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Durability Plan for Coastline Concrete Structures and Design Considerations under Aggressive Environment Conditions

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ABSTRACT: Concrete durability is the ability of concrete to resist environmental impact or chemical attack in a long time without substantial deterioration. The aim of this study is to provide a durability plan for concrete structures to encompass the 100-year design life. The coastline concrete structure design requirements are formulated considering a case study of twin bridges in Ballina regions, Australia, which can be extended to other type of concrete structures. The proposed durability plan incorporates concrete strength, provided cover thickness and construction methods for each structural component. This paper elaborates the service life design of twin bridges and conducts durability design, employing the first principles for environmental loads affecting structural components. The realistic model for chloride induced corrosion, based on diffusion theory and carbon dioxide diffusion model, are also included in this study. In addition, the specifications of concrete and other preventive measures are described. The chloride modelling of the bridges has been conducted using a realistic model based on diffusion theory. It has been revealed that 100 years of design life can be achieved by using 65% slag in the concrete of bridge deck. The findings suggest that fibre boards and supplementary cementitious materials both are notably effective to maintain a durability of 100 years.

Keywords: Durability; Supplementary cementitious materials; Realistic modelling; Diffusion theory

1. Introduction

Many concrete structures are exposed to various types of aggressive environments. Significant challenges and problems can be experienced by concrete structures in long term. These problems include deteriorating processes such as carbonation, alkali-aggregate reaction, chlorination, freezing, and thawing. Bridges provide an essential platform for better communication and transportation in modern societies. A bridge is a common source to get across a river. It is also a way to get across a busy freeway. Therefore, any onshore concrete bridge structure under consideration is required to be designed in a way to withstand environmental resistance and to allow the traffic flow up to hundred years. In this paper a comprehensive durability plan is presented for twin bridges in Ballina region. It is an exclusive plan to ensure 100 years of durability. These Ballina bridges are located 736 km by road from Sydney CBD in the southern side. Both of them contain the same number of piers and abutments. The details of structural members are also present in the drawings to determine the durability of project. The drawings also include the information related to earthquake loading, wind loading and flood data of the adjacent areas of these bridges. The bridges are in the region of Ballina, which is 1 km away from the coastline. The probability of environmental attack increases due to the coastline proximity. Durability plan ensures the overall performance and health of concrete structures. Therefore, the necessity of durability plan is high to achieve a better performance in a long term. There are certain features that should be incorporated in the scope of the durability plan. The major points are:

- Recognizing the potential environmental loads on the structure,
- Identifying appropriate design practices for construction, and
- Establishing suitable measures to achieve the complete design life.

Design life is the period for which bridge, or any structure is necessary to remain useable. The design life for many bridges is intended to 100 years. The realistic modelling based on diffusion theory and CO_2 diffusion model is used to come up with 100 years design life and then comparing with the code AS 5100:2007. The design life incorporates the intended life with appropriate maintenance. To study the environmental effects, previous reports have been reviewed and accordingly a data set was established.

2. Literature Review

Chloride diffusivity along with its design is a major benchmark to check the quality control in durability planning. Concrete compressive strength and chloride diffusivity are the primary element to for measuring the quality checks. Output of research reflects that safety margins of durability less than 2% for 120 years is safe. Durability quality check happened by changing the design value to laboratory classifications specifically for chloride diffusivity [1]. Reinforcement corrosion ratio is high to damage the concrete cover and steel rusting causing cracking, spalling. Corrosion ratio in reinforcement has applied the rule of splash zone, underwater zone and atmospheric zone.

Corroded reinforcements ratio of decreased tropical atolls is higher compared with general marine offshore environment. Chloride concentration of concrete is lower in ordinary Portland cement (OPC) compared with calcium aluminate cement (CAC). To improve the durability in concrete structure lowering the rate of water-to-binder recommended for compressive strength as well. Employing OPC approach is more durable than CAC. The investigations reveal that complicated and harsh environment of tropical atolls corrodes concrete structures severely and the reinforcement corrosion ratio is extensive [2]. Impact of salt spray is moderate on steel (e.g. BSt 420). Focus is required on better engineering to reduce the large losses. There is substantial ductility due to salt spray exposure. The outcomes of accelerated corrosion tests on bare steel bars depicters better qualitative compared with aged concrete steel bars. Deterioration of reinforced concrete structure will occur due to alkali-silica chemical reaction [3]. The tool to describe the relation of chloride ingress into concrete under two different normal & mock environments, the outcome can be defined by Fick's diffusion model. To check the connection of chloride in concrete between normal & abnormal environments also required at different level of chloride content surface of convection zone depth that results in concrete characteristic

& environmental issues [4]. Analysing the chance of corrosion with the time started when considered two separate classifications of environment severity and with depth of concrete underwater including w/c ratio. Starting of corrosion is subject to different parameters. It has been seen that structural surface and the cover depth values are main factors for probabilistic analysis. To check the material resistance against chloride ingress, coefficient of diffusion has a major role. Durability is linked with true values of cover depth which is not commonly used in many practices. This factor gives more accurate results with probabilistic approaches, keeping in view the inherent randomness present on degradation phenomenon that may disturb structural durability. Optimum values of cover depth totally associated with concrete quality against porosity that can be seen in the water-cement ratio. More deep research is needed to consider the conception, maintenance, and reparation to reach the engineering which align with w/c ratio and the cover depth value which ultimately reaches high structural safety [5]. Durability of concrete is not fully linked with design and material; however construction work and quality also does matters. The lower construction quality will be exposed in sever environment in concrete structures. Probability approach of design shall be used to achieve durability. Most of the durability problems can be seen in mismanagement of quality control checks, hence the focus should be given to the quality control to reach desired durability. Experienced based tools to check the quality of concrete construction should be conducted to avoid any quality mismanagement. To reach better controlled durability, proper manual should be issued, explaining the concerns regarding future durability [6,7].

