

**THE FOX & NGAKAWAU RIVER BRIDGES:
ARE 1980S PRE-STRESSED BEAMS CAPABLE OF FULFILLING THEIR DESIGN
LIFE UNDER AGGRESSIVE MARINE ENVIRONMENTS?**

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SUMMARY

The use of precast pre-stressed concrete beams is recognised as a potential vulnerability of bridges in aggressive environments as the structure ages: Few historic designs comply with current durability practice in regard to concrete cover or quality. Moreover, there are technical challenges associated with both arresting further deterioration and restoring structural capacity, should corrosion develop on the pre-stressing tendons. For this reason, it is essential that the risk of reinforcement corrosion on susceptible bridges is well-understood and actively monitored to permit timely and effective intervention.

As reported to the 2012 Conference, Rhys Rogers identified at-risk State Highway bridges based on construction taxonomy, geographic distribution and NZS 3101 exposure classification, whilst undertaking a PhD at Auckland University. This work, which included physical inspection of representative structures, identified two particular 1980s pre-stressed beam bridges – the Fox River & Ngakawau River Bridges on the West Coast – as being likely to experience systemic corrosion of pre-stressing strand within their 100 year design life.

The current paper describes a subsequent fuller investigation commissioned by the NZ Transport Agency (NZTA) to better understand the current and projected future durability performance of the bridges, plus the desirability and timing of any proactive remediation. The investigation also included sampling over estuarine river channels at locations that potentially represent a more severe exposure environment than those accessed by Rogers.

Modelling of predicted life is presented using a generalised solution to Fick's 2nd law that allows for both a time-dependent diffusion coefficient and surface chloride concentration. The inputs employed were statistical values derived from chloride profiles collected both by Rogers and in the current work, including characteristic data calculated as upper 90% one-sided tolerance limits.

The modelling indicates that concerns regarding the probable residual life of the bridges are valid, although arguably overly-conservative. The predicted lives are found to be heavily dependent upon necessary assumptions regarding the rate of chloride accumulation and, crucially, the critical threshold concentration necessary to initiate reinforcement corrosion. The sensitivity of the prediction to these effectively stochastic variables is examined across their likely range of values, and the nature and timing of possible remedial interventions considered.

INTRODUCTION

The recent NZ Transport Agency Research Report 502 'Assessing Pre-tensioned Reinforcement Corrosion within the NZ Concrete Bridge Stock' by Rogers, Al-Ani & Ingham (2013) identified potential durability concerns with the Fox River and Ngakawau River Bridges on the West Coast of the South Island. For both bridges, located in aggressive marine (i.e. NZS 3101 'C Zone') exposure environments, measurements of chloride ingress profiles in their beams led the authors of the report to conclude they would experience corrosion of pre-stressing strand within their 100 year design life and consequently recommend "urgent action" to achieve this intended longevity.

Given the relative youth of these structures (Fox is approximately 25 years old and Ngakawau 22), their importance to the network, and the limited viable repair options for pre-stressed elements once strand corrosion is established, it was considered important to conduct further inspection and sampling to better define the durability threat to each bridge. In particular, the sampling compiled for Research Report 502 was restricted to terrestrial spans: It was conceivable that spans over the estuarine river channels crossed by these bridges represent a more severe exposure environment where chloride deposition is even higher.

Opus Research was commissioned to undertake this additional sampling on behalf of the NZTA. This paper summarises the results obtained and considers the desirability of any proactive maintenance intervention to extend the life of the structures, based on modelled prediction of future performance. Both the existing and new chloride ingress data-sets are considered.

THE BRIDGES

The Fox River Bridge carries S.H. 6 across the river mouth at Woodpecker Bay in the Buller District of the West Coast at R.P. 374/0.00. The bridge was completed in 1988 and consists of six spans, with four pre-stressed 40 MPa f'c GP concrete I-beams per span supported on cast in-situ hammerhead pier caps founded on a conventionally-reinforced octagonal column. Five of the spans use 20 m beams with both pre- and post-tensioning, while beams in the shorter (16 m) sixth span are pre-tensioned only. Figure 1 shows a general view of the bridge. It is situated in an exposed location on an open surf beach, with waves breaking directly beneath the bridge. Spans 3 through 5 are located wholly above salt water at high tide; span 1 is largely situated at beach level above the high tide mark and span 6 is somewhat sheltered from the prevailing wave direction by a small promontory to the west of the bridge



Figure 1. General view of the Fox River Bridge & environs, looking southward in the direction of increasing R.P.

