposure	Splits in elastomeric gland.	QDMR level 1 and 2 inspections at the frequency stated above.	None	Access from deck.	Access into roadway required for close inspeciton of joint.	Traffic management required to close one lane of traffic.
Precast Concrete Bridge Beams	8	Atmospheric Exposure	Cracking and corrosion staining, delamination and spalling	QDMR level 1, 2 and 3 inspections at the frequency stated above.	Chloride and carbonation sampling and testing to confirm durability performance. Delamination survey.	UBIU Parked on hard shoulder. Traffic Management required to close near side lane.	Reduced headroom to river traffic under bridge when UBIU in use.	Close road while inspection in progress.

Table Legend

Table 1	Table 2	Table 3	Table 4	Table 5	Table 6	Table 7	Table 8

Table 6: Replacement of Components

Element			Replacement Frequency (years)	Condition State Guidelines	Design Provisions	Maintenance Plan & Drawing References	Access	Safety Hazards	
Affected Structural Component	Environment No.	Zone						Description	Mitigation
Elastomeric bearings	8	Atmospheric Exposure	25	Hardening of elastomer. Spliting, bulging or tearing of elastomer. Excessive lateral movement of bearing.	Provision of bearing plinths and jacking points for lifting deck and replacement of bearings.	Method statement for bearing replacement and drawings provided. Drawing ref 119563	Paved walkway in front of abutment can be used as working platform.	Collapse of jacks during lifting operations.	Using Flat Jacks
Balustrade	8	Atmospheric Exposure	50	Deep pitting corrosion.	Balustrade holding down bolts are designed to facilitate removal and replacement of corroded or damaged balustrade.	None	Pedestrian footpath.	Hazard to public during replacement activities.	Footpath to be closed to public, and traffic lane nearest woring are to be closed.
Movement Joints	8	Atmospheric Exposure	25	Cracking, splitting or tearing of elastomeric gland.	Stainless steel holding down bolts area provided to facilitate removal and replacement of the movement joint.	None	Access to movement joint from deck surface.	Passing traffic	Traffic management to include safe working area.

Table Legend

Table 1	Table 2	Table 3	Table 4	Table 5	Table 6	Table 7	Table 8

Table 7: Proprietary Product Records

Component Description	Manufacturer Details	Product reference	Servicing Requirements	Warranty	Installation Details
Deck Expansion Joint	Miska	Miska BJ 1 expansion joint	Inspect and remove debris every 12 months. Replace or repair damaged glands.	To be provided by construction contractor.	To be provided by construction contractor.
Bearing	To be provided by construciton Contractor		Inspect, clean and remove debris every 12 months.		
Waterproof membrane	To be provided by construciton Contractor				

Table Legend

Table 1	Table 2	Table 3	Table 4	Table 5	Table 6	Table 7	Table 8

Table 8: Durability Provisions for Critical Components - Construction & Service Phases Records and Departures

Environment		Affected Structural Components	Identified Deterioration Mechanisms	Design Mitigation Measure	Construction Contract Provision	Validation Measure/Tests (departures to be added)	Intervention Level	Construction or Repair Method	Validation Records		Defect Records	
No.	Zone								Location	Description	Location	Description
1	Below River premanent water level	Piles	Chloride Induced corrosion, Sulphate attack of concrete	S80 concrete containing 20% FA and 5% SF. Minimum cover 50 mm. Steam cured. Maximum water cement ratio 0.28. Minimum cover of 50 mm.		Visual inspection for surface defects. Cover meter survey to confirm minimum cover requirements achieved.	Cover less than specified minimum. Cracking inexcess of 0.2mm wide. Honey combing.					
7	Spray Zone	Abutments	Chloride Induced corrosion	S50 concrete containing 20% FA. Minimum cement content 400 kg/m3, maximum water / cement ratio 0.4. Chloride diffusion coefficient of 5e-12 m/s/s. Maximum shrinkage 600 micro strain. Minimum cover of 50 mm.	Thermal modelling to be undertaken prior to construction on site to confirm design assumptions. 7 days wet curing.	Visual inspection for surface defects. Cover meter survey to confirm minimum cover requirements achieved.	Cover less than specified minimum. Cracking inexcess of 0.2mm wide. Honey combing.					
7	Spray Zone	Headstocks	Chloride Induced corrosion	S50 concrete containing 20% FA. Minimum cement content 400 kg/m3, maximum water / cement ratio 0.4. Chloride diffusion coefficient of 5e-12 m/s/s. Maximum shrinkage 600 micro strain. Minimum cover of 50 mm.	Thermal modelling to be undertaken prior to construction on site to confirm design assumptions. 7 days wet curing.	Visual inspection for surface defects. Cover meter survey to confirm minimum cover requirements achieved.	Cover less than specified minimum. Cracking inexcess of 0.2mm wide. Honey combing.					
8	Atmospheric Exposure	Deck units		S50 concrete containing 20% FA. Minimum cement content 400 kg/m3, maximum water / cement ratio 0.38. Chloride diffusion coefficient of 6e-12 m/s/s. Maximum shrinkage 600 micro strain. Minimum cover of 35 mm. Application of waterproof coating to top surface.	Steam curing followed by application of curing compound.	Visual inspection for surface defects. Cover meter survey to confirm minimum cover requirements achieved.	Cover less than specified minimum. Cracking inexcess of 0.2mm wide. Honey combing.					

