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Durability design for tunnels and approaches

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ABSTRACT: The paper provides recommendations on assessment of tunnels exposures as a durability load, and procedures to select appropriate concrete quality and materials to resist those loads. National codes include deemed to satisfy (DtS) prescriptive requirements for limited exposure conditions. This restricts the durability designer's options. Fib Model Code 2020 (2023) discusses the application of decreasing levels of approximation (LoA) as a more expensive but more precise design approach that may lead to significant environmental and structure cost reductions. Durability is increasingly using full probabilistic analysis (FPA) for durability design as a more precise method of predicting design life. This focuses attention on establishing environmental, safety, and life cycle costs (including public and operational costs) to develop a target reliability for the structure's elements at the end of the design life and then designing to meet that reliability. Given a tunnel's life may be extended well past the original service life strategic consideration in the design is also required.

1 INTRODUCTION

The durability loading on an element depends on the exposure. Only the more common exposures are discussed here. When considering tunnel exposure, the three areas of concern are the extrados, the intrados and the approaches. While local conditions might cause a variation along the length of each the key general exposure considerations are discussed here together with factors affecting the resistance to deterioration. A key in assessing exposures for tunnels is that DtS requirements in Australian Codes frequently do not apply to tunnels and hence specific assessment is required for each project.

2 CARBONATION

Carbonation is the process of carbon dioxide ingress reacting with calcium hydroxide to form calcium carbonate. This creates a low pH in the concrete such that reinforcement in carbonated concrete can corrode. In building interiors this is not considered a major issue. In temperate external climates it is classified as only exposure class A to AS 5100.5 (2024) yet the exposure to higher carbon dioxide concentrations and moisture contents in vehicle tunnels is known. The much higher carbonation rates and corrosion risks than in normal atmospheric conditions requires specific consideration. This may be one reason why Austroads (2019) specifies B2 as a minimum exposure inside tunnels, a more severe class than B1 required internally to comply with AS 5100.5 (2024). B2 is only a minimum exposure as a specific tunnel may be determined as having a more severe exposure.

Concrete's carbonation resistance depends on the cement chemistry and penetrability. FPA using carbonation models in fib 34 (2006) can be used to predict carbonation ingress (Jong, 2023). The carbonation resistances, as given in fib Bulletin 34 (2006) are shown in Figure 1.

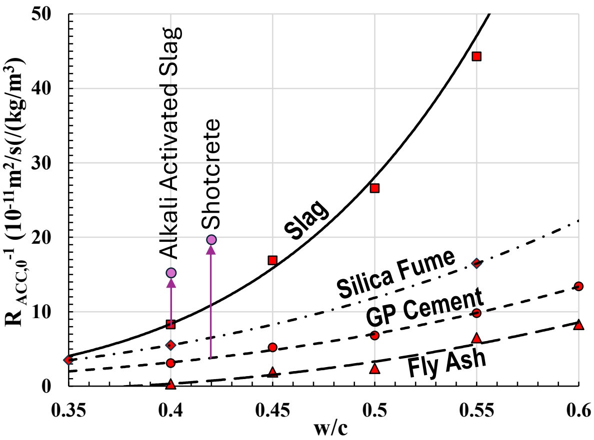


Figure 1: fib $R_{ACC,0}^{-1}$ values. A shotcrete result (from Table 1) shows effect of construction.

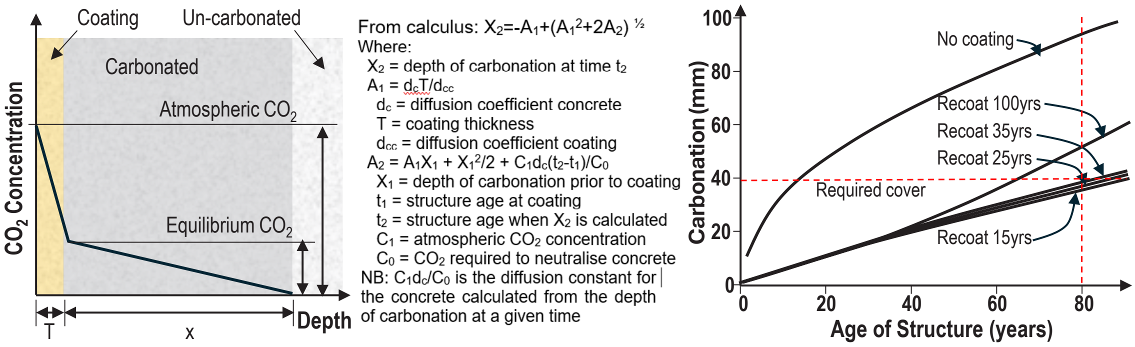
Table 1: Measured RACC,0-1 values for project mixes shows negative effects of curing and placing

