Transport
Roads & Maritime
Services

GUIDE FOR THE PREPARATION OF A DURABILITY PLAN

JUNE 2013

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Foreword

The "Guide for the Preparation of a Durability Plan" (the Guide) is for staff involved in managing the design of new bridges and other structures. It provides a means of appropriately assessing durability and enables a Durability Plan to be delivered for these bridges and structures. The Durability Plan can then be used by designers, constructors and maintainers, to develop the optimum outcomes for their work.

The principles and formats within the Guide can also be applied to existing bridges and structures. The Guide is particularly useful for developing preventative maintenance plans for existing bridges and structures, especially in aggressive environments.

I would like to congratulate the Bridge and Structural Engineering team within the Engineering Technology Services branch of RMS for their dedication and commitment in developing this Guide. It will allow for significant improvement in the preparation and consistency of Durability Plans developed for the many bridges and structures managed by RMS. It will ensure that the durability of the components contained within these bridges and structures meet or exceed their service life, enabling our bridges and structures to service our customers for their full design life without risk of premature failure.

I commend this guide to all staff involved in the design or maintenance of new and existing bridges and structures.

G Evans 2/8/13

Greg Evans
Director Asset Maintenance

1 Introduction

The Roads and Maritime Services (RMS) has recently been provided various Durability Plans containing significant amounts of information which were not considered necessary while some important information was not included.

Some durability designers carry out considerable site specific testing while some conduct hardly any site specific measurement. When the site specific severity of the ground conditions is unknown there is a concern that the design (materials and element geometry) may not provide the design life required for the structure, normally 100 years for bridges.

Australian Standards do provide some recommendations, however the durability recommendations of AS 5100, AS 2159 and AS 3600 are considered to be inadequate by many designers. Some designers continue to use the recommendations of Australian Standards and thus there are substantial differences in the durability standards to the various designs, which increases durability risks.

This document is named 'Guide for the Preparation of a Durability Plan', otherwise known as 'the Guide' elsewhere in this document.

1.1 Purpose of the guide

The purpose of the Guide is:

- To highlight information considered to be important in the preparation of durability plans. Information considered unnecessary is also identified.
- To identify the test requirements that must be carried out to assess the severity of the site environment.
- To encourage the use of similar standards for designs of all major road structures regardless of the form of procurement ie direct construction, design and construct, alliance, or any other forms.
- Emphasize the need and importance of durability modelling on major road structures to ensure the design meets the required design life.

1.2 Application to RMS projects

The Guide will be used in all major infrastructure projects of RMS that require the development of a Durability Plan.

1.3 Who should use the Guide?

The Guide is primarily for durability designers and reviewers involved in the design of bridges and other road structures.

Other users include contract managers, project management team members and asset managers. The principles in the guide may also be useful for external organisations including councils, consultants, other road authorities and contractors.

1.4 Relationship to other RMS documents

All road structure designs must comply with applicable RMS Quality Assurance (QA) specifications and bridge technical directions (BTD). Therefore, the Durability Plan will cross reference to various RMS QA specifications and BTDs. The guide integrates with other RMS documents and specifications. It provides additional information which may not be readily available in relevant RMS policies, manuals, procedures and other documentations.

2 Layout of a Durability Plan

The Durability Plan (DP) should contain the following sections:

- Executive Summary
- Introduction
- Scope
- Definition of Service Life
- Details of Environment (severity)
- Exposure Classification
- Details of Material
- Maintenance Schedule
- References
- Appendices – include results of bore holes or the severity of environment, details of modelling, calculations and other similar information.

3 Contents of a Durability Plan

Details of various sections of a Durability Plan are described below.

3.1 Executive Summary

Executive summary should provide the following information:

- Brief information about the route with number of bridges and major structural elements
- Rough gauge of severity of the environment
- Brief summary of exposure classifications
- Measures adopted to provide the required design life

3.2 Chapter 1 - Introduction

The purpose of this section is to familiarise the reader with the project.

It should contain the following:

3.2.1 Background

Brief description of the project including description of locality, length of the route, location in the state (as a map portion), number of bridges, culverts, any tunnels or large retaining wall etc.

3.2.2 Description of proposed structures

This should be tabulated, suggested headings below:

Table 1: Description of Road Structures			
Asset No	**Asset Brief Description**	**Type and Configuration of Asset**	**Chainage**
[Insert RMS assigned Asset No]	[Insert RMS assigned asset description, as per RMS BIS]	[Insert structure type, configuration and other information of superstructure and substructure etc]	[Insert RMS assigned chainage]

Include any unusual feature of the project or any other information which may be considered useful, such as presence of acid sulphate soil, floodplains or a marine environment.

3.2.3 Form of contract

State form of contract or any other type of project set-up, if known at the time of writing.

3.2.4 Chainage of the route

The chainage of the start, finish and major structural elements to facilitate the discussion in the following sections of the Durability Plan.

3.3 Chapter 2 – Scope and Design Life requirements

3.3.1 Scope

The scope of the activities related to the durability plan should be documented in this section. Any exclusion to the work related to durability design should be clearly mentioned in this section.

3.3.2 Design Life requirements

A list of various elements and the required design lives should be tabulated, see suggested table heading:

Table 2: Design Life Requirements		
Asset	**Element**	**Design Life**
[Insert Asset Type 01]	[Insert Element 01]	[Insert No of years]
	[Insert Element 02]	[Insert No of years]

3.4 Chapter 3 – Definition of Service Life

Service Life should be defined for the various elements. Also, the end of life criteria should be mentioned. Eg for parapets, the end of life can be considered as cracking due to reinforcement corrosion and consequent loss of strength. Service Life is defined in AS 5100.1-2004 as '*A period over which a structure or structural element is expected to perform its function without major maintenance or structural repair*' whereas, ISO 13823:2008 defines it as '*actual period of time during which a structure or any of its components satisfy the design performance requirements without unforeseen major repair*'.

The Service Life of the structure and its components must meet or exceed the Design Life of the structure. Components whose predicted Service Life is less than the Design Life of the structure must be inspectable and replaceable.

In Section 4.2 of AS 5100.1-2004, the Design Life is defined as '*The period assumed in design for which a structure or structural element is required to perform its intended purpose without replacement or major structural repairs*'. Furthermore in the Supplement to AS 5100.1-2004, Design Life is discussed and states that '*This assumption of a nominated design life does not mean that the bridge will no longer be fit for service when it reaches that age*'. Thus, as an asset owner, the expectation is that a well maintained asset would continue to be in service and continue to perform its intended purpose even beyond its design life (100 years to most elements).

Compliance to AS 5100.1 is a requirement of the design. The designer must adopt the Design Life definition of AS 5100.1 and must be stated in this section. Alternatively, if the designer proposes to adopt another definition of Design Life, it must be agreed with the RMS representative and must be clearly spelt out in this section.

3.5 Chapter 4 – Severity of exposures and details of environment

The following information should be properly documented to facilitate the assessment of the severity of the exposure condition and determine the corresponding exposure classification. Some variation in the severity of the exposure is expected along the route and a detail variation is probably not needed in the document. However, some information about the variation along the route should be documented.

3.5.1 Air/atmosphere

The following should be included:

- Temperature range and the variation in a day
- The amount of rainfall
- Average relative humidity (RH)
- Amount of CO2
- Concentration of chlorides or any other pollutant
- Wind speed and direction of wind
- The distance of the structural elements from the coast and the extent of salt spray and wind driven chlorides

3.5.2 Ground

The following should be included:

- Bore hole analysis to determine the severity of the soil/ground.
- Chemical analysis of soil and ground water to measure the concentration of chlorides, sulphates, magnesium, ammonium or other chemical compounds.
- Permeability of soil
- Reduced level (RL) of ground water and floodplain location.
- SPOCAS (suspension peroxide oxidation combined acidity and sulphur) analysis, when acid sulphate soil (ASS) is present, to establish the soil classification and severity as AASS (actual acid sulphate soil) or PASS (potential acid sulphate soil).
- Any other relevant information.

At least one bore hole should be analysed for each of the bridge or major structure to determine the severity of the ground conditions. Also several bore holes should be analysed along the route to assess any variation in the type of soil/ground or groundwater.

3.5.3 Creeks/River/Lake

The following should be included:

- The amount of chlorides, magnesium and sulphates present in the water at the location of bridges.
- Tidal movement and distance from sea
- Reduced water level (RL)

3.5.4 Sea exposure

The following should be included:

- Details of the sea conditions
- Extent of splash activity, any salt spray, wind speed or any other factor which would influence the severity of the exposure condition.
- Any data from condition assessments of existing structures in the local area – helpful in establishing the exposure conditions.

3.5.5 Tunnel or special elements specific to the project

Most of the information described in subsections 3.5.1 to 3.5.4 are relevant to bridges, culverts, retaining walls, noise walls and other elements used in most major projects. It does not cover special elements which are not normally used in the projects, such as tunnel, large retaining wall or other large structural elements. Such elements should be separately covered in the durability plan and how the severity of the micro-environment relevant to the structure will be assessed.

3.5.6 Summary of data

This section should be presented in the body of the Durability Plan in a concise manner. A summary table should be prepared providing a summary of soil analysis for all bridges and all major structures. The table should include information on chainage, bore hole number, pH range, chloride and sulphate concentration, resistivity value, and whether the ground is PASS or not, see suggested table headings below.

Table 3: Summary of Data at Bridge Structures

Asset Brief Description	**pH**	**Sulphate Conc.** (SO4) (ppm)	**Chloride Conc.** (Cl) (ppm)	**Magnesium Conc.** (Mg) (ppm)	**Permeability** (m/s)	**Resistivity** (ohm.cm)	**SPOCAS**

Table 4: Summary of data at other locations

Chainage	**pH**	**Sulphate Conc.** (SO4) (ppm)	**Chloride Conc.** (Cl) (ppm)	**Magnesium Conc.** (Mg) (ppm)	**Permeability** (m/s)	**Resistivity** (ohm.cm)	**SPOCAS**

Other details that should be included in the appendix include:

- Details of the results of the bore analysis
- Results of all bore analysis on which testing is carried out
- Chemical analysis of water of lakes/creeks
- Groundwater results
- Results of SPOCAS analysis
- Other relevant information such as reports on the conditions assessment of existing structures.

3.6 Chapter 5 – Classification of exposures

Relevant deterioration mechanisms should be highlighted for various elements. Sometimes part of the element could be buried and part exposed to atmosphere and consequently the relevant deterioration mechanisms will be different. Thus all relevant deterioration mechanisms should be documented.

There is no need to provide the details of the deterioration mechanisms or the causes of the deterioration or the consequences of the deterioration mechanisms as this information is readily available in the literature.

Subsequently, based on the severity of the environment and the deterioration mechanisms, exposure classification for various elements of different structures should be documented and tabulated, see suggested table headings in Table 5. The rationale behind the selection of exposure classification should be also documented.

Table 5: Limits for determining exposure classifications

Sulfates, (SO_4^{2-})		**pH**	**Chlorides in groundwater**, (ppm)	**Exposure Classification**
In Soil, (ppm)	**In groundwater**, ppm			

Various RMS QA specifications and BTDs, Australian Standards, international guidelines or other references which were used to determine exposure classifications should be documented.

Exposure classification should be summarised and presented in a table, see suggested table headings in Table 6.

Table 6: Summary of Exposure Classification

Asset No	**Asset Description**	**Chainage**	**Exposure Classification**			**Bore hole used for durability assessment**
			Super-structures	**Sub-structures**	**Piles**	

3.7 Chapter 6 – Details of the materials and protective measures

3.7.1 Concrete

3.7.1.1 Materials

Requirements on concrete to achieve the required design life (normally 100 years) in the micro-environment for various structural elements should be documented. Grade of the concrete, binder type, cover values, curing, compaction, cast-in-situ or precast, reinforcement details or any other requirement to achieve its design life should be provided, see suggested table headings in Table 7.

Table 7: Summary of Requirements for Concrete for Exposure Classification U-C*								
Exp. Class	**SCM**	**Cement Content** (kg/m^3)		**W/C Ratio**		**Max Chloride Coeff** (x 10^{-12} m^2/sec)		**Fc.min (d) (MPa)**
		Min	**Max**	**Min**	**Max**	**NT 443**, De	**NT 492**, D_{RMC}	
U-C*								

Tabulate concrete cover values for various elements, see suggested table headings in Table 8.

Table 8: Concrete Cover Requirement to achieve Required Design Life (normally 100 years)						
Exposure Environment		**Exposure Classification**	$F_{c.min\ (d)}$ (MPa)	**SCM (%)**	**Nominal Cover**, (mm)	**Rationale for selecting cover values**
Atmospheric						
Buried						
Marine	Tidal/Splash/Spray					
	Atmospheric					

3.7.1.2 Additional protection measures

Details of any additional (beyond the requirements of Australian Standards and RMS specifications) protective measures to achieve the design life of 100 years should be provided in this section. Some of the protective measures could be coatings (to resist acidic or sulphate attack, improve carbonation or chloride penetration resistance), increase local cover, add water-proofing compounds to concrete or incorporate migratory corrosion inhibitor.

3.7.1.3 Thermal crack control modelling

It is expected that CIRIA C660 modelling will be carried out on all or at least some of the elements where the minimum dimension is 1000 mm. Protective measures, such as insulated formwork, limit on formwork removal time, additional reinforcement and other requirements should be discussed in this section. Also, measures adopted to minimise the risk of restraint shrinkage cracking should be considered and provided in this section.

3.7.2 Steel

Requirement on steel grade, surface treatments or any other requirement should be stated.

3.8 Chapter 7 – Maintenance Schedule

The aim of this section is also to familiarise the asset owner of its responsibilities/duties in ensuring that the design lives of various structural elements are met.

Durability of various elements, their design lives and the maintenance requirements are integral part of the design. Section C6.2 of the Supplement to AS 5100.1-2004, states that '*This assumption of a nominated design life does not mean that the bridge will no longer be fit for service when it reaches that age, or that it will reach that age without adequate and regular inspection and maintenance*'. Thus, AS 5100.1-2004 inherently assumes that some level of periodic inspection and maintenance will be carried out.

The extent of inspection and maintenance should be discussed in this section. The RMS policy PN 158 and relevant RMS BTDs should be considered in drafting the inspection and maintenance regime.

Any specific maintenance requirement which is part of the design should be highlighted in this section.

3.9 Chapter 8 – Summary of Information

A Table should be prepared which summarises information for various structural elements and their components.

The tabulated information should include the expected service life, surrounding environment, material used (concrete grade, cover etc), relevant deterioration mechanisms, exposure classification, expected construction, Structural design requirements, durability requirements and other requirements (formwork removal time, application of curing compound etc), see suggested table headings in Table 9.

Table 9: Summary of Minimum Durability Requirements *[best presented on landscape lay out]*

Element	Design Life (Years)	Environment	Expected Construction with Respect to Durability	Exposure Classification	Expected Curing Method
(Continued)	**Durability Issues**	**Material Requirements for Durability**	**Protective Measures**	**Additional Durability Requirements**	**Comments**

3.10 Chapter 9 – References

Document all references used in the report here.

3.11 Chapter 10 – Appendices

The Durability Plan is intended to be a stand alone document and should not refer to any other document or report. Also, the main part of the Durability Plan should be kept as brief as practical to facilitate the communication of information to RMS, contractor and other teams involved. Thus, any “extra” information should be annexed in the Appendix instead of incorporating in the main document.

The following information should be provided in the assigned appendix number. If the information is not available for a particular appendix, a ‘NOT USED’ note should be indicated.

3.11.1 Appendix A – Results of the chemical analysis of bore holes

Both soil and groundwater data should be included. Results of analysis of water of creeks/lakes along the route should be also provided.

3.11.2 Appendix B – SPOCAS and NAG results

Results of SPOCAS (suspension peroxide oxidation combined acidity and sulphur) analysis should be given here. Also NAG (net acid generation) results should be included.

3.11.3 Appendix C – Condition assessments of existing structures

Report on the condition assessment of existing assets adjacent to the route to indicate the severity of the environment should be a part of this appendix. Also any other condition assessments to validate the limits provided in AS 2159 or other standards should be also included here.

3.11.4 Appendix D – Chloride ingress modelling

Chloride ingress modelling details for tidal, submerged and atmospheric exposure should be provided.

3.11.5 Appendix E – Carbonation modelling

Details of the carbonation modelling should be given here to determine the cover concrete required to provide required design life, normally 100 years for bridges.

3.11.6 Appendix E – Thermal crack control modelling

Measures required to minimise the risk of differential thermal shrinkage and restraint cracking should be assessed in this appendix. Preferred model is CIRIA C660.

4 References

4.1 Roads and Maritime Services

None with direct reference.

4.2 Main sources

- Guideline for the preparation of Road Structures Durability Plans – Queensland Department of Main Roads
- Several existing durability plans for road infrastructure projects, rail project, desalination project

4.3 Other related publications

- F. Blin, S. Furman and A. Mendes, 2011, 'Durability design of infrastructure assets-working towards a uniform approach', *Proceedings of the 18th International Corrosion Congress*, Paper 212, Perth, WA.
- ISO 13843:2008 'General principles on the design of structures for durability', Case postale 56, CH-1211 Geneva 20, Switzerland

5 Attachments

5.1 Attachment A: Sample Durability Plan

A sample Durability Plan has been prepared to illustrate how a durability plan should look like. The sample Durability Plan provides details on what information should be provided in such a document and what information are considered important to RMS. It also indirectly guides the durability designer about the various steps/analysis necessary to ensure the durability performance of a structural element and achieve its Design Life.

Attachment A

Sample Durability Plan

A2B-DU-RP01

A2B-DU-RP01
Durability Plan for A to B Highway

Prepared for

Roads and Maritime Services

Prepared by

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Executive Summary

A durability assessment of structures in A to B highway has been performed. The route is 24.4 km long and involves construction works from the intersection of the Bruxner and Hume Highways south of the city of A to the intersection of Smith Lane and the George Street, north of the city B. Five twin bridges (northbound and southbound), 7 box culverts, numerous pipe culverts and other civil structures are to be constructed as part of the highway and the design life of these structures is 100 years. The structures are mainly to be constructed of reinforced or prestressed concrete.

Durability predictions for reinforced concrete structures and ground improvement works have been made based on the available data for the atmospheric conditions at the site and based on site soil and creek water test results. The greatest threats to the durability of cementitious materials are considered to be any actual acid sulphate soil (ASS) along the route, and elevated sulphate levels in the soil and groundwater. The greatest threats to embedded reinforcement in concrete structures is, in addition to the sulphate / acid sulphate soil conditions, the elevated chloride levels in tidal creek water, and airborne chlorides.

Modelling of chloride diffusion and carbonation of the proposed reinforced or prestressed concrete structures was conducted to assess the risk of reinforcement corrosion for the likely concrete mixes, assuming mild steel reinforcement is used. In addition, the risk of degradation due to sulphates and acid sulphate soils was assessed. The modelling found that suitable concrete mixes can be provided to adequately protect the mild steel reinforcement during the 100 year design life. Recommendations for the necessary concrete mix designs and depths of cover needed to prevent deterioration during the design life are made for the various assets in the different exposure classifications. Recommendations of BRE Digest and ACI 201.2R have been considered in the selection of concrete mixes, strength and type of binder.

For the project specific concrete requirements is based on the RTA Specification B80 – Concrete Work for Bridges. The exposure classification, U, has been expanded to include U-C1 and U-C*, which provide for the more aggressive acid sulphate conditions along the route. Specific concrete requirements are included in the specification, to achieve concrete structures that will have appropriate durability.

Sampling and testing of concrete from the Deep Creek was undertaken to verify parameters used in the chloride diffusion modelling.

