Hay St Perth Art Deco Heritage; Carbonation And Chloride Observations; Restoration And Adaptive Re-use

Anthea L. Airey[1]
[1]Expert Scientist of Airey Taylor Consulting

Abstract: Art Deco architecture of Hay St Perth includes the Bank of NSW Building (1935), Dynon's China Hall Building (1920) and Walsh's Building (1921). Façadism is not always a commercial (or arguably desirable) retrofit option; yet glamorous ground floor tenancies may belie deterioration of the levels above.
Assessing condition is key to alternative remedial strategies. At Walsh's Building, an interior and façade concrete survey (22 cores) found 0-100% carbonation present with influences from render and coating type, orientation, consolidation method (voids) and exposure for any given strength and age. Agreement was found at Walsh's Building and Bank of NSW Building on the minimum corrosion threshold requirements; at the latter a spalled dentil piece featured red rust; examined in two through-sections 0.01% detectable chloride at rebar level was found with 100% carbonation throughout. At Walsh's Building a uniform minimum concentration of 0.01% chloride was found throughout all samples supporting a high initial calcium chloride accelerator dose; 100% carbonated areas subjected to leaks were associated with rusting. The results support acidification by carbonation to be key in reinforcement corrosion initiation. Carbonation from the underside of horizontal surfaces, buffering by lime renders (many still only partially carbonated) and retardation of chloride by render were noteworthy.
Strategies allowing continuous ground floor occupancy are critical enablers to finance restoration works. Some innovative remedial strategies are reported, revitalising and preserving heritage in the most practical way by returning the property to a fully lettable state on all structural levels.

Keywords: Heritage, later-age carbonation, chloride, corrosion threshold, durability, restoration, re-use.

1. Introduction

1.1 Typical Approaches to Art Deco Structures

It is reported that the vacancy rate of heritage[1] structures above the retail (ground) level of the Perth CBD is 42% (1). In smaller structures, this is reported due to retrofit concerns with regard to access with staircases and escalators consuming valuable square meterage of retail which supplies 5 x the rental potential of office space (Table 1). Similar ratios exist in other capitals.

Table 1: Rental Returns before Outgoings, Perth CBD (refs 2a, 2b)

Year	Prime Retail Hay St Mall Precinct $AUD/m^2/annum	Average Retail Hay St Mall Precinct $AUD/m^2/annum	Prime Office (St George's Tce, Central Location) $AUD/m^2/annum	Peripheral Office CBD $AUD/m^2/annum
2013	Good: 1500-2000	800-1250	700-850	400-500
2019	Good: 1500-2000	800-1250	550-775	250-325

Additional concerns are the structural durability of the upper levels to the satisfaction of renovating Engineers and Council. With new concrete very cheaply installed[2] at \$274.60/m^3 and the cost of remediating concrete varying from \$500-1000/m^2 (electro-osmotic treatments)-\$2400/m^2 (traditional repairs)[3] a typical approach to remedying concrete beyond its design life of this era has been façadism. Noteworthy heritage structures flanking the Hay St Mall area incorporating skyscrapers include 108 St George's Tce Bankwest Tower and the recently completed Treasury Building. Use of

[1] The term heritage is used throughout without necessarily referring to its listing status.
[2] A reinforced suspended slab 150 mm thick = \$40/m^2 in Perth, WA (ref 2c)
[3] Dependent on complexity, access costs and inclusions such as galvanic anodes (3).

the outer leaf only of the façade as a folly (with re-exposure of previously indoor concrete) has occurred in the CBD precinct of the neighbouring City of Fremantle.

This paper discusses concrete condition risk assessment, mechanisms and targeted remediation to design out the requirement to substantially demolish concrete structures, bringing the structures from dereliction or C-grade office lets to premium rental condition.

1.2 Concrete Deterioration and Design Life

In Australia of the 1800's construction predominantly involved non-concrete structures of brick and timber.[4] Incorporation of concrete as a building material becoming widespread use in the slabs and facades of the commercial structures by the 1920's. By circa 1950's there was widespread progressive and recurrent steel reinforcement corrosion now better understood through years of study; with rapid (exponential) escalation of costs once so called "concrete cancer" has commenced. This is a corollary of the limit states of concrete damage due to corrosion outlined originally postulated by Tuutti (4) involving time to depassivation[5] (t_0), corrosion propagation (t_1) followed by severe damage (t_2). These costs are now becoming more clearly linked to the cement matrix properties and reinforcement selection in durability models such as Life-365 (5), together with the understanding that serviceability issues such as spalling are a significant concern.

1.3 Carbonation Deterioration Mechanism and Models

Exposure of freshly cast concrete to atmospheric carbon dioxide results in a carbonate reaction product deposited as a front perpendicular to the surface, progressing a distance d in millimetres with the square roof of time t in years (equation 1) and can be viewed by spraying with a pH sensitive dye phenolpthalein changing from pink (high pH ~12-14, uncarbonated) to colourless ("low" pH ~ 9.5) (11). Steel depassivation that can result from this pH drop is one of the three primary causes of reinforcement corrosion (7).

$$d = k\sqrt{t} \qquad (1)$$

To explain variation in the coefficient "k", relative humidity (RH) has been found a key contributor to the fixation of carbonate (16-19). Sufficient humidity, usually in the range 50-95%, is required; in buildings this occurs most rapidly at the drier indoor locations. Pore water condensation is proposed to allow fixation via bicarbonate intermediate (18, 19)[6]. Submerged concrete reactions involving leaching or depositions of calcium carbonate are pH and calcium ion concentration dependent with risks to concrete described by the Langelier Saturation index (6).

Lagerblad (19) researched values for k Equation (1) in field hardened concrete, categorising the results in terms of exposure zones (wet/submerged<buried<exposed<sheltered<indoors) and strength classes (<15 MPa, 15-20 MPa, 25-35 MPa, >35 MPa). In 2017, Wang's temperature term was found to be an over-correction for the Lagerblad rates, applied in a warmer Australian context (15). Micro-climate zones, within the same climatic region, have a more significant effect upon rates of orders of magnitude (indoor concrete 3.5-4.5 x the rate of exposed 25->35 MPa concrete (19).

State of the Art models in aerated concrete are still undergoing development to understand and predict k, with Federation International du Beton (fib) "Bulletin 34 method" rate incorporating surface carbon dioxide concentration, an RH term from an accelerated test under high atmospheric pressure, with a correction factor required to resolve the test method's bias towards permeability (rather than diffusion) and incorporating a "weather function" including rain days that wash off surface adsorbed

[4] The evolution of suspended wood floors to slab on grade is discussed in reference (13).
incorporating BRE (UK) results, a study in South Germany, and a 110 sample CBI study in Sweden
The evolution of suspended wood floors to slab on grade is discussed in reference X.
[5] Tuutti originally described this as corrosion initiation, a term subsequently made more specific by others.
[6] Noting it is difficult to see how this maximises at a steady indoor condition of circa 55-60% RH.

carbon dioxide (17). The difference in partial pressures created by climate averages and CO_2 pollution between the cities is likely to be smaller than some other of the abovementioned microclimate effects.

An alternative TR61 CARBUFF is an empirical model based upon a normalised correlation of long term exposed concrete core test results worldwide. This model, aimed at predicting the durability result of a mix design,[7] assumes the concrete is effectively immersed in an unlimited supply of carbon dioxide from the atmosphere and that the reaction rate relies upon cement chemistry, and resulting porosity from curing technique. This approach better reflects the global carbon cycle where concrete "sequesters" carbon dioxide (21), returning it to limestone, its most stable thermodynamic form; conceptually a "pull" model. Micro-climate zones are not explicitly explored using this calculation.

For the site engineer, carbonation has already occurred, and providing k has been calculated from equation (1), and the concrete will be in the same exposure condition in the future, the remaining design life t_r (years) can be simply calculated from the cover distance d_{cover} less the carbonation front distance d_{CO3} , in millimetres (equation 2).

$$t_r = \frac{(d_{cover} - d_{CO3})^2}{k^2} \tag{2}$$

1.4 Chloride Attack

For CBD buildings of Australian coastal cities the usual source of chloride is atmospherically deposited sea salt (14). Chloride acts to solubilise the surface of reinforcement passive in high pH concrete, which on re-oxidation in water forms an outer crust of expansive rust, releasing the chloride ion to repeat this process catalytically. An increase in salt concentration enhances the conductivity of pore solutions leading to a low resistivity environment where deteriorating currents flow more readily. Ultimately liquid water is required to transport chloride and bring about the rusting process.

In order to commence depassivation, chloride must first migrate to reinforcement depth through a distance of concrete termed "cover". A model relating to the accumulation of chloride at reinforcement is the second derivative of concentration with respect to distance known as Fick's Second Law of diffusion. Given a gradient created by sampling sections perpendicular to the face of concrete exposed (assumed to be the direction of chloride ingress) and test results from crushing, acid digest and titration (12), a chloride profile may be calculated and diffusivity D, a property of the concrete, determined using numerical methods (essentially a sequential series of determinations where the outputs of one estimate form the inputs of the next). The validity of the assumptions requires assessment during the work-up of data. Sensitivities in the methodology include boundary condition assumptions Cs assumption (the surface concentration of chloride) and Co (the original chloride baseline).[8]

It has been reported by Professor Robert Melchers of University of NSW that the probabilistic threshold of corrosion concept is severely flawed in maritime structures where the onset of corrosion appeared related more strongly to the pH drop caused by the leaching out of calcium hydroxide by seawater (22). This could occur on a similar or dissimilar (much accelerated) rate to carbonation depending upon the sorptivity of the concrete, extent of immersion, repetitiveness of the leaching.