The Ngakawau River Bridge is located on S.H. 67 between Westport and Karamea at R.P. 30/8.70. It is a five span bridge, constructed in 1983 from precast pre-tensioned 20 metre span U-beams cast from 40 MPa f'c GP concrete. There are 7 beams per span, supported on hammerhead piers with a central conventionally-reinforced concrete stem column.

The bridge is located over saline water, at the river mouth (Figure 2), which opens into a surf beach. However the bridge is set back approximately 150 m from the coastline and the beachfront provides some buffer against waves breaking directly under the beach. Consequently the exposure environment, whilst legitimately an example of a C Zone classification under NZS 3101 is arguably somewhat less severe than that experienced by the Fox River Bridge. The height of the piers over the channel at high tide is also approximately 2 m greater.



Figure 2. General view of the Ngakawau River Bridge & environs, seen from the Westport (south) side of the bridge.

SIGNIFICANCE OF CHOICE OF CORROSION THRESHOLD

A preliminary review of the chloride analyses on which the original unfavourable durability predictions were based suggested that the duration of residual service life was heavily dependent on the subjective choice of corrosion threshold, i.e. the minimum chloride ion concentration necessary to initiate corrosion. Moreover the thresholds chosen by Rogers et al. (2013) of 0.03% & 0.05% by mass of concrete, whilst in line with some published literature values, are relatively conservative and possibly more appropriate for design purposes than assessing the risk of premature failure of a well-constructed modern structure in service.

Chloride ions are unique and specific agents for the corrosion of mild steel reinforcement; in sufficient concentration they destroy the protective surface oxide layer that ordinarily develops on steel surrounded by alkaline concrete. However, the potential for corrosion to occur is influenced by a great many variables, including but not limited to: cement chemistry; concrete quality (cement content, w/c ratio, curing); ambient environment; and the homogeneity of the cement paste phase at the interface with the reinforcement.

Consequently it is generally recognised that any assumption of a deterministic chloride threshold concentration that will invariably initiate corrosion is an over-simplification, and that this variable is better considered in probabilistic terms. This is illustrated by Figure 3, which shows a graphical representation of ingressed chloride vs. steel corrosion risk published by the BRE Centre for Concrete Construction (2000). The broad swathe of the relationship depicted emphasises the uncertainty in the level of risk associated with any particular level of chloride contamination.

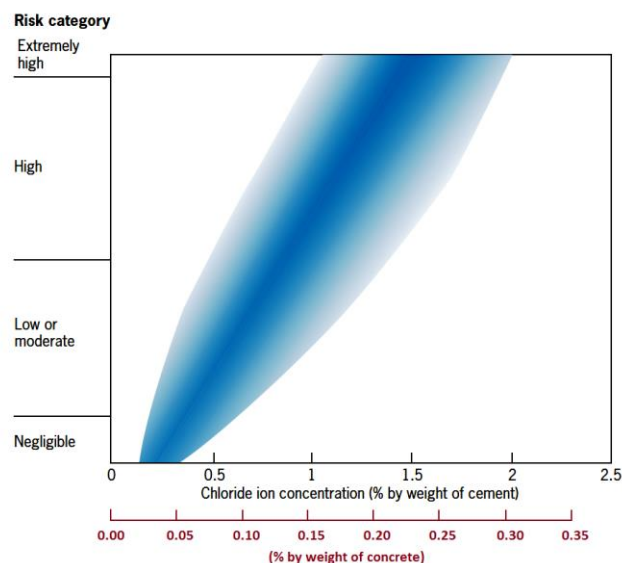


Figure 3. Estimated risk of steel reinforcement corrosion associated with ingressed chloride from the environment (adapted from BRE Digest 444).

Figure 4, adapted from a recent Federal Highway Authority (2012) review article, graphically summarises the maximum and minimum corrosion initiation thresholds reported in the recent literature for corrosion of black steel in concrete. The range of values emphasises that there is considerable subjectivity inherent in selecting a corrosion threshold for service life modelling with chloride profiles collected from existing structures. There is clearly no single 'right' answer.