The soil testing performed to date has been limited to environmental testing to a shallow depth at selected areas of the route to assess acid sulphate conditions, and to traditional testing for durability assessment of concrete assets at the locations of each bridge. Measurement of soil pH is therefore recommended to be performed in situ at all sites in existing soil where concrete structures are to be built, including pits, culverts, pipelines, etc.

1.0 Introduction

1.1 Background

City A is located on the coast of southern New South Wales. It is situated at the mouth of the Richmond River and has a subtropical climate. As part of the Hume Highway Upgrade, a highway is to be built from A to B. The route is 24.4 km long and involves construction works from the intersection of the Bruxner and Hume Highways south of the city of A to the intersection of Smith Lane and George Street, north of the city B. Figure 1 shows a map of the planned route between A and B. Eight twin bridges (northbound and southbound), 12 box culverts, numerous pipe culverts and other civil structures are to be constructed as part of the highway

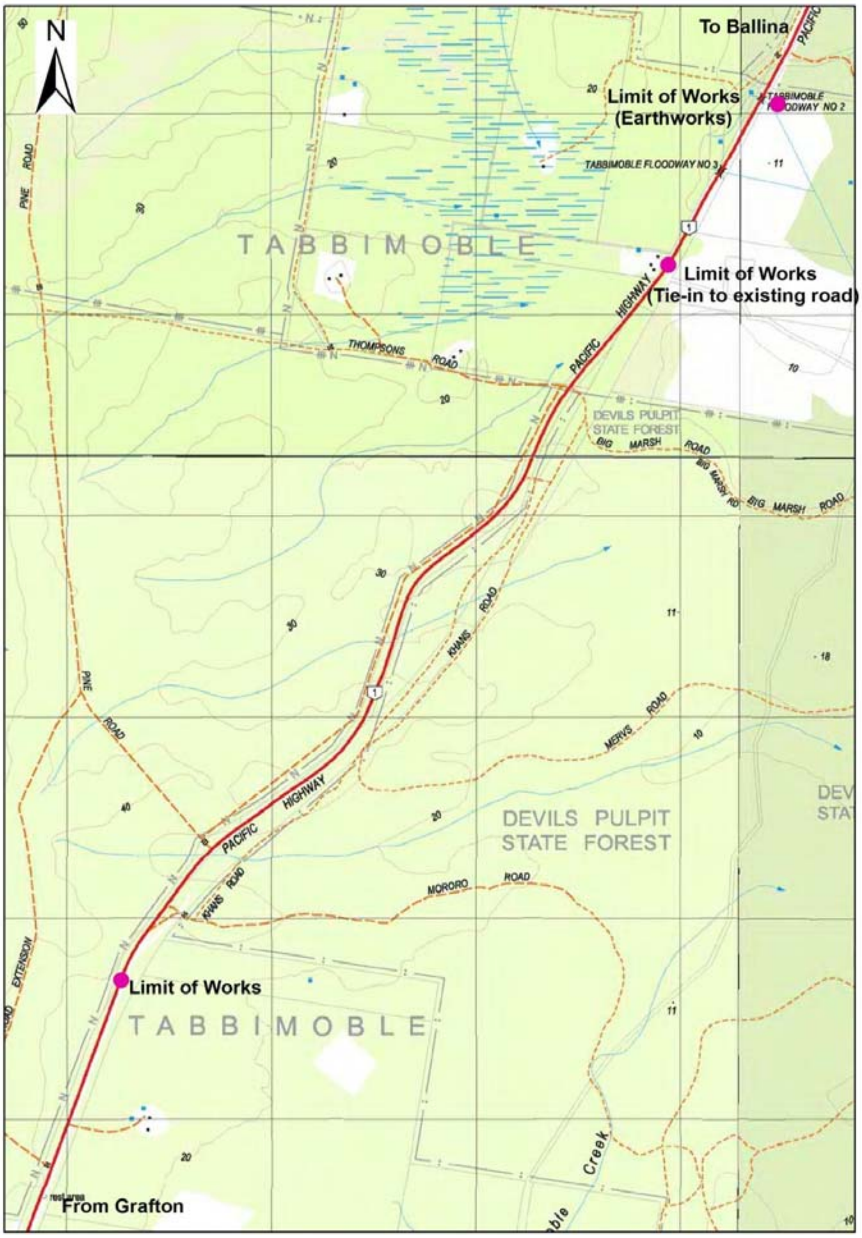

Figure 1. Map of route between A and B (Source: Google Earth).

1.2 Description of Proposed Structures

The structures that are included in the project scope include:

- 5 Twin Bridges
- 7 box culverts
- Retaining walls
- Culverts
- Pipelines and pits
- Ground improvement works

The proposed bridge structures are summarised in Table 1:

Table 1. Description of Bridge Structures

Bridge Details	Description	Chainage
Twin Bridge over Deep Creek	In-situ balanced cantilever, 70 m span over the creek, pier on each bank with bored piles in sacrificial permanent steel casings. Abutments with precast octagonal piles.	138220
Twin Bridge over Smith Road Interchange	Super T, propose bottom up construction of this structure, bored pile foundation at abutments and pad footing at piers.	139600
Twin Bridge over Middle Creek	Planks, precast octagonal piles, piles terminating into headstocks.	141290
Twin Bridge over Tarban Creek	Super T, precast octagonal piles, pile caps above water table.	149580
Twin Bridge over Richmond River	Super T, precast octagonal piles, pile caps above water table.	154750

The drainage structures consist of several box culverts and pipe culverts. Both longitudinal and transverse pipe culverts exist in the project.

1.3 Chainages of the route

The chainage of the start of the route is 136480 north of Maitland and the end of the route is at a chainage of 160880.

2.0 Scope

2.1 Scope of Durability Plan

The purpose of this durability plan is to:

a) Provide a durability review of the structures proposed for the A2B project and identify potential issues affecting durability
b) Analyse and predict the interactions between the structural elements and the exposure environment
c) Provide guidelines to the designers of the assets on how to achieve the required design life

The durability plan covers the various items mentioned below in Table 2.

2.2 Design Life

Table 2 summarises the required design life of the elements to be constructed for A2B highway.

Table 2. Design Life Requirements

Asset	Item	Design Life
Bridges	Piles	100 years
	Pile caps	100 years
	Headstocks	100 years
	Piers	100 years
	Abutments	100 years
	Deck	100 years
	Approach slabs	100 years
Drainage Structures	Drainage structures - accessible	50 years
	Drainage structures – inaccessible	100 years
	Box culvert (crown units and link slabs)	100 years
	Box culvert base slab	100 years
	Pipe	100 years
Fencing	Fencing and Gates	20
Roadscape	Signs - Posts	10
	- Sign Faces	10
	- Surface coating systems	20
	- Fixings and Brackets	40
Road Furniture	Guideposts:- wood, plastic, metal corner cube / other reflectors	8
	Safety Fencing:-corner cube / other reflectors	8
	Guard Rail-steel / timber posts, single / double sided	40
	Wire Barrier	40
	Guardrail Breakaway Terminals (BCT's)	40
	Other Guardrail Terminals	20
	Pedestrian Bollards – Bollards, Refuges	20
	Pedestrian Grab Rails	20

Retaining walls and reinforced soil walls have a design life of 100 years.

3.0 Definition of Service Life

The design life is the period from construction when the asset remains suitable for service without requiring major maintenance. For this project, the definition of service life given in AS 5100.1 has been adopted.

4.0 Severity of Exposures – details of environment

4.1 Air Quality and proximity to the Ocean

The route is about 500 m to 3 km from the coast. According to the National Pollutant Inventory (www.npi.gov.au), the primary pollutants in the area are toluene, xylenes and volatile organic compounds from the Shell Airport Depot and nitrogen and phosphorous associated with cropping and other agricultural activities. The route is not located near major sources of atmospheric pollutants such as smelters or other heavy industry. Thus, exposure to high concentrations of CO, CO_2, NO_x and SO_x is unlikely.

Wind speed and direction influence the movement and dispersion of chlorides, pollutants and aggressive airborne species. Annual wind rose data at 9am and 3pm for the route generally indicate southwesterly winds in the mornings and southerly or north-easterly winds in the afternoons. (http://www.bom.gov.au/climate/averages/wind/selection_map.shtml). There is a high likelihood that the afternoon southerly and north-easterly winds from the coast may bear airborne chlorides.

4.2 Effect of Rainfall

Annual rainfall along the route is around 1,700 mm (www.bom.gov.au) which is regarded as relatively high. This high rainfall will affect the time of wetness of exposed metals and can lead to faster rates of corrosion.

4.3 Effect of Temperature

Review of data available from the Bureau of Meteorology shows the route is in a subtropical climate (www.bom.gov.au/climate/averages/tables.shtml). Monthly mean maximum temperatures range from 19.9 to 28.2°C and monthly mean minimum temperatures are between 8.5 and 19.5°C. The annual mean maximum temperature is 24.4°C and the mean minimum temperature is 14.2°C.

4.4 Effect of Relative Humidity (RH)

Daytime relative humidity along the route is generally within the range of 60-75%.

4.5 Soil and Groundwater Exposure

Site investigations have identified widespread presence of acid sulphate soils along the route alignment (Acid Sulphate Soil Management Strategy-Proposed A2B highway).

Test data for the site show:

- The presence of ASS, PASS, ASR and naturally acidic soils at most locations;
- Soil chlorides, sulphates and pH levels in the ranges of 10 – 830 ppm, 10 – 1270 ppm and 3.5-7.5, respectively;
- Groundwater chlorides, sulphates and pH levels in the ranges 46-2240ppm, 11-425ppm and 4.0-8.6 respectively.

Data on soil and groundwater testing has been compiled in the Appendix A.

4.6 Creek Water Exposure

The project involves several creek crossings and only two creeks (Deep and Tarban Creek) are tidal. Chemical test data for creek water for the site show chloride, sulphate and pH levels in the ranges of 9-2240ppm, <1-22ppm and 5.5-6.9, respectively.

4.7 SPOCAS Analysis

Out of the 10 locations, at eight locations the ground was found to be PASS and at one location the ground was AASS. The chromium reducible sulphur ranged from 0.34 to 0.45% and the titratable peroxide activity ranged from 45 to 110 mole H^+/tonne. The soil was classified as "high permeability soils" as the permeability of the soil ranged from 2 to 5.2 x 10^{-5} m/s.

4.8 Summary of data

Table 3 below gives the summary of results of chemical analysis at bridge structures. Also Table 4 gives the summary of data at other locations.

Table 3. Summary of data at Bridge Structures

Bridge	pH	Sulphate Conc. (SO4) (ppm)	Chloride Conc. (Cl) (ppm)	Magnesium Conc. (Mg) (ppm)	Permeability (m/s)	Resistivity (ohm.cm)	SPOCAS
Twin Bridge over Deep Creek	4.5-5.2	210-320	2000-5200	540-620	$2x10^{-5}$	220-320	S_{cr}=0.1%, TPA=54 moles H^+/tonne PASS
Twin Bridge over Smith Road Interchange	3.8-4.3	680-980	2300-3500	230-320	$3.2x10^{-5}$	1500-2500	S_{cr}=0.3%, TPA=24 moles H^+/tonne PASS
Twin Bridge over Middle Creek	4.7-5.2	580-1240	2200-5400	210-430	$5.2x10^{-5}$	540-890	S_{cr}=0.02%, TPA=14 moles H^+/tonne Not PASS
Twin Bridge over Tarban Creek	4.1-6.2	240-563	2254-8900	320-510	$2.1x10^{-5}$	2200-2350	S_{cr}=0.4%, TPA=42 moles H^+/tonne PASS
Twin Bridge over Richmond River	4.8-6.4	320-890	2100-8540	580-870	$3.4x10^{-5}$	850-940	S_{cr}=0.24%, TPA=31 moles H^+/tonne PASS

Table 4. Summary of data at other locations

Chainage	pH	Sulphate Conc. (SO4) (ppm)	Chloride Conc. (Cl) (ppm)	Magnesium Conc. (Mg) (ppm)	Permeability (m/s)	Resistivity (ohm.cm)	SPOCAS
142580	4.8-6.4	320-890	2100-8540	580-870	$3.4x10^{-5}$	850-940	S_{cr}=0.1%, TPA=54 moles H^+/tonne PASS
144630	4.1-6.2	240-563	2254-8900	320-510	$2.1x10^{-5}$	2200-2350	S_{cr}=0.3%, TPA=24 moles H^+/tonne PASS

5.0 Classification of exposures

5.1 Atmospheric Exposure

The basic exposure classifications stipulated by AS5100.5 are as follows:

- B1 – where a structure is located between 1 km and 50 km of the coastline.
- B2 – where a structure is located within 1 km of the coastline

However, AS5100.5 further notes that where there are strong prevailing winds or vigorous surf, the B1 exposure classification should be increased beyond 1 km. As the route is subject to strong NE/SE winds, high RH and high annual rainfall, the exposure classification zone for B2 is increased up to 2 km from the coast line. Thus for this project, the exposure classifications is :

- B1 – where a structure is located between 2 km and 50 km of the coastline.
- B2 – where a structure is located within 2 km of the coastline

5.2 Buried in ground

Soil analysis data has indicated permeability ranged from 2 to 5.2 x 10^{-5} m/s and therefore the soil is classified as high permeability soil according to Section 3.1 of RTA BTD 2008/12.

PASS have been encountered along part of the route and Table 1b and 2b of RTA BTD 2008/12 has been used to determine the exposure classification.

For the remaining part where PASS has not been observed, Table 6.4.2 (C) of AS 2159 is used to determine the exposure classification. The soil condition “A” is selected as the permeability is from 2 to 5.2 x 10^{-5} m/s. Alternatively Table 4.8.1 of AS 3600 can be also used to determine the exposure classification, however the design life is 50 years and thus a conservative approach should be adopted if AS 3600 is used to determine the exposure classification.

Magnesium content has been found to be less than 100 ppm and thus risk of magnesium sulphate attack is considered to be low.

The sulphate limits in AS 2159 and AS 3600 were compared to the limits given in two well known international guidelines ACI C201-R and BRE Digest. These guidelines are the most respected guidelines for classifying high-sulphate environments. It was found that the limits of AS 3600 and AS 2159 are significantly higher than the limits of two international guidelines and therefore the limits of international guidelines were used to assess the exposure classifications and are given in Table 5.

Condition assessments have been carried out for the two existing structures and the results have indicated that the chloride limits of AS 2159 are not appropriate and therefore the limits were modified and are given in Table 5. The reports are given in Appendix B.

Table 5. Limits for determining exposure classifications

Sulfates (expressed as SO_4^{2-})		pH	Chlorides in groundwater (ppm)	Exposure Classificatio n
In soil (ppm)	In groundwater (ppm)			
< 500	< 300	> 5.5	<2,000	B1
500 – 2,000	300 – 1,200	4.5 to 5.5	2,000 to 6,000	B2
2,000 – 10,000	1,200 – 5,000	4.0 to 4.5	6,000 to 18,000	C
> 10,000	> 5,000	< 4.0	> 18,000	U-C*

Condition assessments have been also carried out on existing structures along the route. These testing were carried out to confirm the exposure classifications established for structural elements along the route. The report is given in Appendix C.

5.3 Structures in water - chloride Ingress

Since some of the piers are located in sea water, modelling has been carried out to assess the severity of the environment. The details of the chloride modelling and the requirements for concrete and cover are given in Appendix D.

5.4 Carbonation of Concrete

The carbonation modelling is given in Appendix E. The requirements for concrete and cover are given in Appendix E.

5.5 Exposure classification for metallic components

The durability requirements of various metallic components exposed to atmosphere has been considered. Some of such structural components are : street light poles, signage, steel wires in fencing, guide posts, safety barriers and other road furniture.

Based on the information given in AS/NZS 4312, the proximity of the A2B to the coast and the number of creek and river crossings the atmospheric corrosivity of the A2B has been classified as C3 - Medium.

5.6 Summary of exposure classification

Based on the criteria given above, exposure classification has been assigned to various structural elements of the bridges. Table 6 gives summary of the exposure classifications for some of the elements. The details of exposure classifications for all elements are given in the respective design reports. Sub-structures refer to abutment, pile cap and piers.

Table 6. Summary of exposure classification

Bridge	Chainage	Exposure classification			Bore hole used for durability assessment
		Super-structures	Sub-structures	Piles	
Twin Bridge over Deep Creek	138220	B2	B2	C	BH15, TP32
Twin Bridge over Smith Road Interchange	139600	B1	B2	B2	BH45
Twin Bridge over Middle Creek	141290	B1	C	U-C*	TP34, 29CB1
Twin Bridge over Tarban Creek	149580	B2	C	C	BH45
Twin Bridge over Richmond River	154750	B2	B2	B2	BH56
	142580	B2	C	C	TP55, BH89
	144630	B1	C	U-C*	BH101
	158600	B2	U-C*	U-C*	BH103

6.0 Details of the materials

6.1 Concrete grade and other requirements

For B1, B2 and C exposure classification, concrete requirements provided in Table B80.6 of B80 will be adopted. For exposure classification of U-C*, the requirements are given in Table 7.

Table 7. Summary of Requirements for Concrete for exposure classification U-C*

Exp. Class	SCM	Cement Content (kg/m^3)		W/C Ratio		Max Chloride Coeff ($\times 10^{-12}$ m^2/sec)		$F_{c.min\ (d)}$ (MPa)
		Min.	Max	Min.	Max	NT 443, D_e	NT 492, D_{RMC}	
U-C*	65% slag	500	600	0.30	0.35	1.2	2.5	60

6.2 Cover values

The minimum acceptable cover is the cover values provided in AS 5100.5 and AS 2159. Table 8 below gives the cover values for various elements. The details of the concrete are given in Section 6.1.

Table 8. Cover required to achieve design life of 100 years

Exposure environment		Exposure Class	$F_{c.min\ (d)}$ (MPa)	SCM (%)	Nominal Cover (mm)	Rationale for selecting cover values
Atmospheric		B1	32	25 FA	65	Greater of the cover values given by carbonation modelling and AS 5100.5
					60-precast	
			40	25 FA	45	
					40-precast	
		B2	40	25 FA	55	Highest cover values given by carbonation modelling, chloride ingress modelling and AS 5100.5
					50-precast	
			50	25 FA	45	
					40-precast	
Buried		B2	50	25 FA	65	Highest of the cover value given by chloride ingress modelling and to withstand sulphate attack and/or acid attack.
					60-precast	
		C	50	65 BFS	70	
					65-precast	
		U-C*	60	65 BFS	70	
					65-precast	
Marine	Tidal, splash, spray	C	50	65 BFS	70	Greater of the cover values given by chloride modelling and AS 5100.5
					65-precast	
	Atmos.	B2	40	25 FA	55	
					50-precast	

1) For precast concrete units it is assumed that steel rigid formworks are used with intense compaction – reduction of 5 mm cover is allowed.
2) Additional cover where cast directly against ground as required by Section 4.10.3.3 of AS 5100.5. Increase the cover by 5 mm for casting against blinding layer due to un-evenness of blinding layer.
3) Increase cover by 10mm for prestressed strand and cables

6.3 Curing

Curing provision A (Performance) according to Section 3.4.1 of B80 will be adopted. Concrete mixes should comply with the sorptivity requirements.

6.4 Additional protective measures

Elements which cannot be easily accessed and therefore are difficult to repair or replace, those items should have longer service life. Dowels will be prepared from stainless steel. Scuppers cast in the deck will be prepared from cast iron coated with an epoxy. Also cast-in ferrules, anchors and fasteners will be prepared from stainless steel.

Piles which are in environment with pH less than 4 will be coated with vinyl Esther coating in addition to the concrete for U-C* exposure classification (S60 concrete with 65% slag and 65 cover for precast). These precast piles will be coated with vinyl Esther before they are driven in ground.

