Basin Reserve Museum Stand
Condition Assessment

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Prepared By Neil Lee

Neil Lee
Concrete Technologist

Reviewed By Sheldon Bruce

Sheldon Bruce
Research Manager - Asset Performance

Opus International Consultants Ltd
Central Laboratories
138 Hutt Park Road, Gracefield
PO Box 30 845, Lower Hutt 5040
New Zealand

Telephone: +64 4 587 0600
Facsimile: +64 4 587 0604

Date: April 2012
Reference: 12 - 5C1964.00 CL
Status: Revision 2

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1 Introduction

This report describes the results of a durability condition assessment and associated materials testing undertaken on the Basin Reserve Museum Stand in March 2012. The work was commissioned by Wellington City Council as a consequence of the Stand being declared an earthquake prone building pursuant to Section 124(1)(c) of the Building Act 2004.

The materials testing is intended to provide inputs for structural assessment and potential seismic strengthening design work, while the durability investigation allows the residual life of the Stand to be estimated with a degree of confidence. It also serves to identify the probable future maintenance demands necessary to safeguard the structures' condition.

The work conducted included the following key components:

- A detailed visual examination of the structure to identify deterioration, including photography of key defects.
- Selection of a subset of the principal concrete elements on the basis of contrasting condition (i.e. sound vs. distressed) for a more comprehensive instrumented analysis to determine current and future corrosion risk. The principal attributes measured and compared were:
    - Depth of protective concrete to reinforcement;
    - Loss of protective concrete alkalinity through reaction with atmospheric CO_2 ('carbonation');
    - Chloride ion contamination.

  Reinforcement corrosion initiated by chloride ion ingress or carbonation is by far the most common deterioration mechanism that occurs to concrete structures under ambient NZ conditions that becomes severe enough to end their functional life.

- Exposure of reinforcing bars at selected locations, particularly where cracks or spalling are evident, to confirm their current corrosion state, extent of deterioration and loss of cross-sectional area.
- Removal of four cores from the building frame to verify concrete compressive strength and quality.
- Visual inspection of the weld integrity of the latticework on the steel columns supporting the pavilion roof, and investigation of the severity of corrosion developing where they are connected into the tops of the concrete columns.

2 Methodology

The condition assessment was undertaken by Neil Lee and David Wong from Opus Central Laboratories' specialist concrete performance section during the period 13 – 16 March 2012.

As far as possible, the nomenclature used to identify particular concrete elements or locations within this report (e.g. 'riser beam', 'Distributing Gallery') is chosen to be consistent with those recorded on the 1924 Wellington City Council construction drawings for the Stand.

2.1 Visual & Auditory Inspection

A detailed visual inspection of the Museum Stand was undertaken, including photographing of key defects and features of interest. A cherry picker and a scissor lift were used to provide elevated work platforms, particularly for inspection of the Sussex Street elevation and the roof support columns. Where spalls, cracking or other evidence of concrete distress was detected, the cover concrete at selected locations was removed using a hammer drill equipped with a chisel bit to expose the underlying reinforcement. These exploratory breakouts allowed the severity of corrosion affecting the bar to be assessed and the consequent loss of cross section estimated.

When a reinforced concrete structure deteriorates through corrosion, the corrosion product formed takes up a larger volume than the steel consumed, building up tensile stresses around the reinforcement and ultimately causing the concrete to fail. This failure can develop as a visible surface crack, typically generated when the ratio of concrete cover to bar spacing is low, or a delamination parallel to the concrete surface, when the cover ratio is relatively greater. Ultimately delaminations are expressed as spalls from the concrete surface but they can be difficult to detect up until this point.

To determine the presence and extent of any delamination, auditory sounding surveys were carried out on all accessible surfaces of the exterior façades and selected concrete elements within the interior of the Stand. A particular emphasis was placed upon checking areas that were showing faint signs of potential deterioration such as the presence of fine cracks or diffuse rust-staining.