Table Legend

Table 1	Table 2	Table 3	Table 4	Table 5	Table 6	Table 7	Table 8

Appendix B

Reinforced Concrete Durability Commentary

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General Overview of Reinforced Concrete Durability Issues

Purpose: This general overview is used for review to identify the project specific durability issues. The content is therefore general for all possibilities.

1. Reinforcement Corrosion

The two principal interactive materials steel reinforcement and concrete are subject to separate but interrelated deterioration processes. Steel embedded in concrete is normally protected from corrosion by the existence of a passive oxide film surrounded by calcium hydroxide ($Ca(OH)_2$), the latter reflecting the highly alkaline environment of fresh concrete where a pH in excess of 12.5 can be anticipated. The two materials thus combine to form a deterioration resistant composite.

The integrity of this composite can be affected by the deterioration of either material or both materials in unison. The deterioration of steel inside concrete is a corrosion process, which can be initiated by one or both of two mechanisms.

a) **Chloride contamination:** the presence of a critical concentration of chloride ions at the reinforcement surface will cause local breakdown of the passive layer, even at a high pH. In structures exposed to a saline environment this is the primary corrosion risk for the reinforced concrete.

b) **Carbonation:** the gradual penetration of atmospheric carbon dioxide in unsaturated concrete will neutralise the protective alkaline environment surrounding embedded steel. If moisture is present, the steel will corrode.

Once initiated, the rate of corrosion is a complex function of several factors including the availability of oxygen and the overall electrical resistance between anodic and cathodic sites. Corrosion is an electrochemical reaction, or more accurately, two half cell reactions:

At the anode $\quad Fe \rightarrow Fe^{2+} + 2e^-$

At the cathode $\quad 1/20_2 + H_2O + 2e^- \rightarrow 20H^-$

These two reactions must be balanced (i.e. the rate of generation of electrons at the anode 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 an assessment of corrosion risk in concrete elements as will be demonstrated below. It should be noted from the above that oxygen is (normally) required at cathodic sites in order for the reactions to proceed.

Secondary reactions at the anodic site convert Fe^{2+} ions into oxide compounds, the exact form of which depends largely on the availability of oxygen at the anode. In atmospheric conditions where oxygen is plentiful, the normal reactions will be generation of a hydrated iron oxide:

$$2Fe(OH)_3 \rightarrow Fe_20_3 \,.\, (xH_2O)$$

These hydrated oxides occupy a greater volume that 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.

This is the typical scenario in an atmospheric reinforced concrete element exposed to saline water. Chloride ions penetrate the cover and adjacent anodic and cathodic sites develop as passivity breaks

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down. Micro cells are formed with anodic sites (usually in the form of pits) and immediately adjacent cathodic sites supplied with oxygen via diffusion of air through the cover. A similar process occurs in atmospheric conditions in which carbonation of the concrete induces corrosion, although a different corrosion product results.

In concrete elements in which the section is totally saturated, the corrosion process is controlled by a lack of oxygen, thus limiting the rate of reaction. Although some oxygen is present in saline water, the slow rate at which it can be diffuse through the concrete cover results in what amounts to oxygen starvation conditions. Thus, although corrosion is often initiated in submerged conditions, it does not proceed at a rate, which leads to significant metal loss, and therefore can be discounted in most cases.