3. Part 1: Structural Members and Environmental Loads

3.1 Identification of Structural Members

The structural components of twin bridges are identified using drawings and listed in the following Table 1 along with their concrete strength grade, methods of construction and cover thickness.

Table 1 Structural members and their construction details

Serial No.	Type of Structural Component	Construction Method	Cover thickness (mm)	Concrete strength grade (MPa)
1	Deck of Bridge	In-situ cast	Sides cover – 55 mm Top cover – 45 mm Bottom – 30 mm	40 MPa
2	Piers (N1 to N3 and S1 to S3)	In-situ cast	70 mm	50 MPa
3	Barriers	Precast	Not available	50 MPa
4	Abutments (Northbound and southbound)	In-situ cast	Face contacting ground – 65 mm Face exposed to atmosphere – 45 mm	50 MPa
5	Piles (No. 550 Octagonal driven)	In-situ cast	50 mm	Segment of Leader pile – 50 MPa Segment of follower pile – 60 MPa
6	Concrete Plank	Pre-tensioned & Precast	35 mm	50 MPa

3.2 Environmental Loads on Structural Members

- To study the environmental effects, previous reports have been reviewed and accordingly a data set was established. This data contains the following details:
- Meteorological data: wind, temperature and relative humidity of Ballina surroundings were obtained by the local meteorological centre. These reports were important in indicating the kinds of environmental attacks that can potentially harm the bridge [8].

Meteorological details are depicted in Table 2.

Table 2 Meteorological data of Ballina region

Rainfall Mean	1845 mm
Relative Humidity	74% - 80%
Mean Temperature	8°-31° Celsius (2019)
Sea Level Rise	1 mm (per generation)
Mean Wind Speed (light winds)	13.5 - 21.5 km/h

- Environmental data: Environmental reports for Ballina region were accessed to study the carbonation-corrosion.
- The rainfall is drastically increasing in that region annually. The Meteorological Bureau of Ballina states that the average rainfall in 2000 was 900 mm, which increased to 1800 mm in 2016. This represents that relative humidity would keep on increasing accordingly in upcoming years [8].
- The air may comprise the 10 billion metric tons by chloride in that region. Out of which 30% may went back to the ground. The chloride content is heading 1450 km (900 miles) away from the ocean region. And this is continuously increasing in the whole region, which creates an alarming situation [9].
- As per AS 5100.5 the twin bridges are in the temperature zone. As the annual temperature is increasing gradually, so a meticulous approach is to be taken during curing practices. The improper curing can lead to plastic shrinkage.
- Testing of Soil: The client has provided the data of borehole for below-ground interaction of environmental loads and types of material (Table 3).

Table 3 Bore log for Ballina twin bridges

Depth (m)	**pH**		**Sulphate (%)**		**Chloride (mg/kg)**	
0	7.20	8.64	0.0630	0.180	684	5930
5	8.49	8.92	0.0109	0.0537	2724	9664
15	6.79	7.75	0.0005	0.0091	3892	5602
20	7.06	7.06	0.0005	0.0005	3816	3816
25	6.86	7.07	0.0005	0.0015	2736	3001
30	6.90	8.31	0.0068	0.0113	3697	3858
35	8.34	8.34	0.0059	0.0059	2248	2248
40	7.20	8.64	0.0630	0.180	684	5930
45	8.49	8.92	0.0109	0.0537	2724	9664

Figure 1 depicts the different types of materials by borehole analysis.

Borehole depth (m)	Material
1	Clay
3	Silty Clay - Medium Plasticity
4	Silty Clay - High Plasticity
5	Silty Sand
13	Gravelly Sand
14	Gravelly Sand Clay
19	Meta-sandstone

Fig.1 Types of materials

The structural members identified previously from drawings are subjected to environmental loads depending on their location i.e., underground, or above ground. The environmental loads on each structural member are depicted in Table 4.

Table 4 Environmental loads on structural members

Serial No.	Structural Member	Environmental Loads
1	Deck of Bridge	Carbonation, Chloride Attacks
2	Piers (N1 to N3 and S1 to S3)	ASR, Chloride and sulphate attacks
3	Barriers	Carbonation, Chloride Attacks
4	Abutments (Northbound and southbound)	Carbonation, Chloride and sulphate attacks
5	Piles	Chloride-induced corrosion sulphate attacks
6	Concrete Plank	Carbonation, Chloride Attacks

4. Part 2: Service Life Design from First Principle

After identifying the structural members, a durability design is conducted from the first principles for environmental loads. The detailed summary of environmental loads is given in Table 5, which are required prior to commencement of design.