Sounding surveys were conducted by dragging a metallic hammerhead across the surface of the concrete and listening for a variance in the reverberation that would indicate 'drumminess'. Good concrete without delamination produces a sharp ringing response, whereas delaminated sections have a duller and more muffled sound, particularly when directly struck.

2.2 Cover Measurements

An adequate depth of concrete cover over mild steel reinforcing is critical to achieving durable concrete structures. This is because chloride ions and carbon dioxide, two of the most destructive agents for reinforced concrete, are atmospheric contaminants that penetrate concrete elements from the surface. Insufficient cover can result in rapid accumulation of these contaminants at the depth of the steel, in concentrations sufficient to initiate corrosion and cause damage within the intended service life of the structure.

The location of the outermost reinforcement and its depth of cover were determined non-destructively using an electromagnetic cover-meter. This instrument uses the variation of flux induced in a magnetic field by the presence of steel reinforcing bars to infer their depth and orientation.

Where reinforcement was exposed through exploratory breakout, physical measurements of the depth of cover were also made to calibrate the cover-meter results.

2.3 Chloride Ion Contamination

Chloride ions in sufficient concentration are unique and specific agents for the corrosion of mild steel reinforcement: They destroy the protective surface oxide layer that ordinarily develops on steel surrounded by alkaline concrete, a process referred to as depassivation. It is generally accepted that depassivation will only occur once the chlorides accumulate beyond a critical threshold concentration in the concrete surrounding the rebar. Thus it is important to understand the distribution of chloride ions in a structure under investigation to determine its corrosion susceptibility.

Chloride contamination can arise from two principal sources. Firstly, chlorides can be incorporated into the fresh mix, either in the form of salt-laden marine aggregates or through use of set-accelerating admixtures based on calcium chloride. Secondly, chlorides may originate from an environmental source, transported as salt-water spray or salt laden aerosol from the coast.

The chloride ion concentration at which corrosion of reinforcement occurs varies according to the quality of concrete and ambient environment. The UK Concrete Society[1] suggests some risk of reinforcement corrosion is present when chloride ion levels are in excess of 0.05% by weight of concrete, and a high risk of corrosion at levels exceeding 0.15%.

To determine chloride profiles, samples of powdered concrete for determining 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 of (ordinarily) 0 – 20 mm below the surface. These holes are subsequently deepened to allow the separate collection of further increments of powdered concrete, of depth and number appropriate to the environmental exposure of the tested element and its reinforcement cover. 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) to express the total chloride content as a percentage of the dry weight of concrete.

2.4 Carbonation

The alkalinity conferred by calcium hydroxide [$Ca(OH)_2$], a by-product of Portland cement's setting reaction, is essential to the passivation process that ordinarily protects mild steel reinforcement embedded in concrete from corrosion. This alkalinity can be lost over time through reaction of the $Ca(OH)_2$ with atmospheric carbon dioxide to produce effectively pH-neutral calcium carbonate or, more rarely, from leaching by 'soft' (i.e. calcium depleted) water. These 'carbonation' reactions, along with chloride ion contamination, are the two principal causes of reinforcement corrosion.

Carbonation depth of selected concrete elements was determined by spraying phenolphthalein indicator[2] solution on the walls of the drill holes created by the chloride sampling; the holes were thoroughly rinsed with water prior to the carbonation test to remove any residual drilling dust. The measured depth below the surface which is not stained purple by the phenolphthalein reagent indicates the region of concrete with a pH of less than 9.0 – 9.3. This is assumed to correspond to the total depletion of $Ca(OH)_2$ in the concrete through carbonation or leaching; any reinforcement that lies within this zone is potentially vulnerable to corrosion.

2.5 Concrete Compressive Strength

A total of four cores, with a diameter of 84 mm chosen to be at least four times the maximum aggregate size in the concrete, were removed from the Stand concrete by water-cooled drilling. One core was taken from the exterior wall of each of the principal building elevations, at standing height above the local ground level. So far as could be ascertained, the exterior walls are monolithic with the perimeter columns, suggesting that the results are also representative of the concrete used in the key structural elements of the Stand.