Piers and pile caps which are in severe marine environment will be fitted with impressed current cathodic protection from the beginning of the service life and will be cathodically prevented.

Piers which are partially buried in acidic ground, the part which is buried will be coated with an epoxy coating such as Nitocote EP 410 or equivalent.

For decks, it is important to follow good practices of hot weather concreting. Use of aliphatic alcohol or erection of wind barriers to retard the evaporation of bleed water will be followed, if considered necessary.

6.5 Differential thermal shrinkage cracking and restraint cracking

CIRIA660 programme will be used to determine what measured need to be adopted to minimise the risk of differential thermal shrinkage cracking and/or other forms of restraint cracking.

For following conditions, analysis using CIRIA C660 will be carried out to determine the measures required to minimise risk of differential thermal shrinkage cracking (internal restraint):

(i) Grade S50 Concrete – minimum dimension exceeds 0.5 metres
(ii) Grade S40 Concrete – minimum dimension exceeds 1.0 metres
(iii) Grade S32 Concrete – minimum dimension exceeds 1.5 metres

The measures adopted to minimise risk of differential thermal shrinkage cracking include :

- Use of insulated formwork. Such formwork must consist of plywood formwork of minimum 18 mm thickness with at least 10 mm thick polystyrene foam attached to it. Alternatively a material or assembly of materials with equivalent insulating properties can be also used.
- Formwork must not be removed until the poured concrete is at least 7 days old
- Formwork to be removed during the warmer part of the day, say 10 AM to 3 PM.
- Use of extra reinforcement on the outside.

For elements subjected to edge or end restraint, CIRIA C660 calculations will be also performed. Such calculation will be carried out if an element is cast on another component and if that component is more than 7 days old. The measures adopted to minimise risk of restraint cracking include :

- Limitation on period between casting of the two elements
- Use of extra reinforcement at the interface

Details of the calculations and recommendations are given in Appendix F.

7.0 Maintenance Schedule

7.1 Overview

Structures will be designed to readily enable items such as bearings, expansion joints and seals, and steel coatings to be maintained or replaced. Where an element is not readily accessible for maintenance or replacement, it will be designed to function for the life of the structure without maintenance.

During the service life of the assets, regular inspection is recommended. This may include but will not be limited to:

(a) Routine Inspections every two years:

- Visual inspections, photographic documentation and reporting on the condition of major bridge elements such girders, headstocks, abutments and their support structures, scour protection, embankments, barriers and railings.

(b) Condition Monitoring (every five years):
This monitoring will involve detailed visual inspection, photographic documentation and reporting on the condition of the major bridge elements as well as measurements of defects such as cracks, settlement and erosion.

Where evidence of deterioration is present, the following testing may also be undertaken:

- Limited sampling and testing of selected materials (e.g. concrete cores or breakouts) to visually inspect reinforcement bars
- Half cell potential surveys to determine corrosion activity of reinforcement.
- Chloride ion concentration measurements using concrete dust samples
- Carbonation testing by progressively drilling a 10mm to 15 mm diameter hole through the concrete cover zone at 2 mm intervals and spraying the hole with phenolphthalein solution.

(c) Servicing and Remedial Action:
This may include:

- Periodic cleaning of drains and desalting of sedimentation ponds
- Tightening of loose bolts and fixings
- Repair or replacement of deteriorated components and materials
- Maintenance, repair and re-instatement of protective coatings
- Timely response to major defects which require prompt servicing and repair.

8.0 Summary of Information

This section covers summary of various structural elements of the bridge.

Table 9. Summary of Minimum Durability Requirements

Element	Design Life (Years)	Environment	Expected Construction with Respect to Durability	Exposure Classification	Expected Curing Method	Durability Issues	Material Requirements for Durability	Protective Measures	Additional Durability Requirements	Comments
Foundations and Piles										
RC Pre-cast Octagon Driven Piles	100	Buried: mainly clay & silty clay; medium to high chloride levels; part sandy clay & sandy gravel.	Pre-cast and driven.	U-C*	Heat accelerated followed by 7d sealed curing	Sulphate attack and corrosion of reinforcement due to chloride ingress.	Min. 65% GGBFS Cem. Content min = 500 kg/m3 w/c max. = 0.35 28d fc.min(d) = 60 MPa Diff. Coeff.1 max. = 1.5 E-12 m2/s	Nom. cover (mm): 50		
RC Cast-In Place Piles with temporary steel sleeve	100	Buried: in steel casing to rock, mainly silty clay (medium to high chloride levels), below RL -30.0 siltstone.	Bored and cast in steel sleeve. Socketed into rock (argillite).	U-C*	wet curing – ground to be kept wet	Sulphate attack, corrosion of reinforcement due to chloride ingress and corrosion of permanent steel casing.	Min. 65% GGBFS Cem. Content min = 500 kg/m3 w/c max. = 0.35 28d fc.min(d) = 60 MPa Diff. Coeff.1 max. = 1.5 E-12 m2/s	Nom. cover (mm): 70		Cover reduced due to steel casing Steel sleeve is insufficient to reduce the exposure classification.
RC Cast-In Place Piles with permanent steel sleeve	100	Buried: in steel casing to rock, mainly silty clay (medium to high chloride levels), below RL -30.0 siltstone.	Bored and cast in steel sleeve. Socketed into rock (argillite).	B2	wet curing – ground to be kept wet	Sulphate attack, corrosion of reinforcement due to chloride ingress and corrosion of permanent steel casing.	Min. 25% PFA Complying to B80	Nom. cover (mm): 70		Cover reduced due to steel casing .

Element	Design Life (Years)	Environment	Expected Construction with Respect to Durability	Exposure Classification	Expected Curing Method	Durability Issues	Material Requirements for Durability	Protective Measures	Additional Durability Requirements	Comments
Piers and headstock										
RC Blade Pier or other piers	100	Atmospheric	Cast in forms on 50 mm thick N20 concrete blinding slab	B1	7d sealed curing	Corrosion of reinforcement due to chloride ingress and carbonation.	Min. 25% PFA 28d fc.min(d) = 40 MPa Complying to B80	Nom. cover (mm) Against Blinding=55 Elsewhere = 50		
Piers in brackish creek or sea water	100	Atmospheric over a creek or sea	Cast in forms on 50 mm thick N20 concrete blinding slab	B2	7d sealed curing	Corrosion of reinforcement due to chloride ingress.	Min. 25% PFA Complying to B80	Nom. cover (mm) Against Blinding=55 Elsewhere = 50		
Piers in tidal or spray or splash zone	100	Wet and dry in a creek or sea	Cast in forms	C	14d sealed curing	Corrosion of reinforcement due to chloride ingress.	Complying to B80	Nom. cover (mm) 70		
Headstock	100	Atmospheric	Cast in forms	B1	7d sealed curing	Corrosion of reinforcement due to chloride ingress and carbonation.	Min. 25% PFA 28d fc.min(d) = 40 MPa Complying to B80	Nom. cover (mm) Against Blinding=55 Elsewhere = 50		
Abutments										
Wing walls and head walls	100	Atmospheric: Open Partially buried: non-aggressive backfill in accordance with RTA Spec B30.	Cast in forms on 50mm thick grade N20 concrete slab.	B2	7d sealed curing	Corrosion of reinforcement due to chloride ingress and carbonation	Min. 25% PFA Complying to B80	Nom. cover (mm): Against blinding = 55 Elsewhere = 50		
Abutment Drainage	100	Buried: non-aggressive backfill	To manufacturer's requirements	NS	NA	Potentially acidic groundwater	To RTA 3556 • Cordrain/18 • Megaflow Strip	Proprietary: Manufacturer / Supplier to ensure performance requirements met		

Element	Design Life (Years)	Environment	Expected Construction with Respect to Durability	Exposure Classification	Expected Curing Method	Durability Issues	Material Requirements for Durability	Protective Measures	Additional Durability Requirements	Comments
							• Geotextile Wrap			
Lateral Restraint Blocks										
Lateral Restraint Block	100	Atmospheric: beneath bridge at abutments and central pier column	Cast in forms	B1	7d sealed curing	Carbonation	Min. 25% PFA 28d fc.min(d) = 40 MPa Complying to B80	Nom. cover (mm): 55		
Bridge Bearings										
Cement Mortar Pad	NA	Atmospheric: Beneath Bridge, between supports and laminated elastomeric bearing	Formed and placed in-situ	B1	NA	Cracking. Voids occurring due to incorrect workability, flowable consistency	Site-mixed & approved cement mortar to RTA B284	Expansive grouts and repair mortars are not permitted		
Laminated Elastomeric Bearing	75	Atmospheric: Beneath Bridge, between mortar pad and keeper plate	NA	NS	NA	Degradation: Ozone Attack	To RTA B281	Proprietary: Manufacturer / Supplier to ensure performance requirements met		
Girders										
1500 Deep Super-T Girder	100	Atmospheric: Beneath deck, on bearings	Pre-cast	B1	Heat accelerated curing	Cracking Carbonation	Min. 25% PFA Cem. Content min. = 370 kg/m^3 w/c max. = 0.46 28d $f_{c.min(d)}$ = 50 MPa	Nom. cover (mm): Outside face = 35 Inside face = 30 Top of flange = 20 Bottom of flange = 30		
Deck Cross Beams	100	Atmospheric: Beneath deck, tied into super-tees	Cast in forms	B1	7d sealed curing	Cracking Carbonation	Min. 25% PFA 28d fc.min(d) = 40 MPa Complying to B80	Nom. Cover (mm): 50		

<table>
<tr><th>Element</th><th>Design Life (Years)</th><th>Environment</th><th>Expected Construction with Respect to Durability</th><th>Exposure Classification</th><th>Expected Curing Method</th><th>Durability Issues</th><th colspan="2">Material Requirements for Durability</th><th>Protective Measures</th><th>Additional Durability Requirements</th><th>Comments</th></tr>
<tr><td colspan="12">Bridge Deck</td></tr>
<tr><td>RC Bridge Deck Slab</td><td>100</td><td>Atmospheric: at edges.
Embedded: between waterproofing membrane and super-tee girders</td><td>Cast in forms</td><td>B1 – Top surface
A – Soffit</td><td>3-d minimum of wet curing</td><td>Drying shrinkage cracking and corrosion of reinforcement due to either carbonation and/or chloride ingress.</td><td colspan="2">Min. 25% PFA
Cem. Content min = 370 kg/m^3
w/c max. = 0.46
28d $f_{c.min(d)}$ = 40 MPa</td><td>Nom. cover (mm):
Sides = 55
Top = 45
Underside = 30</td><td>Curing compound to be applied the same day the slab is poured</td><td>Gap between girders to be bridge with 150 wide tape to form increased local extra cover by 15 mm</td></tr>
<tr><td colspan="12">Bridge Barriers</td></tr>
<tr><td>RC Precast Barrier Panel</td><td>100</td><td>Atmospheric: tied & bolted to deck</td><td>Pre-cast</td><td>B1</td><td>Heat accelerated curing</td><td>Cracking
Carbonation</td><td colspan="2">Min. 25% PFA
Cem. Content min = 400 kg/m^3
w/c max. = 0.44
28d $f_{c.min(d)}$ = 50 MPa</td><td>Nom. cover (mm): 35</td><td>To RTA B153 and B115</td><td></td></tr>
<tr><td>M24 Hex Head Levelling Screw</td><td>100</td><td>Embedded: in Concrete stitch, epoxy glued to deck</td><td>NA</td><td>NS</td><td>NA</td><td>Corrosion</td><td>HDG Steel</td><td>≥ 52.5 µm Zn coating thickness</td><td>HDG to RTA B240
Nom. cover (mm): 55</td><td></td><td></td></tr>
<tr><td>Stitch Concrete</td><td>100</td><td>Atmospheric</td><td>Cast in-situ</td><td>B1</td><td>7d sealed curing</td><td>Drying shrinkage cracking and corrosion of reinforcement due to either carbonation and/or chloride ingress.</td><td>Min. 25% PFA
Cem. Content min = 400 kg/m^3
w/c max. = 0.44
28d $f_{c.min(d)}$ = 50 MPa</td><td>Nom. cover (mm):
40 (min)
60 (max)</td><td></td><td></td><td></td></tr>
<tr><td colspan="12">Traffic Barrier Railing</td></tr>
<tr><td>Traffic Barrier Railing</td><td>40</td><td>Atmospheric: bolted to top of precast bridge barriers</td><td>NA</td><td>C (AS 2312)</td><td>NA</td><td>Corrosion</td><td>HDG Steel</td><td colspan="2">≥ 84 µm
Zn coating thickness
HDG to RTA B220 and B241</td><td>Damaged coating to be reinstated with zinc rich</td><td></td></tr>
</table>

Element	Design Life (Years)	Environment	Expected Construction with Respect to Durability	Exposure Classification	Expected Curing Method	Durability Issues	Material Requirements for Durability		Protective Measures	Additional Durability Requirements	Comments
										primer. Edges to be HDG rounded to 1.5mm radius.	
Anchor Bolt Assemblies (Nut, Washer, Steel Bolt and Anchor Plate)	100	Embedded: in concrete of precast barriers Atmospheric: at ends	NA	C (AS 2312)	NA	Corrosion	HDG Steel	≥ 52.5 µm Zn coating thickness	HDG to RTA B240		
Approach Slab and Dowel Assembly											
RC Approach Slab	100	Buried: underneath asphalt, between abutment wingwalls, blinding slab	Cast in forms on grade N20 concrete blinding slab	B1	7d sealed curing	Cracking Carbonation	Min. 25% PFA Cem. Content min = 370 kg/m^3 w/c max. = 0.46 28d $f_{c.min(d)}$ = 40 MPa	Nom. cover (mm): Against blinding = 55 Elsewhere = 45 Fill to be screened and non-aggressive.			Reduced cover due to blinding slab & sheltered environment
20 DIA Stainless Steel Dowel	100	Embedded: in concrete of abutment	NA	NS	NA	Corrosion due to either carbonation and/or chloride ingress.	Stainless steel Grade 304 to ASTM A276				
20 Thick Cellular Polystyrene Sheeting	NA	Embedded: between concrete of abutment and approach slab	NA	NS	NA	Degradation: General wear and tear; Microbial attack; Oxidation	Polystyrene Sheeting			Class H in accordance with AS 1366.3.	
Drainage											

Element	Design Life (Years)	Environment	Expected Construction with Respect to Durability	Exposure Classification	Expected Curing Method	Durability Issues	Material Requirements for Durability	Protective Measures	Additional Durability Requirements	Comments
225 DIA FRC Drainage Pipe	40	Atmospheric: under bridge	Pre-cast	B1	NA	Drying shrinkage cracking, degradation of concrete due to exposure to aggressive chemical spillages.	FRC	As per manufacturer's requirements and AS5100.5.		
FRC 225/100 Saddle Tee Adaptor	40	Atmospheric: under bridge, water, road spillages	Pre-cast	B1	NA	Drying shrinkage cracking, degradation of concrete due to exposure to aggressive chemical spillages.	FRC	As per manufacturer's requirements and AS5100.5.	FRC 225/100 Saddle Tee Adaptor	
Cast Iron Inlet Pipe	100	Atmospheric: water, road spillages Embedded: in deck concrete	NA	C (AS 2312)	NA	Corrosion	Epoxy Coated Cast Iron		Exposed surface not a risk, other surface embedded in concrete.	
R10 Galvanised Steel Bar	40	Atmospheric: on deck surface, water, road spillages	NA	C (AS 2312)	NA	Corrosion	HDG steel ≥ 84 µm Zn coating thickness	HDG to RTA B241	Across outlet pipe to prevent ingress of large articles. To be replaced if missing or damaged	
Expansion Joints										
Granor Etic EJ 160 Cast In Finger Type Expansion Joint System	NA	Atmospheric	NA	C (AS 2312)	NA	Wear and tear due to traffic.	NA	Proprietary: Manufacturer / Supplier to ensure performance requirements met.		

8.1 Other Recommendation Related to Durability Plan

In addition to the details of the concrete requirements, following recommendations are also given related to the durability plan and the project.

- It is recommended the Durability Plan be used as a living, working document throughout the project.

- At the detailed design stage of the project, further review is recommended to be performed of the recommendations for concrete structures after the actual concrete mixes have been decided. The diffusion coefficients for the proposed mixes and other mix information will be examined to confirm the design life can be achieved by the mixes in the relevant exposure classifications. Further details would be added to the Durability Plan, where relevant, for more specific available information.

- Inspection, sampling and testing of selected existing reinforced concrete structures is recommended at key areas along the route, including those where acid sulphate conditions were previously identified at severe levels and where structures would be subjected to tidal brackish water conditions, or are permanently located directly over brackish water channels or creeks. The findings would provide input to the Durability Plan and amendments would be made, accordingly, even if only to document the survey and confirm the suitability of the current approach.

- Creek and ground water is recommended to be sampled and tested to identify potential durability hazards for reinforced concrete structures. The data would be added to the Durability Plan and recommendations would be updated accordingly, if needed.

- Site pH testing is recommended as the work proceeds to identify any local strongly acidic areas, with pH < 3.5. Very few, if any, areas are anticipated to be in this category, however, and if any are present, based on the available soil test results; they are likely to only affect the ground to a shallow depth. The test method should be in accordance with accepted soil testing for concrete structures durability assessment (as distinct from environmental testing). Special measures will need to be taken at any locations of extreme low pH, such as soil replacement, cement or lime stabilisation, or other protection for reinforced structures that would otherwise be placed in direct contact with the soil.

- At the detailed design stage the durability plan is recommended to be used to develop detailed lists of assets and asset components with specific requirements for concrete grade, cement type, cover, curing, and specific construction or maintenance works that will be required for the structure to achieve the design life.

- During construction, it is recommended that any changes to designs and any construction issues that may influence durability or future maintenance be recorded on asset and asset component lists. This would include recording relevant items from review of RFI’s and NCR’s, in addition to those assessed from review of “as built” details. The updated asset information is recommended to be used to prepare, at the end of the construction period, asset data sheets for each asset with durability information and monitoring and maintenance recommendations. The asset data sheets envisaged would be suitable for use with GIS and would be able to be tailored to suit existing RMS asset management databases.