4.1 Chloride-Induced Corrosion

The bridge structure is located on the coastline; this closeness of the bridge makes the structure favourable to the chloride attacks. It is given that there is no atmospheric air borne chloride present but there are other parameters responsible for chloride ingress in concrete like mixture proportions, type of cement and existence of reinforcing bars. The carbon dioxide in the presence of moisture reacts with chloride salts and gives free chlorides. These chloride ions diffuse into the concrete and affect its strength. Piles and piers are considered as submerged elements of a bridge structure. Piles have three places where the exposure conditions vary.

1. Submerged portion in ground
2. Submerged portion in water
3. Portion above sea water

Table 5 Environmental loads summary and their effects [10]

Serial No.	Location	Environmental Loads	Consequences	Issues In design
1	Underground	Chloride in creek water	Rate of diffusion is low due to high quantity of Oxygen in creek water	Chloride attack mostly occur to the piers due to their exposure
2		Sulphate attack	Causes the volume of concrete to expand and contract due to change in its elastic properties. End result is cracking in concrete	Piers are more vulnerable to sulphate attack
3		Alkali Silica Reactions	It causes random cracking and deteriorates the surface. High Loss in serviceability	For this bridge structure, it is not accepted to possess visible surface cracking
4	Atmospheric or above ground	Chloride-induced corrosion	Rate of diffusion of chlorides increase with the increase in concentration of chloride ions in atmosphere It increases the corrosion rate of steel	There is no airborne chloride present in atmosphere as given.
5		Corrosion due to Carbonation	It decreases the serviceability and strength of structure exposed to atmosphere	The members of bridge exposed to atmosphere are most effected e.g., bridge deck, pile cap and portion of piles
6		Pollutants	Interacts with the surface and delaminates	Protective layers to avoid pollutants causing cracking
7		Temperature and moisture	Tends to expand and contract the concrete	Exposed elements protected by paint

The chloride diffusion for outer exposed parts of the bridge to atmosphere is more than in comparison with submerged components. This happens due to the presence of high concentration of oxygen in water. The submerged elements have higher values of chloride concentration. Concrete having strength of 30 MPa or more has chloride concentration ranging from 1%-2 % by cement weight [11]. Pier 1 on Northbound is experiencing the highest quantities of chlorides with 2280mg/kg. The chloride penetration effect through using SCMs can be decreased by controlling the cover design for the elements of structure submerged in water. The concrete deck is assumed of experiencing the chloride diffusion and the chloride concentration is taken as 2.8 kg in a meter cube for modelling [12]. We have done the chloride induced modelling of concrete using realistic model based on diffusion theory.

4.2 Realistic Model for Chloride Concentration

The realistic model can be used for the twin bridges due to the variability of factors like unified environment, uniform quality and absolute cover which may affect the service life of [13]. As we know the covers from the provide drawings, we can calculate the time required for chloride to pass through that cover by using following modelling relation.

$$t = \frac{x^2}{2D_a}\left[erf^{-1}\left(1 - \frac{C_x}{C_s}\right)\right]^{-2} \qquad (1)$$

Where: t = Time, x = Depth, D_a = Diffusion coefficient, C_s = Concentration of chloride at surface,

C_x = Concentration of chloride, f = Compressive strength

By using the correction factors for stress, exposure time and temperature as well as microclimatic conditions, we can derive improved model as follow.

$$t = \frac{x^2}{2k_1k_2k_3D_a}\left[erf^{-1}\left(1 - \frac{C_x}{\xi\theta C_s}\right)\right]^{-2} \qquad (2)$$

Some of the values can be suggested for the various factors mentioned above as shown in table 6.

Table 6 Suggested factors for realistic model of chloride induced corrosion

Factors	**Suggested**
	$D_e = k_{1.}k_2.k_3.D_a$
Exposure Time Factor k_1	k_1 is $\left(\frac{t_{e1}}{t_{e2}}\right)^{\alpha}$ where α = 0.4 (Lim, 2000). α = 0.59 for grade 50 GP concrete
Temperature Factor k_2	2 for every 10^o C Temperature rise above 23^oC, 0.5 for every 10^o C temperature drop below 23^o C. Linear interpolation (Evardsen and Mohr, 2000).
Crack-freed stress factor k_3	0.70-0.76 for compression members, 1.1 for tension zone in un-cracked flexure member
Limiting crack width to cover W_{cr} /C ≤0.01 Stress factor k_3	1.2 for crack in tension zone
	C_e = €θ Cs
Limiting crack width to cover W_{cr} /C ≤0.01	θ= 1.0 for crack-freed member such as columns and piles
Crack factor width factor θ	θ = 1.2 for members with limiting flexural crack such as beams
Microclimatic load factor €	€ = 1.0 for fully submerged zone. € = 1.15, 1.08,1.02 for grade 32, 40 and 50 in tidal zone
Limiting value of C_e	1.5, 1.0 and 0.8 for grade 32, 50 and 60 respectively. [14]

4.3 Observations for Chloride Induced Corrosion

By using the borehole data mentioned in Table 3, it can be observed that for piles submerged below the ground the chloride concentration is in the range of 5000-12000 mg/kg according to the depth. It means that the chloride induced corrosion will be more severe in these regions as compared with the rest of the areas.