The compressive strength of the concrete core samples was measured in dry condition in accordance with Section 9 *Determination of Strength in Compression of Drilled Cores* from NZS 3112:Part 2 *Tests Relating to the Determination of Strength of Concrete*.[3] Cores were trimmed to a standard 2:1 length to diameter ratio whenever possible. Where this was not achievable, the compressive strength was normalised to this aspect ratio using the correction factors given in ASTM C 42 *Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete*.[4]

3 Findings

3.1 Concrete Quality

To assess the quality of concrete used in construction of the Museum Stand, four concrete cores were removed for measurement of compressive strength, one from each of the four principal building elevations. The results are presented in Table 1 and demonstrate a high degree of variability, with unconfined compressive strengths ranging from 14.5 to 37.5 MPa.

All four core specimens were free of cracks or other defects that might produce erroneously low strength results and displayed well-consolidated concrete with minimal excess voidage, estimated visually as less than 0.5%.[5] The grading of the aggregate, which appears to consist of rounded to sub-angular natural (river-run or seashore) greywacke gravels, varied widely between the cores. For example, the eastern elevation core contained almost no aggregate greater than 10 mm diameter, whereas the southern and northern samples have a more regular size distribution including coarse aggregate. The western elevation sample is characterised by a unusually high cement paste fraction in comparison with the other three cores, which may explain the strength differential.

Table 1. Concrete compressive strength results.

Sampling Location	Unconfined Compressive Strength (MPa)
Northern elevation wall	23.0
Eastern elevation wall	14.5
Southern elevation wall	22.5
Western elevation wall	37.5
Mean Strength	***24.0 MPa***

Judged on these results, it seems reasonable to assume that the concrete in the Museum Stand generally is both highly variable and of relatively low quality. It is also important to note that while the core samples were well-consolidated, instances of deficiencies in concrete compaction were observed during the condition assessment that would further reduce its strength; Figure 1 shows an example. It is therefore recommended that conservative assumptions are made regarding concrete strength for the purposes of any seismic assessment conducted.

Figure 1. Poor concrete compaction in the base of concrete perimeter column (Sussex Street elevation; basement archive room).

3.1.1 Durability Requirements

The Museum Stand is located approximately 1 km to the south of the nearest shoreline. Although in the direction of the prevailing wind, this is sufficiently distant to place the structure in the B1 Exposure Classification for concrete durability under NZS 3101:2006 *Concrete Structures*.[6] This is a moderately aggressive environment in the coastal perimeter where this is some possibility of exposure to chloride ions, but in which the life of concrete structures is generally dictated by their resistance to carbonation. Protection of reinforcement in this environment is considered by NZS 3101 to be adequately provided by a combination of both an appropriate depth of cover concrete and also concrete quality (i.e. strength).

The design strength of the concrete used in the construction of the Museum Stand is unknown but the default cover indicated by the construction drawings is 1½″ (approximately 38 mm). For reinforcement provided with this nominal cover, NZS 3101 currently mandates 25 – 30 MPa specified compressive strength concrete to achieve a 50 year service life, with 45 – 50 MPa necessary for a 100 year life. Considering the core strength results reported in Table 1 and their variability, much of the Stand concrete would not meet current code requirements for a 50 year life and certainly would not be expected to be durable for 100 years.

However, the durability performance demand is reduced by the fact that many of the key structural elements, including a large minority of the columns, the stringer beams supporting the tiered seating and the girder beams exist within a fully enclosed interior environment. In this relatively benign A1 exposure classification, NZS 3101 considers that provision of adequate cover alone is generally sufficient to ensure durability: For a 25 MPa concrete, the lowest strength considered, the minimum required cover is for a 100 year life is 35 mm.[6]

3.2 Exterior Concrete Condition

3.2.1 Sussex Street Elevation

By far the most significantly deteriorated concrete element of the structure is the Sussex Street (western) elevation, which is characterised by the development of significant concrete spalls around windows and window reveals. Typical examples of this damage are shown in Figure 2; all of the window surrounds on this elevation are affected to a greater or lesser degree.