Location	Strength (MPa)	Cement (kg/m^3)	Cement Type	w/c	Curing	Mean DoC in ACC tests (mm)	$R_{ACC,0}^{-1}$ (units as fig 1)	Outcome
Tunnel segments	75	420	7% FA	0.40	Steam	1.35	1.03	OK
Wall shotcrete	30	415	GP	0.42	Compound	6.00	20.4	Coat
Flat slab soffit	50	420	GP	0.37	Heat cured	2.75	4.29	Coat
Extruded planks soffit	45	445	28% FA	0.17	Heat cured	2.25	2.97	OK
Diaphragm wall	30	415	GP	0.43	None	4.25	10.2	OK
Extruded hollow core	45	445	28% FA	0.17	Heat cured	4.00	9.07	Coat
Precast flat slab	50	445	GP		Heat cured	6.00	20.4	Coat
Precast cladding	40	348	GP	0.47	Air cured	4.50	11.5	Coat
50MPa Alkali Activated Slag from a building project.						5.17	15.1	-

For vehicle tunnels Beushausen (2021) recommends a carbon dioxide loading of 1000-5000ppm. A reasonable conservative estimate should be established for the project. It should be verified in service. On one road tunnel the carbon dioxide concentrations at the tunnel exhaust was estimated as 1296 ppm average maximum over the life of the structure with a relative humidity of 65%. Caution is required in using this RH as waster penetration may cause a higher RH in the concrete. At the exhaust various material were used and the performance of each had to be assessed. An initial durability assessment was undertaken using data in as a guide. However, because the data was not specific to the project mixes the carbonation resistance (RACC) was measured by the ACC Test in fib Bulletin 34 (2006). Results are given for the tunnel segments and various other elements in the exhaust structure in Table 1.

The CO2 concentration was applied to determine the cover required to achieve the required 80 years to depassivation. A propagation period of 20 years was assumed but it may not always be conservative. Where this was more than the actual cover a coating was determined to be required.

The coating effectively reduces the carbon dioxide concentration at the concrete surface as shown in Figure 2a. The reduction is a function of the relative penetrability of the coating and the concrete and the consumption of carbon dioxide in the concrete that fixes the zero for the concentration gradient.



a) Schematic of how the CO_2 concentration reduces through the coating and carbonated concrete to provide a flux of CO_2 at the uncarbonated concrete where the carbonation reaction is taking place. Also shown is the formula for calculating the effect of the coating on the carbonation rate.

b) Example of carbonation depth with time assuming different coating re-application intervals. Carbonation depth at 80 years without coating is 86mm plus 10mm partial factor but actual cover is only 40mm. 35yr replacement interval of 450 µm thick coating gives adequate protection.

Figure 2: Carbonation With Coating

Given the huge investment in a tunnel's construction, the high repair cost due to the public and operational cost, and the relatively low cost of improvement to durability design, the following could be mandatory design requirements for vehicle tunnels:

— Prediction and monitoring of the CO_2 content of the tunnel, exhausts and approaches.
— Test concrete for carbonation resistance where the exposure is above normal atmospheric.
— Specific determination of reliability required for each element by the project team. For example, soffits may have a higher reliability required than walls.
— Full probabilistic or partial factor modelling of carbonation depth with time.

3 ATMOSPHERIC TUNNELS BELOW GROUND WATER

There are two key zones for consideration, extrados and intrados tunnel exposures.