Table 7 Observations for chloride induced corrosion

Depth	pH	Soil condition	Chloride concentration range
3 m to 8 m	8.64 to 8.76	Clay	5000-12000 mg/kg
17 m to 19 m	6.79	Silty sand	5000-12000 mg/kg

4.4 Calculations

The Following equation was used to calculate the amount of chloride concentration at a specific location.

$$C_x = C_s\,[1 - \mathrm{erf}(\frac{x}{2\sqrt{D_a t}})] \qquad (3)$$

In this case the cover above is 60 mm, a check should be performed to figure out the amount of chloride at 60 mm and then compare this to the chloride threshold level (CTL) = 0.05, Cs=1.4, α = 0.55.

For chloride, the first step is to calculate the diffusion coefficient for 100 years (D100 years). This can be carried out using the following formula:

$$D_{100} = 0.90E - 12(\frac{1}{100})\alpha \qquad (4)$$

The blinder that is chosen is fly ash with a w/b ratio of 0.4. As seen from Table 4,

$$D_1 = 0.90E - 12\text{m2/s},$$

Therefore:

$$D_{100} = 0.90E - 12\ (\frac{1}{100})\alpha$$

$$D_{100} = 7.15\text{E} -14$$

The t is 100 years. This need to be converted into seconds.

T = 60 X 60 X 24 X 365 X 100

T – 315360000

Finally,

$$C_x = 1.4\ [\ 1 - erf\left(\frac{0.06}{\sqrt{7.15E -\ 14\ X\ 3153600000}}\right)]$$

$$C_x = 0.0066 < 0.05$$

Since the calculated C_x for the piles is less than CTL, the piles is deemed to comply with chloride induced corrosion.

Table 8 Chloride concentration against different types of cement

Blinder	w/b	$f28$ (MPa)	Cs	D 1 year ($10m^{12} m^2$ / s)	Alpha	Critical Chloride
GP	0.6	0.5	38.0	0.8	7.93	0.60
GP	0.4	63.0	2.8	1.15	0.55	0.12
High Slag	0.6	33.5	1.6	2.87	0.75	0.05
High Slag	0.4	45.5	0.5	1.24	0.60	0.03
Fly Ash	0.6	25.5	1.5	2.80	0.85	0.10
Fly Ash	0.4	62.5	1.4	0.90	0.55	0.05

By looking at the initial drawing given, the smallest cover required is the 55mm which is for the concrete planks. After applying the same formula as above, but using a cover of 55mm instead of 60mm:

$$C_x = 1.4 \left[1 - erf\left(\frac{0.006}{\sqrt{7.15E - 14 X 3153600000}}\right)\right]$$

$$C_x = 0.013 < 0.05$$

This means that the smallest cover of 55mm is also sufficient again chloride induced corrosion. According to this result, it is acceptable to say that all components of the twin bridge will be acceptable to chloride induced corrosion.

4.5 Chloride Modelling

It is established via modelling that choice of concrete and concrete cover compared with chloride ingress is much better, as depicted below in Figure that chloride ingress is at upper layer of concrete cannot reach to reinforcement even after 99.8 years. This slowdown to chloride is established in below graph that chloride ingress percentage is lower as 0.10% after 100 years. Figure 02 shows the effect of chloride concentration on reinforcement with time.

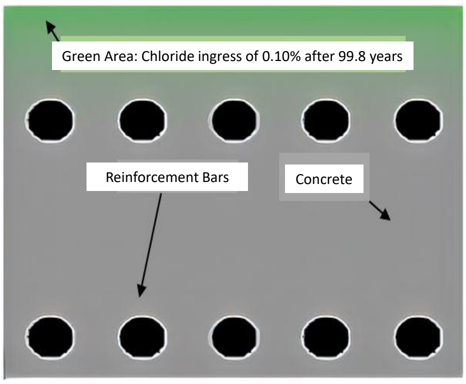

Fig.2 Effect of chloride concentration on reinforcement with time

4.6 Carbonation-Induced Corrosion

The Carbonation corrosion is witnessed various parts old structures in the region of Ballina. The report issues by environmental authorities have indicated that the carbon induced corrosion may also lead to the expansion of the structure by steel corrosion. These expansions have high impacts on strength and cause cracks in the structure. The following members of the structure will show the possible effects of carbonation-induced corrosion:

- Planks precast: The already formed layer of carbonation will prevent the further carbonation in the affected members of the structure.
- Concrete in deck: Carbonation may slightly affect the deck concrete, although it is not prone to carbonation due to thick 10 mm bitumen fibre board and 75 mm bituminous topping, protecting it underside.

The following materials are taken into consideration to highlight the effects the effects of carbonation induced corrosion in our bridge structure.

- Ordinary Portland cement or GP Cement
- Slag by 65% in replacement with Cement
- Fly Ash (FA) 25% replaced with cement

4.7 CO_2 Diffusion Model

The carbonation depth of the members that are prone to the carbonation is calculated by using the following relation.

$$d_c = Kt^{0.5} \tag{5}$$

Where: K is rate of carbonation or coefficient (mm/year$^{0.5}$),

T = Initial time,

dc = Depth of carbonation

The calculations for concrete plank and abutments by using Ordinary Portland cement or GP Cement, Fly Ash (FA) 25% replaced with cement and Slag by 65% in replacement with Cement respectively are shown below.

- By using general purpose cement (GP) in concrete having strength more than 35 MPa and exposed conditions.
- Calculation of depth of carbonation by applying 25% fine aggregates (FA) in concrete having strength more than 35 MPa in exposed conditions.
- Calculation of depth of carbonation by applying 65% slag in concrete having strength more than 35 MPa in exposed conditions.