This spalling has two sources, one of which has probably been a contributor to the existence of the other. Firstly, the window frames are constructed from mild steel which has begun to corrode severely at certain locations. This may be partially due to inconsistent maintenance of the protective paint coating on the exterior of the frames. However, there is also a particular point of vulnerability introduced by the installation detail, in the form of short steel tabs positioned around the perimeter of the window frames. These sit directly upon the face of the cast surface of the concrete and presumably serve as locating devices. They are then encapsulated in an unreinforced 3 coat solid plaster render approximately 25 – 30 mm deep to give a fair-faced wall surface.

Any protection initially conferred by the protective alkalinity of the plaster render has subsequently been lost through carbonation and the frame tabs have served as loci for initiation of corrosion. This is illustrated by the images reproduced in Figure 3, which show the results of exploratory concrete breakouts at spall locations. The expansive pressure created by the formation of voluminous corrosion product at these tabs has encouraged the plaster to debond at its interface with the concrete substrate. As a result there are large areas of delaminating drummy sheet plaster between the ground floor and gallery windows on all of the window bays.

Corrosion of the window frames and the subsequent plaster delamination will have permitted water to permeate the main body of the concrete. This in turn has encouraged the development of corrosion on many of the reinforcing trim bars in the window surrounds. The was revealed by exploratory breakouts at spalls, examples of which are shown in Figure 4, Figure 5, Figure 6 & Figure 7.

In all cases where exploratory breakout was undertaken at spalls on the Sussex Street elevation, the underlying reinforcement was revealed to be heavily corroded. Loss of cross section typically exceeding 50% and frequently the corrosion was so severe that the bars no longer possess any structural integrity. On the basis of these findings, the minimum maintenance requirement or this elevation is the removal of the window frames and systemic reconstruction of the reinforced concrete around window openings and reveals.

(i)

(ii)

(iii)

(iv)

Figure 2. Examples of spalling developed around window surrounds on the Sussex Street elevation:

- **(i) Beside the Bay 9 (southern-most) gallery window**;
- **(ii) Either side of column between windows in Bays 6 & 7;**
- **(iii) Above ground floor window in Bay 1;**
- **(iv) Above the gallery window in Bay 7.**