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10.0 Appendices

10.1 Appendix A – Summary of Bore Hole Analysis

Chainage	Bore Holes	pH	Resistivity ohm.cm	EC mS/cm (1:5)	SO_4 mg/kg or ppm	Cl^- mg/kg or ppm	Mg mg/kg
113420	TP101	5.4	18000	0.06	8	33	1400
113720	TP104	5.1	26000	0.04	4	6	470
113830	BHCUT103	6.9			10	410	33
113880	TP106	5.5	40000	0.03	4	5	290
113880	TP106	5.2	20000	0.05	41	24	820
114000	TP108	5.6	29000	0.04	2	11	250
114270	TP109	5.7	36000	0.03	<5	5	510
114270	TP109	5.7	31000	0.03	7	5	720
114270	TP109	5.7	34000	0.03	<5	7	540
114365	TP9604	5.7	33000	0.03	2	5	460
114720	TP9605	5.2	26000	0.04	8	13	300
114720	TP9605	5.1	16000	0.06	52	39	980
115050	TP201A	6	48000	0.02	3	4	280
115120	TP202A	5.5	38000	0.03	3	6	210
115120	TP202B	5.6	19000	0.05	35	35	990
115255	6.5			25	510	54	250
115265	5.4	16000	0.06	21	43	680	510
115265	5.7	24000	0.04	7	8	440	720
115270	6.07		1.32				540
115330	5.3	32000	0.03	16	11	290	250
115330	5.2	23000	0.04	20	21	330	510
115365	5.8	42000	0.02	3	6	320	720
115365	5.5	33000	0.03	16	18	440	250
115770	5.5	38000	0.03	2	6	160	510
115770	5.7	50000	0.02	2	6	230	720
116000	5.3	40000	0.03	3	7	650	540
116000	7.1	2300	0.43	63	490	1800	460
116620	6.6						300

Chainage	pH in H_2O	Resistivity ohm.cm	EC mS/cm (1:5)	SO_4 mg/kg or ppm	Cl^- mg/kg or ppm	Mg mg/kg	Soil AS 2159
116800	6.33		0.29	4	59		
116820	5.6	20000		<5	11	510	B
116820	5.3	38000		9	9	210	B
116890	5.6	31000		3	5	270	B
116890	5.4	22000		7	10	390	B
117140	5.5	34000		4	3	340	B
117140	5.5	32000		3	12	330	B
117245	5.2	19000		3	10	460	B
117245	5.7	25000		18	41	1300	B
117340	5.4	28000		3	11	630	B
117340	5.6	56000		3	5	600	B
117460	5.3	33000		5	5	410	B
117460	5.1	32000		5	5	600	B
117580	5.2	23000		4	4	250	B
117580	5	16000		3	3	500	B
117850	6.4						
117700	6.54		0.25	13	36		
118720	7.89	5319.149	0.188				
118720	8.59	5882.353	0.17				
118730	5.7	11111	0.09	150	370	244	B
119070	7.2						
119270	4.8		0.48	440	460	10	B
120330	5.7	24000	0.04	6	12	400	B
120440	5.7	30000	0.03	3	10	240	B
120440	5.3	18000	0.06	39	44	790	B
120440	5.3	13000	0.08	18	73	930	B
120810	5.7	38000	0.03	10	5	460	B
120810	5.4	37000	0.03	11	4	280	B
120900	5.28		0.06	68	50		
120940	5.5	42000	0.02	8	4	260	B
120940	4.4	22000	0.05	35	22	310	B
121015	5.7	59000	0.02	3	2	270	B

Chainage	pH in H_2O	Resistivity ohm.cm	EC mS/cm (1:5)	SO_4 mg/kg or ppm	Cl^- mg/kg or ppm	Mg mg/kg	Soil AS 2159
121015	5.1	56000	0.02	18	3	270	B
121270	5.5	40000	0.03	7	5	550	B
121270	5.5	19000	0.05	39	28	490	B
121270	4.6	5000	0.20	28	190	550	B
121770	6	18500	0.54	10	159	12	
121770	5.2	13000	0.08	27	43	930	B
121770	4.8	5600	0.18	55	200	890	B
121770	4.6	4200	0.24	88	280	890	B
121790	5.8	2000	0.50	440	680	13	B
121790	6.9	1493	0.67	410	850	12	B
122050	5.8	26300	0.38	8	106	10	
122720	8.1						
123480	n/a	12700	0.79	32	217	530	
123480	5.5	1695	0.59	770	420	35	B
123560	5.5	10400	0.96	331	90	38	
123560	6.4	2564	0.39	360	380	67	B
123560	4.7	7143	0.14	240	30	21	B
123610	4.7	11400	0.88	185	-4	33	
123610	4.7	11200	0.89	196	208	32	
123660	5.2	4300	0.23	310	65	1500	B
123660	6.7	1900	0.53	450	490	2500	B
123660	6	910	1.10	400	1400	2000	B
123660	5.1			25	830	260	
123750	4.5	1100	0.95	1500	240	2600	B
123750	7.7	1600	0.63	530	520	1900	B
123750	5.1						
123750	5.1			2500	570	320	
123810	5.6	7100	0.14	160	28	1700	B
123810	6.9	1200	0.83	290	1100	1100	B
123810	6	910	1.10	260	1500	1100	B
123810	5.5			740	210	95	
123810	5.5			740	210	95	

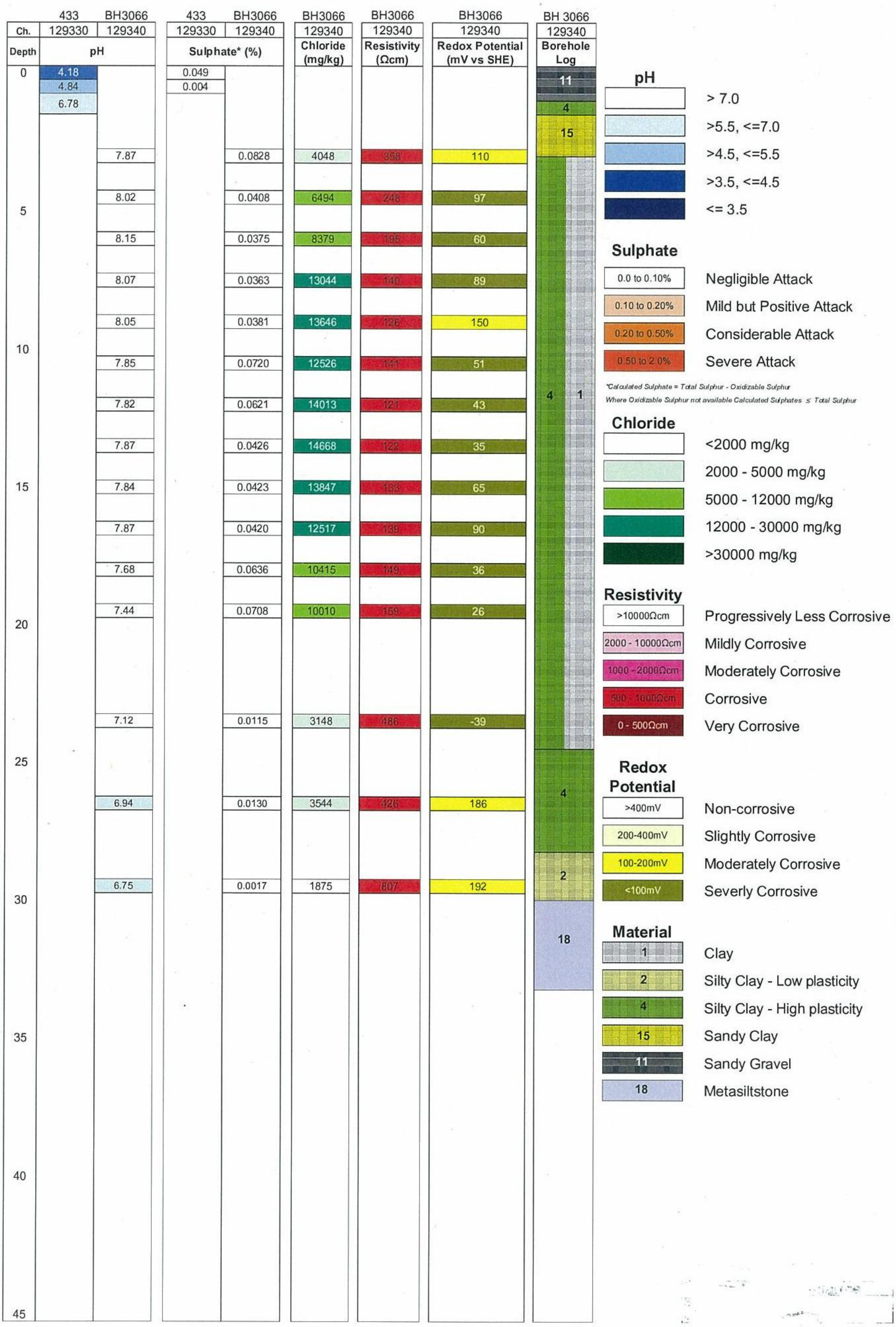
Chainage	pH in H_2O	Resistivity ohm.cm	EC mS/cm (1:5)	SO_4 mg/kg or ppm	Cl^- mg/kg or ppm	Mg mg/kg	Soil AS 2159
123870	7.6	600	1.70	1800	500	5000	A
123870	6.8	830	1.20	170	1500	1900	B
123960	5.5			1100	220	130	
124050	5.6	36000	2.81	1287	352	161	
124050	4.9		1.20	3070	240	377	B
124050	8.7	1000	1.00	1140	1000	87	B
124070	4.4						
124140	7.3	1100	0.89	970	380	4000	A
124140	8.5	830	1.20	260	780	4700	A
124140	5.1			1100	220	120	
124230	7.3	1100	0.87	1000	380	3700	B
124230	8.8	910	1.10	400	840	5900	B
124230	3.6			1800	440	180	
124230	3.6			1700	520	190	
124270	4.6			1400	330	150	
124500	7.9	910	1.10	1500	380	4200	A
124500	9.1	1200	0.85	340	820	5700	A
124500	8.7	530	1.90	130	2500	6200	B
124500	8.5	320	3.10	150	4200	4900	B
124500	5.2			740	140	93	
124700	4.9	6000	1.66	746	89	83	
124700	6.7	910	1.10	1500	200	4100	A
124700	8.9	500	2.00	220	2400	6200	B
124900	4.4	3900	2.54	1530	141	219	
124900	8		0.74	1420	250	170	B
124900	9.2		0.74	280	970	124	A
124900	4.9		1.27	3610	130	506	A
124900	4.4	39000	2.54	1530	141	218.7	
125000	7.3	1100		1200	120	4500	B
125000	8.8	1000		660	580	4700	B
125000	9.4	2100		12	600	3700	B

Chainage	pH in H_2O	Resistivity ohm.cm	EC mS/cm (1:5)	SO_4 mg/kg or ppm	Cl^- mg/kg or ppm	Mg mg/kg	Soil AS 2159
125000	3.6	25000		2600	150	300	
125060	8.1	1400		880	160	4100	A
125060	8.6	1000		1000	550	5400	B
125060	6.4	1000		1500	150	4700	B
125090	3.4	480	2.10	4700	120	3800	B
125090	8	910	1.10	1100	450	5800	A
125090	8.9	1700	0.60	50	830	4400	B
125090	5.3	56000		700	150	85	
125150	8.3	560	1.80	910	1200	6200	B
125150	3.1	590	1.70	3900	190	3600	B
125150	8.1	910	1.10	1200	430	4700	B
125150	5.3	44000		910	220	110	
125240	4.1			2500	230	290	
125270	2.82						
125290	7.2						
125330	6.2			37	17	11	
125370	4.4						
125400	7.3						
125570	7.58						
125630	5.3		0.20	350	50	22	B
125630	5.8		0.77	1750	160	196	A
125630	8.3		1.20	710	1800	50	B
125720	6.3		1.08	3280	30	320	A
125720	7.3		1.21	2720	380	278	A
125800	8.1	1400	0.73	930	200	4500	A
126000	4.9	12000	0.08	29	9	3800	A
126000	7.8	2200	0.46	110	73	1600	A
126000	8.5	910	1.10	23	1000	5800	B
126000	6.1			3.9	19	6.1	
126010	7.4						
126150	4.8	2600		390	11	3600	A
126150	7.8	5000		58	96	4000	A

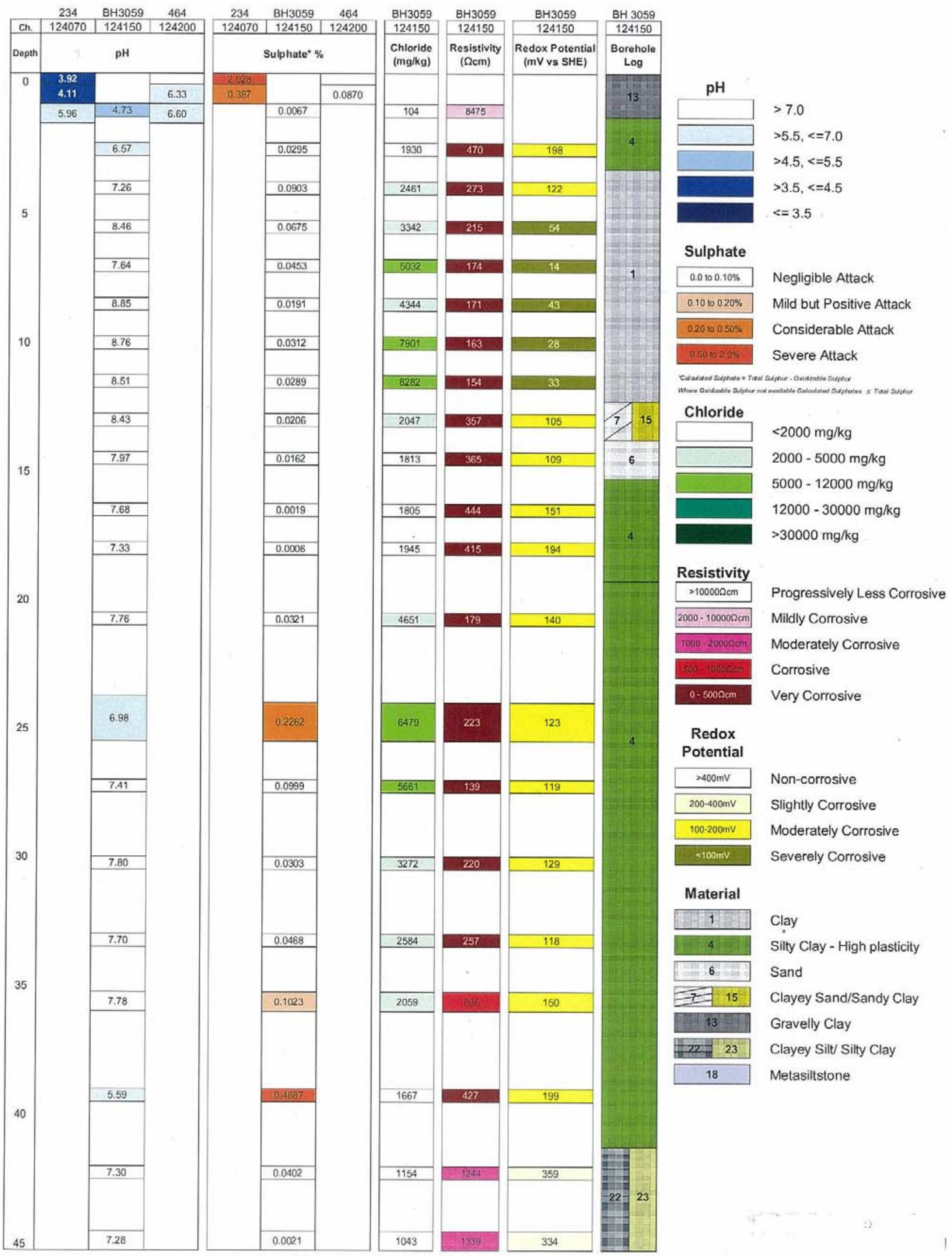
Chainage	pH in H_2O	Resistivity ohm.cm	EC mS/cm (1:5)	SO_4 mg/kg or ppm	Cl^- mg/kg or ppm	Mg mg/kg	Soil AS 2159
126770	6.33		0.99				
126820	5.4						
127060	6.3						
Levee	5.5	24000	0.04	36	2	1700	B
Levee	4.1	5900	0.17	<5	50	460	B
Levee	5.3	12000	0.08	91	7	4100	B
Levee	7.3	7700	0.13	3	2	3700	B
Levee	8.5	5300	0.19	8	4	15000	B
Levee	8.1	7100	0.14	3	3	3300	B
Levee	7.6	29000	0.03	3	3	3800	B
Levee	8.3	8300	0.12	6	2	3600	B
Levee	11.97		2.08	236	78		
Levee	6.23		0.07	64	50		
Levee	7.61		0.10	9	27		
Levee	6.28		0.44	81	38		
north of KB	6.15		0.41	58	65		
north of KB	6.87		0.14	85	111		