This discussion is consistent with earlier work by this author on car parks exposed only to the atmosphere or possibly occasional salt spray; that if caused by chloride, the threshold of corrosion was as low as 0.02% by weight of concrete in fully carbonated concrete (or alternatively, initiated by carbonation as the primary mechanism); whereas that purely associated with chloride front was closer

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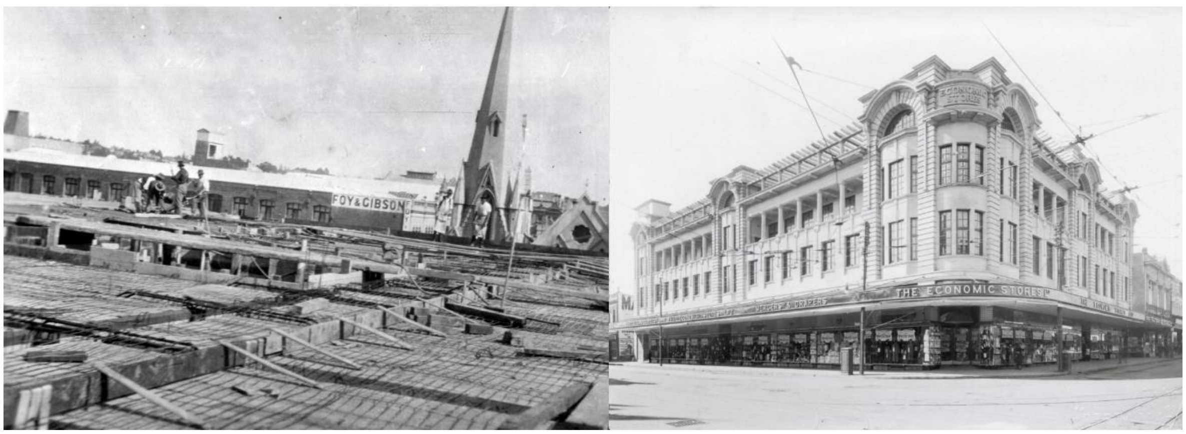
[7] The four inputs are "buffer capacity" compared to Portland Cement (=1), C3A content of the cement, equivalent period of wet curing and mean relative humidity of the environment governs the result.
[8] This is the approach most commonly used to predict the durability of hardened concrete with respect to chloride attack, reference 18 outlines the equations and work up method used herein.

to 0.04% by weight of concrete (14). RILEM have recently completed a review, favouring that apparent carbonation-induced corrosion is due to the increase of Cl/OH^- ratio, and release of previously bound chloride, upon carbonation and this effect can precede arrival of the carbonation front (23), also observed by us (14).

The fib's recommendation for the probabilistic corrosion threshold is 0.6% by weight of cement; with lowest threshold of 0.2% and higher of 2.0% (18). A significant non-environmental source of chloride is calcium chloride is used as an accelerator to ensure the cement reaction commences. AS 1379 stipulates a maximum chloride content of 0.8 kg/m^3 (circa 0.03% by weight of concrete) from this cause (8).[9]

The 2017 forensic analysis of a potential spall condition (case study 2) agreed with key findings of a more extensive study of 2010-2011 (case study 1) prompting this report.

2. Case Study 1 – 726 Hay St



(a) Layup of reinforcing mesh over beams 1921
State Library Image Number 007188d

(b) Re-constructed Economics Store, 1923
State Library Image Number 007183d

Figure 1. Re-construction of the Economics Store, 726 Hay St Perth

2.1 History and Objectives

In 1920 a fire in Perth occurred destroying the previous Economic (department) Store at 726 Hay St and by 1921 a new Economic Store was under construction according to an inter-war Art Deco design by Architect Talbot Hobbs with Lead Contractor C. W. Arnott (24). It was further renovated according to a 1961 architectural design by Oldham Boas Ednie Brown and Partners to add an extra level housing an indoor tea room at the roof, and new escalators.[10]

The construction of 5 levels plus a basement was assessed by Airey Taylor Consulting (ATC) in 2004 as part of the Metrorail City project identifying it to be a primarily steel-framed structure encased in concrete, with secondary reinforced concrete beams with reinforced concrete floors (25). This and other studies by ATC found that in circa 1961 a new slab at level 4 (roof) had been suspended over the old roof slab upon which to establish the tea rooms and new concrete roof structure upon which a plant room was built at level 5. Over time, leaks at the escalator skylight allowed water to move within a double slab arrangement on level 3 roof/level 4 slab from the extensions, down the restraint cracks soaking through the slabs and thence into beams and columns at levels 3, 2 and 1. Thus, ordinarily indoor concrete had been subject to a number of years of weathering. Particularly at level 3 this resulted in delamination of the underside of the beams (Figure 3 (c)).

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[9] Minimisation of chloride further occurs in "performance" concretes such as "B80" concretes (9).

[10] This latter renovation not reported in popular posts such as Wikipedia.

At the time of commencing detailed condition inspections it was determined that the internal suspended slabs all suffered from a regular series of long linear cracks at the location of the underlying major beams and columns. This is a defect of a slab cast at a later date than the beams due to restraint by the beam during shrinkage of the slab. The beams themselves exhibited linear cracking due to this.

At the time we surveyed the building in 2009 only the ground level was tenanted, in similarity to the host of properties referred to in reference (1). It was intended by a new owner in 2009 to renovate the structure back into a combined retail commercial building with parking in the basement level. A progressive examination occurred of the structural elements 2010-2011 as follows.

2.2 Slab Assessments

A summary of all slab results are presented in Table 2. The cores extracted were photographed, sprayed on site with phenolphthalein solution, further photographed and logged for observations. "Bluemetal" local escarpment quarried granite aggregate was employed in all cores, of maximum 40 mm in 1961 and 30 mm in the 1921 roof and floor samples, 40 mm in columns, with large aggregate pieces noted in the balcony 3 slab of 65 mm. Cores 1-6, 8-10 were further independently split and tested at SGS laboratory for carbonation testing (11) and chloride profile (12).

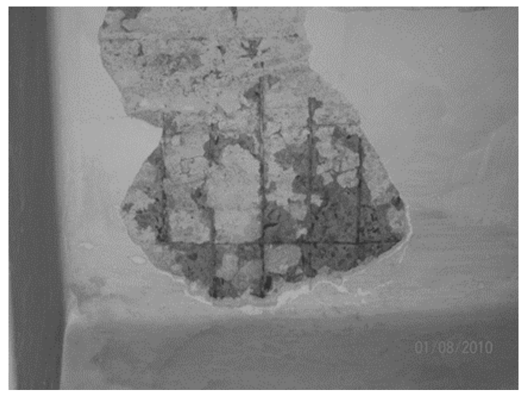


Figure 2. Reinforcing Mesh Level 2 ceiling

The 1921 slabs contained a rectangular mesh with a lower diameter wire found 2.8 mm on the soffit side welded to the top 4.52 mm (core 2) which appeared galvanized. Checks in less damaged areas confirmed a galvanised mesh similar to "Clinton Electrically Wired Fabric" (refer Figure 1 (a), Figure 2) which is possibly the earliest usage of this material in Australia (26). A dissimilar, yet to be identified helical wire mesh was found in the level 3 balcony soffit (core 9).

2.2.1 Slab Carbonation Lifetime Projection

The core survey revealed the 1961 (49 years of age) reinforcement of the level 5 plant room floor slab protected by a roof to be in a shiny passive state by observation in carbonated concrete from the soffit side with the front adjacent from the top side. The carbonation coefficient of 5.7 mm/√year (soffit side), significantly greater than 1.6 mm/√year from the top side; the latter adjusted to 4.5 mm mm/√year if the painted topping is considered part of the cover consistent indoor concretes. A low background of chloride at the limit of detection at 0.01% was present as would be expected in an enclosed structure. This concrete had been cast directly onto wood bonded to a bitumen layer, both retrieved during coring. Although in good condition, the concrete was formally at the end of its design life at the design age due to carbonation.

The original roof at level 3 (89 years of age) was understood to have had this same wood floor over a layer of bitumen, on its top side (probably the deck of the original roof garden upon which the level 4 slab was cast). In other areas this upper slab was missing and the 1921 slab exposed directly to the elements with a waterproofing layer of bitumen.

A partially carbonated concrete roof sample also featured unequal carbonation and had rates of circa half that of the 1961 concrete. Of interest was a sample beneath a layer of bitumen and the central wood floor where the topside carbonation (30.5 mm, k = 3.2 mm√year) was almost double the concrete from the soffit side (19 mm, k = 2.0 mm/√year). The carbonation coefficients for that partially carbonated slab sample appear in reasonable agreement with Lagerblad's data (22) on "sheltered" circa 25-35 MPa concrete (32 MPa found on site). The difference between "sheltered" and "indoor" concrete would largely be one of relative humidity.

Concrete samples that had fully carbonated included exposed slab locations (k = 6.6-7.4 mm√year) and the kitchen (k = 7.7 mm√year). The kitchen had the bitumen membrane common to other areas, was further sheltered by the wood floor above, however did experience leaks. A high carbonation coefficient was also noted for the partially carbonated level 3 suspended slab (k=8.5 mm/√year, soffit side). If the adhered fully carbonated toppings are included in the calculation as viable cover, the k-constants are further boosted to a range 8.2-10.4 mm√year.