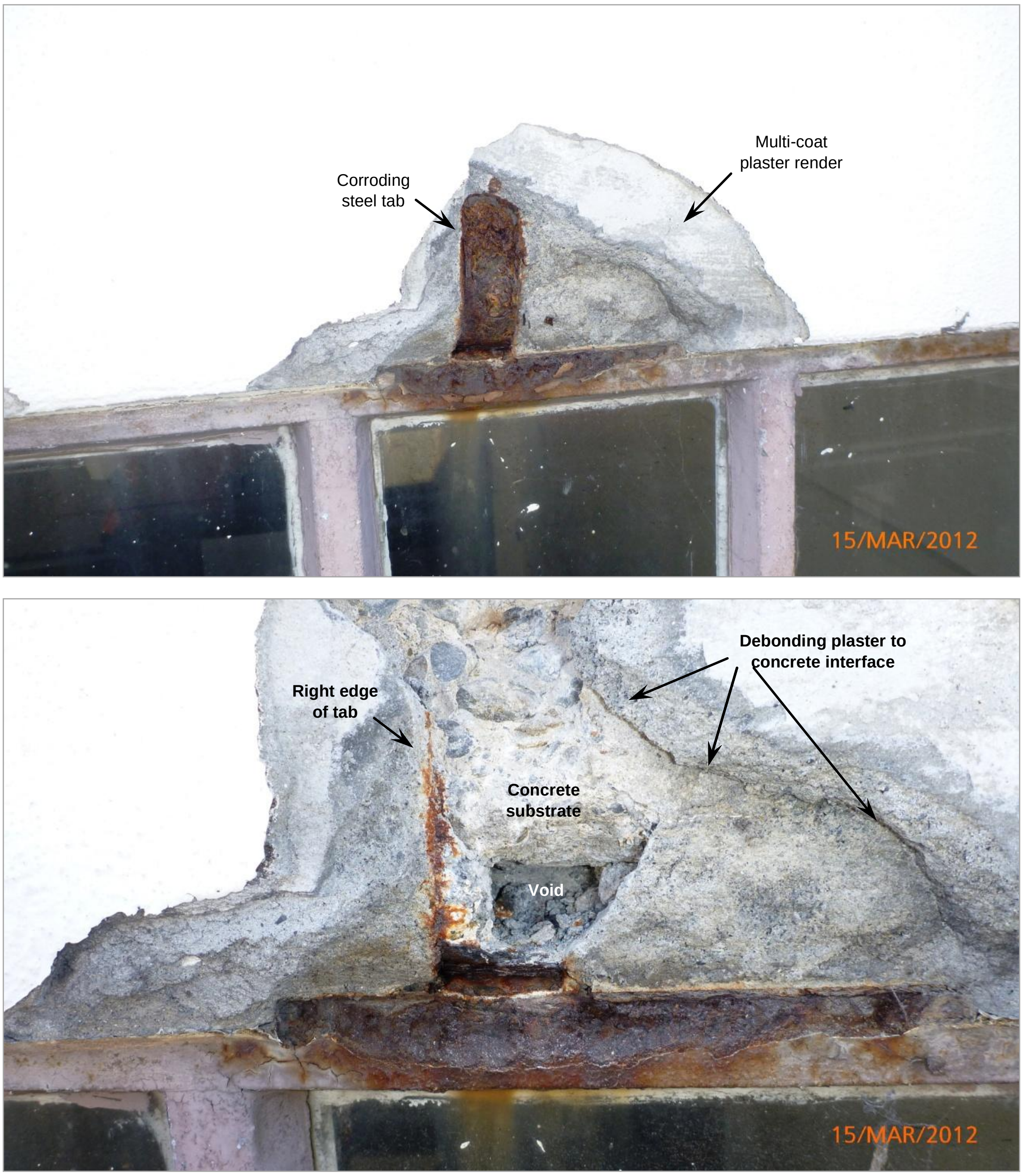


Figure 3. Examples of window frame corrosion and associated spalling on the Sussex Street elevation above the ground floor window on Bay 2. Note in the bottom picture that the ca. 30 mm gap between the top of the frame and the window opening in the concrete has been incompletely filled with mortar; the resulting void may have allowed moisture to accumulate directly above the unprotected frame, explaining the intense corrosion at this position.

Figure 4. Completely corroded lintel bar above ground floor window (2nd bay from North). Note that aside from the plaster render, there is minimal protective concrete cover to this bar.