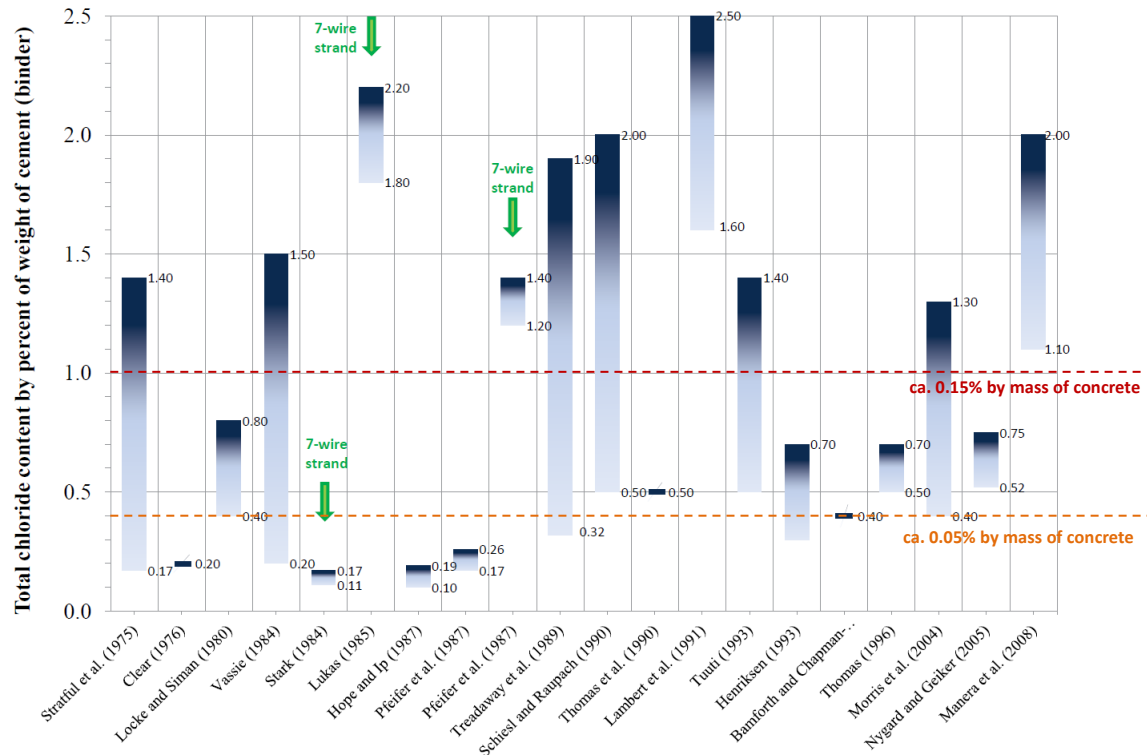


Figure 4. Examples of published threshold concentrations for chloride-induced corrosion of carbon steel reinforcement. Note that the scale is by weight of cement.

Considerations Specific to Pre-Stressing Strand

Corrosion of pre-stressing steel is a greater concern than that of conventional reinforcement due to the possibility that localised reduction in the cross-sectional area of the strand will result in an abrupt failure. The high pre-tensioning stresses also render the strand more vulnerable to stress corrosion cracking and, where the loading is cyclic, to corrosion fatigue.

For these reasons, the maximum permissible chloride content in fresh concrete is lower for casting of pre-stressed elements than that permitted for conventionally-reinforced concrete. Under NZS 3101 these figures are 0.50 kg/m³ and 0.80 kg/m³ respectively, or approximately 0.022% & 0.035% by mass of concrete respectively depending on the mix density. Despite this, one of the few reported studies examining the performance of pre-stressing tendons on exposure to chloride ions found that unstressed strand had a corrosion threshold up to six times greater than conventional black steel reinforcement (Pfeifer et al., 1987). When stressed, the strand was more susceptible to corrosion but still showed markedly better resistance to chloride contamination than conventional mild steel reinforcement. The cause of this unexpectedly good corrosion resistance is believed to be a residual chemical film of rod treatment and wiredrawing lubricants, particularly zinc phosphate and calcium stearate, which remain on the surface of the strand even after high temperature tempering for stress relief (Osborn et al., 2008). However since such residual films are detrimental to bond development they are ideally minimised during manufacturing and their presence and efficacy as corrosion inhibitors cannot be relied upon.

On balance, while the potential consequences of pre-stressing strand corrosion are more severe than those associated with conventional reinforcement, there is no reason to suggest that the strand itself is more susceptible to chloride-induced corrosion under normal service conditions. Therefore for the purposes of analysis, the critical thresholds suggested by the UK Concrete Society (1984) were adopted. This document indicates that chloride contamination of 0.05% by mass of concrete around the reinforcement represents 'some risk' of corrosion, while concentrations in excess of 0.15% constitute a 'high risk'.

SAMPLING UNDERTAKEN

On each bridge, instrumented testing of the pre-tensioned beams in three spans was undertaken to verify reinforcement cover and determine chloride ingress profiles and the extent of carbonation. The spans were chosen to represent a range of possible variation in micro-exposure environment across each bridge and complement the existing test data obtained by Rogers et al. (2013); three beams were selected in each span across the length of each bridge. Table 1 details the overall sampling coverage obtained.

Table 1. Identification of pre-stressed beams sampled on the Fox & Ngakawau River Bridges.