When concrete has fully carbonated the k-constant calculated is the minimum, hence results may be variable purely due to this. Nevertheless groupings can be seen. A very significant rate of carbonation must have been present on both the level 3 floor and roof in order for rates to have reached 7-10 mm√year or greater (full penetration > 70 mm).

A possible contributor is the bitumen membrane. Support for this hypothesis is shown in Sample 3 where the progress of the front adhered *via* a topping to bitumen, is double that from the soffit side. This is suggestive that the bitumen component was contributing to the front. Bitumen is carbon rich and if applied in any manner involving heat can presumably oxidise the organic components, producing carbon dioxide in addition to other gases. Additionally, the bitumen cores were odorous at time of sampling indicating decomposition.

The level 3 suspended floor slab was indoor and did not have a bitumen membrane present and exhibited the reverse, and more expected[11] trend with concrete carbonating more rapidly from the soffit side with a small excluded un-carbonated zone nearer the top side. The rate of carbonation was still substantial, but from the underside. Such high rates are broadly applicable to lower grade hardened concrete (15-20 MPa 9 mm√year), nevertheless the sample beneath the wood floor indicates an interesting possible mechanism.

Cementitious toppings within the building at roof level and at level 5 had fully carbonated. A protective sealing effect of bitumen was present on the balconies. Where the bitumen coating was present, only localised carbonation to full depth had occurred at bitumen defects, compared to the general lack of carbonation where the bitumen was adhered and undamaged. As such, un-carbonated concrete beneath the topping may be due to a protective effect of the bitumen, rather than the topping. This suggests that decomposition of the bitumen at level 3 roof may have influenced the top side carbonation, rather than the initial hot application.

The slabs were reinforced with a mesh which was on the soffit side at a depth of approximately 10 mm (varying 0-20 mm). To this was adhered a thick 10 mm plaster (Figure 2). To investigators surprise a layer of sand was found in and above the plaster adhered to the concrete suggesting the concrete may have been cast with a fine blinding layer on a plaster already prepared on the formwork. The low cover approach may have been used as a shrinkage management measure of the concrete surface or possibly intended as a combined plaster reinforcement in addition to the concrete reinforcement. The Battye Library photo (Figure 1(a)) confirms this low cover. At time of inspection, areas not affected by water the plaster was still adhered and reinforcement was found to be quite shiny (passive) or grey consistent with galvanising.

What was interesting in the roof samples was that despite other materials bonded to the concrete it had relentlessly carbonated from both sides. As noted in Table 2 some intra-surface carbonation suggests the bond layer to a cementitious topping has a sufficiently high permeability to allow a carbonation path through a composite. From the soffit side, carbonation had occurred, and to a

[11] Horizontal surfaces have more rapid carbonation rates which is attribute to these being still zones where CO_2 can accumulate.

greater degree of intrusion through fissures (voids). Table 2 reports the main front, in addition to the depth of carbonation in the fissures recorded as notes.

For most concretes the remaining life to the soffit reinforcement was zero. For the roof and lower slab concretes all the reinforcement observed had been carbonated many years prior and was stable; it appeared to required leaks to supply sufficient liquid water to initiate corrosion. At the balconies, some well embedded carbonated reinforcement had commenced rusting alongside a minute void (sample 10) yet others appeared passive, all at a low background of chloride and in minor voids that appeared to arise due to construction method. This indicated a fair degree of passivity unless wet. Given these circumstances the only cores that had "mid-range" projections of interest were the balcony 1 and balcony 3 slabs. Balcony 1 had approximately 80 years life left in its top reinforcement, whereas balcony 3 had less than 1 years predicted for both reinforcements.

The sample 10 exampled found in other members that voids due to consolidation quite often ran around the actual reinforcement itself, posing more of a hazard than outlined by these formalised projections from typical matrix character and reinforcement location.

2.2.2 Slab Chloride Lifetime Projections

Chloride diffusion calculations were performed and are reported in Table 2. As there were no un-carbonated slabs at reinforcement depth, a time to corrosion due to chloride attack alone cannot be evaluated, as it is combined with carbonation (the exception was a sample observed on the façade).

Selection of testing increments was conducted to allow evaluation of C at reinforcement depths in the present day. To establish a diffusion curve three-four sections of each core were tested inclusive of the bonded cementitious toppings. The boundary condition $C(x, t=0) = C_0$ by means of a deeply located section likely outside the range of chloride ingress was found to be 0.01% for all cores. The surface concentration assumed a steady state in equilibrium established by the render with the environment, feeding chloride into the sample, Cs at top of concrete was estimated by fitting the data points with an excel exponential function extrapolated to the y-intercept (Top of concrete X=0); the data point from the topping was included in the curve at negative X-abscissa for this exercise. For cores 2 and 3, the topping layer was of a lower chloride concentration, than the concrete. This is quite curious, and could not be explained by converting to kg/m^3 chloride with assumed density 1800 kg/m^3, which only boosted the "concrete equivalent" chloride by about 1.2 x (insufficient to increment the surface results significantly). In order to progress the calculation a value of 0.06 % was found from a nearby façade arch sample, and this was used, which appeared consistent. Diffusivity D was then calculated according to the method outlined in reference 15.

The values found for D were influenced by the low degree of chloride accumulation, for example it can be said that for samples 1, 9 and 10 Fick's Law does not apply due to effective protection of the surface layer and/or sheltering, with a constant chloride at background level within. The diffusion constants arrived at due to the minor amount of chloride present in the render of sample 1 indicates if such concretes were exposed to further leaking many years would need to elapse prior to the upper or lower reinforcement being at risk due to chloride attack. Samples 2 and 3 of the level 3 roof slab ($D = 1.58 \times 10^{-13}$, 89 years) rely upon an estimate of Cs extrapolated from sample 19 (refer façade). Level 3 balcony slab (0.475×10^{-13} (89 years) and Sample 5 south-west corner of level 3 roof slab (0.63×10^{-13}, 89 years) rely on their own data suggesting the "true" value of the external concretes. It is known that diffusivity is not constant and decreases with age likely due to ongoing later hydration (which depends upon post-supply of liquid water), but it is expected this effect to cease within a few decades. For example TfNSW B2 performance concretes of 3.5×10^{-12} m^2/s at 58 days (9) and a 1969 swimming pool concrete evaluated by us (0.95×10^{-12} m^2/s (15)). 726 Hay St constants in exposed locations support an ongoing decrease of diffusivity of Portland cement with significant retardation of C_s due to painting and renders (compared with D's of exposed concretes evaluated in high chloride exposure/marine conditions).

With *D* known *t* was varied to the time at which the chloride threshold at reinforcement exceeds a "probabilistic threshold of corrosion". The probabilistic threshold of corrosion assumptions were as follows. Light corrosion was noted in fully carbonated samples 5, 8 and 9 at the background level of chloride at 0.01 % by weight of concrete, suggesting this to be the onset of corrosion due to carbonation and exposure to leaks, rather than purely chloride attack from the leaks. A result of 0.02% chloride by weight of concrete surface rust was evident of sample 3 concrete with more vigorous consumption of reinforcement occurring at sample 2 (0.04% by weight of concrete). Accordingly, in estimating the risk of reinforcement corrosion due to chloride, a lower threshold of corrosion due to chloride was set at 0.02% by weight of concrete and an upper bound at 0.04%. This is in good agreement with some previously reported observations in the range by us (14) and also the fib lower estimate (0.2% by weight of cement)[12]; yet lower than the 0.6% generally referenced fib by weight of cement (18).

For chloride ≥0.02 % by weight of concrete at the reinforcement in the present day were already rusting thus the remaining service life was zero. The slab reinforcement with background values of 0.01 % by weight of concrete near the soffit, and remote from the chloride source at the top of concrete projections for time to reinforcement were dominated by Cs (surface concentration of chloride). If Cs was less than either threshold, this means the life expectation may extend significantly (infinitely according to mathematics), and this is reported as "> 100 years". There was an obvious dichotomy of those concretes exposed to the leaks that had accumulated ≥0.02 % and those that had not. Only one sample, level 3 balcony 2 had an appreciable level of chloride that could result in corrosion at some future point in time; this was protected by cover to the reinforcement so this too was expected to last > 100 years.

2.3 Column Condition

Three concrete encased steel beam columns were cored; the results indicated carbonation occurring from the steel encased member side with 1921 concrete k = 2.1 mm√year. Column augmentation had occurred on level 3, which supplied an interesting multiply carbonated sample of k = 2.8 mm√year.

Concurrent carbonation of the layers of sample 3 could be seen to occur. This is supportive of the notion of a buffering effect occurring (rather than the top layers acting simply as cover). A linear combination of front distances through these different cementitious materials may be a more useful model:

$$k \propto \frac{\Delta X_1 k_1 + \Delta X_2 k_2 + \Delta X_3 k_3}{\Delta X_1 + \Delta X_2 + \Delta X_3} \qquad (3)$$

Where ΔX_n = distance of carbonation front through respective calcareous materials (mm)
n = layer 1, 2, 3, etc.
k_n = carbonation coefficient of respective calcareous material (mm√year)

This hypothesis would require further development and evaluation – it has been enumerated for a column and some façade samples in tables 3 and 4 respectively.[13] The derived constant is simply a weighted average value of k that is carbonating as a composite material.