An intermediate condition arises in concrete elements which are water excluding. In this case saline water on one side can penetrate the cover and create anodic sites. The potential difference between the two grids of steel, and oxygen access from the inside wall, means the cathodic sites can develop. Because oxygen is not readily available at the anodic sites, the secondary reaction is severely limited and the formation of expansion oxides is minimal. However metal loss will still occur, with eventual structural consequences if the corrosion rates are significant.

This process is known as macro cell corrosion and has been considered the theoretical corrosion risk for United Kingdom North Sea hollow leg concrete platforms. Although this mechanism undoubtedly occurs it is difficult to assess its rate. The rate is dependant on the total electrical resistance of the corrosion circuit.

Another mechanism for reinforcement corrosion is stray DC traction currents from nearby rail and tram systems. Modern traction systems are generally designed to minimise the levels of stray current leakage to earth from the rail. This is generally achieved by the installation of insulators under the rail to increase the resistance of the rail to earth.

Metallic structures buried in the ground may be subject to stray current interference, and subsequent corrosion, in the vicinity. The susceptibility of a structure to stray current corrosion will be dependant upon the geometry and size of the structure, any coatings applied to the structure, the conductivity of the environment surrounding the structure, the proximity of the structure to both the traction rail line and the traction electrical substation.

Stray DC currents will find the lowest resistance path back to the source. (i.e. where the power is generated at the substation). Modern traction vehicles utilise regeneration braking to increase the efficiency of the operation traction system. Regenerative braking utilises the energy “stored” in the moving traction vehicle, to regenerate power into the system when the rail vehicle is braking. This implies that each rail vehicle will act as a substation in the system under braking. The magnitude of the current “escaping” is a function of the length of the rail, the leakage resistance to earth of the rail, and the current loading per unit length of the track.

When current arrives at the reinforcement, the reinforcement electrical potential will become more negative than the steel’s natural potential. At a reinforcement discontinuity, the current leaves the structure and returns into the soil on its circuitous path to the substation (supplying the positive return), resulting in a more positive electrical potential at the point of discontinuity. This results in stray current corrosion at the site of steel discontinuity.

2. Concrete Deterioration

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Concrete deterioration can occur without reinforcement corrosion, albeit the processes involved may eventually initiate or accentuate corrosion mechanisms. Main concerns are:

a) Alkali Aggregate Reaction (AAR)

b) Sulfate Attack

c) Acid Attack

d) Microbiologically Induced Attack

e) Aggressive Carbon Dioxide

f) Magnesium Attack

g) Delayed Ettringite Formation (DEF)

2.1 Alkali Aggregate Reaction (AAR)

Alkali aggregate reaction (AAR) is a chemical process in which alkalis, present in cement, combine with certain compounds in the aggregate when moisture is present. The reaction produces an alkali-silica gel that can absorb water and expand to cause cracking and disruption of the concrete. For alkali-silica reaction and damaging expansion of the resulting gel to occur in hardened concrete, it is necessary to have the occurrence of the following three elements:

a) A sufficiently concentrated pore solution

b) A proportion of reactive silica or silicate in the aggregate

c) Sufficient moisture in the concrete

Previous project experience has indicated that North Queensland aggregates exhibit varying degrees of reactivity. Special assessment of aggregates for AAR is normal practise in major projects and approved concrete suppliers should have petrographic assessment records for review prior to mix design approval.

In instances where laboratory testing of aggregates (peterographic analysis) indicates a certain degree of reactivity measures to reduce the risk of ASR deterioration shall be incorporated in the mix design. These include either or a combination of the following (as listed in Table 4 SAA HB79-1996) using an alternative aggregate source, limiting the alkali content within mix and using blending cements.

2.2 Sulfate Attack

Two principal chemical reactions between sulfates present in the groundwater and the concrete are attributed to sulfate attack:

a) A reaction between the sulfates and calcium hydroxide to produce calcium sulfate (or gypsum). The volume of the gypsum produced is more than double the volume of the calcium hydroxide reaction. This gives rise to internal stresses, which break down the concrete.

b) Consumption of lime as above lowers pH, allowing sulfate to react with destabilised aluminate minerals in the current paste to form an expansive mineral, ettringite, which results in breakdown of the cement paste.

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These reactions lead to expansion and disruption of the concrete. Ultimately, the concrete is reduced to a soft and friable state.

Factors that influence the rate of sulfate attack of concrete include:

a) Concentration and type of sulfate and the pH of the water;

b) Mobility of the sulfate contaminated water;

c) Penetrability, cement type and content, water/cement ratio of the concrete.