Fox River Bridge			Beam No.			
Span No.	(North) Δ+ R.P. ↓ (South)	1 2 3 4 5 6	downstream	←	upstream	
			4	3	2	1
			◇	◇	◇	
			□	□		
			◇	◇	◇, □	□
			◇	◇	◇	

Ngakawau River Bridge			Beam No.					
Span No.	(North) ↑ Δ+ R.P. (South)	5 4 3 2 1	downstream	←	upstream			
			7	6	5	4	3	2
						□		
			◇		◇			◇
			◇		◇			◇
			□			□		
			◇		◇			◇
								□
			◇		◇			◇

Key: ◇ Beams sampled for this study
□ Beams sampled by Rogers et al. for Research Report 502
(note some beams sampled in multiple locations)

To determine chloride profiles, samples of powdered concrete for measuring chloride content were obtained by drilling 26 mm diameter holes at selected locations and quantitatively collecting the resulting powder. Two holes were drilled at each location and their powders combined; this minimises any distortion due to variable aggregate distribution within the concrete and helps ensure a representative sample is obtained. The holes were initially drilled to collect powder from a nominal depth increment of 15 mm below the concrete surface, with the precise depth depending upon the nature of the element and the expected cover over the reinforcement. The holes are subsequently deepened to allow the separate collection of a further four increments of equal depth to a total sample depth of 75 mm nominally. Obtaining the chloride concentration as a function of depth in this fashion both gives an indication of the likely source of the chlorides and also allows the corrosion risk to be related to the cover distribution of the reinforcement.

The resulting powders were analysed by x-ray fluorescence spectroscopy (XRF) by a specialist IANZ accredited laboratory to express the total chloride content as a percentage of the dry weight of concrete.

CHLORIDE INGRESS MODELLING

To characterise the chloride contamination and estimate future performance, the measured chloride concentration profiles vs. depth were modelled using a least-squares non-linear regression procedure to fit the collected data to Crank's error function solution to Fick's 2nd law of diffusion:

$$C_{(x,t)} = C_i + (C_s - C_i) \left(1 - \operatorname{erf} \left(\frac{x}{2\sqrt{D_c t}} \right) \right) \quad [1]$$

where

$C_{(x,t)}$ is the chloride concentration at depth x and time t ;
 C_s is the notional Surface Chloride Concentration;
 C_i is the initial chloride concentration in the concrete;
 D_c is the Achieved Chloride-Ion Diffusion Coefficient; and
 erf is the Gaussian error function.

The ‘Surface Chloride Concentration’ (C_s) can be thought of as a measure of the force driving the chloride ions into the concrete and reflects the severity of the exposure environment. The ‘Apparent Diffusion Coefficient’ (D_c) represents the concrete’s resistance to penetration, reflecting its intrinsic durability under the conditions of exposure. This technique has been widely adopted by concrete technologists to predict corrosion risk due to chloride penetration (RILEM, 1999), despite being recognised as largely empirical in nature.

Using the derived values for D_c & C_s , it is possible to calculate the future development of the chloride profiles at each sample site, assuming these time-weighted average values are accurate indicators of future performance. This calculation can be extended a step further to estimate the residual life of the concrete structure if two further assumptions are made. These are: (i) that end of life is signalled by the outermost reinforcement beginning to corrode; and (ii) that a specific minimum level of chloride contamination reaching the reinforcement will trigger this corrosion.

To remain mathematically valid, Crank’s equation [1] requires that C_s remains invariant, or at least establishes a constant value very quickly in comparison with the total time of exposure. Recent sampling of experimental long-term exposure specimens suggests this is not necessarily the case in practice, potentially introducing significant errors in life prediction based on such a simple model (Lee & Scott, 2012).

To evaluate this effect a more sophisticated solution developed by Mejlbro (Frederiksen et al., 2009) that allows for both a time-dependent diffusion coefficient and surface chloride concentration was also employed. The surface chloride concentration is explicitly linked to the concrete type and exposure environment by incorporating the value of the chloride ion diffusion coefficient as follows:

$$C_{(x=0,t)} = C_s = S \cdot \{(t - t_{ex})D_{(t)}\}^p \quad [2]$$

Under this alternative boundary condition, the solution to Fick’s 2nd law is:

$$C_{(x,t)} = C_i + (C_s - C_i) \cdot \psi_p(u) \quad [3]$$

where

$$C_s = S_{ref} \left(\frac{t}{t_{ref}} \right)^{(1-\alpha)p} \quad [4] \quad \& \quad u = \frac{x}{2 \sqrt{t \cdot D_{av, \psi(t_{ref})} \cdot \left(\frac{t}{t_{ref}} \right)^{-\alpha}}} \quad [5] \quad \text{for } t, t_{ref} \gg t_{ex}$$