Table 9 Observations for chloride induced corrosion

Rate of carbonation (K) For Relative humidity >80%	1	(mm/year$^{0.5}$)
Corrosion rate For cover = 50 mm	2	µm/year
Intensity of corrosion to crack	100	µm
Carbonation Depth Calculation		
For normal cover	35	mm
Propagation time, T_p	35	Years
Initiation time, Ti	65	Years
Depth of carbonation for precast planks, dc	8.062258	mm
*This shows that cover of plan is sufficient by considering GP cement for carbonation		
Calculation for Atmospherically Exposed Abutments		
For nominal Cover	45	mm
Propagation time, T_p	45	Years
Initiation time, Ti	55	Years
Depth of carbonation, dc	K * (t) $^{0.5}$	mm
Depth of carbonation for exposed abutments	7.416198	mm
*This shows that cover of abutments is sufficient by considering GP cement for Carbonation		

Table 10 Observations for chloride induced corrosion

Rate of carbonation (K) For Relative humidity >80%	5	(mm/year$^{0.5}$)
Corrosion rate For cover = 50 mm	2	µm/year
Intensity of corrosion to crack	100	µm
	A) Carbonation Depth Calculation	
For normal cover	35	mm
Propagation time, T_p	35	Years
Initiation time, T_i	65	Years
Depth of carbonation for precast planks, dc	40.311289	mm
This shows that cover of plank is NOT enough, when using 25% fine aggregates for carbonation.		
	B) Calculation for Atmospherically Exposed Abutments	
For nominal Cover	45	mm
Propagation time, T_p	45	Years
Initiation time,T_i	55	Years
Depth of carbonation, dc	K * (t) $^{0.5}$	mm
Depth of carbonation for exposed abutments	37.080992	mm
*This shows that cover of abutments is sufficient by using 25% FA for carbonation.		

Table 11 Observations for chloride induced corrosion

Rate of carbonation (K) For Relative humidity >80%	7	(mm/year$^{0.5}$)
Corrosion rate For cover = 50 mm	2	µm/year
Intensity of corrosion to crack	100	µm
	A) Carbonation Depth Calculation	
For normal cover	35	mm
Propagation time, T_p	35	Years
Initiation time, T_i	65	Years
Depth of carbonation for precast planks, dc	56.435804	mm
*This shows that cover of plank is NOT enough by using 65% slag for carbonation.		
	B) Calculation for Atmospherically Exposed Abutments	
For nominal Cover	45	mm
Propagation time, T_p	45	Years
Initiation time, T_i	55	Years
Depth of carbonation, dc	$K * (t)^{0.5}$	mm
Depth of carbonation for exposed abutments	51.913389	mm
*This shows that cover of abutments is NOT sufficient by using 65% slag for carbonation.		

The precast barriers would not undergo carbonation corrosion, as they do not possess any reinforcement. In above calculations, it is clear by carbonation modelling that when concrete is casted by using GP cement with keeping nominal cover for elements, it prevents carbonation. The above calculations show that cover is also not sufficient for 100-year design life by replacing cement with fly ash and slag. Although, the regional and meteorological clearly showed that the carbonation is not a huge issue to the bridge structure, however we would redesign the structure, which would withstand the adversity of slag and fly ash on concrete carbonation induced corrosion [15]. By performing above calculations, it can be observed that cover is not sufficient for some conditions. It is to be noted that by changing cover, we will be changing the structural capacity of the structure so we will prefer not to change the cover. Instead, we will change the concrete mix.

4.8 Reduction of Carbonation

If we want to reduce carbonation without changing cover requirements, then we can suggest following recommendations.

- Concrete should be made by blending cement having a minimum cement content of 240 kg/m^3
- The cement used for making concrete should be a shrinkage limited one or the general-purpose cement which should conform to the specifications.
- If it is desired to prevent carbonation in drier regions of bridge then the compensation of carbonates can be made by increasing general purpose cement proportions.

4.9 Sulphate and Acid Sulphate

In the report provided by the Roads and Maritime Services (RMS) acidic sulphate soils is present in Pacific Highway. Along with that the borehole data also confirms Pier 1, Northbound is exposed with sulphate content. Table 12 outlines the content of each member exposed with the sulphate soil in ground.

Table 12 Observations for chloride induced corrosion

Part, Member	Content of Sulphate (mg/kg)	Classification of Exposure
Abutment a, SB	671	Not Severe
Abutment a, NB	49	Not Severe
Abutment a, NB	91	Not Severe
Pier 1 SB	15601	Very Severe
Pier 2 NB	1001	Moderate
Pier 3 NB	1002	Moderate

The mechanism of sulphate ions transport is diffusion. The sulphate ions diffuse into the concrete core and effect the concrete. The diffusion rate of sulphate is 1/100 as compared to the chloride attack. Though the rate is exceedingly small, but the effect of harming the concrete is severe. The ettringite (delayed ettringite formation, DEF) is formed into the concrete due to sulphate attack. This leads to the expansion and concrete expands up to 227% of its original volume. The elastic properties of concrete loss due to the gypsum formation by the sulphate attack. The coastal location of twin bridges makes it vulnerable to both sulphate and chloride, considering this both would also interact with each other. The interaction of chloride and sulphate ions reduces the effects produced by the sulphate ions. In the presence of chloride, the ettringite and gypsum are soluble. These soluble precipitates out of the concrete. Barriers and Planks are other precast member must be put in use after they get dried. This allows protective layer against sulphate attacks to be formed by carbonation [15]. The submerged piers and ground contacted abutment surfaces are discussed further in the report.