3.1 *Extrados Exposure*

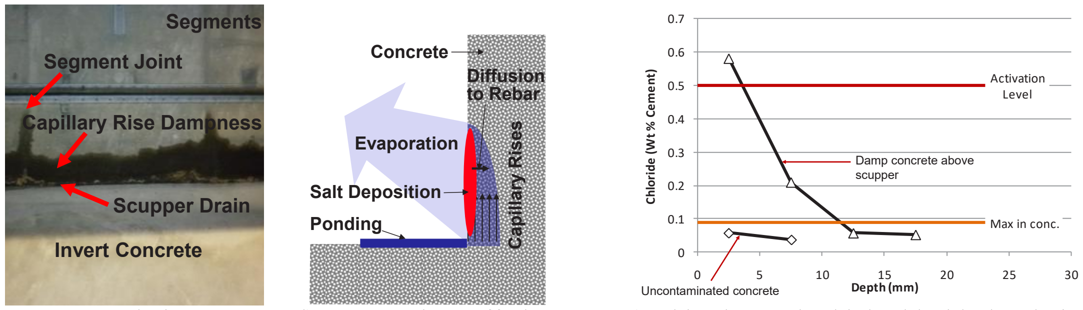
The extrados exposure severity depends on the nature and pressure of the attacking water. Although there are standard classifications for different ground conditions great care is required to assess the chemical attack on concrete and corrosion resistance of the reinforcement.

In some countries the groundwater sulphate levels can be very high. While this is very aggressive to GP cements high slag blends (>60% slag) in high performance concrete can provide excellent resistance to chemical attack, even from brines and acid sulphate soils. This is recognized in Australian standards but not in some European standards. Two key parts to design for exposures are the chemical resistance of the cement system and the penetrability of the concrete.

Reinforcement corrosion on the extrados is not a general concern as below the groundwater the pores will be saturated and hence all voids around the reinforcement, the normal location for corrosion commencement, will have a high pH from leached hydroxides (Papworth, 2018). This paper also notes that in immersed conditions localised corrosion can initiate early in a structures life and continue slowly such that loss of bar section isn't significant for up to 50 years. Such corrosion does not become visible as rust stains and spalling.

3.2 *Intrados Exposure*

Intrados exposure must consider how the durability design will cope with leaking cracks and joints. It is common that tunnels leak. Even water with low chloride levels that leaks into the tunnel and spreads over the intrados can give rise to high chloride levels on the surface due to 'evaporative concentration' of salts at the concrete surface. Chloride penetrates the concrete by a variety of mechanism involving water within the concrete pores. Mechanisms are described in Figure 2 to Figure 4.

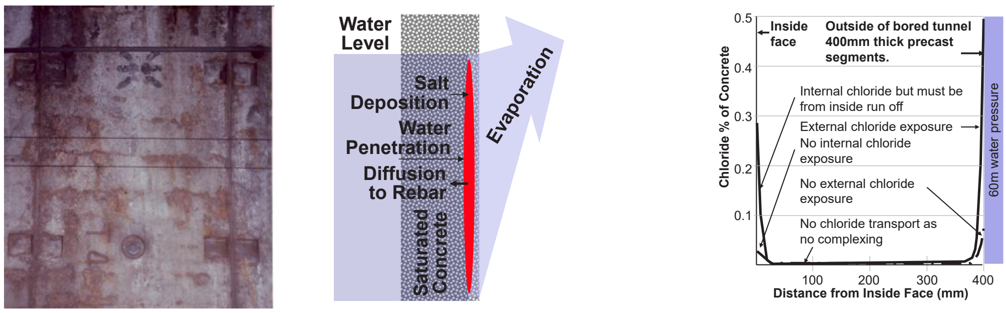


c) Scupper drains carry ground water leakage. Capillary rise can cause water to rise.

d) Evaporation off the concrete leaves behind salts. Concentrated surface chloride then diffuses back to the rebar.

e) This shows the high chloride levels just above a scupper drain. Constant evaporation can lead to very high salt levels and can also cause salt etching and sulphate attack.