The type of sulfate is most important in respect to the extent and rate of sulfate attack of concrete. For example, magnesium sulfate has a more aggressive action than other sulfates and decomposes the hydrated calcium silicate cement phases in addition to reactions (a) and (b) described above. Ammonium sulfate is more aggressive than sodium sulfate because of the increased solubility of the gypsum in ammonium sulfate solutions and formation of the ammonia gas which allows reaction (a) and (b) proceed to completion.

The pH of the sulfate solution is important, since more extensive and more rapid rate of attack occurs at low pHs. Acid dissolution (neutralisation) of the cement paste occurs in combination with sulfate attack. The type(s) of acid also influences the degree and rate of the attack.

The flow rate of groundwater affects durability since a mobile groundwater will remove the products of reaction and replenish the sulfate attack site with more aggressive media. Specific flow rates are not quoted in the literature and standards, although standards such as AS 2159-1995 differentiate the aggressiveness of groundwater according to “high” permeability and “low permeability” soils.

Concrete quality aspects such as cement type and content, penetrability and water/cement ratio have a marked affect on the degree and rate of sulfate attack. Blast furnace slag, fly ash and silica fume blended cement based concretes have been found to be more resistant to sulfate attack than sulfate resisting Portland cement concretes. Increasing cement content and decreasing water/cement ratio has the effect of lowering the water penetrability of a concrete, which leads to decreased sulfate attack.

Depending on the form of concrete construction, shrinkage and thermal cracking aspects limit the extent to which cement content can be increased. Similarly, construction issues such as placability, compactibility, etc limit the extent to which the water/cement ratio of a concrete can be decreased. The penetrability of a concrete is also affected by construction aspects such as compaction, method of cure, curing period, etc.

Low sulfate concentration in the ground water within the construction area of a project will result in sulfate attack not being a major environmental risk.

2.3 Acid Attack

Acids in general are considered to be problematical for concrete when the pH falls below 6.5 for prolonged periods. The key break down mechanism is outlined below.

The reduction of pH due to consumption of calcium hydroxide destabilises the calcium aluminate hydrates and the calcium silicate hydrates in the cement paste, resulting in breakdown of these cementing minerals.

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Ultimately the integrity of the concrete is diminished and the surface concrete gradually erodes with time. Factors that influence the rate of acid attack of concrete include:

a) Concentration and type of acid;

b) Mobility of the acid contaminated water;

c) Penetrability, cement type and content, water/cement ratio of the concrete

d) Aggregate type (not significant with local aggregates)

The type of acid is most important with respect to the extent and rate of the attack of concrete. For example, sulfuric acid attack is particularly destructive since mechanisms discussed in the Section 2.2 Sulfuric Attack, will also be active.

The flow rate of ground water affects durability since a mobile groundwater will remove the products of reaction and replenish the acid attack site with more aggressive media. Specific flow rates are not quoted in the literature and standards, although standards such as AS 2159-1995 differentiate the aggressiveness of groundwater according to “high” permeability and “low permeability” soils.

The comments made on concrete quality for Section 2.2 Sulfuric Attack also apply to Acid Attack.

2.4 Microbiologically Induced Attack

a) **Aerobic Reactions** (oxygen utilising)

Oxidation of elemental sulfur by micro organisms to generate sulfuric acid is generally attributed to bacteria of the genus thiobacillus. These organisms are known to exist in mud, seawater, sewage, and boggy places. The pH ranges from 1.0 to 5.0, depending on the species of thiobacillus present.

b) **Anaerobic Reaction** (living without oxygen)

The most common type of anaerobic bacterium in soil is the sulfate reducing bacterium (SRB). The most common species is Desulfovibrio Desulfuricans which functions in the absence of oxygen but only when iron is present. Sulfate is reduced by the bacterium to sulfide, thus forming iron sulfide (commonly known as pyrite) and hydrogen sulfide (H_2S) gas. This situation is obviously very corrosive to iron which is sacrificed as pyrite. If the soil conditions change to an aerobic condition (for example, temporarily during construction), the H_2S can spontaneously oxidise in the presence of water to form sulfuric acid.

c) Coexistence of Aerobic and Anaerobic Bacteria

The bacteria coexist within the same soil mass such that if conditions change from anaerobic to aerobic conditions or vice-versa the dormant bacteria strain takes over. Thus any construction activity which allows access of oxygen to the soil may sustain aerobic bacterial activity. This may create a short term risk during construction (where hydrogen sulfide generated by SRB gets converted to sulfuric acid).

d) Sulfuric Acid Attack of Concrete caused by Microbiological Organisms

The deterioration of concrete under the action of sulfuric acid generated under aerobic conditions has been well documented under the extreme case of sewer exposure. The problem of concrete attack by aerobic and anaerobic bacteria has been recognised by the oil industry where microbiological activity is enhanced in storage tanks due to the presence of oil and by elevated temperatures resulting from oil storage. The influence of microbiologically induced attack of concrete in soil has not been as well

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documented as the cases of sewer and oil storage exposure environments. Nevertheless the problem in soils and groundwater has been recorded.