[12] Equates to. 0.028% by weight of 2360 kg/m^3 with14% cement content. A "factor" of 7-8 is oft rounded to 10 for convenience.
[13] Alternatively, these different classes of materials evaluated according to the TR61/CARBUFF approach and given a normalised constant in similarity to the alternative concrete types. TR61/CARBUFF allows linear combinations of constants.

Table 2: Slab Assessment

	Site Observations													Measurements and Test Results												Calculation of remaining time to T_0												
Sample ID	Description	Original Exposure Category (I = Indoor, S = Sheltered)	Construction Year/Decade assigned	Total Thickness of Composite Sample d_1 (mm)	GPR Report - slab thickness d_2 (mm)	Concrete sample length d_3 (mm)	Distance to top reinforcement/diameter d_4 (mm)	Cover (top reinforcement) d_5 (mm)	Diameter top reinforcement (mm) d_6 (mm)	Top Reinforcement Condition	Distance to soffit reinforcement or mesh d_7 (mm)	Diameter soffit reinforcement or mesh d_8 (mm)	Reo Condiiton soffit reinforcement	**1. Sample Profile and Layer Identity** (Paint = P, Bitumen = B Topping = T, Cement-Sand Topping = CS, Wood (jarrah) = W, Sand + Plaster = SP) **2. Profile (mm) 3. Carbonation front (mm).** *Layer containing reinforcement in italics.* ΔX1 (mm)	ΔX2 (mm)	ΔX3 (mm)	ΔX4 (mm)	ΔX5 (mm)	ΔX6 (mm)	ΔX7 (mm)	Compressive Strength Original (Adjusted) MPa	**Chloride Concentration C_n (kg/m^3) at X_i** - *concentration at reinforcement in italics*; **Distance from TOC X_i (mm) - centre of increment.** $C_T\Delta X$	C_1/X_1	C_2/X_2	C_3/X_3	Cs (interface with topping). C0 = 0.01 % by weight of concrete Cs_1	Diffusivity, D m2/s x10-13 D	Remaining t, Top Reinforcement (Cl corrosion threshold = 0.02 % by weight of concrete) TOC $t_{0,Cl}$	Remaining t, top Reinforcement (Cl corrosion threshold = 0.04 % by weight of concrete) TOC $t_{0,Cl}$	Remaining t, Soffit Reinforcement (Cl corrosion threshold = 0.02 % by weight of concrete) SOC $t_{0,Cl}$	Remaining t, Soffit Reinforcement (Cl corrosion threshold = 0.04 % by weight of concrete) SOC $t_{0,Cl}$	Concrete + Topping $k_1 = (\Delta X_1 + \Delta X_2)$ /SQRT(ts - tcon) k_1	Carbonation Coefficient TOC $k_2 = \Delta X_2$/SQRT(ts – tcon) k_2	Carbonation Coefficient SOC $k_3 = \Delta X_{SOC}$/SQRT(ts – tcon) k_3	Resulting Exposure Category Assigned (I = Indoor, S = Sheltered)	CO_3 of top: Remaining $t_1 = (d_4 - \Delta X_1 - DX_2(3))^2/k_1^2$ reinforcement t_0 T+TOC (years)	CO3 of top: Remaining $t_2 = (d_5 - \Delta X_2(3))^2/k_2^2$ reinforcement t_0 TOC (years)	CO_3 of soffit: Remaining $t_3 = (d_7 - \Delta_{SOC})^2/k_3^2$ reinforcement t_0 SOC (years)
1	Level 5 1961 slab ("new" roof slab)	I	1961	170	150-170	145	25	5	8	Shiny	25	8	Shiny	T	C			W				0.03	0.01	0.01	*0.01*	0.02	0.32	>100 Cs ≤ 0.02	>100 Cs ≤ 0.04	>100 Cs ≤ 0.02	>100 Cs ≤ 0.04	4.5	1.6	5.7	I	0.0	5.0	0
														0-20	*20-31.5*	31.5-140	*130-170*	145-170				0-20	*20-45*	45-70	70-95													
														FC	FC 11.5	UC	FC (40)					-11.5	12.5	37.5	62.5													
2	Level 3 original roof slab 2.5 m from N/E window (leaks)	I	1921	115		68	Not Found/Detected				0,3	2.80, 4.52	Light grey coloured mesh, rust estimate 5% - 25%. Outer layer thinned from 4.5 mm (top) to 2.8	B	T	C						0.02.	0.04	0.04		0.06	1.58			0	0	10.4	7.7	7.7	S			0
														0-10	10-35	*35-108*						10-35	35-65	*65-105*														
															FC	FC						-5	65	100														
3	Level 3 original roof slab central beneath wood floor	I	1921	110		65	Not Found/Detected				10	4	Surface rusted	B	T	C		SP				0.01	0.04	0.02		0.06	1.58			0	>100	3.2	0.0	2.0	I			0
														0-10	10-30.5	30.5-81	81-100	101-110				10-30	30-60	*60-100*														
															FC	UC	FC (19)	FC (19)				-10	12.5	37.5														
4	Level 3 original roof slab "exposed deck". Odour.	I	1921	96		70-75	Not Found/Detected				13	4	Mesh absent a short way under neighbouring spall	B	S	T	C				39 (32)											9.5	7.4	7.4	I			0
														0-5	5-6	6-26	*26-96*																					
																FC	FC																					
5	Level 3 original roof slab south-west corner	I	1921	100		70	Not Found/Detected				20	4	Shiny 0 % - 10% surface rusted	B	T	C	S					0.04	0.02	0.01		0.03	0.63			>100	>100 Cs ≤ 0.04	NE*	7.4	7.4	I			0
														0-11	11-17	*17-87*	Noted					11-17	17-47	*47-90*														
															6 FC[1]	FC (60)						-3	15	51.5														
														Note 1: Except UC band @ 5.5 mm																		*Not estimated due to complex front in topping						
6	Level 3 original roof south-east on line of columns	I	1921	85		76	Not Found/Detected				0	0	Rusting	B	T	C					29.5 (23)											8.2	6.6	6.6	I			0
														0-8	8-23	*23-85*																						
															15 FC	FC (62)																						
7	Level 3 suspended floor slab	I	1921	100	110	100	Not Found/Detected				10	4	Not observed	C			SP																1.1	8.5	I			0
														0-10	10-20	*20-100*	100-110																					
														FC	UC	FC																						
8	Level 3 Balcony 1 slab (broken off core)	S	1921	170	240	85	100	32 mm (meas) 20 (GPR)	5	Shiny	3	5	Not observed	B	T	B	T	C					0.02	0.03	0.01	0.03	0.48	>100 (distance)	>100 Cs ≤ 0.04	>100 Cs ≤ 0.02 (est)	>100 Cs ≤ 0.04 (est)	6.7	3.4	N/A	S	78.1	89	0
														0-10	10-35	35-37	37-68	*68-145*					68-93	*93-118*	*118-145*													
															UC[1]		UC[2]	UC[3]					12.5	37.5	62.5													
														Note 1: Except at bitumen defect, 35 mm. Note 2: 10 mm localised intrusion. Note 3: Faint band 4 mm in render/concrete interface.																								
9	Level 3 Balcony 2 slab drilled to ceiling	S	1921	210	200	81	157	32 (meas) 20 (GPR)	10	Shiny	3,8	5	In void: shiny	B	T	B	T	T	C				0.01	0.01	0.01	NT	N/A	>100 Cs ≤ 0.02	>100 Cs ≤ 0.04	>100 Cs ≤ 0.02	>100 Cs ≤ 0.04	N/A	3.4	3.2	S	>100	>100	0
														0-10	10-50	50-51	51-80	80-125	125-150	*150-210*			125-150	150-175	175-200													
															FC		UC[5]	UC[5]	UC[5]	FC (30)[4]			12.5	37.5	62.5													
														Note 4: Contains fully carbonated fissure from SOC to 45 mm depth																								
														Note 5: Surface carbonated at interface of sections.																								
10	Level 3 Balcony 3 slab	S	1921	210	200	105	22	12 (meas) 20 (GPR)	5	Rust on slightly voided side	3,8	5	In void: shiny	P	T	T	C						*0.01*	0.01	0.01	NT	N/A	>100 Cs ≤ 0.02	>100 Cs ≤ 0.04	>100 Cs ≤ 0.02	>100 Cs ≤ 0.04	2.33	2.3	4.8	S	0.74	0.74	0
														0-1	1-3	3-10	*10-45*	45-85	*85-115*				10-35	35-60	60-85													
															UC	UC	30 FC	UC	UC				12.5	37.5	62.5									(void)				(est)

Key: I= Indoor. GPR = reinforcement detection by ground penetrating radar. ND = Not detected. NO = Not observed. N/A = Not applicable. FC = Fully carbonated. FC (Value) = Fully carbonated (distance from soffit side). UC = Uncarbonated. TOC = top of concrete. SOC = Soffit of Concrete. Est = estimate. ts = year of sampling. tcon = year of construction.