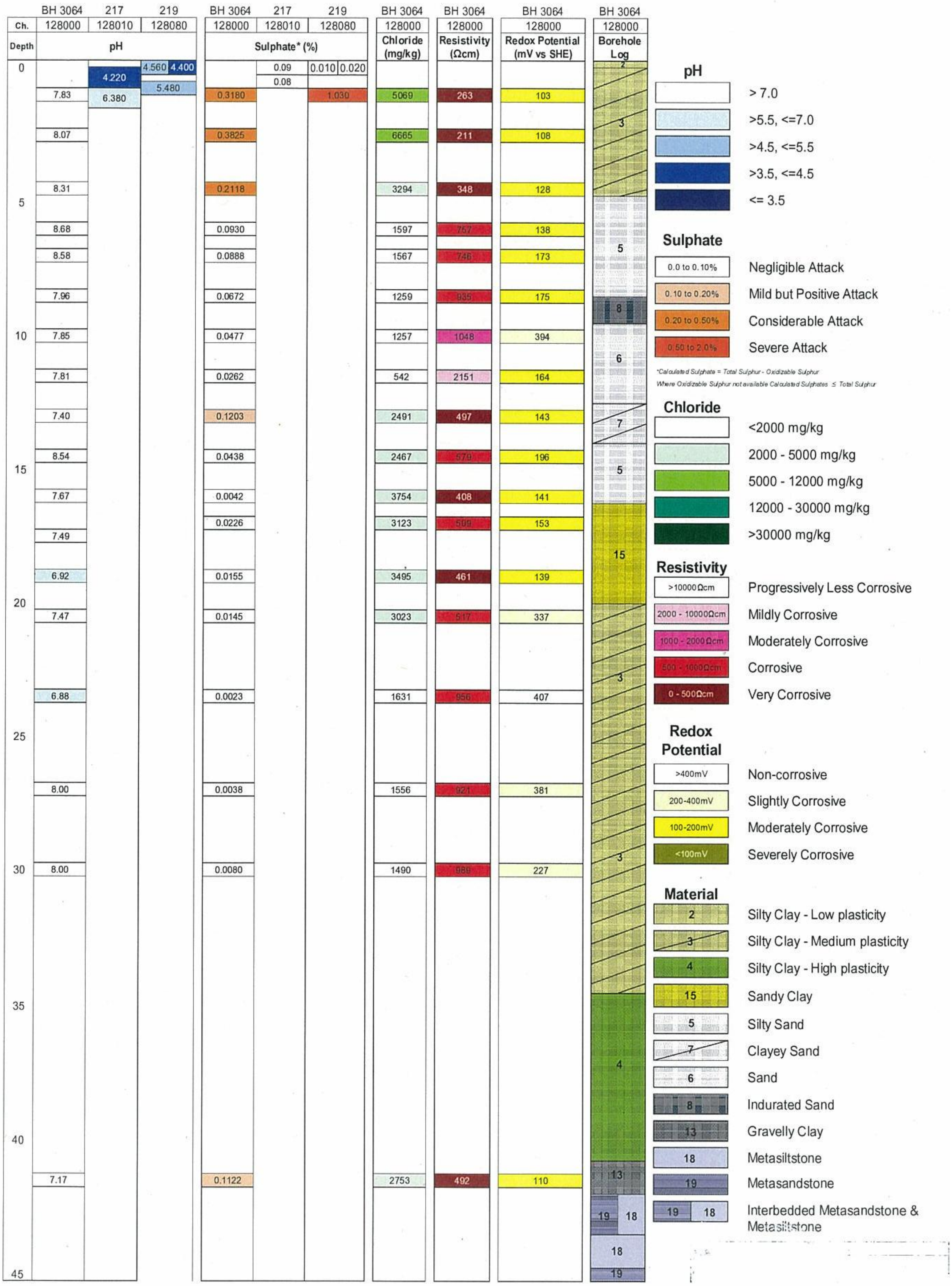
<u>Formatting Criteria</u>

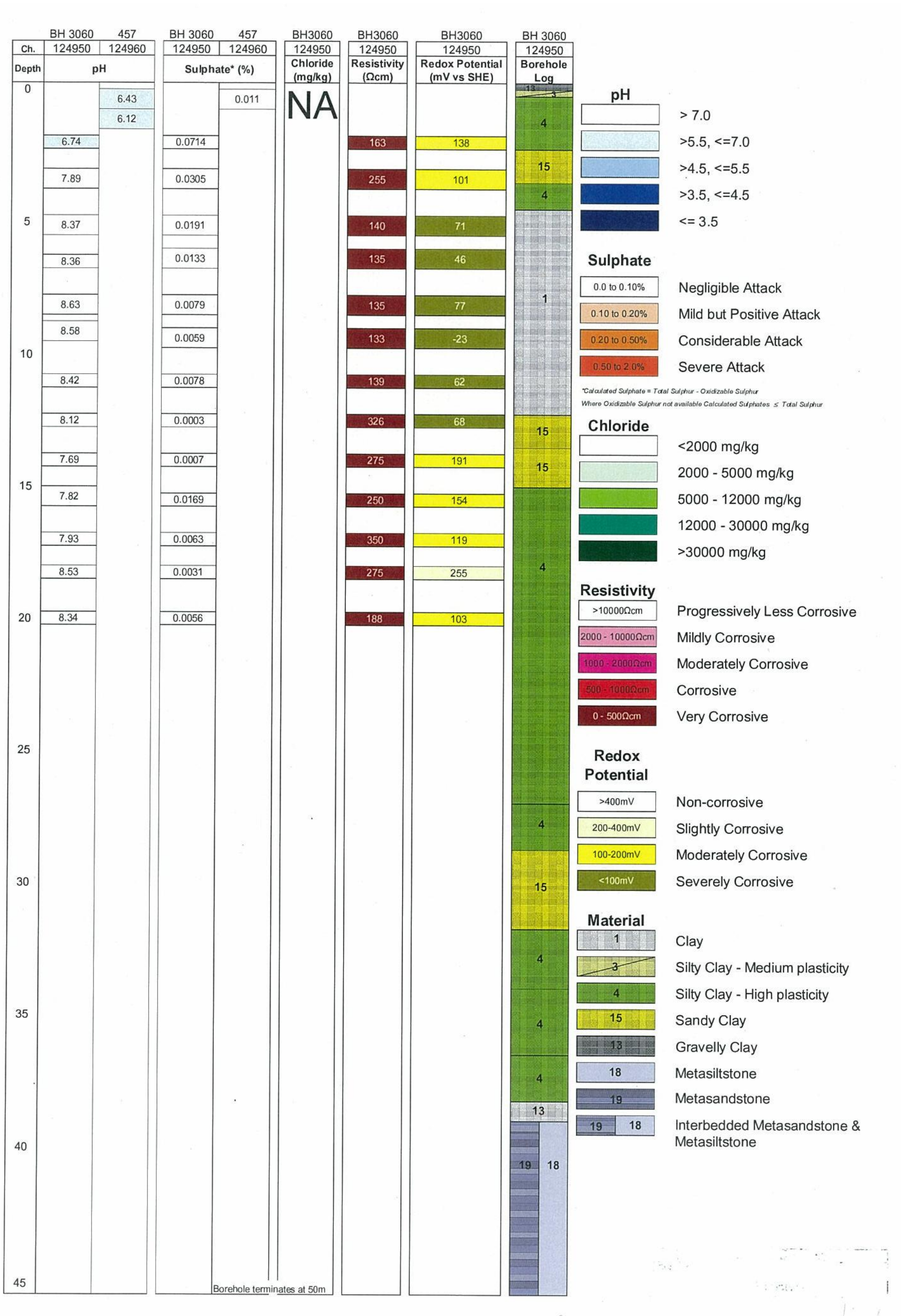
pH		SO_4		Cl		Mg		Resistivity	
	< 3.5		> 6000		> 30000		> 1000		< 1000
	3.5 - 4.5		3000 - 6000		12000 - 30000		>1000		< 5000 and > 1000
	4.5 - 5.5		1500 - 3000		6000 - 12000				> 5000
	> 5.5		< 1500		< 6000				



Ch.
Depth
433 129330 pH
BH3066 129340 pH
433 129330 Sulphate* (%)
BH3066 129340 Sulphate* (%)
BH3066 129340 Chloride (mg/kg)
BH3066 129340 Resistivity (Ωcm)
BH3066 129340 Redox Potential (mV vs SHE)
BH 3066 129340 Borehole Log
0 5 10 15 20 25 30 35 40 45
4.18 4.84 6.78
7.87 8.02 8.15 8.07 8.05 7.85 7.82 7.87 7.84 7.87 7.68 7.44 7.12 6.94 6.75
0.049 0.004
0.0828 0.0408 0.0375 0.0363 0.0381 0.0720 0.0621 0.0426 0.0423 0.0420 0.0636 0.0708 0.0115 0.0130 0.0017
4048 6494 8379 13044 13646 12526 14013 14668 13847 12517 10415 10010 3148 3544 1875
358 248 195 140 126 141 121 122 183 139 149 159 486 426 807
110 97 60 89 150 51 43 35 65 90 36 26 -39 186 192
11 4 15 4 1 4 2 18
pH
> 7.0
>5.5, <=7.0
>4.5, <=5.5
>3.5, <=4.5
<= 3.5
Sulphate
0.0 to 0.10% Negligible Attack
0.10 to 0.20% Mild but Positive Attack
0.20 to 0.50% Considerable Attack
0.50 to 2.0% Severe Attack
*Calculated Sulphate = Total Sulphur - Oxidizable Sulphur
Where Oxidizable Sulphur not available Calculated Sulphates ≤ Total Sulphur
Chloride
<2000 mg/kg
2000 - 5000 mg/kg
5000 - 12000 mg/kg
12000 - 30000 mg/kg
>30000 mg/kg
Resistivity
>10000Ωcm Progressively Less Corrosive
2000 - 10000Ωcm Mildly Corrosive
1000 - 2000Ωcm Moderately Corrosive
500 - 1000Ωcm Corrosive
0 - 500Ωcm Very Corrosive
Redox Potential
>400mV Non-corrosive
200-400mV Slightly Corrosive
100-200mV Moderately Corrosive
<100mV Severly Corrosive
Material
1 Clay
2 Silty Clay - Low plasticity
4 Silty Clay - High plasticity
15 Sandy Clay
11 Sandy Gravel
18 Metasiltstone


Ch.
Depth
234 124070
BH3059 124150
464 124200
pH
Sulphate* %
Chloride (mg/kg)
Resistivity (Ωcm)
Redox Potential (mV vs SHE)
BH 3059 124150
Borehole Log
0
5
10
15
20
25
30
35
40
45
3.92
4.11
5.96
4.73
6.33
6.60
6.57
7.26
8.46
7.64
8.85
8.76
8.51
8.43
7.97
7.68
7.33
7.76
6.98
7.41
7.80
7.70
7.78
5.59
7.30
7.28
2.028
0.387
0.0870
0.0067
0.0295
0.0903
0.0675
0.0453
0.0191
0.0312
0.0289
0.0206
0.0162
0.0019
0.0006
0.0321
0.2262
0.0999
0.0303
0.0468
0.1023
0.4887
0.0402
0.0021
104
1930
2461
3342
5032
4344
7901
8282
2047
1813
1805
1945
4651
6479
5661
3272
2584
2059
1667
1154
1043
8475
470
273
215
174
171
163
154
357
365
444
415
179
223
139
220
257
427
1244
1339
198
122
54
14
43
28
33
105
109
151
194
140
123
119
129
118
150
199
359
334
13
4
1
7
15
6
4
4
22
23
pH
> 7.0
>5.5, <=7.0
>4.5, <=5.5
>3.5, <=4.5
<= 3.5
Sulphate
0.0 to 0.10% Negligible Attack
0.10 to 0.20% Mild but Positive Attack
0.20 to 0.50% Considerable Attack
0.50 to 2.0% Severe Attack
*Calculated Sulphate = Total Sulphur - Oxidizable Sulphur
Where Oxidizable Sulphur not available Calculated Sulphates ≤ Total Sulphur
Chloride
<2000 mg/kg
2000 - 5000 mg/kg
5000 - 12000 mg/kg
12000 - 30000 mg/kg
>30000 mg/kg
Resistivity
>10000Ωcm Progressively Less Corrosive
2000 - 10000Ωcm Mildly Corrosive
1000 - 2000Ωcm Moderately Corrosive
500 - 1000Ωcm Corrosive
0 - 500Ωcm Very Corrosive
Redox Potential
>400mV Non-corrosive
200-400mV Slightly Corrosive
100-200mV Moderately Corrosive
<100mV Severely Corrosive
Material
1 Clay
4 Silty Clay - High plasticity
6 Sand
7 15 Clayey Sand/Sandy Clay
13 Gravelly Clay
22 23 Clayey Silt/ Silty Clay
18 Metasiltstone


BH 3064 217 219
Ch. 128000 128010 128080
Depth pH
Sulphate* (%)
Chloride (mg/kg)
Resistivity (Ωcm)
Redox Potential (mV vs SHE)
Borehole Log
0 5 10 15 20 25 30 35 40 45
4.220 6.380 4.560 4.400 5.480
0.09 0.08 0.010 0.020 1.030
7.83 0.3180 5069 263 103
8.07 0.3825 6665 211 108
8.31 0.2118 3294 348 128
8.68 0.0930 1597 757 138
8.58 0.0888 1567 746 173
7.96 0.0672 1259 935 175
7.85 0.0477 1257 1048 394
7.81 0.0262 542 2151 164
7.40 0.1203 2491 497 143
8.54 0.0438 2467 579 196
7.67 0.0042 3754 408 141
0.0226 3123 509 153
7.49
6.92 0.0155 3495 461 139
7.47 0.0145 3023 517 337
6.88 0.0023 1631 956 407
8.00 0.0038 1556 921 381
8.00 0.0080 1490 989 227
7.17 0.1122 2753 492 110
2 3 5 8 6 7 5 15 3 3 4 13 19 18 18 19
pH
> 7.0
>5.5, <=7.0
>4.5, <=5.5
>3.5, <=4.5
<= 3.5
Sulphate
0.0 to 0.10% Negligible Attack
0.10 to 0.20% Mild but Positive Attack
0.20 to 0.50% Considerable Attack
0.50 to 2.0% Severe Attack
*Calculated Sulphate = Total Sulphur - Oxidizable Sulphur
Where Oxidizable Sulphur not available Calculated Sulphates ≤ Total Sulphur
Chloride
<2000 mg/kg
2000 - 5000 mg/kg
5000 - 12000 mg/kg
12000 - 30000 mg/kg
>30000 mg/kg
Resistivity
>10000Ωcm Progressively Less Corrosive
2000 - 10000Ωcm Mildly Corrosive
1000 - 2000Ωcm Moderately Corrosive
500 - 1000Ωcm Corrosive
0 - 500Ωcm Very Corrosive
Redox Potential
>400mV Non-corrosive
200-400mV Slightly Corrosive
100-200mV Moderately Corrosive
<100mV Severely Corrosive
Material
2 Silty Clay - Low plasticity
3 Silty Clay - Medium plasticity
4 Silty Clay - High plasticity
15 Sandy Clay
5 Silty Sand
7 Clayey Sand
6 Sand
8 Indurated Sand
13 Gravelly Clay
18 Metasiltstone
19 Metasandstone
19 18 Interbedded Metasandstone & Metasiltstone


Ch.
Depth
BH 3060
457
124950
124960
pH
Sulphate* (%)
BH3060
Chloride (mg/kg)
Resistivity (Ωcm)
Redox Potential (mV vs SHE)
BH 3060
Borehole Log
0
5
10
15
20
25
30
35
40
45
6.43
6.12
6.74
7.89
8.37
8.36
8.63
8.58
8.42
8.12
7.69
7.82
7.93
8.53
8.34
0.011
0.0714
0.0305
0.0191
0.0133
0.0079
0.0059
0.0078
0.0003
0.0007
0.0169
0.0063
0.0031
0.0056
NA
163
255
140
135
135
133
139
326
275
250
350
275
188
138
101
71
46
77
-23
62
68
191
154
119
255
103
Borehole terminates at 50m
pH
> 7.0
>5.5, <=7.0
>4.5, <=5.5
>3.5, <=4.5
<= 3.5
Sulphate
0.0 to 0.10% Negligible Attack
0.10 to 0.20% Mild but Positive Attack
0.20 to 0.50% Considerable Attack
0.50 to 2.0% Severe Attack
*Calculated Sulphate = Total Sulphur - Oxidizable Sulphur
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Chloride
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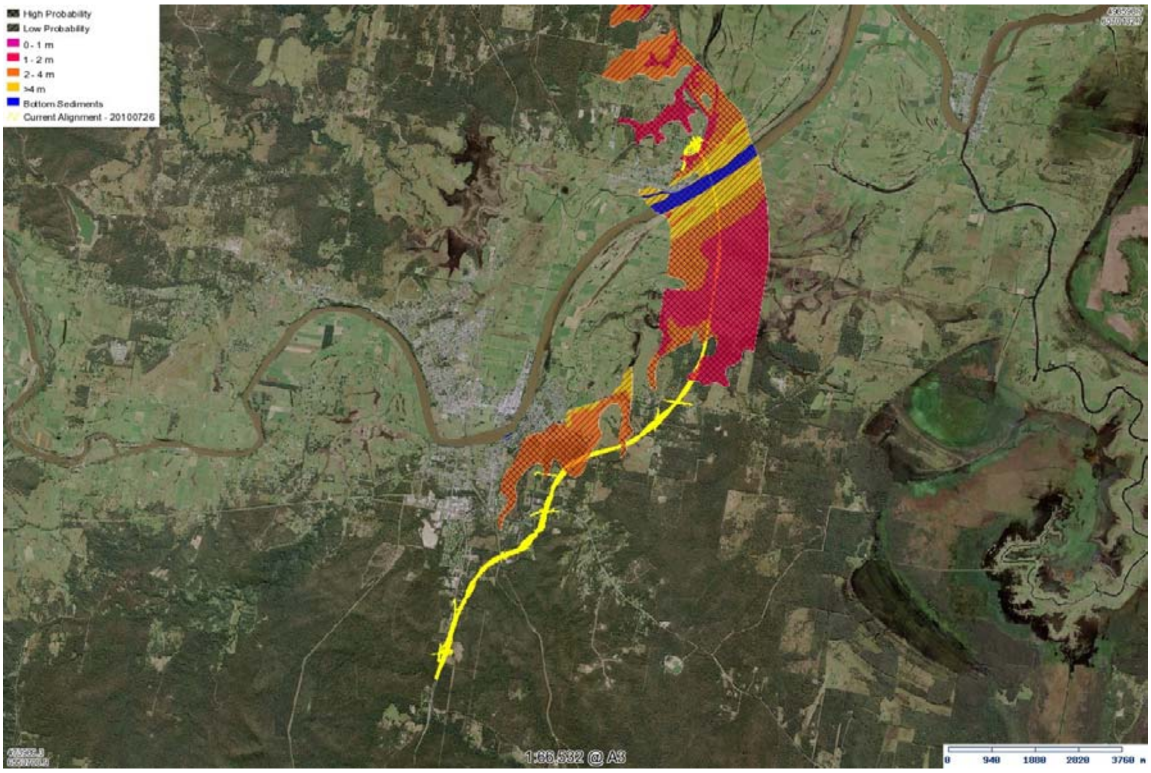
2000 - 5000 mg/kg
5000 - 12000 mg/kg
12000 - 30000 mg/kg
>30000 mg/kg
Resistivity
>10000Ωcm Progressively Less Corrosive
2000 - 10000Ωcm Mildly Corrosive
1000 - 2000Ωcm Moderately Corrosive
500 - 1000Ωcm Corrosive
0 - 500Ωcm Very Corrosive
Redox Potential
>400mV Non-corrosive
200-400mV Slightly Corrosive
100-200mV Moderately Corrosive
<100mV Severely Corrosive
Material
1 Clay
3 Silty Clay - Medium plasticity
4 Silty Clay - High plasticity
15 Sandy Clay
13 Gravelly Clay
18 Metasiltstone
19 Metasandstone
19 18 Interbedded Metasandstone & Metasiltstone

10.2 Appendix B – Results of SPOCAS and NAG testing

Acid Sulphate Soil Risk Map



Results of the SPOCAS Analysis

Chainage	pH in KCl	pH in H2O2	TPA mol H+/t	TAA mol H+/t	TSA mol H+/t	%SP	%SKCl	%SPOS	%STPA	Classification
115345	9.00	6.80	<5	<5	<5	0.04	<0.005	0.04	<0.01	
117020	9.60	7.10	<5	<5	<5	0.23	0.04	0.19	<0.01	PASS
117365	8.40	7.10	<5	<5	<5	0.01	<0.005	<0.005	<0.01	
118730	4.55	4.40	20	30	<2	<0.01	<0.01	<0.01	0.03	
119270	4.25	4.09	96	84	12	0.06	0.06	<0.01	0.15	PASS
120840	4.71	4.85	26	28	<2	<0.01	<0.01	<0.01	0.04	
121790	4.85	4.61	24	22	2	<0.01	0.01	<0.01	0.04	
121790	5.94	5.65	<2	2	<2	<0.01	<0.01	<0.01	<0.01	
123480	4.98	5.08	14	12	2	0.01	0.03	<0.01	0.02	
123560	4.51	4.13	82	64	18	0.01	<0.01	0.01	0.13	PASS
123560	5.53	5.31	<2	10	<2	<0.01	<0.01	<0.01	<0.01	
123660	4.50	4.80	10	30	<5	0.03	0.02	0.01	0.02	

123660	5.30	6.00	<5	10	<5	0.03	0.03	<0.005	<0.01	
123660	4.90	5.70	<5	17	<5	0.03	0.03	<0.005	<0.01	
Chainage	pH in KCl	pH in H2O2	TPA mol H+/t	TAA mol H+/t	TSA mol H+/t	%SP	%SKCl	%SPOS	%STPA	Classification *
123750	6.90	6.40	<5	<5	<5	0.03	0.02	<0.005	<0.01	
123480	6.11	6.47	<2	<2	<2	<0.01	<0.01	<0.01	<0.01	
123560	6.68	5.01	<2	<2	<2	<0.01	<0.01	<0.01	<0.01	
123600	6.34	6.21	<2	<2	<2	<0.01	<0.01	<0.01	<0.01	
123810	4.80	5.00	<5	22	<5	0.02	0.01	0.01	<0.01	
123810	5.90	6.20	<5	<5	<5	0.02	0.02	<0.005	<0.01	
123810	4.90	5.90	<5	15	<5	0.02	0.02	<0.005	<0.01	
123870	6.90	6.40	<5	<5	<5	0.02	0.02	0.01	<0.01	
123870	7.50	4.90	<5	<5	<5	0.17	0.02	0.15	<0.01	PASS
123960	5.50	6.10	<5	10	<5	0.014	0.01	<0.005	<0.01	
123960	7.20	2.10	430	<5	430	1.30	0.10	1.20	0.68	PASS
124050	4.95	2.81	174	12	162	0.57	0.12	0.45	0.28	PASS
124050	7.78	2.54	434	<2	436	0.98	0.03	0.95	0.7	PASS
124050	6.55	4.03	36	<2	38	0.50	<0.01	0.05	0.06	
124140	6.20	2.40	240	<5	240	1.10	0.10	1.00	0.39	PASS
124140	8.70	3.00	150	<5	150	1.90	0.09	1.80	0.24	PASS
124230	7.00	2.40	280	<5	280	1.30	0.08	1.30	0.46	PASS
124230	8.60	4.30	35	<5	35	0.97	0.06	0.91	0.06	PASS
124430	5.28		11	8			0.03			
124430	7.25		263	0			0.04			PASS*

10.3 Appendix C – Condition assessment of structures along the route to assess the severity of exposures

Also condition assessments to validate the limits of chloride and sulphate concentrations given in AS 2159 and other standards should be included in this Appendix.