4.10 Alkali-Silica Reactivity and Ettringite

In alkali-silica reaction with concrete aggregates wherein, magnesium, sodium, potassium hydroxides react with mineral content in the crack or hole openings of concrete. They form silica gel and cause cracking and expansion. The cracklings can occur in different parts of the structure with different patterns. In lightly loaded elements it occurs as map-cracks, such as in precast barriers. In highly loaded elements like abutments and piers, it occurs in the principal stress direction. The alkali-silica reactivity is considered as a serviceability issue. The use of alkali metals should be on priority for better design. Apart from that Australia is has less content of alkali in cement which makes it less vulnerable to alkali-silica reactivity (ASR). Moreover, the use of Supplementary Cementations Material (SCM) like slag and fly ash are used to diminish the effect of ASR. Along with that the use of nonreactive aggregate eliminates the adverse impact of ASR. The surface abrasion caused by the ASR increases the deterioration of concrete surface and increase serviceability problems. There are procedures and treatments in this report to handle all these issues. The surface abrasion caused by the ASR increases the deterioration of concrete surface and increase serviceability problems. There are procedures and treatments in this report to handle all these issues. The bridges under consideration are S3 category of acceptability of damage due to ASR. It means that the twin bridge must be designed to avoid any visible cracking caused by ASR during the complete Hundred-year design life.

4.11 Preventive Measures

The following preventive measures can be taken to avoid the alkali silica reactions.

- Use of supplementary cementations materials (SCMs) like natural pozzolans, fly ash, fume of silica, and GGBFS. SCM can be used as 25% for fly ash, 10% for silica fume, 40-70% for blast-furnace slag by mass of cement.
- The access of external alkalis and moisture should be controlled by using a sealer which can be paint or any moisture repellent surface. The surface should be re-applied after certain period.
- Use river gravel for this purpose because it consists of minimum reactive aggregates.
- Application of barium and lithium salts in the form of admixtures can be used.
- Use of low water to cement ratio and air entrainment of concrete to control the expansion due to ASR.

4.12 Bacteria-Induced Corrosion

The concrete can be affected by microbial activity in underground surfaces. There are two possibilities of happening this activity. The anaerobic bacteria create corrosive and disruptive sulphuric acid that causes harm to the concrete. Secondly, the bacteria named 'Thiobacillus ferrooxidans' oxidize the pyrite. These both create sulphates and corrosive sulphuric acids which have adverse effects on the concrete [16].

5. Part 3: Specifications of Concrete and Other Protection

5.1 Specifications of Materials

5.1.1 Classification of exposure and material design

The exposure of structures is classified in the standards given by Table 4.3 of AS 3600 [17]. It states that the structural members that are below water and submerged will be considered in B1 classification. Whereas the structural members outside the water and exposed to external atmosphere are subjected to B2 classification. The structural members of the twin bridges can be analysed for the exposure classification. The piers of the bridges are constantly under chloride and acid sulphates exposure due to chlorides present in soil. In this case, the effect of sulphates is reduced to negligible extent due to presence of chloride because the sulphate salts will be dissolved and come out of the concrete. Hence, the piers will only be subjected to the attack of chlorides and we will analyse this situation and give remedial preventive actions for this situation. The pier headstocks are present in the zone where water will continuously splash on structure, so it is considered in C2 classification. The piles are classified in U due to the absence of data of exposure. Table 13 gives the classification of exposure for different elements of bridges.

Table 13 Exposure classification of structural components

Location	Classification based on Exposure				
	Piles	Pier	Planks	Deck	Abutments
Southbound	U	C2	B2	B2	B2
Northbound	U	C2	B2	B2	B2

The exposure classification mentioned above helps in determining the concrete grade for the structural members. For this purpose, AS 5100.5 (Table 4.10.3-A) is used to determine concrete grades for cast-in-situ concrete members [18]. Table 14 provides the nominal cover for different exposure classifications of cast-in-situ concrete members.

a. Nominal cover where standard formwork and compaction were used

Table 14 Nominal cover for cast-in-situ concrete members

Exposure Classification	Nominal Cover for concrete of characteristic compressive strength(f_C) not less than (mm)			
	25 MPa	32 MPa	40 MPa	≥50 MPa
A	35	30	25	25
B1	-	45	40	35
B2	-	-	55	45
C	-	-	-	70

Note: Increased value is required if Clause 4.10.3 (c) applies

In case of precast concrete members, AS5100.5 provides us with a different table to determine concrete grades for precast concrete members based on their exposure classification. Table 15 provides the nominal cover for different exposure classifications of precast concrete members [18].

b. Nominal cover where rigid formwork and intense compaction were used

Table 15 Nominal cover for precast concrete members

Exposure Classification	Nominal Cover for concrete (TABLE 4.10.3(B)of characteristic compressive strength(f_C) not less than (mm)			
	25 MPa	32 MPa	40 MPa	≥50 MPa
A	25	25	25	25
B1	-	35	30	25
B2	-	-	45	35
C	-	-	-	50

It is necessary to give special attention while casting the concrete and achieving the required cover. There is provision of allowance of 20mm for cast-in-situ and 5mm for precast members given in Australian Standard AS3600:2009.