Consequently the chloride accumulation in the concrete $C_{(x,t)}$ at any depth x and time t can be completely specified providing four experimentally determined parameters are known: α , p , $D_{av, \psi(t_{ref})}$ & S_{ref} . The indices α and p describe the rate of change with time of the average diffusion coefficient and the surface chloride concentration respectively and the other two parameters are point-wise measures of these properties after some reference period of exposure. Note that while D_{av} is mathematically related to the value of D_c derived from fitting Crank’s equation to the measured chloride profiles they are not numerically equal due to the removal of an assumption of a steady driving force created by a constant surface chloride load.

Derivation of Inputs for Modelling

The most critical sampling locations could not be determined absolutely by inspection because the beams are not currently showing any evidence of distress. Therefore the only reasonable approach to a robust risk assessment is sufficient testing to permit a statistical evaluation of achieved diffusion coefficients and surface chloride concentrations. Characteristic, in addition to mean values, of these parameters were adopted for subsequent modelling with appropriate tolerance limits to avoid over-estimation of durability performance.

In recognition that pre-stressed concrete elements are very difficult to maintain once corrosion begins to propagate, the appropriate characteristic value was selected as the 90% upper one-sided tolerance limit for both D_c & C_s at the 90% confidence level (NIST/SEMATECH, 2012): Assuming a normal distribution, 90% of the beams are expected to display better durability performance than the case represented by this characteristic value.

Table 2 shows the statistical values for D_c & C_s adopted for modelling the projected longevity of the pre-stressed beams on each bridge. These values derive from analysis of both the latest Opus sampling and the earlier chloride profiles recorded by Rogers et al. (2013); Figure 5 shows the complete Fox River Bridge data set represented graphically, with annotated bounds representing the upper 90% tolerance limits.

Table 2. Summary of chloride diffusion parameters measured on the pre-stressed beams of the Fox & Ngakawau River Bridges, including data from Rogers et. al (2013).

Statistic	Apparent Diffusion Coefficient, D_c (mm ² /year)		Surface Chloride Concentration, C_s (%w/w on concrete)	
	Fox	Ngakawau	Fox	Ngakawau
Mean	7.0	45.0	0.131	0.069
Standard deviation	3.7	29.0	0.061	0.040
Characteristic Value 1-sided upper tolerance limit; 90% of population at 90% certainty	13.7	98.2	0.241	0.143

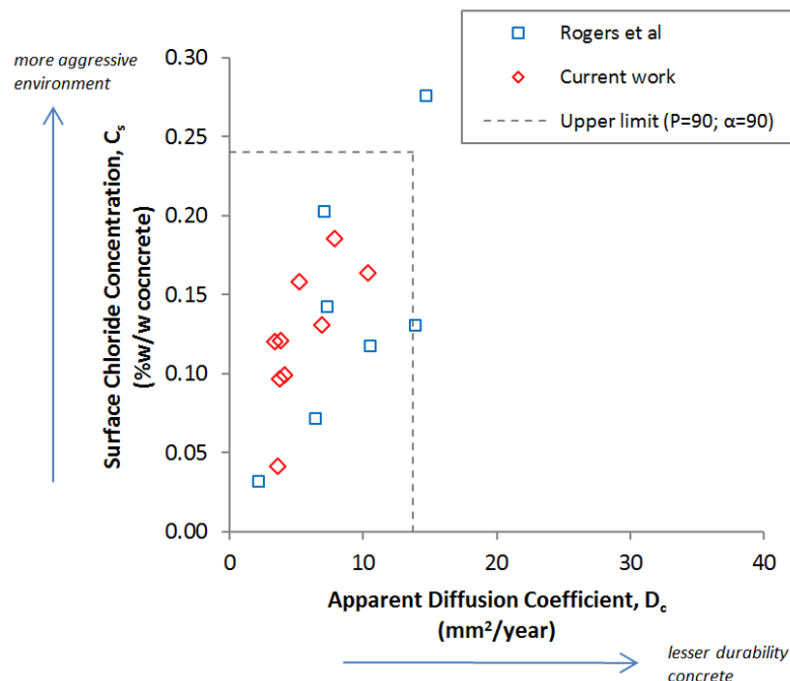


Figure 5. Graphical plot of all individual chloride diffusion measurements undertaken on the Fox River Bridge pre-stressed beams, including data from Rogers et al. (2013). The dashed values indicate the bounds used to obtain characteristic values for life prediction modelling.