10.4 Appendix D – Modelling to calculate chloride penetration in concrete

According to Fick's law, the concentration $C_t(x,t)$ at depth x and time t can be given by equation 1, where *erfc* is a complementary error function. C_i or the initial concentration of chloride ions in concrete was assumed to be zero.

$$C_t(x,t)/C_s = erfc\left(x/2\sqrt{D_a \times t}\right) \qquad (1)$$

where

x = depth (m)
$C_t(x,t)$ = chloride concentration at cover depth (% by mass of concrete)
C_s = surface chloride concentration at time t (% by mass of concrete)
erf^{-1} = inverse of error function
D_a = apparent diffusion coefficient at time t (m^2/sec)
t = time for chloride to reach $C_t(x,t)$ at cover depth x (seconds)

Variations on Equation 1 have been proposed to account for phenomena such as chloride binding, convection, intermittent exposure, moisture content, and temperature (e.g., Saetta et al., 1993; Andrade et al., 1997; Hansen and Saouma, 1999; Xi and Bažant, 1999; Tang and Nilsson, 2000; Nilsson, 2000; Ababneh et al. 2003). However, Equation 1 generally describes the diffusion process reasonably well for most circumstances in uncracked concrete.

The changes in diffusion coefficient with time can be modelled according to Equation 2 as given in ACI Life 365:

$$D_t = D_{t_1}\left(\frac{t_1}{t}\right)^m \qquad (2)$$

where
D_t = diffusion coefficient at time t (m^2/s)
D_{t1} = diffusion coefficient at time of testing t_1 (m^2/s)
t_1 = time at test (s)
t = time (s)
m = age factor depending on mix proportions

There is experimental data available for the age factor "m" (e.g., Mangat and Molloy, 1994; Bamforth, 1999; Thomas and Bamforth, 1999; Lee and Chisholm, 2005; Nokken et al., 2006). In addition, the ACI Life 365 model provides guidance on selection of the age factor. Lee and Chisholm (2005) have reviewed the subject of the age (time reduction) factor, its variability and the sensitivity of the predicted time to corrosion initiation to this parameter. In addition, Lee and Chisholm (2005) summarise the approaches that different models take to estimate of the age factor.

Equation 2 can be integrated to determine the average diffusion coefficient, as given in Equation 3:

$$D_A = D_{t1}.t_1^{\,m}\,\frac{(t_e^{\,(1-m)} - t_s^{\,(1-m)})}{(1-m)(t_e - t_s)} \qquad (3)$$

where
D_A = average diffusion coefficient (m^2/s)
D_{t1} = diffusion coefficient at time of testing (m^2/s)
t = time (s)
t_1 = time at testing (s)
m = age factor ≠ 0, 1
t_s = age at start of exposure (s)
t_e = age at end of exposure (s)

Although Equations 2 and 3 predict ongoing reduction of the diffusion coefficient, it is expected that after 30 years the diffusion coefficient will remain constant since hydration will be virtually complete. Therefore, a time weighted average diffusion coefficient (D_{TWA}) has been calculated assuming reduction of the diffusion coefficient over the first 30 years according to Equation 2, followed by a constant value thereafter. A similar principle is used in ACI Life 365. The time weighted diffusion coefficient was calculated according to Equation 4:

$$D_{TWA} = \frac{\sum_{i=1}^{n} D_{ti}.t_i}{\sum_{i=1}^{n} t_i} \qquad (4)$$

where D_{TWA} = time weighted average diffusion coefficient (m^2/s)
D_{ti} = diffusion coefficient at time t_i (m^2/s)

Equation 5 below gives the chloride concentration at various depths incorporating the variation in diffusion coefficient with time due to maturity of the concrete.

$$C_t(x,t)/C_s = 1 - erf\left[\left(x/2\sqrt{\left(\frac{1}{1-m}\right) \times D_a \times \left(\frac{t_o}{t}\right)^m \times \left[t - \left(\frac{t}{t_s}\right)^m \times t_s\right]}\right)\right] \qquad (5)$$

where
m = maturity coefficient
t_o = age of trial mix at the time of testing
t_s = time of commencement of exposure to chlorides
$C_t(x,t)$ = chloride concentration at cover depth (% by mass of concrete)
C_s = surface chloride concentration at time *t* (% by mass of concrete)
erf[1] = error function
D_a = apparent diffusion coefficient at time t (m^2/sec)
t = time for chloride to reach $C_t(x,t)$ at cover depth *x* (seconds)

The piers and piles in creek water below the tidal/splash zone, are classified as “submerged” elements. Chloride ingress from brackish creek water could potentially cause initiation of reinforcement corrosion. Once initiated, the corrosion rate is predicted to be low since water saturated concrete has restricted oxygen availability. The chloride threshold concentration is also predicted to be up to one order of magnitude higher for submerged reinforced concrete compared with atmospheric (Bertolini et al., 2004). Frederiksen (2002) summarises published data on threshold values for submerged marine concrete and recommends concentrations of 1.0-2.0% by weight of cement for concrete with a water/cementitious material ratio = 0.4 and 0.6-1.5% by weight of cement for concrete with a water/cementitious material ratio = 0.5. The recommended threshold concentrations given by Frederiksen (2002) also depend on fly ash and silica fume content as shown in Table D1. This compares with the typical value of 0.06% by weight of concrete assumed as an initiation threshold for non-submerged conditions. Based on the environmental conditions that define the submerged zone, the time to corrosion for this zone was calculated to determine the optimal treatment to achieve the 100 year design life.

Table D1. Recommended Chloride Threshold Concentrations (Frederiksen, 2002)

Concrete Type	w/cm	Threshold Concentration					
		Marine Submerged Zone		Marine Splash Zone		Atmospheric Zone	
		% wt. cm	% wt. concrete	% wt. cm	% wt. concrete	% wt. cm	% wt. concrete
100% CEM I	0.5 (~S40)	1.5	0.22	0.6	0.09	0.6	0.09
5% SF	0.5 (~S40)	1.0	0.15	0.4	0.06	0.4	0.06
10% SF	0.5 (~S40)	0.6	0.09	0.2	0.03	0.2	0.03
20% FA	0.5 (~S40)	0.7	0.11	0.3	0.05	0.3	0.05
100% CEM I	0.4 (~S50)	2.0	0.36	0.8	0.14	0.8	0.14
5% SF	0.4 (~S50)	1.5	0.27	0.5	0.09	0.5	0.09
10% SF	0.4 (~S50)	1.0	0.18	0.3	0.05	0.3	0.05
20% FA	0.4 (~S50)	1.2	0.22	0.4	0.07	0.4	0.07

Note: CEM I = Portland cement assumed approximately equivalent to Australian Type GP
SF = silica fume
FA = fly ash
w/cm = water/cementitious material ratio by mass
The % wt. concrete has been calculated from the % wt. cm assuming the cementitious content is 15% and 18% for S40 and S50, respectively
Assume w/cm = 0.5 approximately equivalent to S40 and w/cm = 0.4 equivalent to S50

It was conservatively assumed that the initial (56 day) diffusion coefficient for Grade S50 concrete with a minimum cementitious content of 420 kg/m^3 and maximum water/cementitious material ratio of 0.4 is ~2 x 10^{-12} m^2/s. Similarly, the assumed initial diffusion coefficient for Grade S40 concrete with minimum cementitious content of 370 kg/m^3 and maximum water/cementitious material ratio of 0.46 is ~6 x 10^{-12} m^2/s. It is recognised that actual values may vary and testing is recommended to verify the assumed diffusion coefficients. Based on the assumed initial diffusion coefficients, the time weighted average values over 100 years were calculated for different concrete mix options. These are summarised in Table D2.

Table D2. Assumed Time Weighted Average Diffusion Coefficients

Supplementary Cementitious Material	% Supplementary Cementitious Material	Age Factor "m"	D_{TWA} (m^2/s) S40	D_{TWA} (m^2/s) S50
None	0	0.2	2.1 x 10^{-12}	7.0 x 10^{-13}
Fly Ash (FA)	25	0.4	7.4 x 10^{-13}	2.5 x 10^{-13}
Blast Furnace Slag (BFS)	65	0.57	3.1 x 10^{-13}	1.0 x 10^{-13}
Silica Fume (SF)	8	0.36	9.2 x 10^{-13}	3.0 x 10^{-13}

10.4.1 Submerged Zone - Chloride Ingress

Assuming brackish creek water at all locations, the estimated surface chloride concentration for submerged concrete is 0.35% by weight of concrete. This value was based on investigations in other brackish creeks but should be verified by core testing on existing structures in the Ballina area exposed to creek water. The chloride concentration versus depth of cover was predicted for the different concrete mixes at 100 years and the results are presented in Figures D1 and D2. In the plots the conventional corrosion threshold and the minimum value (given by Frederikson (2002)) for concrete with water/cementitious material ratio =0.4 (S50) and 0.5 (S40) in submerged marine environments are indicated.

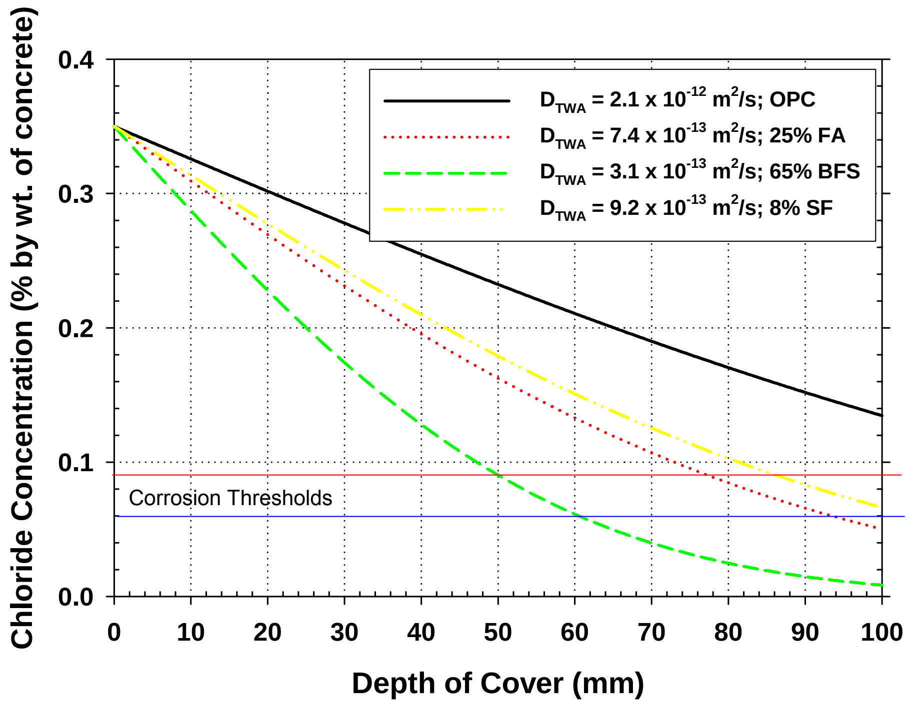


Figure D1. Predicted Chloride Ingress for Different S40 Concrete Mixes at 100 Years.

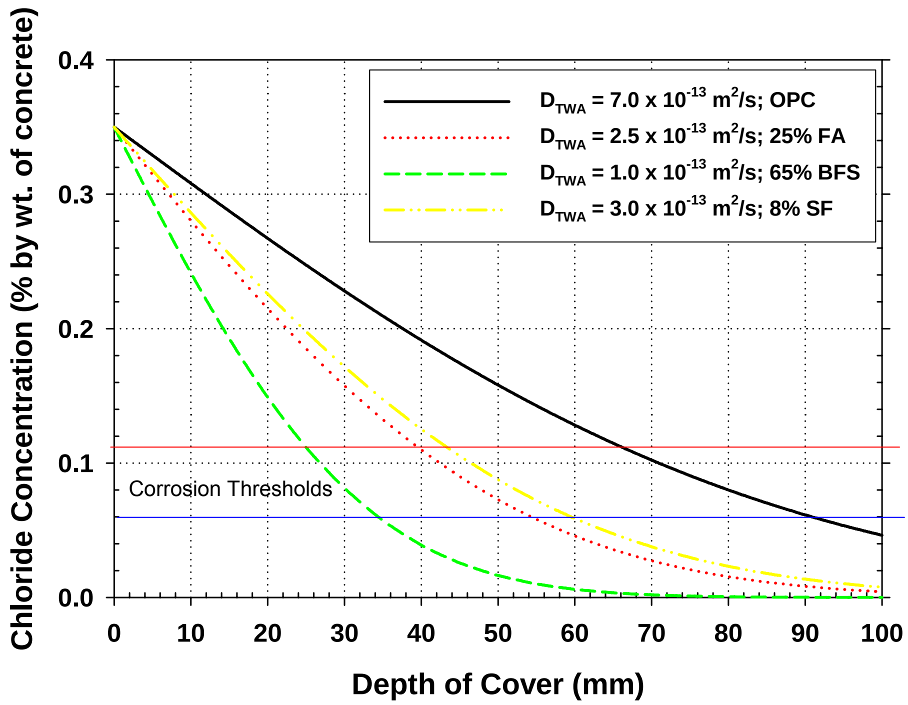


Figure D2. Predicted Chloride Ingress for Different S50 Concrete Mixes at 100 Years.

Figures D1 and D2 indicate the superiority of the S50 mixes containing supplementary cementitious materials for submerged conditions. Assuming the higher corrosion threshold is applicable, all types of S50 mixes considered would be acceptable provided the depth of cover is appropriate. Mixes with 25% fly ash, 65% slag or 8% silica fume would be preferred and RTA B80 specification requires the use of blended cements for C exposure classifications. For S40 concrete in Figure D1, only the mixes with 25% fly ash or 65% slag appear suitable at realistic depths of cover (i.e., < 75 mm). Note that the above predictions assume a serviceability limit of corrosion initiation period of 100 years and that the corrosion propagation period has been neglected. Hence, the modelling is considered to be conservative. It is also noted that the above predictions are deterministic and do not account for the inherent variability in concrete properties, depth of cover, etc that occur in reality. Therefore, a reliability approach to chloride ingress prediction could be performed when more details for proposed mixes for the Ballina Bypass project become available.

The RTA B80 specification requires corrosion inhibitors for C exposure classifications involving chlorides. The modelling above indicates that inhibitors are not required for the concrete to achieve the 100 year design life, provided appropriate quality concrete, mix proportions and depth of cover are used. We would not place high reliance on any potential benefits of corrosion inhibitors in concrete since there are also concerns regarding the long-term effectiveness of inhibitors and their ability to remain functional over a life of 100 years. Chloride modelling has confirmed mild steel reinforcement can be adequately protected by appropriate concrete cover and therefore stainless steel is not required for structures in brackish water to achieve the design life. Furthermore, where the concrete cover must be restricted, e.g. to 50 mm, as commonly occurs for precast piles, the use of suitable binder will ensure the pile can achieve the design life. Provided appropriate binders are used in the concrete mixes and cured adequately for all piles in brackish conditions, corrosion inhibitor and stainless steel reinforcement would be unnecessary.

10.4.2 Tidal Zone - Chloride Ingress

Tidal zone environments are considered more aggressive than submerged environments due to higher potential surface chloride concentrations associated with wetting and drying and higher corrosion rates. The surface chloride concentration for brackish water environments is expected to be approximately 0.4% by weight of concrete. Verification by testing on existing structures in equivalent environments in Ballina is recommended. Modelling of chloride ingress for S40 and S50 mixes as defined above was performed and the results are shown in Figures D3 and D4. The typical threshold value of 0.06% by weight of concrete is indicated, along with the minimum threshold value for a splash environment suggested by Frederikson (2002) in Table D1. The latter environment is considered to be more aggressive than the creek tidal environment but is included for reference.

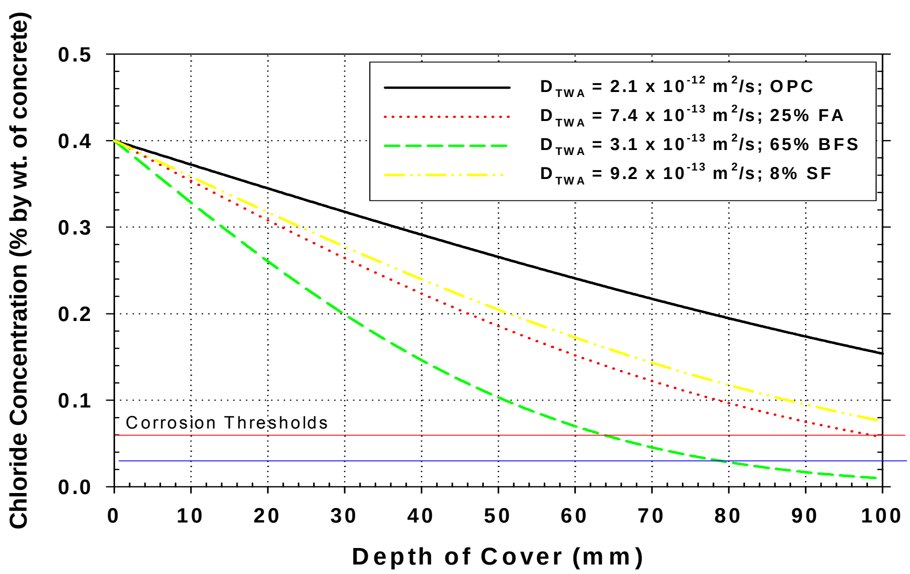


Figure D3. Predicted Chloride Ingress versus Depth of Cover for Different S40 Mixes at 100 Years.

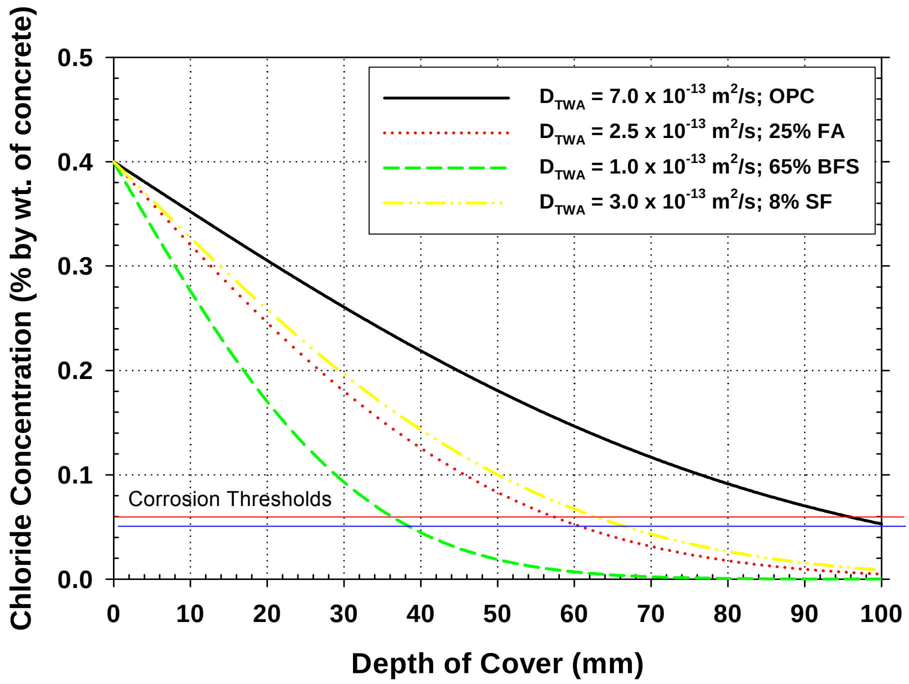


Figure D4. Predicted Chloride Ingress versus Depth of Cover for Different S50 Mixes at 100 Years.