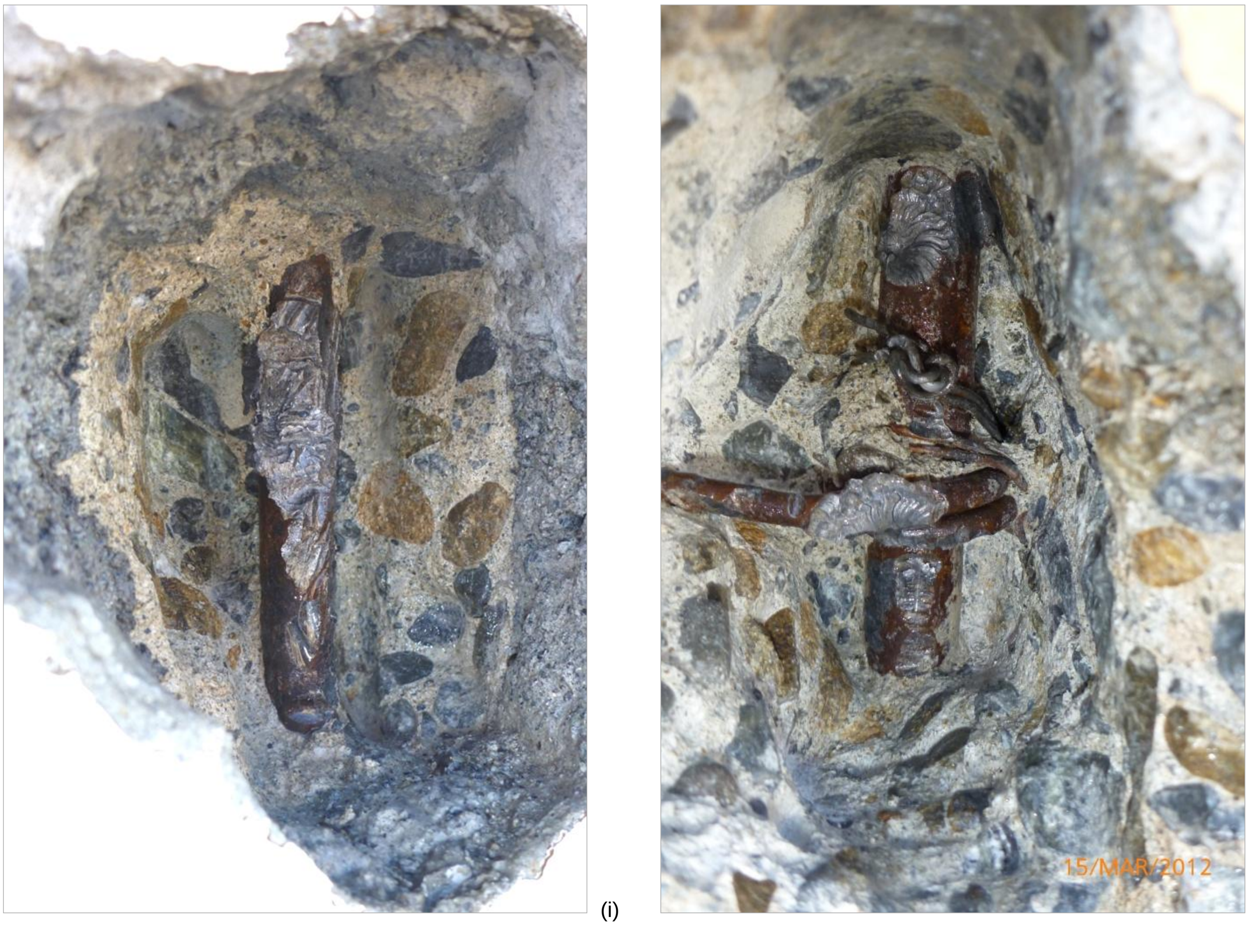
Figure 5. Heavily-corroded trim bar above the Bay 6 gallery window. The reinforcement is encapsulated in poor quality concrete which affords little corrosion protection to the steel; in particular note the rubbly material between the dotted lines, which presumably represents a badly-executed cold joint in the construction process.

Figure 6. Corroding bars exposed at spall adjacent to Bay 9 gallery window (refer to Figure 2(i) for view of spall prior to breakout). The reinforcement is embedded in loosely-compacted concrete, which has afforded little protection against corrosion.

Figure 7. Exploratory breakout of spalls adjacent to the left side of the Bay 3 window reveal. This diagonal hoop reinforcement appears to tie vertical trim bars around the window openings into the main column reinforcement. The original bar diameter appears to be ½″ but loss of section due to corrosion in the worst area approaches 50%. The bar is almost on the surface of the concrete, directly below the plaster system.

It must be acknowledged that all of the concrete reinforcement spalling uncovered on this elevation pertain particularly to the window openings and the presence of corrosion is a direct result of construction deficiencies (i.e. poor compaction and/or substandard covers). Elsewhere on the Sussex Street façade the concrete appears sound. The vast majority of visible defects are traceable to shrinkage cracking of the plaster coat, which is primarily a cosmetic defect.

However, exploratory breakouts conducted in sound areas demonstrate evidence for incipient systemic deterioration: Even bars in well-consolidated concrete display signs of active surface corrosion, albeit of currently superficial extent, as illustrated by the images in Figure 8.