5.2 Concrete Works and Thermal Modelling

Concrete mix design and construction methods inserted in concrete works modelling program along with related site parameters to check the durability of this bridge. The modelling was performed for 100 years' time to fit the design life supplies. Granulated ground blast furnace slag (GGBFS) is famous to be utilized in lowering the temperature differentiation due to heat in concrete pours by hydration. Along 180mm nominal thickness of bridge deck, it has not been the main aspect. The effectiveness of SCM can be seen in below picture very evidently. It can be evidently seen in the Figure 3; temperature differentiation is due to heat of hydration is satisfactory having concrete pour being done in the morning in summer season.

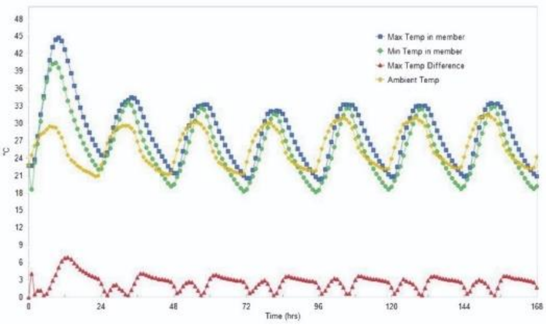

Fig3. Effect of temperature on concrete

5.3 Concrete Specifications

After specifying the concrete cover for the structural members based on their exposure classification, we can also specify the design requirements of concrete mix design to fulfil durability requirements. For this purpose, we can get assistance from the RMS B80 guide. The durability requirements of concrete based on exposure classification of structural members are given in Table 16.

Table 16 Requirements of durability for cast-in-situ & precast concrete members

Exposure Classification	Minimum Cement Content (kg/m^3)	Minimum Cement Content (kg/m^3)	Minimum Water/ Cement Ratio (by mass)	Minimum Water/ Cement Ratio (by mass)	Maximum Chloride test coefficients at 20^{0}C (x 10^{-12}m^2/sec) NT Build 443 (D_3)	NT Build 492 (D_{RMC})	Minimum Strength for durability f_c min(d) (MPa)	Action Required
				Cast-in-place concrete				
A	320	400	0.56	0.4	N/A	N/A	25	N/A
B1	320	450	0.50	0.4	N/A	N/A	32	N/A
B2	370	500	0.46	0.32	3.5	8.0	40	Use Blended cement with minimum 25% FA or 50% BFS
C	420	550	0.40	0.32	2.0	4.0	50	Use Blended cement with minimum 65% BFS
U					In accordance with Annexure B80/A1			
				Precast concrete				
A, B1	320	600	0.5	0.28	N/A	N/A	40	N/A
B2	370	600	0.46	0.28	3.5	8.0	60	Use Blended cement
C	420	600	0.40	0.28	2.0	4.0	60	Use Blended cement

Applying the design requirements of concrete cover and concrete mix design, incorporating environmental loads and exposure classification of structural members, we can suggest design specifications for the durability plan of bridges to satisfy 100-year design life. Details are depicted in Table 17.

Table 17 Design specifications for 100 years design life

Serial No.	Structural Component	Exposure Classification	Environmental loads	SCMs	Nominal cover	Water to cement ratio	Concrete Grade	Cement content
1	Deck of Bridge	B2	Carbonation, Chloride Attacks	GGBFS (65%)	30mm	0.32-0.46	55 MPa	370-500 kg/cum
2	Piers (N1 to N3 and S1 to S3)	C2	ASR, Chloride and sulphate attacks	GGBFS (65%)	70mm	0.32-0.40	50 MPa	420-575 kg/cum
3	Barriers Precast	B2	Carbonation, Chloride Attacks	Fly ash (25%)	No reinforcement	0.32-0.46	50 MPa	370-500 kg/cum
4	Abutments (Northbound and southbound)	B2	Carbonation, Chloride and sulphate attacks	Fly ash (25%)	45mm	0.32-0.46	50 MPa	370-500 kg/cum
5	Piles (No. 550 Octagonal driven)	U	Chloride-induced corrosion sulphate reactions	GGBFS (65%)	50mm	0.28-0.35	60 MPa	420-575 kg/cum
6	Concrete Plank (Pre-tensioned and precast)	B2	Carbonation, Chloride Attacks	Fly ash (25%)	35mm	0.32-0.46	55 MPa	370-500 kg/cum

The suggested concrete grade for deck is 55 MPa and it is conforming the requirement for abrasion which is at least 32 MPa. Similarly, the input for water-cement ratio for chloride modelling is used 0.34 in Life-365 (modelling software), while performing chloride induced corrosion modelling this lies in the range of 0.32-0.40 as indicated above.