Figures D3 and D4 suggest that tidal creek water exposure zones require an S40 mix with 65% slag and 70 mm minimum cover or one of the considered S50 mixes with supplementary cementitious materials (i.e., 8% silica fume and 70 mm cover, 25% fly ash and 65 mm cover or 65% slag and 50 mm cover).

The modelling above indicates that corrosion inhibitors are not necessary provided appropriate quality concrete, mix proportions and depth of cover are used. The analysis also indicates that the use of stainless steel reinforcement is not necessary for the structures to achieve the design life, provided a S40 mix with 65% slag and 70 mm minimum cover or one of the S50 mixes with 25% fly ash and 70 mm cover or 50 mm cover with 65% slag, is used.

10.5 Appendix E – Carbonation modelling to assess the requirements of covercrete

The rate of carbonation is expressed typically expressed by Equation 1.

Depth of Carbonation (mm) = $C.t^{0.5}$ (1)

where C = carbonation rate or coefficient (mm/year$^{0.5}$)
t = time (years)

The carbonation rate can be expressed as a function of the controlling factors and these are described by Lay et al. (2003) and Maage and Smeplass (2001). However, this requires knowledge of associated input parameters which are not able to be clearly defined at this stage. Hence, the simpler Equation 1 is used in this instance.

In order to predict the depth of carbonation it is necessary to consider an appropriate estimate of the carbonation coefficient in the service environment for the proposed concrete. Table E1 summarises published carbonation coefficients for concrete mixes similar to those considered for the Ballina Bypass.

Table E1. Published Carbonation Coefficients for Concrete Mixes

Concrete	Curing (days)	Carbonation Coefficient (mm/yr$^{0.5}$)	Test Method	Source
OPC, 46 MPa, w/cm = 0.41	7	2.0	Accelerated (4% CO_2, 23^{o}C, 50% RH)	Ho and Lewis (1987)
OPC, 42-50 MPa, w/cm = 0.41-0.45	7	1.0-2.2	Accelerated (4% CO_2, 23^{o}C, 50% RH)	Ho and Lewis (1987)
20% FA, 46 MPa	1	8.5	Laboratory 23^{o}C 50% RH	Ho and Lewis (1987)
20% FA, 46 MPa	1	4.5	Outdoors Melbourne, N vertical	Ho and Lewis (1987)
20% FA, 46 MPa	1	3.0	Outdoors Melbourne, S inclined	Ho and Lewis (1987)
40% FA, 43 MPa	7	5.0	Accelerated (4% CO_2, 23^{o}C, 50% RH)	Ho and Lewis (1987)
20% FA, 42-50 MPa, w/cm = 0.41-0.45	7	2.5-3.8	Accelerated (4% CO_2, 23^{o}C, 50% RH)	Ho and Lewis (1987)
25% FA, 41 MPa	7	2.8	Accelerated (4% CO_2, 23^{o}C, 50% RH)	Ho and Lewis (1987)
OPC, w/cm = 0.5	1	6.0	Outdoors Sheltered, Canada	Burden (2006)
OPC, w/cm = 0.5	7	2.0	Outdoors Sheltered, Canada	Burden (2006)
OPC, w/cm = 0.5	28	0.5	Outdoors Sheltered, Canada	Burden (2006)
30% FA, w/cm = 0.5	1	8.0	Outdoors Sheltered, Canada	Burden (2006)
30% FA, w/cm = 0.5	7	5.0	Outdoors Sheltered, Canada	Burden (2006)
30% FA, w/cm = 0.5	28	2.5	Outdoors Sheltered, Canada	Burden (2006)
OPC, w/cm = 0.4	1	5.0	Outdoors Sheltered, Canada	Burden (2006)
OPC, w/cm = 0.4	7	1.0	Outdoors Sheltered, Canada	Burden (2006)
OPC, w/cm = 0.4	28	0.0	Outdoors Sheltered, Canada	Burden (2006)
30% FA, w/cm = 0.4	1	7.0	Outdoors Sheltered, Canada	Burden (2006)
30% FA, w/cm = 0.4	7	3.0	Outdoors Sheltered, Canada	Burden (2006)
30% FA, w/cm = 0.4	28	1.0	Outdoors Sheltered, Canada	Burden (2006)
OPC, w/cm = 0.4	28	1	Laboratory 20^{o}C 60% RH	Collepardi et al. (2004)
OPC, w/cm = 0.5	28	4	Laboratory 20^{o}C 60% RH	Collepardi et al. (2004)

25% FA, w/cm = 0.4	28	3.0	Laboratory 20°C 60% RH	Collepardi et al. (2004)
25% FA, w/cm = 0.5	28	5.9	Laboratory 20°C 60% RH	Collepardi et al. (2004)
15% BFS, w/cm = 0.4	28	0.7	Laboratory 20°C 60% RH	Collepardi et al. (2004)
15% BFS, w/cm = 0.5	28	2.8	Laboratory 20°C 60% RH	Collepardi et al. (2004)
50% BFS, w/cm = 0.4	28	4.5	Laboratory 20°C 60% RH	Collepardi et al. (2004)
50% BFS, w/cm = 0.5	28	5.2	Laboratory 20°C 60% RH	Collepardi et al. (2004)

Note: FA = fly ash, BFS = Blast Furnace Slag, OPC = ordinary Portland cement

In addition to mix design and materials, Table E1 indicates the importance of adequate curing to achieve low carbonation rates. The data in Table E1 can be used to estimate the carbonation rate for predictive purposes and the estimated carbonation rates used in the modelling are summarised in Table E2. An atmospheric CO_2 concentration of 0.04% and curing period of 7 days have been assumed and the data below have been derived from results for comparable conditions.

Table E2. Estimated Carbonation Coefficients for Modelling

Concrete Mix	Carbonation Coefficient (mm/yr$^{0.5}$`)
S40, OPC	2.0
S40, 25% FA	5.0
S40, 65% BFS	7.0
S50, OPC	1.0
S50, 25% FA	3.0
S50, 65% BFS	5.0

The carbonation predictions are presented in Figures E1 and E2.

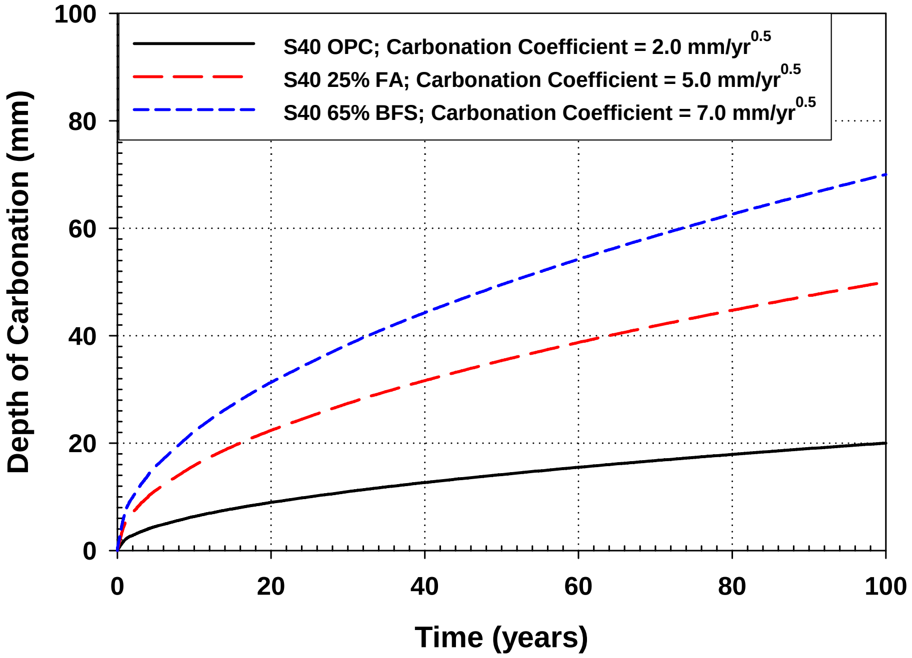


Figure E1. Predicted Depth of Carbonation versus Time for S40 Concrete.

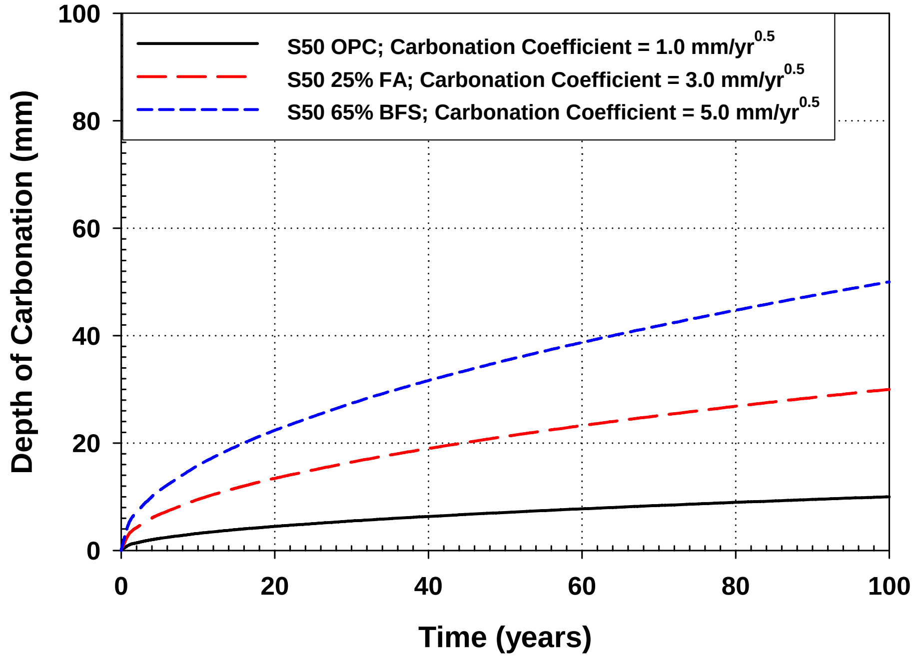


Figure E2. Predicted Depth of Carbonation versus Time for S50 Concrete.

Figures E1 and E2 predict corrosion initiation by a carbonation front reaching the depth of steel. A corrosion propagation period of ~10-20 years may occur before cracking and spalling are evident. The outcomes of the predictions are summarised in Table 23 in terms of required minimum cover. The values are rounded up to the nearest 5 mm and an absolute minimum cover of 30 mm is used regardless whether a lower value is predicted to be adequate.

Table E3. Estimated Required Minimum Depths of Cover for Carbonation Resistance

Concrete Mix	Required Minimum Depth of Cover (mm)
S40, OPC	30
S40, 25% FA	50
S40, 65% BFS	70
S50, OPC	30
S50, 25% FA	35

10.6 Appendix F – Results of CIRIA C660 modelling

1. Conditions and Assumptions of the Prediction

1.1 Dimensions of the Columns:

The concrete columns are 1.8 meters thick, 4.0 meter long at bottom and 5.6 meters at the top, and 8.5 meters high.

1.2 Reinforcement Details:

For long face of the columns, the rebar details are as follows:

(1) Horizontal bars:
- Top section: N12 @ 150 mm spacing, with 45 mm cover thickness
- Bottom section: N16 @ 150 mm spacing, with 65 mm cover thickness

(2) Vertical bars:
- Top section: N24 @ 105 ~ 140 mm spacing, with 57 mm effective cover thickness
- Bottom section: N28 @ 100 ~ 105 mm spacing, with 81 mm effective cover thickness

1.3 Concrete Details:

Concrete to use has a 0.37 w/c ratio and contains GP cement 400 kg/m^3 from Kandos Cement and fly ash 125 kg/m^3 (about 23.8%) from Liddell Power Station. As there is no reliable data available on the hydration heat value of the cement, a value 323 kJ/kg was predicted in the analysis.

Coarse aggregates and coarse sand from Wolffdene are identified as a basalt rack and fine sand is quartz sand from Dubbo. The concrete density is estimated about 2335 kg/m^3. Specific heat 1.0 kJ/kg·ºC, thermal conductivity 2.1 w/m·ºC, and thermal expansion coefficient 10*10^{-6}/ºC are estimated basalt coarse aggregates and coarse sand.

1.4 Existing Ground

Two columns will be poured on the spread footing of 1.2 metre thickness, which placed on a basaltic rock. There will be a construction joint between the footing and columns.

1.5 Conditions of the Pour

The time of concrete pour is assumed to be 6:00 am and pour is estimated to occur at September 2008, as suggested by designers. There will be slightly difference in analysis results if the concrete is poured in different time and season. According to the historic observation data from Bureau of Meteorology, following temperatures are predicted, maximum 24 ºC, minimum 12 ºC and mean 18 ºC. Average wind speed is 5.3 m/s and humidity is 65%. Usual concrete placing temperature without any cooling or heating measure is estimated as 5 ºC above the mean temperature of the day, i.e. 23 ºC in this case. In case a higher concrete placing temperature experienced in field condition or a lower placing temperature required to control the maximum peak temperature and the maximum temperature differential to the target values, thermal analysis on the varying concrete placing temperature is provided in this report.

1.6 Formwork/Insulation Options

Four formworks/insulations or combinations are considered in this analysis to control particularly the maximum peak temperature and maximum temperature differentials at the assumed weather condition. They Include:

(1) Steel Form of Any Thickness, with an estimated surface conductance of 26.8 w/m^{2}C

(2) Plywood Form 18 mm, with an estimated surface conductance of 5.46 w/m^2C
(3) Plywood Form 37, with an estimated surface conductance of 3.46 w/m^2C
(4) Plywood Form 18 mm + Polystyrene foam 10 mm or the equivalent (steel form + Polystyrene foam 15 mm), with an estimated surface conductance of 1.81 w/m^2C

2. Results of Thermal Analysis

The results of the thermal analysis including the temperature control, edge restraint calculation, and the cracking potential are presented in this section.

2.1 Temperature Control in Concrete Columns

As an example, the temperatures in the centre and on the surface of the columns and the temperature differential between the centre and surface are given in Figure 1, plotted against the time after placing, for 18 mm plywood form with usual concrete placing temperature of 23 ºC. The relevant temperature profiles for the moments of the peak centre temperature and the maximum temperature differential are given in Figure 2. It can be seen that peak temperature is 81 ºC, which is only slightly higher than the required maximum 70 ºC and the maximum temperature differential is 34 ºC, which is much higher than the required maximum 20 ºC. The columns would crack and its durability would be impaired with the cracks and possibly with the ASR and DEF.

The predicted results, including the maximum peak temperatures, the maximum temperature differentials, and related temperature drops in the centre and on the surface, for various initial placing temperature and formworks/insulations are given in Table F1 and also plotted in Figure F1 to F2 against the placing temperatures. They are discussed in the following paragraphs.

2.1.1 Steel Form

For steel formwork, the required 70 ºC maximum temperature can be achieved with a maximum placing temperature below 23 ºC. However, the corresponding maximum temperature differential is 53 ºC, which is much higher than the required 20 ºC to control the cracking caused by the internal restraint. A lower placing temperature to 13 ºC could only reduce the maximum temperature differential to 47 ºC. The temperature drop for the placing temperatures of 23 ºC is 62 ºC in the centre and 19 ºC on the surface. It is not recommended to use steel form in this case.

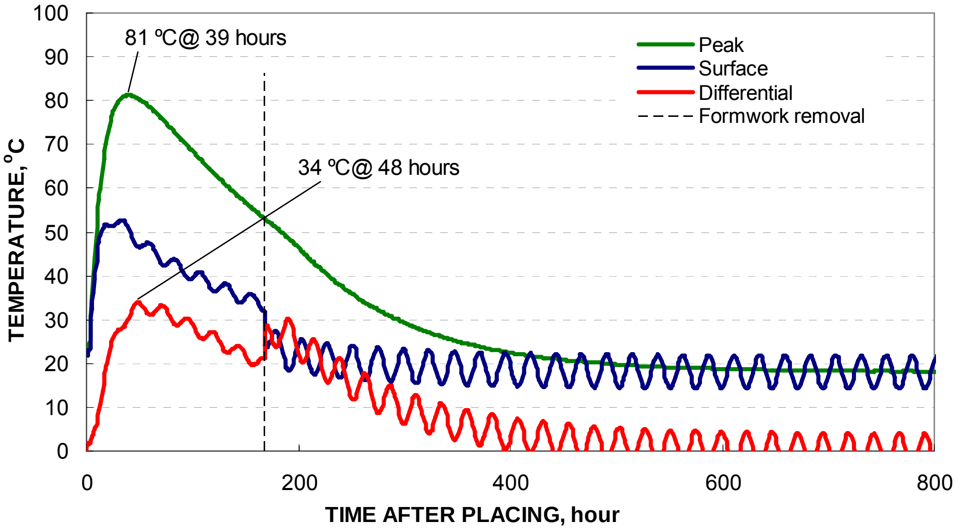


Figure 1, Temperature of concrete columns placing at 23 ºC in 18 mm plywood form

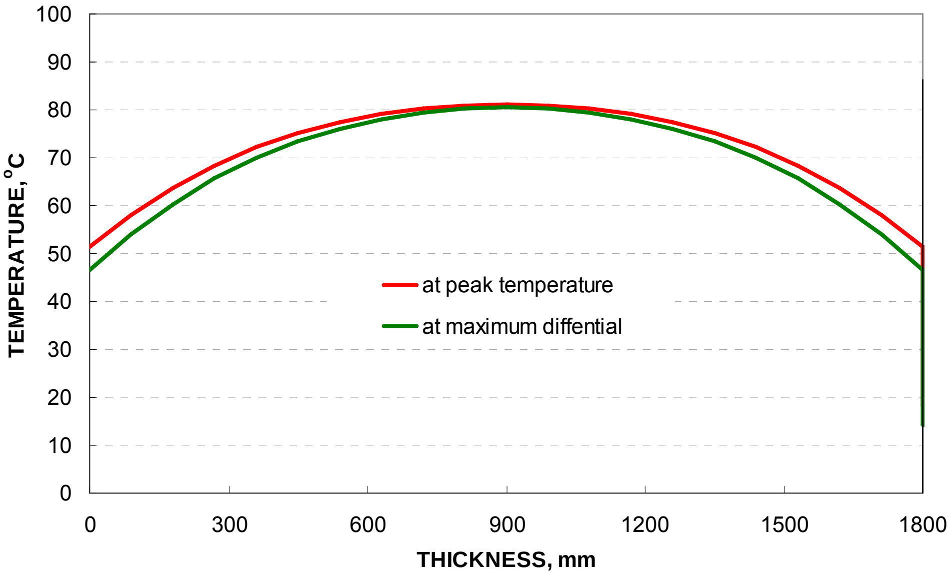


Figure 2, Temperature profiles of concrete columns for the moments of the peak temperature and maximum differential, placing at 23 ºC in 18 mm plywood form

2.1.2 Plywood Form 18 mm

For 18 mm plywood, the required 70 ºC maximum temperature can be achieved with a maximum placing temperature below 22 ºC. The corresponding maximum temperature differential is 34 ºC. A lower placing temperature of 13 ºC can only reduce the maximum temperature differential to 28 ºC. The temperature drop for the placing temperature of 22 ºC is 63 ºC in the centre and 35 ºC on the surface. It is not recommended to use 18 mm plywood form in this case.