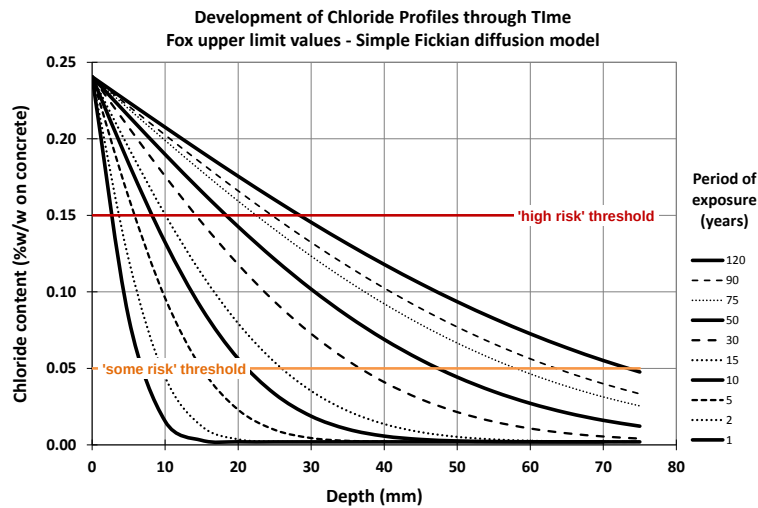
ANALYSIS & RESULTS

The results of modelling various scenarios for the example of the Fox River Bridge beams are presented in Figure 6, which shows the projected development of the chloride contamination vs. depth profiles through the beams at a series of different exposure periods (age).

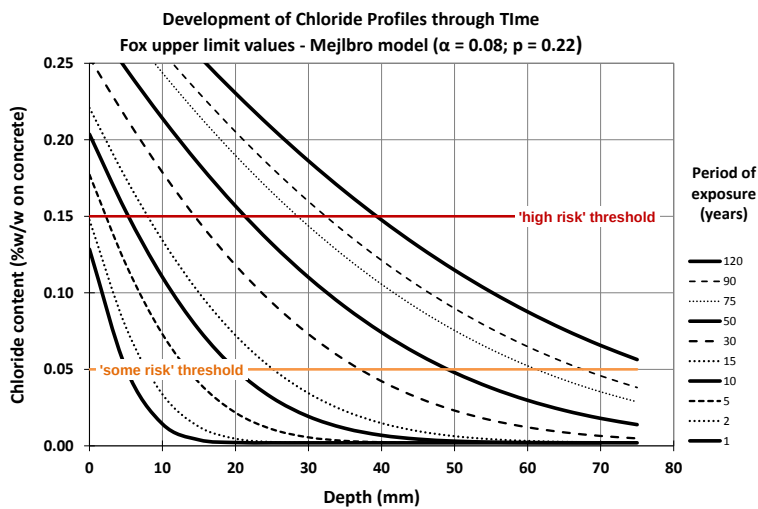
The first plot (i) in Figure 6 employs the characteristic upper bound values calculated for D_c & C_s in conjunction with the simple Crank's equation solution, in which both the diffusion coefficient and surface chloride concentration are assumed to be invariant with time.

The second plot (ii) repeats this calculation using the more generalised Mejlbro model, which permits a time-dependent reduction in diffusivity and the gradual accumulation of surface-deposited chlorides. Because the rate of change indices α & p cannot be determined from chloride profiles collected at just a single age, accurate values appropriate for the Fox River Bridge concrete and its exposure environment are unknown. The choice of these parameters is therefore somewhat subjective and while the Mejlbro model is more realistic than Crank's solution it is not necessarily more accurate given this limitation. Under this scenario, the hypothetical time to corrosion initiation is typically reduced at any given cover depth because the improvement in concrete diffusivity through time does not quite compensate for the increased driving force created for diffusion as the surface chlorides accumulate.

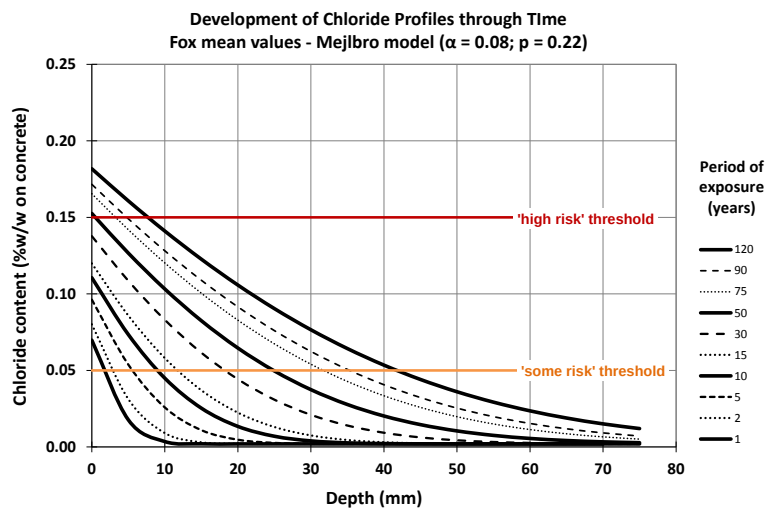
Finally in the third plot (iii), the Mejlbro model is re-run with the average values of D_c & C_s as inputs. In this case, the predicted residual life before corrosion initiation increases significantly relative to the other two scenarios, due to the removal of any statistical uncertainty associated with these values.