A further problem follows the breakdown of the surface concrete; namely the risk of reinforcement corrosion (which is discussed above). This can be caused by either the ingress of chloride from the saline water (which break down the normally stable protective iron oxide film that forms on reinforcing steel) or due to loss of alkalinity due to the acid reacting with the calcium hydroxide.

Sulfide ions can also promote reinforcement corrosion in the same way as chloride ions and the previous discussion on corrosion applies. Given the nature of the two ions, chloride is likely to penetrate the concrete faster than sulfide ions. Defence against chloride induced corrosion can therefore be considered sufficient to deal with this risk.

2.5 Aggressive Carbon Dioxide

The presence of carbon dioxide can emanate from, for example, microbiological processes of vegetative decay of plant debris, or the interaction of sedimentary calcareous rocks with ground waters. The effect of aggressive carbon dioxide is to produce a carbonic acid solution that has a high capacity to dissolve lime, leading to leaching of lime from the cement paste. The tendency for leaching will also be dependent on the hardness of the groundwater.

2.6 Magnesium Ion

The significance of magnesium lies in its ability to be readily exchanged for calcium within the cement binder. This results in the calcium silicate hydrates significantly losing their binding properties, leading to breakdown of the cement binder.

The estimation of what represents a critical concentration of carbon dioxide in waters which may come in contact with concrete is not simple. The calcium carbonate saturation index (Langelier Index) and Ryznar Stability Index gives an indication of the aggressivity of the carbon dioxide.

The aggressivity of the water is also determined from the Overall Corrosion Index value.

The Overall Corrosion Index is defined as the sum of the Leaching Corrosion (LCI) and the Spalling Corrosion Index (SCI). The aqueous parameters considered important in respect to LCI and SCI are:

a) pH, calcium carbonate saturated pH, calcium ion, total ammonium ion, magnesium ion, total sulfate ion and chloride ion.

Based on these aqueous parameters, corrosion indices may be calculated which will dictate the corrosivity of the water to concrete and the capacity to dissolve lime from the cement paste.

2.7 Delayed Ettringite Formation (DEF)

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 65° C or less, DEF is not likely with normally available cements. For concrete temperatures greater than 65° C but less than 80°C, cement chemistry needs to be examined to

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determine the likelihood of DEF. Hence, to avoid DEF it is recommended by MTG that the maximum temperature of the concrete during the manufacturing of precast elements should not exceed 65°C.

3. Concrete Cracking

Concrete by its nature has a propensity to crack. Cracking may occur due to effects of lack of curing, cement hydration (drying shrinkage and thermal cracking) or through response to applied tensile loads. As concrete is generally designed to carry load in compression the presence of controlled cracking of reinforced concrete is not necessarily detrimental to its structural capacity. It is more likely to have an effect on the serviceability and long term durability of the structure. Cracks that form during construction should be subject to durability review for the ability to achieve the design service life without premature deterioration.

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Appendix C

Modelling of Chloride Ingress

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1) General

The prediction of ingress of aggressive salts into concrete containing steel reinforcement is an extremely useful tool when designing concrete for long-term durability. Established methods of prediction, based closely on empirical modelling techniques described in a DETR report (1999), have been used to develop an ingress model that is representative of corrosion initiation and propagation seen in real structures. This section describes the procedures used to assess the risk of chloride-induce reinforcement corrosion.

Conditions for exposure of the bridge elements during its service life of 100 years in terms of diffusion are examined, along with basic estimations of mass transport mechanisms of ingress.

2) Background

Established techniques based on Fick's second law of diffusion and empirical relationships between concrete properties developed by TRL (1997) and DETR (1999) were adapted and used to predict chloride ingress and corrosion to the serviceability limit state defined as the cracking of the cover to the steel. The model employed in this study is based on an understanding of the transport processes involved and the principal influencing factors, with a validation of the model against the consensus view of current behaviour by interpreting performance studies into the deterioration of reinforced concrete structures in different environments around the world. The model enables the user to study the effect of a number of boundary conditions on concrete mixes and formulate the most suitable basic mix design that is durable over its specified service life under the assumed environmental conditions.