Figure 2: Capillary Rise, Evaporative Concentration and Diffusion

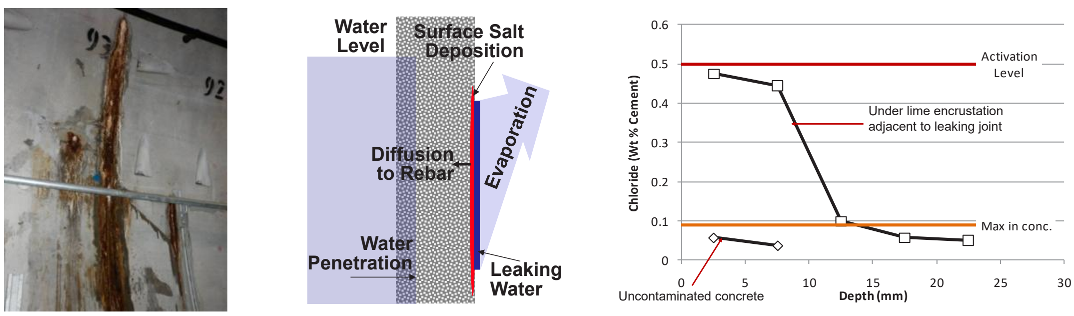


a) In concrete where water can penetrate the lining thickness salts will deposit at the surface leading to sulfate attack or reinforcement corrosion.

b) If evaporation is greater than flow rate no water becomes apparent but salt build up could still be occurring.

c) Edvardsen (2004) measured the chloride profile through a 400mm thick tunnel wall subject to a 60m pressure head. High chloride on the inside face occurred where there was high chloride externally even though no chloride penetrated.

Figure 3: Water Penetration and Evaporative Concentration



a) Saline water penetrates at cracks and joints as leakage and is then swept across the concrete surface.

b) It builds up as encrustations. Evaporation concentrates the salt which then diffuse inward.

c) This shows the chloride concentration beneath a surface encrustation due to leakage after just a few years.

Figure 4: Leakage, Evaporative Concentration, and Diffusion

Although not developed for tunnels a chloride loading (C_s) can be used in the fib Bulletin 34 (2006) chloride ingress model to provide some guidance for ingress of chlorides. For elements in marine and coastal exposures there is guidance on C_s (Papworth, 2025/1), but there are no guides for groundwater exposure. Edvardsen (2004) found C_s of 1.5 weight percent of cement (WPC) from leakage but this isn't related to time or external chloride concentration. This is very low and based on Papworth (2025/1) is less than might be expected for coastal atmospheric exposure. It is recommended that design be based on the chloride loading expected for the extrados or the severest atmospheric chloride level of the tunnels exposed ends or intakes as shown in Table 2.

Table 2: Exposure Classes[1] for Reinforcement Corrosion in Chloride Exposures for Tunnels and Approaches

	Internal Exposure Classes			Proposed External Exp. Class		Proposed Internal Exp. Class	
External Exposure	AS 4312	AS 5100.5	AGRT02-19	Papworth (2025/1)[2]	Surface Cl^- WPC	Papworth (2025/1)[2]	Sur-face Cl^- WPC
Brine	-	U	-	XS2a (-/-/U)	N/R[3]	XS1f(-/-/U)	4.0[4]
Seawater	-	U	C	XS2a(C/-/B2)	N/R[3]	XS3b(C/-/U)	3.5[5]
Beach front	C5	B2+	B2	XS1d(B2/C5/B2)	3.0	XS1d(B2/C5/B2)	3.0
High Coastal	C4	B2	B2	XS1c(B2/C4/B2)	2.0	XS1c(B2/C4/B2)	2.0
Med. Coastal	C3	B1	B2	XS1b(B1/C3/B1)	1.0	XS1b(B2/C3/B1)	1.0

1. Environment at the concrete surface. Tunnels have different reliabilities but the same exposure class.
2. The bracketed figures are (AGRT02-19, 2019 /AS 4312, 2019/AS 5100.5,2024).
3. The mechanisms controlling reinforcement corrosion are not related to chloride level.
4. Taken as XS1f rather than XS1e as the condition is likely to be continuously wet and hence have an aging factor for wet exposure rather than a higher value typical of a coastal type exposure.
5. This value is for a typical splash zone. Papworth (2025/1) recognises higher surface chloride levels for more severe exposures and consideration should be given to those if there is evidence to support it.

Modelling using C_s can determine when a critical chloride level at the reinforcement will be reached using he principal measure, i.e. chloride migration. This should be measured initially on cores from blocks cast using proposed construction practices, and subsequently on the structure. Concrete's chloride migration reduces over time. It is important to ensure the aging factors used are calibrated for the country of use. Papworth (2021) describes an approach for this.