(i) (ii)

Figure 8. Condition of reinforcement in sound concrete: (i) $^{3}/_{8}''$ diameter bar in middle of wall panel between Bay 2 windows and (ii) $^{5}/_{8}''$ diameter bar with $^{3}/_{16}''$ stirrup in adjacent perimeter column C27. Both bars display faint signs of active corrosion, suggesting they have been depassivated.

The initiator of corrosion in this sound concrete appears likely to be a combination of risk factors created by both chloride ion contamination and carbonation, as further discussed in section 3.4.

3.2.2 Other Façades

The other elevations of the stand are in noticeably better condition than the Sussex Street façade. Several large (1 – 4 m^2) drummy areas were detected during sounding surveys of the north and south walls that may be indicative of developing reinforcement corrosion. However, investigation suggested these were instead due to repeated hygral and thermal movement strains exceeding adhesion of the plaster to its concrete substrate, creating delamination. Plaster cracking as a result of bond failure is visible in many locations (Figure 9). These cracks are often rust-stained, but this is due to the cracks acting as concentration points for evaporation of run-off water that has been iron contaminated by adjacent corroding fixtures. The plaster system generally is in poor condition and likely to require replacement or substantive remediation in the medium term.

Figure 9. Cracking in the solid plaster finish applied to the northern elevation.

Where spalls are present their origin is similar in nature to those previously described, i.e. they reflect a pre-existing vulnerability created by poor construction practice, exacerbated by a location that provides ready access to moisture to drive the corrosion reaction at an appreciable rate. Figure 10 shows an example: There was very little protection for the principal reinforcement in the spring beam defining one edge of the northern access stairs when originally constructed. The subsequent closing in of the underside of the stairs with a sheet-clad timber-frame without an adequate flashing detail of the junction has allowed the underside of the beam to stay wet, producing severe corrosion.

Figure 10. Spall on the inner spring beam of the northern stand access stairs.

The outer face of the cast in-situ drainage channels that cap the tops of the north and south ends of the stadium also displays several small spalls related to issues of low concrete quality and cover.

Exposure of reinforcing bars within sound concrete on these elevations produced results similar to those illustrated by Figure 8, i.e. superficial active corrosion indicating that the reinforcement is depassivated either through chloride contamination or carbonation but does not exist in an environment aggressive enough to promote appreciable rates of corrosion. This relatively benign condition is attributed to the plaster render and decorative paint coating on the walls assisting to maintain the concrete in a reasonably dry state.

3.2.3 Bleachers (Terraced Seating)

It was estimated that 25 – 30% of the exterior faces of the riser beams used to construct the tiered seating area are affected by low cover stirrups which are often effectively exposed on the surface of the concrete (Figure 11(i)). Electromagnetic cover-meter measurements made on a population of 71 stirrups indicated a median cover of 16 mm. This compares with a measured concrete carbonation depth of 35 mm (Appendix A) leaving both the stirrups and the confined principal reinforcement susceptible to corrosion. This deficiency, compounded by further examples of poor concrete consolidation, is reflected in a number of spalls and existing patch repairs to the fronts and corners of the bleacher terraces and steps, as illustrated by the examples shown in Figure 11. Although this is an irritation rather than a serious structural issue, ongoing and increasing expenditure to maintain the condition of these elements can be expected. Fortunately the relatively dry environment provided by the covering roof serves to reduce the rate of corrosion of the affected bars. Nevertheless, section losses of up to 30% were observed on the most severely corroded reinforcement, as can be seen in Figure 12.

(i)

(ii)

(iii)

(iv)

Figure 11. Minor but ubiquitous examples of concrete defects developed on the exposed external faces of the bleachers formed by the riser beams, due primarily to sub-standard concrete cover depths.