The rate of diffusion is described by the diffusion coefficient. This parameter gives the flux of a species (quantity passing through a unit area per unit time) per unit concentration gradient (Glass et al, 2000).

$$C_x = C_{sn}\left(1 - erf\frac{x}{2\sqrt{D_c t}}\right) \qquad \textbf{(1)}$$

The solution to Fick's diffusion equation can be derived by the use of an error function (Bamforth, 1997):

- where: D_{ca} is the apparent diffusion coefficient (m^2/s) at time t(s)
- C_x is the chloride concentration at depth, x(m) after exposure time t(s)
- C_{sn} is the notional surface level of chloride
- erf is the error function
- C_{sn} and D_{ca} are derived from best-fit analysis of a chloride profile after exposure time t.

Empirical models are based on the analysis of empirical data, and many chloride profiles follow the form given by equation 1, even though underlying assumptions on transport processes may be incorrect (Glass et al, 2000). Therefore, although the chloride profile is characteristic of a diffusion process, it is not necessary to assume that it was caused solely by diffusion. Capillary suction caused by wetting and drying the surface of the concrete or mortar will accelerate the ingress of chlorides but will produce a chloride profile that fits the diffusion curve.

The apparent diffusion coefficient, by the very nature of the form of the curve measured in real structures, is regarded in this study as representing all relevant transport processes operating in combination for concrete in aggressive environmental conditions. This is because it is believed that the apparent diffusion coefficient and surface concentration are both increased by cyclical wetting and

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drying. Therefore, use of surface concentrations and apparent diffusion coefficients as measured in real structures will take account of capillary suction during wetting and after drying.

Chloride-induced corrosion in the model is divided into an initiation phase (i.e. time to the point where chlorides are at sufficient levels to cause corrosion at the reinforcement) and a propagation phase (time required for the incremental addition of chlorides at the reinforcement to increase the corrosion rate to the level where the tensile strain capacity of the concrete is exceeded by the strain caused by the volumetric expansion of corrosion products). The propagation phase therefore assumes that the rate of corrosion is related closely to the concentration of chlorides available at the site of corrosion (Bamforth, 1997). The model uses the results from laboratory testing of concrete samples under no load conditions. The effect of the tensile stresses induced in the cover concrete due to loading of the segments may be to reduce the amount of additional stress that the cover concrete can tolerate before cracking due to the progressive build-up of corrosion products on the reinforcement near the face. It is therefore necessary to add a safety factor to the service life required in order to provide a margin for these uncertainties.

3) Approach to the modelling process

a) General

The approach to the modelling process can be divided into three distinct phases:

- Establish design service life and serviceability limit state
- Establish the cases to be modelled and define the input parameters based on the assumptions, including the exposure environments and their potential severity
- Undertake predictive modelling to assess durability and service life of selected mixes.

b) Specification of service life and serviceability limit state

It is understood that the bridge elements are to be designed with the following conditions in mind:

- Service life of 100 years
- Serviceability limit-state under chloride ingress is the onset of cracking of the concrete induced by corrosion of the reinforcement
- A safety factor of 1.15 has been added to the 100 year design life to give a target design life of 115 years for durability design options.
- The chloride modelling exercise has concentrated on the splash zone on the piles, headstock soffits, and abutment base as this considered the environment where the greatest build-up of salts is possible if, due to the successive wetting and drying episodes take place resulting in a increased deposition of chloride salts.
- The analysis takes into account that the concrete should be defect-free for a minimum of 25 years, and have a basic design life of 100 years. For this reason, in order to minimise the risk, modelling has concentrated on establishing the service life performance under the worst case combination of conditions that could occur on the bridge.

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4) Cases modelled

a) Concrete mixes

The type of concrete mix to be used the bridge elements was outlined in Table 10.

5) Assumptions

a) Temperature

Concrete temperature during service life has been assumed at 28°C from analysis of local climatalogical data.