The last factor in resistance to chloride induced corrosion is the chloride level at which reinforcement depassivation occurs (C_{crit}). It is known that C_{crit} varies with exposure, voids at the bar: concrete interface, metal type, cement type and many other factors. The low and broad C_{crit} distribution described for all concrete in fib Bulletins 34 (2006) and 76 (2015) resulted as the effect of other parameters could not be determined. Papworth (2025/2) describes the status of a fib Bulletin being prepared. It gives C_{crit} values for different steels, cements and exposures.

C_{crit} for black steel is often quoted as 0.4 WPC but frequently users fail to recognise this as the characteristic value for C_{crit}. The distribution expressed in Bulletin 76 is given as a mean value of 0.6 WPC with standard deviation of 0.15 as shown in Table 3. This is consistent with a characteristic value of 0.4 WPC.

Table 3: Recommended Lowest C_{crit} Distributions (Papworth, 2025/2)

Steel type and exposure	Mean (μ)	Std Dev (σ)	a	b	Charact.
Carbon steel in atmosphere	0.60	0.15	0.2	2	0.35
Carbon steel in saturated concrete	2.00	0.15	0	3	1.75
Prestressing cables, posttensioned tendons	0.40	0.15	0	0.5	0.20
Galvanised reinforcement	0.95	0.15	0	2	0.70
Stainless steel in atmospheric conditions					
304	4.00	2.00	0	5	0.70
316, 2304, 2101	7.00	2.00	0	9	3.70
Steel fibres	0.95	0.15	0	2	0.70

The broad C_{crit} distribution in Bulletin 76 (2015) is currently taken to apply to all cement types and concrete grades and qualities. However, it has also been shown that corrosion may not commence in some circumstance until much higher concentration are reached. This is clearly the case in immersed situations where Papworth (2025/2) notes theory, laboratory simulations and practice all show that in immersed situation the critical chloride content is much higher. Critical chloride levels proposed are also shown in Table 3.

Base chloride level (the chloride concentration in the as built concrete) can be critical because it determines the allowable increase in chloride before the critical chloride level is reached. On one project the contractor failed to prove that the specified base chloride level was being achieved, and it was six months before the engineer undertook their own tests and found the chloride level was much higher than specified, even though on other projects in the area it was being achieved. The high base chloride level reduced the predicted service life considerably.

4 WATER TUNNELS

The exposure severity of the inside face of water tunnels depends on what the tunnel carries. Concrete is inherently resistant to fresh water however tunnels with running water can accelerate leaching reducing performance. The commentary to AS 3735 (2001) provides a comprehensive guide to the aggressivity of water.

Seawater intake tunnels are subject to the seawater attack and possibly low pH where dosed to reduce marine growth. CIA Z7/02 (2018) provides two exposure classes for immersed concrete, i.e. for above and below pH 6. The use of a low exposure class (e.g. AS5100.5 B2) in seawater tunnels may be inappropriate. Permanent seawater immersion will lead to high chloride penetration, but it does not lead to rebar corrosion. However, if the tunnel is closed for maintenance years after construction such that the concrete around the reinforcement becomes unsaturated widespread corrosion could commence and once started may not be easily stopped.

Precast segmental tunnels are commonly used for brine outfalls of desalination plants. The chloride and sulfate loading, particularly magnesium sulfate, can be twice that of seawater tunnels and at these levels magnesium sulfate attack has been proposed as a major issue for silica fume and slags. However close inspection of test data shows the concern arises from sample size effects and brines attack concrete at only twice the rate of seawaters very low attack rate.

5 TUNNEL APPROACHES

At some point along the atmospheric tunnel's approach the exposure changes from a tunnel to that of the surrounding atmosphere. Where the tunnel exposure is more severe than that due to the atmosphere at the approach a decision has to be made on the extent to which the tunnel exposure will be spread to the approaches. There is no data to define this but it is recommended that for tunnels where wind spreads contaminants to the approaches that a minimum of 100m be required for approaches to have the same exposure as the tunnels. In the case of carbon dioxide spread may be limited as exhaust fans remove most of the gases. The tunnel portal and all exhaust chambers should be considered part of the tunnel. Where approaches confine the dispersion of fumes the tunnel exposure might be spread to at least 50m from the portal. Data is limited in this regard, and it would be valuable to monitor carbon dioxide levels in and around vehicle tunnels.