5.4 Effect of SCMs and Curing on Durability of Concrete

There will be high heat of hydration due to mass concrete of piers, abutments, and deck slab. For prevention of occurrence of thermal cracks, a suitable amount of supplementary cementitious materials (SCM) are added in the concrete mix but also steps are taken to ensure the effective and proper curing. Loss of moisture is prevented by curing; steam curing is recommended for pre-cast members whereas wet curing for 14 days for cast-in-situ members is recommended. In the region of high wind, moisture loss can also happen. To avoid excessive bleeding wind barriers must be erected as this will make also curing more effective [19]. There are lots of advantages of adding SCMs in the concrete mix design. The negative effect of the increase in depth of carbonation due to the addition of SMs will be weighed out due to its more beneficial effects. The design life of 100 years will be obtained in aggressive and harsh weather conditions, slag is added up to 65%. Chloride induced corrosion prevented by the addition of fly-ash. Another advantage of SCMs is that the concrete mix will be less permeable and more durable. Further, sulphate attack is prevented by the addition of slag and fly-ash as both will reduce chemical diffusion rate. There will be less alkali-silica reaction due to these supplementary cementitious materials (SCMs) [20].

5.5 Performance Based Specifications

Performance based specifications are required for the bridge to last for 100 years. They are done to help attain design life and their costs for maintenance are generally high. Some of the requirements to meet by using performance-based specifications are compressive strength, water absorption, surface abrasion, chloride diffusion and sorptivity [17].

The Performance based specifications are as follows.

- The concrete needs to have a compressive strength of 50MPa
- Diffusion coefficient, $D_{28=3X10^{12}} m^{\frac{2}{s}}$ and $D_{365=1X10^{12}} m^{\frac{2}{s}}$
- RTA sorptivity is 14 mm
- Volume of Permeable Voids (%) is 13%
- Rapid Chloride Permeability is 1000 Coulomb

Following are the summary of performance-based specifications required for our bridge.

Table 18 Design specifications for 100 years design life

	D_{28} ($^{-12}m^2/s$)	$Sorptivity_{28}$ (mm)	VPV_{28}(%)	$RCPT_{28}$ (Coulomb)
	Precision from relevant standards or other sources			
Sources of Precision	ASTM C1566	ASTM C1585	ATSM C642	ASTM C1202
Reportable to	0.001	0.1 x 10^{-4}	0.1	NA
Repeatability CV	14%	6%	2%	12%
Correlation D_{365} of a GP & 3GB cement concrete (Cement Concrete & Aggregates Australia 2009)				
Range of value correlation coefficient, R	10-60, 74%	15-100mm, 76%	6-13, 12%	1000-5000, 76%
Critical Range: One order change in D_{365} from 1 to 10 m^2/s	3	14	13	1000
Increment used in classification	RMS 1.5	RTA B80 12mm	VicRoads 1-2%	C1202 1000

5.6 Preventive Actions for Corrosion of Concrete

5.6.1 Use of coatings

Polyurethane based exterior weather coating by the trademark of SIKA/FOSROC for additional protection of concrete surface on piers, abutments and deck slab is good prevention from corrosion. This gives a strong and rapid hindrance to the entrance of forceful synthetic chemical substances into the structure while all the while enabling the structure to relax and breath. We can develop the required thickness in micrometres by applying multiples layers of coating of these. During the application procedure, we may use the technical sheets provided by the manufacturers.

5.6.2 Cold and expansion joints treatment

While constructing the bridges, joints are more sensitive and foreseeable. If there is a time gap in concrete placing then joints will form, and these are cold joints. For the accommodation of movements due to contraction and expansion of structural components, expansion joints are formed [21]. However, for the diffusion of harsh elements, joints are the most vulnerable points. Surface retarders are used at the end to avoid cold joints. The use of this will avoid forming any joints at next when placing new concrete with old one. It is critical to treat the expansion joints. To seal the expansion joints different sealants of approved trademark (SIKA/FOSROC) are used. The sealants are used to avoid ingress the chemicals into structures through the expansion joints provided in structure of bridge. Hence the movements in the bridge structure due to expansion and contraction are also not compromised [22].

5.7 Preventive Actions for Corrosion of Steel Reinforcements

5.7.1 Control of cracks using AS5100 & AS 3600

Structural integrity is severely damaged if cracks are oriented along with the reinforcement. Shear and bending strength of the structure is adversely affected by these cracks. Alkali-silica reaction can produce the cracks that are in same alignment along the steel reinforcement bars. To achieve the required design life, AS3600 provides the different strategies to reduce and control these cracks. For instance: one technique is provided in Section 8.6 of AS 3600-2009.

5.7.2 Galvanization / cathodic protection

Zinc-Rich epoxy coating on steel reinforcement prevent it from deteriorating and further ingress of chemicals. The manufacturer provides the technical details for these coatings, and they should be implemented. Another option is the application of galvanized reinforcement than ordinary reinforcement.

6. Conclusions

After comprehensive literature review and establishing the design requirements through modelling some suggestions are presented for marine structures. Throughout the entirety of the coastline concrete structure and design specifications for aggressive environments this document should be considered as a living document for alkali-silica reactions. It is suggested that to identify the potential durability risks for members in tidal splash zone and submerged structures, conduct testing of ground water and creek. It is endorsed that due to the marine exposure and acid sulphate conditions, pre-cast reinforced concrete structures tested and inspected before installation. Proper inspection of edge reinforcement detailing is required as per scope and specifications. Construction of coastline concrete structure in aggressive environment conditions demands the quality control and insurance at all the stages. Concrete Works modelling for environmental loads i.e., chloride ingress, thermal effects, and carbonation: has confirmed the design life of 100 years for concrete structures as per given specific design requirements.

7. Disclosure Statement

The authors would like to mention that there is no potential conflict of interest.

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