Table 3. Façade Assessment

	Site Observations										Measurements and Test Results								Calculation of remaining time to T_0						Remaining Life Projections					
Sample ID	Description	Original Exposure Category	Construction Year/Decade assigned	Depth of Member	Concrete sample length	Distance to reinforcement	Reinforcement cover	Reinforcement diameter	Reinforcement condition	1. Sample Profile and Layer Identity (Painted Render = PR, Render = R, Concrete = C) 2. Profile (mm) 3. Carbonation front (mm)				1. Chloride Concentration C_n (kg/m^3) at X_i 2. Profile (mm) 3. Distance from TOC X_i (mm) - centre of increment				Cs (interface with topping)	Diffusivity, D m2/s x10-13	Remaining t, Top Reinforcement (Cl corrosion threshold = 0.02 % by weight of concrete)	Remaining t, top Reinforcement (Cl corrosion threshold = 0.04 % by weight of concrete)	Remaining t, Soffit Reinforcement (Cl corrosion threshold = 0.02 % by weight of concrete)	Remaining t, Soffit Reinforcement (Cl corrosion threshold = 0.04 % by weight of concrete)	Carbonation Coefficient Painted Render $k_1 = (\Delta X_1 + \Delta X_2(3))/SQRT(ts\text{-}tcon)$	Carbonation Coefficient TOC $k_2 = \Delta X_2/SQRT(ts - tcon)$	Linear Combination Carbonation Coefficient $k_3 = \Delta X_1 {*} k_1 + \Delta X_2 {*} k_2 + \Delta X_3 {*} k_3$	Final Exposure Category	Remaining $t_1 = (d_3 - \Delta X_1 - DX_2(3))^2/k_1^2$ CO_3 of top reinforcement	Remaining $t_2 = (d_4 - \Delta X_2(3))^2/k_2^2$ CO3 of soffit reinforcement
				d_1 (mm)	d_2 (mm)	d_3 (mm)	d_4 (mm)	d_5 (mm)	d_6 (mm)	ΔX1 (mm)	ΔX2 (mm)	ΔX3 (mm)	ΔX4 (mm)	$C_T\Delta X$	C_1/X_1	C_2/X_2	C_3/X_3		D	TOC $t_{0,Cl}$	TOC $t_{0,Cl}$	SOC $t_{0,Cl}$	SOC $t_{0,Cl}$	k_1	k_2	k_3		t_0 TOC (years)	t_0 SOC (years)
									Surface rust	PR	C			0.09	0.03	0.02	0.01												
19	Façade arch - south - top	E	1921			67	57	5	Black coloration	0-10	10-16	*16-235*		0-10	10-37	*37-64*	64-91	0.06	1.11	0	0	0	0	0.4	0.6	0.5	E	>100	>100
									No splitting	1-3.5	6 FC	UC		-5	23.5	50.5	123												
				281	281					PR	R	C		0.01	0.01	0.01	0.01												
19	Façade arch - south - from soffit	S	1921			55	35	N/A	Rust particle/reo debris remains. Concrete spall present.	0-12/15'	15-20	20-25 20-45[1]	*45-264*	0-20	*20-47*	47-74	74-101	0.01	N/A					1.58	0.5	1.6	S	>100	>100
										FC	UC	5 C	UC	-10	47	60.5	87.5												
										Note 1: Down edge of spall, possible original fissure or surface crack																			
										PR	R			0.04															
a	Façade arch - north-west cornice render	E	1921							0-6	6-12	12-15		0-15				0.04	N/A					0.63					
										2-6 FC	UC	FC		7															
										Note 2: Render carbonated from underside in addition to top side.																			
										PR	R			0.04															
b	Façade arch - north-west archrender	E	1921							0-9	9-15			0-15				0.04	N/A					0.63					
										9 FC	UC			7															
										T	C			0.03	0.02	0.02	0.02												
20	West balcony wall - inner	S	1921	339	327	60 (est)	45(est)	NO	NO	0-10	10-15	*15-319*		0-10	10-37	*37-64*	64-91	0.03	0.16	0	>100	0	>100	1.05	0.5	1.1	E	>100	N/A
										FC	4-5 FC	UC		-5	13.5	40.50	67.50												
										T	C																		
20	West balcony wall - outer	S	1921	339	327	60 (est)	45(est)	NO	NO	0 - 15-20	20-24/27	24-319												2.11	0.7	1.1	S	>100	N/A
										FC	4-7 FC	UC																	
										T	C			0.02	0.01	0.01	0.01												
21	Ring Beam - 1920's (north face)	E	1961	160	160	75	57	5	Shiny	0-11	11-22	*22-81*		0-11	11-38	38-45	*45-72*	0.02	0.16	>100	>100	>100	>100	0.4	1.6	1.5	E	>100	N/A
										1-3.5	6-11 FC	UC		-5	23.5	50.5	123												
										T	C			0.01	0.01	0.01	0.01												
22	Ring Beam - 1960's (North face)	E	1961	160	160	60	45	5	Shiny	0-11	11-22	*22-81*		0-11	11-38	*38-45*	45-72	0.02	N/A	>100	>100	>100	>100	0.4	1.6	1.5	E	>100	N/A
										1-3.5	6-11 FC	UC		-5	23.5	50.5	123												

Key: E= Exposed. S = Sheltered. GPR = reinforcement detection by ground penetrating radar. ND = Not detected. NO = Not observed. N/A = Not applicable. FC = Fully carbonated. FC (Value) = Fully carbonated (distance from soffit side). UC = Uncarbonated.

TOC = top of concrete. SOC = Soffit of Concrete. Est = estimate. ts = year of sampling. tcon = year of construction. Italicized ranges are those containing reinforcement.

Table 4: Column Assessment

	Site Observations												Measurements and Test Results								Remaining Life Projections					
Sample ID	Description	General Exposure Category	Construction Year/Decade assigned	Width Dimension (square column)	Depth to I-beam (GPR)	Concrete sample length	Distance to first reinforcement top of sample (mm)	Cover distance to first reinforcement (mm)	First reinforcement diameter (mm) and condition	Distance to second reinforcement (mm)	Cover distance to second reinforcement (mm)	Diameter second reinforcement and condition	1. Sample Profile and Layer Identity (Painted Plaster = PP, Plaster = P, Concrete = C) 2. Profile (mm) 3. Carbonation front (mm)								Carbonation Coefficient Plaster $k_1 = \Delta X_1(3)/SQRT(ts-tcon)$	Carbonation Coefficient Render $k_2 = (\Delta X2(3))/SQRT(ts-tcon)$	Carbonation Coefficient 1961 Concrete $k = (DX3(3))/SQRT(ts-tcon)$	Carbonation Coefficient 1923 Concrete only $k = d/SQRT(t)$	Remaining $t_1 = (d_4-d_5)^2/k_3^2$ (CO_2) concrete edge to first reinforcement	Remaining $t_2 = (d_3-d_7)^2/k_2^2$ (CO_2) I-beam edge to second reinforcement
				d_1 (mm)	d_2 (mm)	d_3 (mm)	d_4 (mm)	d_5 (mm)	d_6 (mm)	d_7 (mm)	d_8 (mm)	d_9 (mm)	ΔX1 (mm)	ΔX2 (mm)	ΔX3 (mm)	ΔX4 (mm)	ΔX5 (mm)	ΔX6 (mm)	ΔX7 (mm)	ΔX8 (mm)	k_1	k_1	k_3	k_4	t_1	t_2
15	Basement Column	S	1921	720	160	160	60	60 (GPR) 51 (assumed)	NO	120 (assumed)	40	NO	PP[1]	1921 C										2.1	>100	100
														0-20[2]	20-140	140-160										
														0-30[3]												
														20 FC	UC[4]	FC (20)[5]										
													Note 1: Site notes "render missing".													
													Note 2: Generalised front, large aggregate pieces affect distance & surrounds.													
													Note 3: Through fissure. Note 4: An inclusion, and some tiny aggregate pull-out													
													Note 5: Front varies white - light pink													
17	2nd Floor Column	S	1921	540	100	120	70-80 (GPR)	75	NO	45	75	N/A	PP	1921 C							large	N/A	N/A	2.1	>>100	139
													0-2	0[6]	20-120	100-120										
														0-20[7]												
													UC	20	UC[8]	FC (20)										
													Note 6: Top surface uncarbonated with patches of very fine carbonation.													
													Note 7: Localised porous area. Note 8: Variable intrusion noted.													
18	3rd Floor Column	S	1921-1961	430	ND	160	46	35	4 Shiny	135	51	5	PP	R	R	1961 C		1921 C			0.29	1.29	2.71	2.8	32.9	0
													0-2	2-3	3-11	11-30	30-84	84-110	110-150	150-160						
													2 FC	1 FC	FC	19 FC	UC	FC	UC[1]	UC-FC[2]						
													Note: Full carbonation graduated peak at very fine crack to reo depth													
													Note 2: Irregular shallow carbonated surface at I-beam face													

Key: E= Exposed. S = Sheltered. GPR = reinforcement detection by ground penetrating radar. ND = Not detected. NO = Not observed. N/A = Not applicable. FC = Fully carbonated. FC (Value) = Fully carbonated (distance from soffit side). UC = U

TOC = top of concrete. SOC = Soffit of Concrete. Est = estimate. ts = year of sampling. tcon = year of construction. Italicized ranges are those containing reinforcement.

2.4 Beam Condition

The building was built using elastic rather than plastic theory and in modern terms this meant it was significantly over-engineered with a frame of major beams and columns that were concrete encased steel. The largest reinforcement beams of the basement were 550 mm deep x 470 mm wide. It was noteworthy that in many locations damage was initially mechanical from the installation of a fire sprinkler system (Figure 3(a)). At the level 3 there were a significant density of minor reinforced beams of 300-320 mm depth x 200 mm width supporting the roof slab. The beam design relied upon a triple 20 mm round bar reinforcement tied into columns. This could not be cored from the underside without structural damage. Measurements were taken of the cover and an estimate made of the residual service life from vertical edge carbonation measurements. Of particular interest were “thin beams” found on level 3 which had been supplemented with brickwork in 1961 probably for fire insulation reasons, which had only 20 mm cover from the vertical edge. At level 4 delamination of the bottom layer of concrete of encased I-beams had occurred. The concrete encasement there was also tested.