The temperature of concrete in service can have a profound effect on performance with respect to its resistance to attack from aggressive salts. Changes in temperature principally affect the rate of diffusion and the threshold value to corrosion initiation with respect to chloride migration. The model employed in this study assumes as a basis that diffusion coefficient and threshold value are estimated at 20°C. An increase in temperature has the effect of increasing the rate of diffusion and lowering the effective threshold value to initiate corrosion, generally following the Arrhenius equation:

$$Factor = \exp\left[\frac{E}{R}\left(\frac{1}{293} - \frac{1}{(T+273)}\right)\right]$$

- where E is the activation energy (KJ/mol), R is the molar gas constant (0.008314 KJ/mol °K) and T is the temperature (°C).
- The Arrhenius factor requires that reaction rate effectively doubles for each increase in temperature of 10°C. This factor has been accounted for in the model for in-service temperatures stated above.

b) Other

The model assumes no workmanship defects or deterioration in the concrete prior to installation.

6) Input parameters

a) Introduction

The required inputs for prediction are as follows in Table C.1.

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TableC.1 Input parameters used for modelling

Parameter	Design Value	
Surface Chloride Concentration (% by weight concrete)	Atmospheric	0.5 %
	Splash zone	2.0 %
	Tidal Zone	1.0%
	Submerged	1.0%
Background chloride level (% by weight concrete)	0.01%	
Average Temperature	28°C	
Grade of Concrete	S50 for insitu and precast concrete.	
Corrosion Activation Threshold* (% by weight concrete)	0.06% for reinforcement 0.04% for prestressed reinforcement	
Cement content (kg/m3)	Minimum 400 kg/m^3	
Cover	50 typically (35 deck units)	
Bar diameter	Element specific	

b) Chloride surface concentration (C_{sn})

For the purposes of prediction, it is essential to determine the loading of chlorides on the structure as this will establish the initial concentration gradient. The level of salts which build up on the surface of concrete is determined to a large extent by the location of the structure, orientation of the surface and ambient conditions. As a consequence variations in the surface concentration of salts are likely.

Under severe conditions of exposure, it appears that the surface level of salts establishes itself very quickly in relation to the expected life of the structure, and remains approximately constant thereafter (DETR, 1999). In less severe exposure conditions the rate of build up on the surface can take much longer. It must be appreciated that in the design service life of the bridge, even if the surface concentration takes a few years to build up to its maximum value, this surface level will still be effective for most of its life.

The use of blended cement concretes exhibit increased effective chloride levels at the concrete surface due to their improved hardened properties and greater chloride binding capacities. A modification factor of: 'Csn +15%' has therefore been applied to increase the chloride surface levels when modelling blended cements (TRL, 1997).

Surface chloride concentrations are thought to accumulate through a number of processes, by migration of chloride-laden salt water through the body of the concrete by diffusion under a concentration gradient capillary suction and wick action caused by surface evaporation.

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c) Chloride threshold value

The chloride threshold value (i.e. the concentration of chlorides at which corrosion at the steel surface is initiated) used in the analysis has been estimated, for reinforced concrete, at 0.06% by weight of concrete, based on evidence supported by a number of workers (Browne,1982; Vassie, 1984; Bamforth & Chapman-Andrews, 1994; Glass and Buenfeld, 1995).

d) Background chloride levels

All mixes assume a background chloride level of 0.01% (by weight of concrete).

e) Apparent chloride diffusion coefficient

The use of apparent chloride diffusion coefficients estimated from empirical relationships represents a potential source of error in the modelling. Diffusion coefficients for a given concrete can vary considerably, and differences of up to one order of magnitude have been recorded within structures containing the same concrete (TRL,1997). Durability predictions are therefore based on a range of conservative values where possible, using upper and lower levels of severity for boundary conditions and assumptions.

Diffusion coefficients for concretes containing Portland cement were derived using empirical relationships with water/cement ratio. Relationships between the reduction in diffusion coefficients for different binder types at different levels of replacement were extracted from the literature (TRL, 1997) and embedded within the model. The apparent diffusion coefficients used in the analysis for the two working temperatures factored using the Arrhenius equation and are presented in Table C.2.

Table C.2 Apparent chloride diffusion coefficient derived for the modelling

Concrete type	Apparent diffusion coefficient (Dca_{20}) (x 10^{-12} m^2/s) at 20°C (Initial Case)	Temperature corrected Dca_{20} (x 10^{-12} m^2/s) at 28°C
20% FA mix	0.05	0.13
20% FA and 5% SF mix	0.03	0.09

f) Design Crack Widths

Cracks in concrete will promote the ingress of deleterious agents into the body of the concrete, effectively by-passing the protective cover concrete. The number, spacing and width of the cracks all influence the corrosion risk. The codes of practice acknowledge this through the provision of design crack widths depending on the exposure environment. Crack widths of 0.3mm or greater present a high risk of corrosion whereas widths of 0.1mm present a low risk on the basis that the crack may not penetrate the full cover depth and some self healing may occur.