(i)



(ii)



(iii)

Figure 6. Hypothetical future development of chloride contamination in the Fox River Bridge pre-stressed beams considering various scenarios: (i) a conventional Fick's law model using upper bound characteristic values for an invariant D_c & C_s ; (ii) upper bound values for D_c & C_s which are allowed to vary with time; and (iii) a time-dependent model employing mean rather than characteristic values.

Figure 7 & Figure 8 summarise the results for both bridges for what proved to be the most pessimistic scenarios, i.e. use of the Mejlbro solution with upper bound values for D_c & C_s . The plots show the chloride accumulation through time at specific depths pertinent to the reinforcement cover distribution (nominally 40 mm design cover to the pre-stressing strand for both bridges).

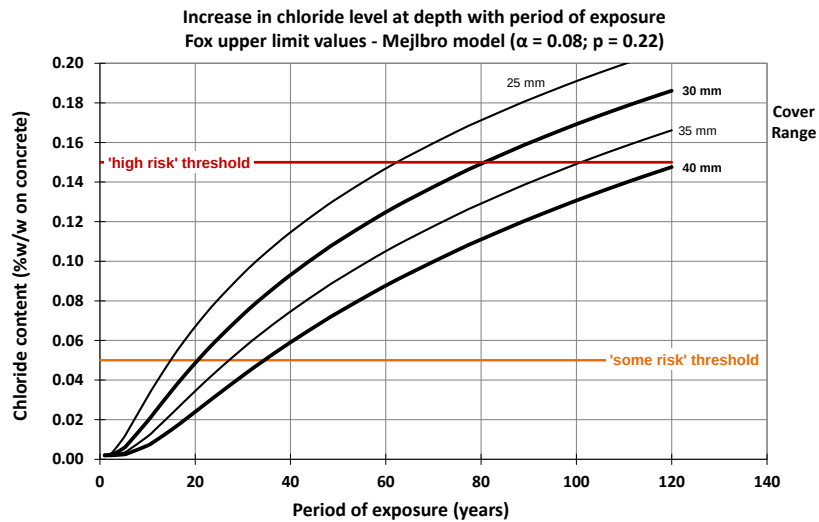


Figure 7. Modelled evolution of chloride ingress with time in the Fox bridge beams, based on statistical upper bound values for diffusivity & surface chloride accumulation.

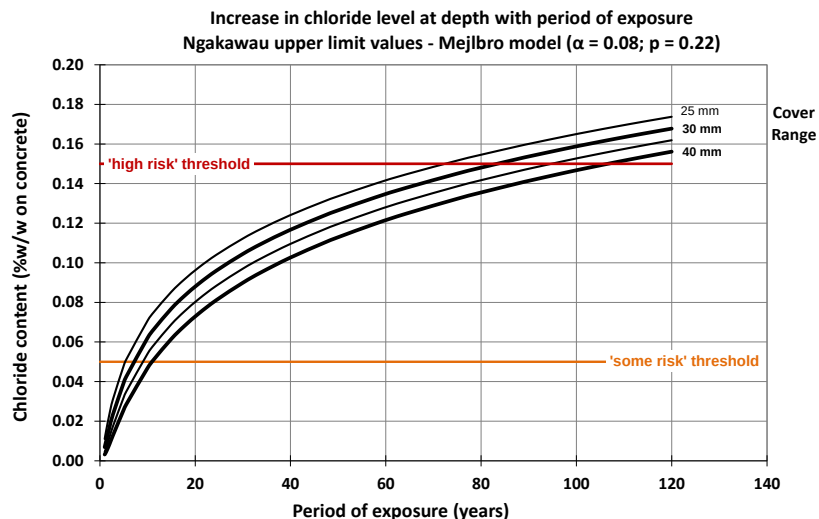


Figure 8. Modelled evolution of chloride ingress with time in the Ngakawau bridge beams, based on statistical upper bound values for diffusivity & surface chloride accumulation.