2.1.3 Plywood Form 37 mm

For 37 mm plywood form, the required 70 ºC maximum peak temperature can be achieved with a maximum placing temperature of 21 ºC. The corresponding maximum temperature differential is 27 ºC, which is still higher than the required 20 ºC. A lower placing temperature of 13 ºC can only reduce the maximum temperature differential to 23 ºC. The temperature drop for placing temperatures of 21 ºC is 62 ºC in the centre and 40 ºC on the surface. It is recommended to do thermal analysis to see whether cracking risk is high or not with 37 mm plywood form.

2.1.4 Plywood Form 18 mm + Polystyrene Foam 10 mm

For combination of 18 mm plywood form + 18 mm polystyrene foam or the equivalent (steel form + 15 mm polystyrene foam), the maximum peak temperature of 70 ºC can be achieved with a maximum placing temperature of 20 ºC while the corresponding maximum temperature differential reduces to 17 ºC, is lower than the required 20 ºC. A lower placing temperature of 13 ºC can reduce the maximum temperature differential slightly to 15 ºC. The temperature drop for placing temperatures of 20 ºC is 62 ºC in the centre and 47 ºC on the surface. It is recommended to use these combinations of forms and insulation in this case, provided that the risk of early age thermal cracking is proven low in the analysis.

Table 1, Temperatures of concrete placed at various temperatures with different forms/insulations

Placing Temperature, ºC	Peak Temperature, ºC	Time, hours	Temperature Differential, ºC	Time, hours	Temperature Drop, ºC	
					Centre	Surface
(1) Steel Form of Any Thickness <25 mm, to be removed after 1 day						
27	84	30	55	46	66	22
25	82	32	54	46	64	21
23	80	34	53	46	62	19
21	77	37	51	47	59	17
19	75	41	50	47	57	16
17	72	45	48	47	54	15
15	69	50	45	47	51	15
13	67	54	42	48	49	14
(2) Plywood Form18 mm, to be removed at 8 days						
27	85	34	36	48	67	39
25	83	37	35	48	65	36
23	81	39	34	48	63	35
21	79	43	33	50	61	34
19	77	47	32	71	59	33
17	74	52	31	71	56	31
15	72	56	30	72	54	29
13	70	60	28	72	52	27
(3) Plywood Form 37 mm, to be removed at 12 days						
27	86	37	28	50	68	45
25	84	39	28	71	66	43
23	82	43	27	71	64	42
21	80	45	26	72	62	40
19	77	50	25	72	59	39
17	75	54	24	72	57	37
15	73	58	24	72	55	35
13	71	65	23	73	53	33
(4) plywood 18 mm + Polystyrene foam 10 mm or the equivalent (steel form 15 mm + Polystyrene foam), to be removed at 23 days						
27	87	43	19	72	69	53
25	85	45	18	72	67	52
23	83	50	18	72	65	50
21	81	52	17	73	63	48
19	79	56	17	73	61	46
17	76	60	16	96	58	44
15	74	67	16	96	56	43
13	72	72	15	96	54	41

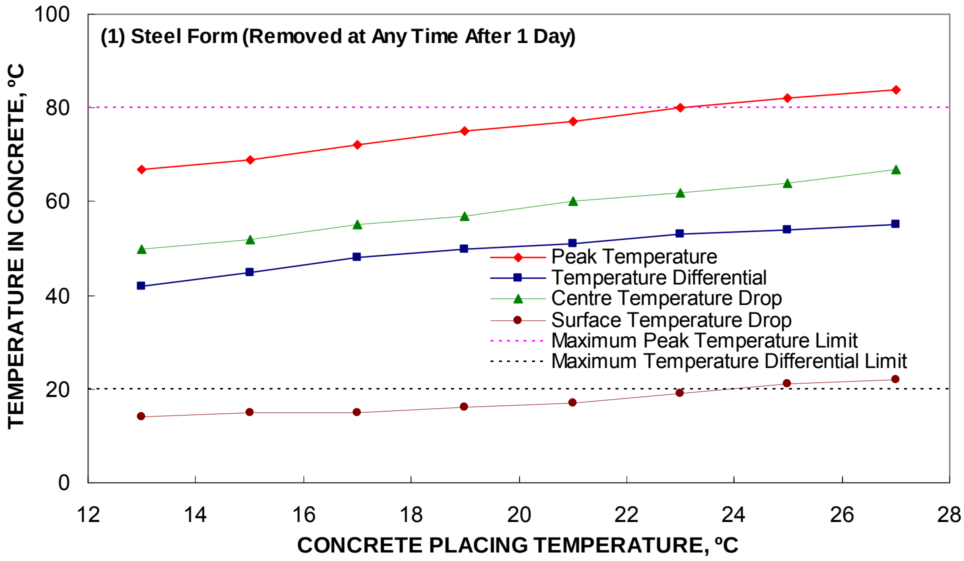


Figure 3, Predicted temperatures of concrete columns in steel form placed at various temperatures

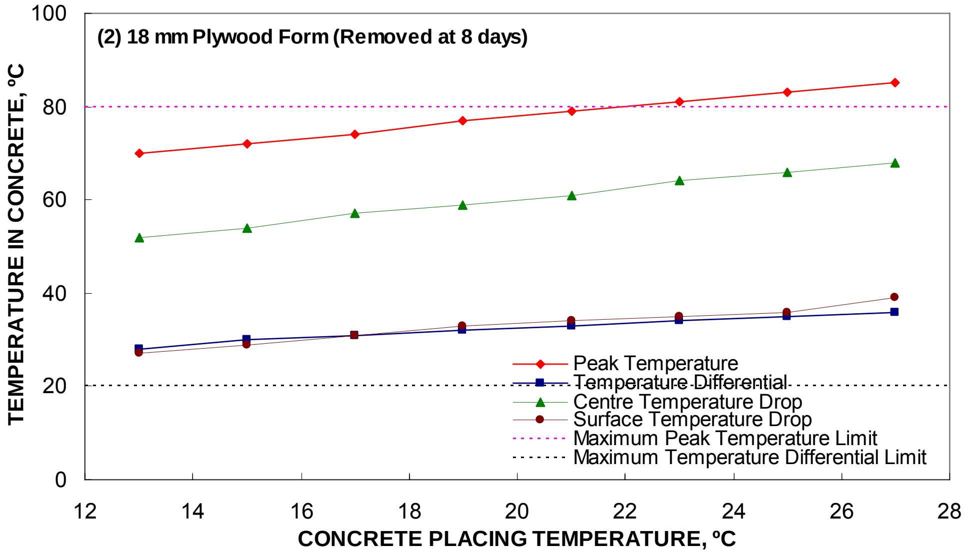


Figure 4, Predicted temperatures of concrete columns in 18 mm plywood form placed at various temperatures

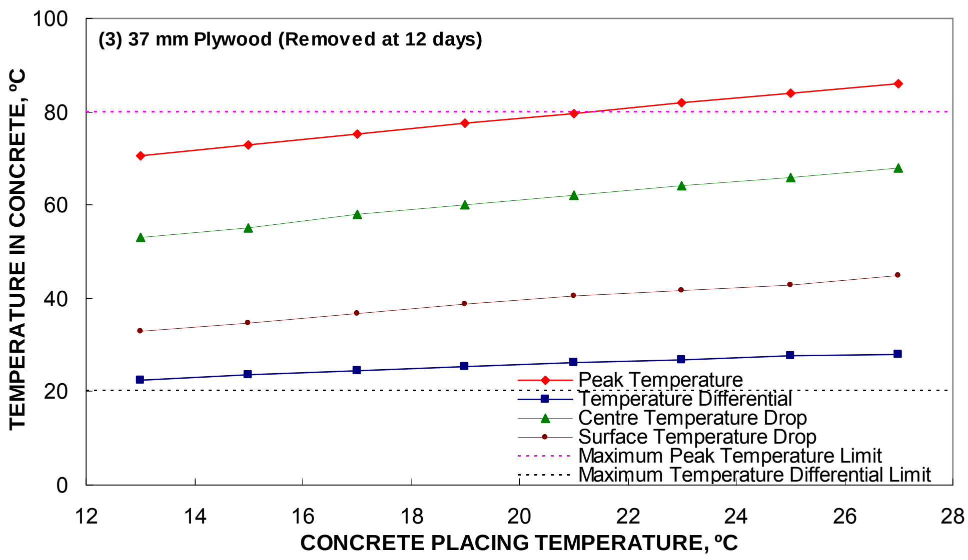


Figure 5, Predicted temperatures of concrete columns in plywood form 37 mm placed at various temperatures

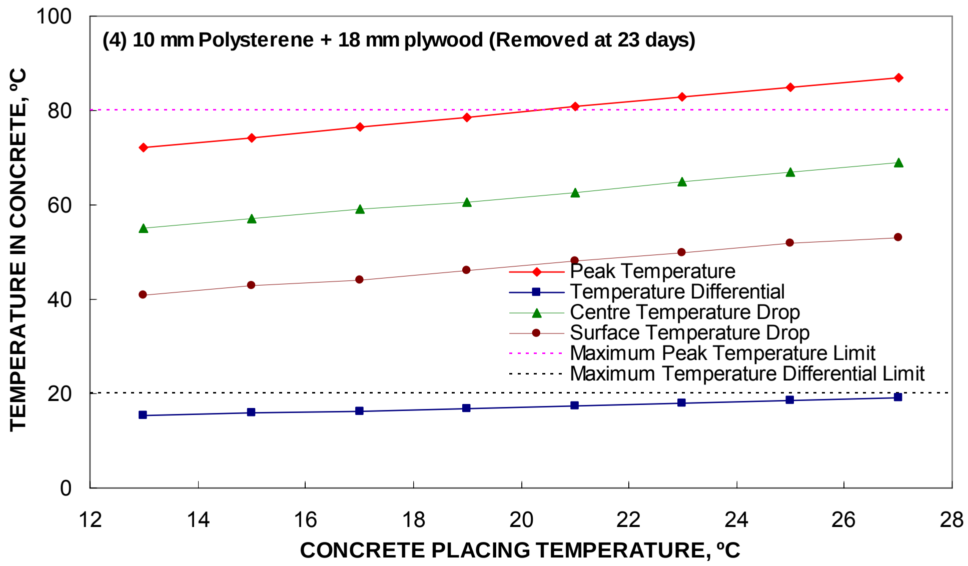


Figure 6, Predicted temperatures of concrete columns in plywood form 18 mm + polystyrene foam 10 mm (or equivalent steal polystyrene combination) placed at various temperatures

2.2 Edge Restraint from Footing

The restraint from the footing to the concrete columns above increases with the concrete stiffness (modulus of elasticity) and the size (thickness and width) of the footing. The stiffness (modulus of elasticity) of concrete increases significantly with its age especially in early period of concrete. As it is not clear how soon the columns will be constructed after the footing is constructed, a maximum of 7 day period is assumed as a conservative value. In case of an early construction of columns can be achieved, less cracking potential would be expected.

The restraint factor from the concrete footing of 7 days old to the new pour column is predicted as shown in the Figure 7 below. It can be seen that the restraint factor is 0.28 at the joint and reduces to 0 at 2.5 meter above.

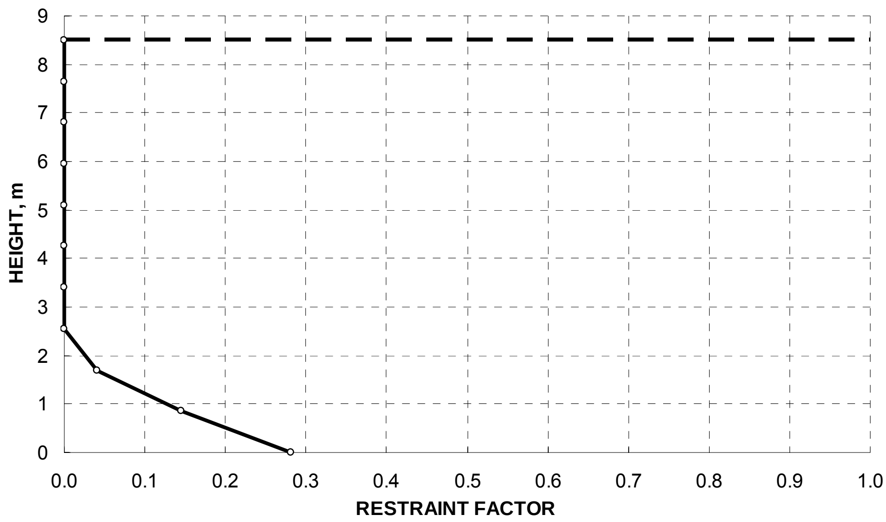


Figure 7, Restraint factor from 7 days old concrete footing to now columns

2.3 Predicted Cracking Potentials Due to Internal Restraint

Cracking potentials, due to the internal restraint, in concrete columns with all four types of forms (insulations) are predicted for a few levels of the concrete placing temperature, at which the maximum peak concrete temperature can be controlled below the required maximum temperature of 70 ºC. The cracking risk is evaluated with the CIRIA660 model. The width and spacing of the potential cracks on the surface and in the centre of the concrete columns are predicted in both horizontal and vertical directions.

2.3.1 Steel Form

The predicted cracking potentials for concrete columns with steel form are given in Table 2. Concrete columns are predicted to have a high cracking risk in the centre and on the surface due to internal restraint. There will be vertical cracks of 0.08 ~ 0.13 mm width and horizontal cracks of 0.06 ~ 0.09 mm width on the surface, depending on the position and the placing temperature. The cracks, especially the horizontal ones, can be considered as not significant. In the centre of the concrete columns, there will be vertical cracks of 0.17 ~ 0.40 mm width and horizontal cracks of 0.17 ~ 0.41 mm width, with crack being wider at top section. The crack width in the centre can be considered as significant and will affect adversely the structural integrity and durability. It is not recommended to use the steel form in this case.

2.3.2 Plywood Form 18 mm

The predicted cracking potentials using 18 mm plywood form are shown in Table 3. Concrete columns are predicted to have a low cracking risk in the centre and on the surface due to internal restraint, except that placed at 22 ºC and on the surface. There will be vertical cracks of 0.03 ~ 0.07 mm width and horizontal cracks of 0.02 ~ 0.04 mm width on the surface, which can be considered as not significant. In the centre of the concrete columns, there will be vertical cracks of 0.05 ~ 0.15 mm width and horizontal cracks of 0.07 ~ 0.15 mm width. The crack width in the centre may have some adverse effects on structural integrity and durability of the columns. It is not recommended to use 18 mm plywood form in this case at a placing temperature higher than 17 ºC.

2.3.3 Plywood Form 37 mm

The predicted cracking potentials for the concrete columns with 37 mm plywood form are given in Table 4. Concrete columns are predicted to have a low cracking risk due to internal restraint, with a negligible crack width of 0.01~0.03 mm on the surface and in the centre. It is not recommended to use 37 mm plywood form at a placing temperature below 21 ºC, provided that there will be low cracking risk due to the edge restraint from the footing.

2.3.4 Plywood Form 18 mm + 10 mm Polystyrene Foam

The predicted cracking potentials for the concrete columns with 18 mm plywood form+10 mm polystyrene foam are given in Table 5. Concrete columns are predicted to have a very low cracking risk. The predicted crack width is 0.00 mm in all cases. Therefore, it is recommended to use 18 mm plywood form+10 mm polystyrene foam or the equivalent (steel form +15 mm polystyrene foam) in this case at a placing temperature below 20 ºC, provided that there will be low cracking risk due to the edge restraint from the footing.

Table 2, Cracking potential prediction on concrete columns due to internal restraint with steel form

Section	Bottom			Top		
Placing Temperature, ºC	23	19	13	23	19	13
Temp Drop, ºC	53	50	42	53	50	42
1. Horizontal Bars (Vertical Cracks)						
Rebar Diameter, mm	16	16	16	12	12	12
Rebar Spacing, mm	150	150	150	150	150	150
1.1 Crack on the surface						
Cover, mm	65	65	65	45	45	45
Cracking Risk	High	High	High	High	High	High
Crack spacing, mm	1276	1276	1276	1136	1136	1136
Crack width, mm	0.13	0.12	0.09	0.12	0.11	0.08
Crack Time, days	2	2	2	2	2	2
1.2 Crack in the Centre						
Cover, mm	819	819	819	843	843	843
Cracking Risk	High	High	High	High	High	High
Crack spacing, mm	8763	8763	8763	11050	11050	11050
Crack width, mm	0.27	0.24	0.17	0.40	0.36	0.26
Crack Time, days	13	13	13	13	13	13
2. Vertical Bars (Horizontal Cracks)						
Rebar Diameter, mm	28	28	28	24	24	24
Rebar Spacing, mm	105	105	105	140	140	140
2.1 Crack on the Surface						
Cover, mm	81	81	81	57	57	57
Cracking Risk	High	High	High	High	High	High
Crack spacing, mm	825	825	825	815	815	815
Crack width, mm	0.09	0.08	0.06	0.08	0.08	0.06
Crack Time, days	2	2	2	2	2	2
2.2 Crack in the Centre						
Cover, mm	791	791	791	819	819	819
Cracking Risk	High	High	High	High	High	High
Crack spacing, mm	5017	5017	5017	6523	6523	6523
Crack width, mm	0.28	0.24	0.17	0.41	0.37	0.26
Crack Time, days	13	13	13	13	13	13

3. Conclusions

3.1 The predicted concrete peak temperature reduces with reducing concrete placing temperature for all four types of forms (insulation). To control a maximum temperature below 70 ºC for the expected durability, the concrete placing temperature should be lower than 23, 22, 21 and 20 ºC for steel form, 18 mm plywood, 37 mm plywood and the insulated form (18 mm plywood + 10 mm polystyrene or equivalent) respectively.

3.2 For above required concrete placing temperatures, the predicted maximum temperature differentials are 53, 34, 26, 17ºC for steel form, 18 mm plywood form, 37 mm plywood and the insulated forms, reducing with increasing level of insulation.

3.3 The predicted restraint factor from the underneath footing has a maximum value of 0.28 at the joint and reduces with the distance from the joint if the concrete footing is less than 7 days old when concrete columns is cast.

3.4 Concrete columns with steel form are predicted to have a high cracking risk due to the internal restraint especially in the centre of column. The crack width is 0.06 ~ 0.13 mm on the surface and 0.17 ~ 0.41 mm in the centre.

3.5 Concrete columns with 18 mm plywood form are predicted to have a low cracking risk due to the internal restraint except that at a high placing temperature and on the surface. The crack width is 0.02 ~ 0.07 mm on the surface and 0.05 ~ 0.15 in the centre.

3.6 Concrete columns with 37mm plywood form are predicted to have a low cracking risk due to internal restraint. The crack width is 0.01 ~ 0.03 mm on the surface and is 0.00 ~ 0.03 mm in the centre, which are negligible.

3.7 Concrete columns with insulated form (18mm plywood form + 10 mm polystyrene foam) are predicted to have a very low cracking risk due to the internal restraint, with all crack widths being 0.00 mm.

3.8 Concrete columns with all forms are predicted to have a low cracking risk on the surface due to the edge restraint from the footing. The crack width is 0.00 ~ 0.04 mm and increases with an increasing level of form insulation. However, they have a high cracking risk in the centre with a cracking width being 0.12 ~ 0.21 mm, except that at a very low placing temperature (13 ºC). Use of three additional horizontal N16 rebars per layer (with vertical spacing of 150 mm) can reduce the crack width to 0.08 ~ 0.13 mm.

4. Recommendation

4.1 Plywood form of 37 mm thickness and the maximum concrete placing temperature below 21 ºC are recommended in this case, with a low cracking risk expected due to the internal restraint.

4.2 Use of three additional horizontal N16 rebars (vertical spacing of 150 mm) is recommended to reduce the crack width in the centre due to the edge restraint from the footing.

4.3 Concrete columns should be cast within 7 days after footing has been cast.

4.4 The 37 mm plywood form should be removed after 12 days to avoid surface cracking.