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g) Findings

The output of the modelling exercise is the determination of a range of years for the serviceability limit states for the bridge components, based on the ingress of chlorides. Table C.3 presents the results of the modelling results.

Table C.3 Predicted design service life for chloride ingress into concrete from the internal surfaces at 50 mm minimum cover

Additional Protective Measures	Predicted Service Life at 35°C (Years)	
	20% FA	20%FA 5%SF
None	90	115

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GHD

201 Charlotte Street Brisbane QLD 4000
GPO Box 668 Brisbane QLD 4001
T: (07) 3316 3000   F: (07) 3316 3333   E: bnemail@ghd.com.au

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Document Status

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		Name	Signature	Name	Signature	Date

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Appendix E: Blank Summary Tables

Table 1: Structure Durability Outline

Critical Durability Components	Service Life					
	Duration (years)	Contract Reference	Potential Mode of Failure	Deterioration Mechanism	Factors Controlling Rate of Deterioration	End of Life Criteria (Expected governing failure mechanism)

Table Legend

Table 1	Table 2	Table 3	Table 4	Table 5	Table 6	Table 7	Table 8

Table 2: Component Exposure Assessment (All Components)

Environment		Environmental Details	Affected Structural Components	identified Deterioration Mechanisms	Code Requirements	Assess if Code Sufficient	Identified Additional Requirements
No.	Zone						

Table Legend

Table 1	**Table 2**	**Table 3**	**Table 4**	**Table 5**	**Table 6**	**Table 7**	**Table 8**

Table 3: Durability Provisions for Critical Components - Design Phase

Environment Classification		Affected Structural Components	Identified Deterioration Mechanisms	Design Mitigation Measure	Construction Contract Provision	Validation Measure/Test
No.	Zone					
0	0	0	0			
0	0	0	0			
0	0	0	0			
0	0	0	0			
0	0	0	0			
0	0	0	0			

Table Legend

Table 1	**Table 2**	**Table 3**	**Table 4**	**Table 5**	**Table 6**	**Table 7**	**Table 8**

Table 4: Maintenance Intervention Assumptions (to achieve service life)

Element			Servicing			Cyclic			
Affected Structural Component	Environment No.	Zone	Activity	Frequency	Access Provision	Intervention Level	Activity	Frequency	Access Provision
0	0	0							
0	0	0							
0	0	0							
0	0	0							
0	0	0							
0	0	0							

Table Legend

Table 1	Table 2	Table 3	Table 4	Table 5	Table 6	Table 7	Table 8

Table 5: Inspection & Access Provisions

Element			Condition State Trigger Levels	Inspection Requirement	Supplementary Requirements to BIM	Access	Safety Hazards	
Affected Structural Component	Environment No.	Zone					Description	Mitigation
0	0	0						

Table Legend

Table 1	Table 2	Table 3	Table 4	Table 5	Table 6	Table 7	Table 8

Table 6: Replacement of Components

Element			Replacement Frequency (years)	Condition State Trigger Levels	Design Provisions	Maintenance Plan & Drawing References	Access	Safety Hazards	
Affected Structural Component	Environment No.	Zone						Description	Mitigation
0	0	0		0	0				

Table Legend

Table 1	Table 2	Table 3	Table 4	Table 5	Table 6	Table 7	Table 8

Table 7: Proprietary Product Records

Component Description	Manufacturer Details	Product reference	Servicing Requirements	Warranty	Installation Details
0			0		

Table Legend

Table 1	**Table 2**	**Table 3**	**Table 4**	**Table 5**	**Table 6**	**Table 7**	**Table 8**

Table 8: Durability Provisions for Critical Components - Construction & Service Phases Records and Departures

Environment		Affected Structural Components	Identified Deterioration Mechanisms	Design Mitigation Measure	Construction Contract Provision	Validation Measure/Tests (departures to be added)	Intervention Leve	Construction or Repair Method	Validation Records		Defect Records	
No.	Zone								Location	Description	Location	Description
0	0	0	0	0	0	0	0					

Table Legend

Table 1	Table 2	Table 3	Table 4	Table 5	Table 6	Table 7	Table 8