Some surprising results ensued, which was an absence of carbonation on painted, plastered vertical faces of both the 1921 and 1961 beams and column 2. Carbonation had occurred on the interface of the steel beam and the concrete encasement of the 1961 beam[14]; but not the plastered side. Some finely carbonated in some voids in the 1921 concrete. The generally poor condition of the underside of beams on level 3 and steel encased beams of level 4 was attributed to poor consolidation allowing surface corrosion to delaminate the underside when wet; steel had lost no substantial section enabling re-use.

Table 5: Beam Assessment

<table>
<tr><th rowspan="3">Sample ID</th><th colspan="8">Site Observations</th><th colspan="5">Measurements and Test Results</th><th colspan="5">Remaining Life Projections</th></tr>
<tr><th rowspan="2">Description</th><th rowspan="2">Original Exposure Category</th><th rowspan="2">Construction Year/Decade assigned</th><th>Depth of Beam</th><th>Width of Beam</th><th>Concrete sample length</th><th>Distance to bottom reinforcement (GPR)</th><th>Distance from edge of beam to reinforcement (GPR)</th><th colspan="5">1. Sample Profile and Layer Identity (Painted Plaster = P, Concrete = C) 2. Profile (mm) 3. Carbonation front (mm)</th><th rowspan="2">k_1 = Carbonation Coefficient Plaster $\Delta X_1(3)$/SQRT(ts-tcon)</th><th rowspan="2">Carbonation Coefficient Rendered Edge $k_2 = \Delta X_2(3)$/SQRT(ts-tcon)</th><th rowspan="2">Carbonation Coefficient exposed edge $k_3 = \Delta X_4$/SQRT(ts-tcon)</th><th rowspan="2">CO_3 Remaining $t_1 = (d_5 - \Delta X_2(3))^2/k_1^2$ of rendered edge to reinforcement</th><th rowspan="2">CO_3 Remaining $t_2 = (d_5 - \Delta X_2(3))^2/k_2^2$ of reinforcement from soffit (est)</th></tr>
<tr><th>d_1 (mm)</th><th>d_2 (mm)</th><th>d_3 (mm)</th><th>d_4 (mm)</th><th>d_5 (mm)</th><th>ΔX1 (mm)</th><th>ΔX2 (mm)</th><th>ΔX3 (mm)</th><th>ΔX4 (mm)</th><th>ΔX5 (mm)</th></tr>
<tr><td rowspan="4">11</td><td rowspan="4">Basement Beam</td><td rowspan="3">I</td><td rowspan="3">1923</td><td rowspan="3">450</td><td rowspan="3">270</td><td rowspan="3">160</td><td rowspan="3">50</td><td rowspan="3">80</td><td>P</td><td colspan="4">C</td><td rowspan="3">0.70</td><td rowspan="3">1.9</td><td rowspan="3">N/A</td><td rowspan="3">>100</td><td rowspan="3">>100</td></tr>
<tr><td>0-7[1]</td><td>7-25</td><td>25-50</td><td>50-130</td><td>130-160</td></tr>
<tr><td>3</td><td>18 FC</td><td>UC</td><td>UC[2]</td><td>UC</td></tr>
<tr><td colspan="17">Note 1: Render extends to 12 mm in places, av used in calc. Note 2: Very fine layer of carbonation along boney fissure</td></tr>
<tr><td rowspan="7">12</td><td rowspan="7">Third Floor Thin Beam</td><td rowspan="4">I</td><td rowspan="4">1923</td><td rowspan="4">320</td><td rowspan="4">150</td><td rowspan="4">75-100 (broken off)</td><td rowspan="4">50</td><td rowspan="4">20</td><td>P</td><td colspan="2">C</td><td></td><td></td><td rowspan="4">0.75</td><td rowspan="4">0.0</td><td rowspan="4">N/A</td><td rowspan="4">>100</td><td rowspan="4">96</td></tr>
<tr><td rowspan="2">0-7[3]</td><td>0-20[4]</td><td rowspan="2">20-100</td><td></td><td></td></tr>
<tr><td>0-80[5]</td><td></td><td></td></tr>
<tr><td>FC</td><td>UC</td><td>UC</td><td></td><td></td></tr>
<tr><td colspan="17">Note 3: Slight pink coloration of render at interface with concrete.</td></tr>
<tr><td colspan="17">Note 4: Fine surface carbonation and some pink under render, localised carbonation at void of surface aggregate.</td></tr>
<tr><td colspan="17">Note 5: Mostly uncarbonated, except patch 20 mm from edge and 80 mm through boney fissure. Some of the fissure uncarbonated.</td></tr>
<tr><td rowspan="4">13</td><td rowspan="4">Third Floor Thin Beam</td><td rowspan="3">I</td><td rowspan="3">1923</td><td rowspan="3">350</td><td rowspan="3">120</td><td rowspan="3">120</td><td rowspan="3">50</td><td rowspan="3">20</td><td>P</td><td colspan="3">C</td><td></td><td rowspan="3">0.75</td><td rowspan="3">0.0</td><td rowspan="3">3.2</td><td rowspan="3">>100</td><td rowspan="3">39</td></tr>
<tr><td>0-7[6]</td><td>0-70[7]</td><td>70-90</td><td>90-120</td><td></td></tr>
<tr><td>FC</td><td>UC</td><td>UC</td><td>FC (30)</td><td></td></tr>
<tr><td colspan="17">Note 6: Render fully carbonated, slightly pink on interface with concrete. Note 7: Very fine surface carbonation immediately beneath render. Note 8: Uncarbonated; localised carbonation 20 mm from edge near boney fissure, and 80 mm through fissure. Some of the fissure uncarbonated.</td></tr>
<tr><td rowspan="3">14</td><td rowspan="3">Fourth Floor Concrete Encasement of Steel Beam</td><td rowspan="3">I</td><td rowspan="3">1961</td><td rowspan="3">400</td><td rowspan="3">250</td><td rowspan="3">110</td><td rowspan="3">60</td><td rowspan="3">75</td><td></td><td></td><td colspan="2">C</td><td></td><td rowspan="3"></td><td rowspan="3">0.0</td><td rowspan="3">1.4</td><td rowspan="3">>100</td><td rowspan="3">>100</td></tr>
<tr><td></td><td></td><td>0-100</td><td>100-110</td><td></td></tr>
<tr><td></td><td></td><td>UC</td><td>FC (10)</td><td></td></tr>
<tr><td colspan="19">Key: I= Indoor. GPR = reinforcement detection by ground penetrating radar. N/A = Not applicable. FC = Fully carbonated.</td></tr>
<tr><td colspan="19">FC (Value) = Fully carbonated (distance from soffit side). UC = Uncarbonated. TOC = top of concrete. SOC = Soffit of Concrete. Est = estimate.</td></tr>
<tr><td colspan="19">ts = year of sampling. tcon = year of construction.</td></tr>
</table>

[14] A >35 MPa indoor concrete is expected to have a coefficient of circa k = 3.5 mm√year, which agrees with observations.

(a) Mechanical damage, basement (b) Water damage, level 3 (c) Deep beam delamination

Figure 3. Damage to Beams, 726 Hay St

2.5 Façade Condition

In Perth the prevailing sea breeze is from the south-west. The most exposed condition for chloride was thought to be an arch at the top of the south-west façade exhibiting some top and soffit side reinforcement corrosion. A through-core was taken through this 280 mm arch. Chloride contamination was found on the top-side, a characteristic diffusion profile commencing from the render at 0.09% by weight of concrete. Reinforcement at 57 mm was sampled as part of this section with corrosion was just commencing at the upper reinforcement at 0.02% by weight of concrete. The carbonation front was 6 mm into the concrete and thus the corrosion threshold could be said to be purely from chloride attack. Chloride diffusion modelling from this good example of diffusion mechanism revealed D to be 1.1 x 10^{-13} m^2/s. At the underside of the same arch there was complete shelter from chloride with levels at a constant 0.01 % by weight of concrete. The soffit reinforcement at 55 mm depth (35 mm cover – confirmed from cessation of rust stains/debris) was completely rusted out and a spall had occurred due to this in the past. Carbonation front assessments were 5 mm into the concrete and a further 20 mm into the spall, indicating carbonation an important promoter of rusting at this location. The reinforcement was however a further 10 mm in from the carbonation front along the spall edge suggesting a mechanism similar to that discussed in the introduction (14, 23).

Some efforts were made to use render as a de-facto indicator of weathering compared to the south-west arch – a north-west cornice and north-west arch render were sampled. Chloride levels in both were approximately half that of the south façade at 0.04% by weight of concrete. The renders were also carbonated. The north-west cornice lime render had carbonated on both sides a distance of 2-6 mm, compared with the façade arch on the top side 9 mm (k = 0.63 mm√year).

Two ring beams of the north façade were assessed – one from 1961 concrete the other 1920's. Both had background level chloride concentrations at 0.01% by weight of concrete. Together with the render observations this suggested a lower chloride risk on this façade. Carbonation ingress was likewise significantly lower on these façade samples than the indoor samples with a partial carbonation of the render samples (1-3.5 mm of 11 mm, k = 0.4 mm√year) and 6 mm penetration into concrete (k = 0.4 mm√year). Reinforcement drilled was shiny indicating a passive state.