It is apparent that the estimated residual life to corrosion initiation is very dependent on the supposed value of the corrosion threshold. The inference of a premature threat of pre-stressing corrosion is entirely valid for the critical corrosion threshold of 0.05% by mass of concrete assumed by Rogers et al. (2013) in their analysis. However, as a practical limit state denoting the end of the serviceable life of a pre-stressed concrete member, we believe a critical threshold of 0.05% is possibly too conservative, even allowing for the limited remediation options available once corrosion commences.

In Opus' extensive experience carrying out condition inspections of deteriorating bridge structures, it is rare to observe significant propagation of corrosion at such low chloride concentrations in practice. In precast elements, the reinforcement should also be well-encapsulated in cement-rich concrete with abundant alkalis, which also helps safeguard the critical chloride:hydroxide ion ratios.

For this reason, we believe that modelling of deterioration with a 0.10% threshold is a pragmatic choice that better reflects the likely in-service performance of the bridge beams. Assuming this threshold, the results obtained suggest that the most probable residual life before corrosion-induced spalling could be expected to develop in the poorest performing beams is a minimum of 47 years for Fox and 22 years for Ngakawau. If mean rather than characteristic durability performance values are modelled, the predicted residual life for both bridges extends beyond 100 years before corrosion of the critical pre-stressing strands becomes pervasive. By this point, the structural capacity of the beams would potentially be gravely compromised however.

CONCLUSIONS & CONSIDERATION OF POSSIBLE REMEDIAL MEASURES

This investigation has expanded the available data pertaining to the current and projected future durability performance of the bridge beams on the Fox and Ngakawau River Bridges. It has also served to alleviate any concern that the previously unsampled bridge spans across the river channels may be subject to appreciably more severe exposure conditions. Modelling of predicted life has been undertaken using statistical values for achieved diffusion coefficients and surface chloride concentrations, including characteristic figures based on upper 90% one-sided tolerance limits, derived from all of the available chloride profiles.

Given a minimum residual life of at least 22 years for the poorest performing beams, and a reasonable expectation of a substantially greater longevity, the warning given by Rogers et al. (2013) that "*Urgent action is required to arrest imminent or existing pre-tensioned reinforcement corrosion in these bridges*" is believed to be somewhat overstated.

Accordingly a strategy of 'watchful waiting' (i.e. taking no immediate remedial action) was recommended for the bridge beams to the NZ Transport Agency. However, cognisant of their worst-case residual life predictions and that their durability design would be considered deficient under the current NZS 3101 "Concrete Structures Standard", the beams should be re-sampling in 10 years' time to confirm ongoing satisfactory performance (or sooner if routine service inspections detect any evidence of corrosion in the intervening period). A strategy of watchful waiting avoids any upfront maintenance cost whilst still allowing sufficient time for intervention if unanticipated problems develop.

Application of a silane-based hydrophobic impregnator prior to the onset of corrosion has become a common practice overseas for attempting to optimise the life of deteriorating reinforced concrete bridges. These compounds produce a hydrophobic (water repellent) silicone lining on the capillary pores within the concrete, preventing chloride-contaminated moisture from being drawn inwards and retarding the rate of ingress towards the reinforcement. The silane impregnant does not form a film across the concrete surface and conceal the development of future cracking or spalling and nor does it require any commitment to on-going maintenance. For this reason, it was considered the only surface treatment for which a reasonable engineering justification exists for application to the bridge beams.

A review of best practice for the surface treatment of concrete bridges (Freitag & Bruce, 2010) indicates that because silanes need to be applied regularly throughout the life of the structure they will be most cost-effective if applied when corrosion is imminent but no damage has yet occurred (i.e. late in the initiation phase, just prior to the onset of corrosion).

On the basis of the projected future development of the chloride profiles, it was believed to be too early in the lifecycle of the bridges for a silane treatment to be justified. There are also some concerns with the effectiveness of the treatment on dense precast concrete due to problems achieving adequate penetration depths. However, silane application should certainly be considered if resampling of beams in the future shows significant progression of the chloride fronts.

Migrating corrosion inhibitors are surface-applied chemical treatments that directly interfere with the corrosion process either by encouraging the reformation of passivating films around the steel or immobilising the corrosive species and preventing them migrating to the reinforcement. It is difficult to provide a definitive statement regarding the cost-benefit of their application to the bridges because of an absence of hard evidence for their effectiveness and longevity. Nevertheless given the risk associated with corrosion of pre-stressing strand, corrosion inhibitors may have some utility in the future management of the bridges in conjunction with a silane treatment. Evidence indicates they are most effective in low to moderately chloride contaminated concrete. It was suggested to the NZTA that the potential benefit from their application should be reviewed following the recommended re-sampling in 2022.

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