6 CONCRETE RESISTANCE

The concrete's resistance to the exposure load is generally determined by the chemistry of the cement system, the penetrability of the concrete and the corrosion resistance of the reinforcement in its environment.

6.1 *Cement System and Exposure Type*

GP cement is the predominant cement used and is suitable for tunnels where exposure is benign, and heat of hydration is not problematic. Supplementary Cementitious Materials (SCM's), typically fly ash, slag and silica fume, provide different beneficial properties when blended with GP cement. Performance of SCM varies from country to country and hence local knowledge of materials and measurement of their performance is critical.

6.2 *Concrete Penetrability*

Three aspects determine the time to penetration of a front that will depassivate reinforcement, i.e. mix design, quality of construction and cover.

On one project the author investigated honeycombing of insitu cast tunnel concrete was a regular event throughout construction. The issue was that the high slump of the concrete mix reverted quickly leaving a thixotropic concrete difficult to place and compact. The cause was a function of the cement system and admixtures. Modern superplasticizer, with slag and silica fume cements particularly, can be a problem if there is insufficient initial slump. Marosszeky (2023) points out how complex achieving a quality concrete is. It is concluded that concrete needs to be considered as a special process with execution plans that are thoroughly assessed.

Quality control also impacts the cover achieved. For example, spacers at the right centres and assurance the cage will not move under fresh concrete loading are key to success. However, there is also a misunderstanding on the variability of cover. AS5100.5 allows for a 5mm negative tolerance on cover. That is a 3mm standard deviation taking minimum cover as a 95% confidence level. On one project, where other concerns led to widespread cover measurements on various segment types the standard deviation was found to be consistently 9-10mm. The nominal cover of 70mm was reduced to an actual minimum cover of 52mm. Attention to achieving and verifying early cover is a lesson learned time after time. A factor to bear in mind in verification is that NDT cover often underestimates cover by 2-3mm. It can be worth calibrating the covermeter against taped cover in drill holes.

Cracks that leak with saline water are not only an issue due to evaporative concentration but also due to the risk of accelerated corrosion at cracks. Papworth (2025/3) notes that rapid corrosion at cracks can occur if cracks exceed a certain threshold width. Contributing factors are high stress in the reinforcement, lack of initial passivation, inability of diffusing hydroxyl ions to counteract acid conditions.

7 DESIGN INPUTS

7.1 *Levels of approximation*

Papworth (2021) notes that fib Model Code (2024) says the design strategy should be specified and that besides design by avoidance of degradation the LoA's can be applied for the verification of the design service life. LoA is introduced to encourage more complex design solutions where practical, in order to give more cost efficient and/or sustainable solutions. Probabilistic methods are likely to provide suitable approaches, particularly for major structures such as tunnels.

Some codes (e.g. AS 5100.5) note that the code may not always provide a safe solution. This is particularly true where the actual exposure does not closely match the code definition, a common issue with tunnels. Hence probabilistic methods which consider a broader range of variables can be used for tunnels to resolve doubt about the deemed-to-satisfy solutions and provide more economic solutions by providing a method of design for higher performance materials.

7.2 *Reliability Based Design*

A significant issue for DtS provisions is that they must be based on a certain reliability (inverse function of probability of failure) over the design life but that reliabity is not stated in the codes. Reliability for code DtS is most likely linked to Serviceability Limit State (SLS) failures, i.e. failure requiring limited cost of repair such as patching. SLS state failures are typically allowed to have a PoF of 5-10%. Reliability is a balance between the consequences of failure and the cost of improved performance. Model Code 2020 notes the validity of ISO 2394 for determination of target reliability,

A consideration for all tunnels is access for inspection and the cost of shutdowns. The project team will have to consider what reliability is required to cater for the specific situation. On one tunnel the original design had no access for inspection. This increased reliability required, and hence performance requirements, significantly as durability failure could lead to collapse without warning. Access was ultimately introduced for cost effectiveness.

Full Probabilistic Analysis (FPA) is undertaken by using distributions for all key variables in a model (equation) that is agreed to represent the actual situation. Using a Monte Carlo type analysis the reliability can be calculated. Use of small variations in distributions for variables like cover and temperature to better represent the actual tunnel gives a predicted reliability that more closely matches the real situation.