The West balcony wall, mid-way between the two facades had a relatively high chloride background of 0.02 % by weight of concrete; due to its constant nature C_0 was assumed to be 0.02% by weight of concrete. If the chloride threshold was assumed to be 0.02%, the remaining life would be zero years; if a higher threshold assumed then greater than 100 years expectation. This example indicates issues with sensitivities of the models when both the threshold and results are low. Carbonation results had a low level of penetration once more at 5 mm into the concrete.

In summary carbonation rates in exposed condition on the west and north sides are significantly lower than indoor concretes as expected from wash off by rain, An even lower rate of 0.5-0.6 mm √year rate was seen in the weathered south-west arch, nevertheless corrosion on the soffit side due to this had occurred. Chloride accumulation at the west and south facades that may challenge the durability had occurred, however the north façade exhibited considerable remaining durability due to low levels of chloride and shallow carbonation depth.

Unlike internal plasters, it was clear from the partial carbonation of bonded renders that concurrent carbonation of the render and concrete, was occurring. As such the render could not be considered true cover, however may be having some effect on the ultimate carbonation rates of concrete. Render carbonation rates varied 0.4-2.11 and concrete 0.5 – 1.6 and these appeared inversely related (if render rate high, concrete low and vice versa) suggesting some composite behaviour. A linear combination of distance carbonated through both tends but consistent with exposed condition estimates of k = 1-1.5 mm √year. An exception is the top of the façade arch which likely experienced significantly more rain than the other concretes.

2.6 Remedial and Project Approach

The variability of concrete repair costs can be a concern. Contractors may justifiably charge by square meter given a particular depth, or by litre or cubic meter (7). To quantify all the concrete in the beams and columns that required repairing testing, visual inspection and measuring cover in order to work out the width and depth of breakout in a side by side arrangement with inspectors from a tenderer in order to produce a quantum reviewed both by Engineer and a potential Contractor; with an indexed drawing displaying the locations and specification repair methodology cross reference. At tender this assisted fixed price offers for the concrete repairs of the order of $2 million, acted as a working document for the Contractor, and assured that sufficient breakout was to occur for a quality repair.

All concrete repairs included the recommended methodology of breakout to a saw cut edge beyond the extent of reinforcement, and to a minimum depth behind reinforcement to ensure the repair had mechanical in addition to material bond anchor of repair mortars (7). Dry abrasive blasting with garnet was used for reinforcement cleaning and pre-soak treatment (plus acrylic bonding agent, where applicable) prior to reinstatement and curing in forms and/or taped polythene. In terms of re-passivating, a conservative approach was utilized in the chloride contaminated areas with galvanic anodes; the Owner did not wish to revisit concrete repairs in the event of leaks.

A patch repair was inappropriate for the deep beams. To restore their integrity following reinforcement cleaning and application of epoxy zincrich primer a form and pour methodology was specified requiring a pre-soak let out with a tap, followed by reinstatement with a super fluid, high strength, low alkali, micro-concrete reinstatement agent was poured in with venting for air to be displaced so that voids in the concrete do not form, through specially prepared formwork with spouts. The forms were to be left on for the cure period (Figure 5 (a) and (b)).

Following the revelations regarding the amount of historic topping and re-sealing that had gone before, all the external balconies and roofs were treated with toppings and products containing a reputable Australian brand of inorganic crystalline self-healing agents. This ensured that any materials further adhered, such as tiles, the inevitable break-down of polymeric components used in membranes and tile adhesives due to mechanisms such as UV attack, hydrolysis or osmotic pressure, would result in a temporary leak that would self-heal. The Owner supported this initiative despite lower cost alternatives proposed by the Contractor.

The slabs were considered an issue due to full carbonation. Leaks can occur even in well-sealed internal spaces due to the break-down of bathroom membranes, and in an upwards direction due to humidity in bathrooms and kitchens.

Design options considered for the internal slabs included demolition and re-casting, rejected due to the volume of material to be removed in the busy CBD. A successful re-alkalisation trial was conducted, however due to the low number of available electrical units, this would add to project timeframes for the 8000 m^2 to be treated. The underside of the concrete had partly adhered plaster and rusted mesh which would need some form of reconciliation, adding a further process per square meter.

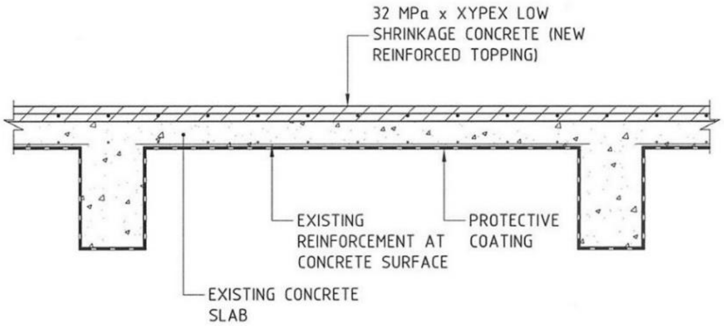


Figure 4: Slab Encapsulation Strategy

The building itself suggested the strategy (Figure 4). We had noted that the reinforcement mesh at the soffit side. This did place a natural limit on the volume of concrete that could delaminate on to the ceiling below. Further, to make use of original over-engineering – by placing a reinforced topping slab containing crystalline self-healing agents, over the scabbled surface of the original slab to cantilever over the closely spaced beams (Figure 5 (d)). Services were installed at a distance from the beams and soffit to ensure the past issues with mechanical damage were not revisited and leaks well separated from the concrete (Figure 5 (a)). The soffit was also blasted to remove plaster and clean any rusted reinforcement to a shiny condition before re-encapsulating with a surface tolerant epoxy to exclude air and water (Figure 5 (c)). The epoxy was of a glass flake type to reinforce the paint and bridge minor amounts of future slab particle shedding.

The solution meant that the process of any future bathroom installations a redundancy measure of self-healing agents would occur in the event of any long term break down of the membrane on either side of the concrete.

The internal ceilings concealed the soffit remediation works (Figure 5 €). The façade was visited with traditional concrete repair (9) and coated with acrylic elastomeric paints with colours to complement the spire of the Wesley Uniting Church on the opposite side of Murray St (Figure 5 (f)).

(a) Form and Pour Beam repair (b) Beam following stripping (c) Protection of surfaces and spray application of GFR epoxy coating

(d) Installation of topping slab (e) Fitout level 3 (f) Façade upon completion

Figure 5. Key activities and results of concrete remediation

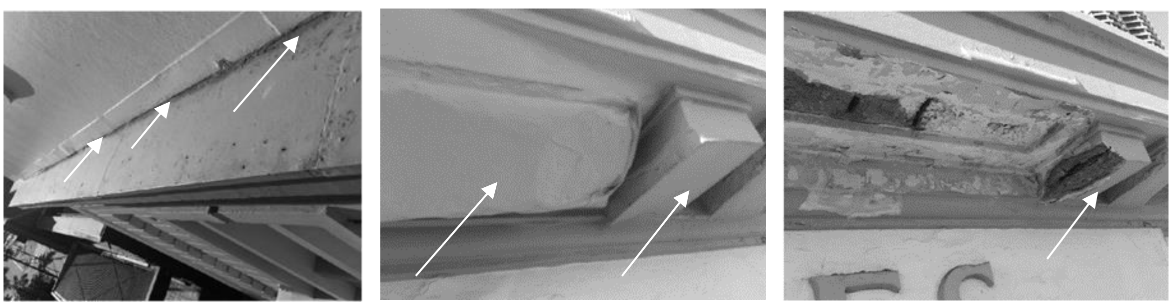
3. Case Study 2 - Bank of NSW (and Dynon's China Hall)

3.1 History and Objectives

The Bank of NSW advertises at its entry "Established 1817" indicating the inauguration of the bank itself, with the actual construction in 1935. The building structure is complete in its original form. Its neighbour, Dynon's China Hall was a large emporium replaced by the Dynon's Plaza precinct. What has been retained is essentially the entry, also now referred to as Dynon's China Hall. The issues concerning the Owners were due to the continued aging of the concrete street facades and propensity for spalling from the upper cornices that functioned as awning forming part of the entablature.

Figure 6. Bank of NSW Building

At Bank of NSW corrosion at the base of a discontinuous flashing on the upper cornice (acting as an awning for the windows below) was noted (Figure 7 (a). A cherry picker employed to investigate finding the bulging on soffit side due to water (Figure 7 (b)). Tap testing found a series of loose and delaminated pieces of concrete soffit and steel reinforcement within (Figure 7 (c)).



(a) Water ponding at discontinuous flashing

(b) Bulging paintwork

(c) Following removal of loose concrete – arrow indicates sample location

Figure 7. Upper Cornice Investigation

3.2 Sampling and Results

A loose section of a dentil was knocked off and retained ("the spall" Figure 8 overleaf) and the three through-sections of concrete tested by phenolphthalein, representing a cover section (piece #2) and two sections including the reinforcement (Piece # 3 and #1 – right). The concrete was found to be fully carbonated to maximum depth of 150 mm from rendered edge. There were two renders noted, a light coloured outer render 2-5 mm thick and a dark render 3-12 mm thick. Some very light pink stains were noted in the outer layer. The chloride results are reported in Table 6.

Table 6. Chloride Results

Sample Location	Depth (mm) of Section	% Cl by weight of concrete
From Short Edge	0-20 (render)	0.03 (render)
	20-35	0.01
	35-50	0.01
From Long Edge	10-50	0.01

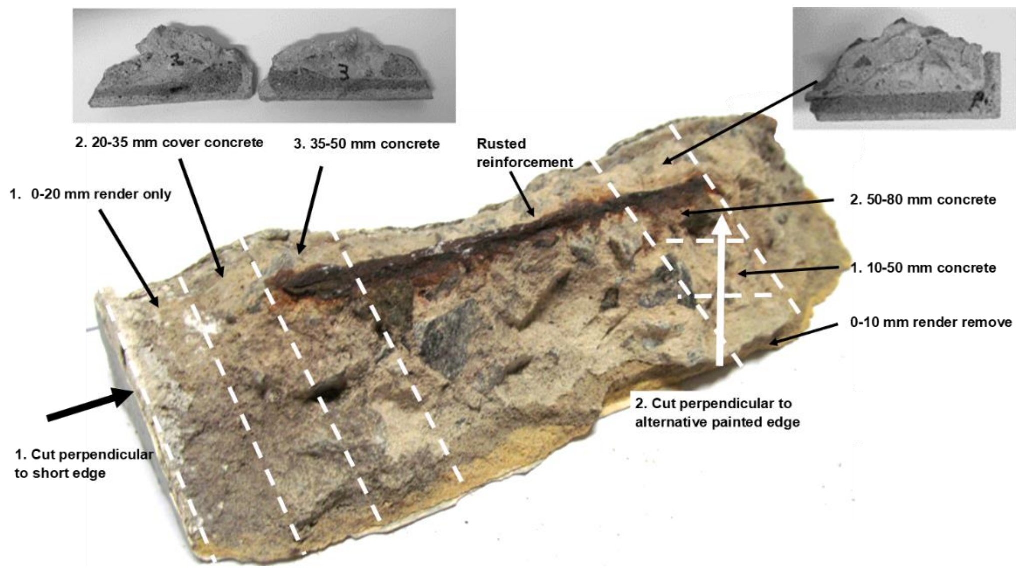


Figure 7. Cutting plan to examine causes of upper cornice spall development and spray results[15]

3.3 Discussion

The Bank of NSW results strongly support that carbonation was required for the depassivation rusting and subsequent near-delamination of the spall. The dentil had long term exposure to the original chloride present as an accelerator, however there have been no spalls apparently due to this in the previous eighty two years. Unlike the Dynon's China Hall building, there was no protective render on the topside, and the flashing did not protect the topside or back corner. It is clear that there was abundance of water traversing the awning in recent years. The critical factor in rusting appears this water in conjunction with the fully carbonated state.

3.4 Remedial and Project Approach

As an outcome of the condition assessments, several options were provided for concrete remediation, steel and brickwork repair. These included electro-osmotic treatment, traditional repair with a form of cathodic protection (anodes). Replacement with a precast or polyurethane facsimile is also possible, although typically a last resort.

Scaffolding can represent up to 25% of construction cost. The design of a façade scaffolding system shared over the two properties (Figure 8) aimed to gain synergies featured a high degree of cantilevering from towers key shopfronts with regard to detailed investigation, trade attendance, disposal of waste, and repainting. This involved considering the height for labourers to effect these works, with manageable and minimal changes. By packaging this as a separate contract to the main works contract, the Owner would have more control of the appearance of the scaffolding, and duration of scaffold hire.

[15] Not visible due to carbonated state and black and white illustrations.

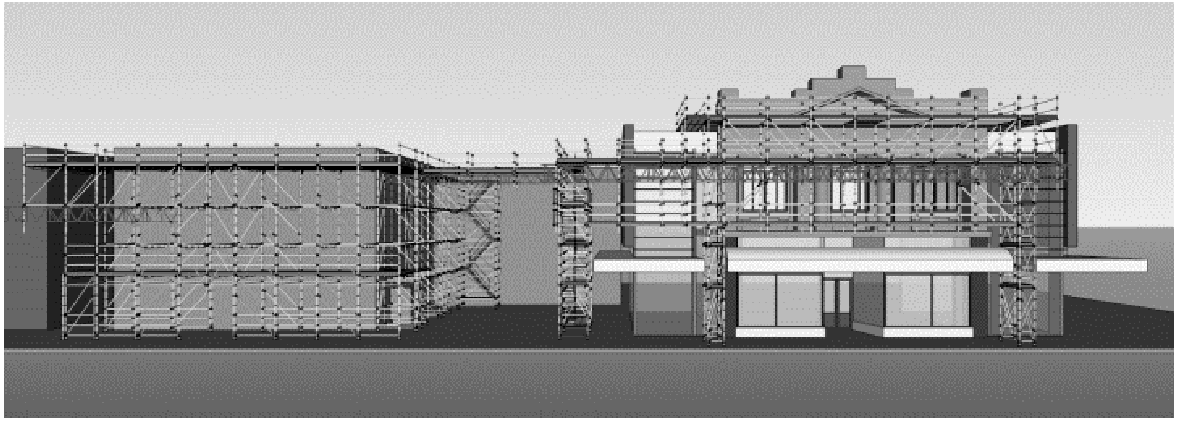


Figure 8. Proposed scaffolding solution courtesy Caledonia Group

4. Project Costs and Investment

For heritage to be preserved, construction methodologies employing strategies to keep tenants operational and therefore protect the owner present and future cashflows are key project enablers. At 726 Hay St this was due to dry trades employed and an encapsulation strategy that avoided the demolition of the suspended concrete floors and coincidentally preserved the heritage concrete. A full restoration is yet to occur at 899 and 915 Hay Streets.

A slump in Perth occupancy rates occurred in 2010-2011 and the 726 Hay St project had significant duration, of over two years completed in late 2012, when the Premier cut the ribbon in an opening ceremony bringing 5,668 m^2 of new office space online. Vacancy rates in premium office space varied from 2.9% in 2013 to just under 20% from 2013-2019 with gross income dropping and rising costs, net income dropping from \$620 (2013) -418/m^2 (2019). Considering a project cost reported by others of \$15-19 million and conservatively 80% occupancy of the 726 Hay St office space, a payback[16] on the renovation considering the new office space only, would require 6 years.

Effectively, there was also a rental rate increase of the retail component due to the quality of the result has since been reported by Lease Equity that an improvement from a facility rated as peripheral (\$800-\$1250 /m^2/annum retail, 2019), premium rents are now being commanded (\$3000-\$4000 /m^2/annum retail, ((2)) with 16-18% rental increases locked in (27).

5. Conclusions

Results from both 726 and 899 Hay St structures strongly support carbonation as the later age corrosion initiator for steel reinforcement in concrete given uniform backgrounds of chloride (0.01%). With the tools available to the investigating scientist, a clear phenolpthalein result at reinforcement level should be seriously considered as the conclusion of concrete design life, due to the propensity for corrosion to occur rapidly once wet. Particularly, for overhead concrete roof structures in public buildings that should offer longevity, an anti-carbonation strategy to preserve design life beyond that proposed by standards should be considered in new design works due to sudden cessation of passivity when carbonated and wet. Remedial approaches however are available to mitigate this risk as demonstrated herein, these come at a cost.

The rates of carbonation had a big spread, these were in many cases consistent with micro-climate zones identified by Lagerblad (22). Painted, lime-based plasters appeared capable of forming a seal against carbonation, at least on vertical surfaces. Lime containing renders buffered their own carbonation front on vertical and horizontal surfaces. This did not, however appear to protect voided

[16] Payback is a simple tool that does not take into account interest.

concrete behind an intact layer of render. In these long term observations of both structures, effective capture of environmental chloride was occurring in the lime cement renders.

For facades, strategies such as electro-osmotic realkalisation and desalination may be of benefit if the render is still bonded. Replacement of existing chloride contaminated render with a matched mix design, is also a possibility to rejuvenate and further protect original concrete by removing Cs, the surface concentration of chloride with propensity to diffuse, and distancing the concrete from capillary suction effects.

At 726 Hay St bitumens used for waterproofing at the balconies had a protective effect against carbonation however at the roof also appeared to accelerate the rate; this latter issue also associated with decomposition. It is unclear if the effects of modern torch-on membrane systems have been assessed for this risk; this would be a worthwhile and readily tested exercise.

The construction methodologies in this era of early adoption of concrete at 726 Hay St showed considerable efforts to address durability issues by the constructors. Galvanised mesh to counter corrosion for example was manufactured in the USA (26) and was likely imported via the UK. Other reinforcement had significant cover.

To enable restoration and re-use project financials must be commercially viable, and for the vast number of structures awaiting remediation there is a perceived responsibility by government to fund. Strategies discussed herein to assist true commercial viability include retaining ground floor tenants during the works through "dry trades" and sensitive scaffold design and site management, stabilizing costs by take-off of quantities prior to the works (rather than a significant portion of the project being dealt with as a latent condition); independent scaffolding contracts, selection of methodologies to minimize the number of treatments of any square meterage, preservation measures durable in themselves, and maximum retention of original building fabric to achieve societal expectations of a quality restoration. If our industry can apply these disciplines, owners are in a better position to justify their projects from the square meterage gains of the "upper floors" and leveraging retail/office and prime/average rental ratios. There is a role for the architectural and engineering consultants to optimize these strategies prior to the financial load of the construction project commencing.

6. Acknowledgements

Images sourced from the State Library of Western Australia and reproduced with the permission of the Library Board of Western Australia.
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