At this stage reliability requirements for Ultimate Limit State (ULS) failures, such as prestressing failure, would require much higher covers than currently used. Hence it is currently considered that ULS durability failures should be dealt with by avoidance.

7.3 *Life Cycle Cost Analysis*

The target reliability is selected by the project team for safety, budget, and life cycle cost (LCC) reasons. The durability of tunnels is often of particular importance because of safety and the cost of repair. Direct repair cost may be small compared to public and/or operational costs. The interval for maintenance type repairs comes from FPA with the cost at each interval determining the current cost.

7.4 *Specifying End of Life Requirements*

How target reliability is specified for the element at the end of its design life and for a specific limit state determined by the condition required at the end of service life. Given a tunnel's life may be extended well past the original service life this requires strategic consideration in the design. Reliability, and maintenance and repair costs, go into assessing the whole-of-life cost for different construction options.

8 CONCLUSIONS

In high value complex projects like tunnels safe DtS provisions are unlikely to provide the most economic and sustainable design. Using a higher LoA such as FPA will provide safer and more reliable structures. However, data on tunnel exposure is limited. Better understanding of the key distributions that determine deterioration rates in tunnels would enhance the modelling. It is proposed that relatively cheap monitoring systems be used to monitor new tunnels so that their performance can be assessed with time and to provide better data for new tunnel designs.

9 REFERENCES

AS 3735 Commentary. 2001. Concrete structures for retaining liquids – Commentary. . Standards Australia, Sydney.

AS 4312. 2019. Atmospheric corrosivity zones in Australia. Standards Australia, Sydney.

AS 5100.5. 2024. Bridge Design Part 5: Concrete. Standards Australia, Sydney 2024

Austroads, 2019. Guide to Road Tunnels (AGRT02-19)

Beushausen. H., Ndawula. J., Helland. S., Papworth. F., Linger. L. 2021. Developments in defining exposure classes for durability design and specification. *Fib Structural Concrete,* 2021

CIA Z7/02. 2018. Durability Exposure Classifications. *Concrete Institute of Australia Concrete Durability Series* Sydney 2018

Dewah. H. 2007. Effect of sulfate concentration and associated cation type on concrete deterioration and morphological changes in cement hydrates. *Construction and Building Materials,*21, 29–39.

Edvardsen C.K. 2004. Deterioration modelling – Model Verification Through In-Situ Tests – Great Belt Link (Denmark). *First International Symposium. Safe and Reliable Tunnels.* Prague 2004

Fib Model Code. 2023. Model Code for Concrete Structures (2020). International Federation for Structural Concrete (fib) Switzerland. 2024

Fib Bulletin 34. 2006. Model Code for Service Life Design. Féderation internationale du béton, Lausanne, Switzerland, June 2006.

Fib Bulletin 76. 2015. Benchmarking of deemed-to-satisfy provisions in standards: Durability of reinforced concrete structures exposed to chlorides", Féderation internationale du béton, Lausanne, Switzerland, June 2006.

Jong. H., Khan. I., and Papworth. F. Comparison of Full Probabilistic Modelling and Deemed to Satisfy Requirements for Concrete Carbonation Induced Corrosion. Concrete Institute of Australia, Adelaide, *Concrete 2023*

Marosszeky, M. Papworth, F. and Munn, B. Creating Reliability in Concrete Construction—Achieving Quality Within a Complex Sociotechnical System. *Concrete 2023*, Concrete Institute of Australia Virtual Conference, Australia.

Papworth. F. 2018. Durability design for concrete immersed in seawater or brine. Australasian Corrosion Association. *Corrosion 2018*, Adelaide 2018.

Papworth. F. 2021. fib Model Code 2020 - Condition Limit States and Target Reliabilities for Durability Design. Concrete Institute of Australia, Adelaide, *Concrete 2021*

Papworth. F. 2025/1 Modelling for Improved Durability Design - Boundaries for Marine, Coastal and Inland Saline Waterways Exposure Classes. Concrete Institute of Australia, Adelaide, *Concrete 2025*.

Papworth. F., Andrade. C., Lollini. F. 2025/2 Selection of a Critical Chloride Level for Full Probabilistic Modelling. *Corrosion and Materials Degradation.* 2025

Papworth. F. 2025/3 Forensic Assessment of Cracks. Concrete Institute of Australia. Hybrid web presentation. Melbourne 2025.