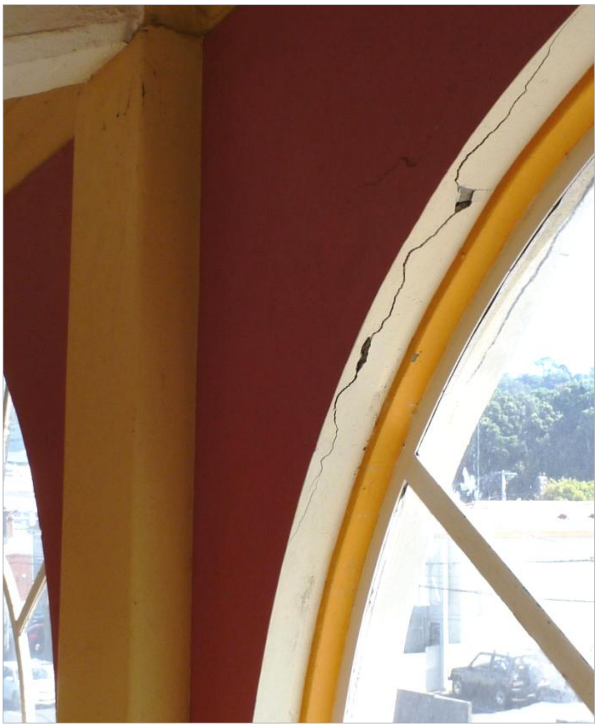
Figure 12. Corrosion along the top of bleacher riser beams as a result of a combination of poor concrete quality, consequent carbonation, and insufficient cover. Section loss on the principal reinforcement estimated as 30%.

3.3 Interior Concrete Condition

The interior of the Museum Stand houses the Cricket Museum and associated archives in the Basement (Basin Reserve level), dressing rooms, toilets and storage space on the Ground Floor (Sussex Street level), and various small rooms along the Distributing Gallery (1^{st} floor level) that provides interior access to the bleachers.

As might be expected in an enclosed and predominantly dry interior environment, the condition of the concrete elements is generally good within the interior of the stand. The most striking spalls occur on the window surrounds in the southern half of the Distributing Gallery along the wall facing Sussex Street (Figure 13). These defects exactly replicate those observed on the exterior façade, i.e. they arise as a result of water leakage around window frames allowing corrosion propagation on reinforcement that has become depassivated because it is embedded in low quality/cover concrete with little resistance to carbonation. The severity of corrosion on the reinforcement is somewhat less in these spalls than on the exterior, with section loss on one 5/8″ diameter bar exposed in the worst-case estimated at 30% (Figure 14).

6th bay



7th bay

Figure 13. Examples of spalling at window reveals along the Distributing Gallery wall at the 6th & 7th window bays.

Figure 14. Exposed reinforcement responsible for spall in Figure 13(i). Cover to the $^5/_8$″ dia. vertical bar is 47 mm (including plaster layer) but the concrete is rubbly and deficient in cement paste. The purple colour is due to phenolphthalein indicator.

The most common defects visible on the interior are small spalls to the soffits and bottom edges of the riser beams and to the secondary beams supporting the Distributing Gallery floor. Representative examples of these spalls, a large number of which have been previously addressed by patch repair, are shown in Figure 15 & Figure 16 respectively.

Figure 15. Examples of developing and patch repaired spalls on the undersides of riser beams. The spalls typically occur along the lines of shrinkage cracks or construction joints in the concrete that permit water entry from the exterior terraced seating.

Figure 16. Spalling of the secondary beams that support the Distributing Gallery floor.

In terms of concrete quality and cover on these elements, there is no distinction between those areas hosting spalls and those in which the concrete is currently sound. In both cases, covers tend to be low (15 – 30 mm typical) and the carbonation front has progressed beyond the reinforcement depth. Reinforcement exposed by exploratory breakout in sound concrete typically displays evidence of depassivation, with superficial active corrosion visible, as illustrated by Figure 17. It is probable that whether a spall develops is simply controlled by water availability to drive corrosion; many spalls on the riser beams are aligned with stained shrinkage cracks that suggests percolation of water from the exterior terraces.

No deterioration was noted on the principal structural elements within the interior, i.e. the columns, stringer beams and main girders. This is attributable to their generally more carefully controlled and deeper reinforcement covers. However the maximum carbonation depths measured on these elements exceeds the minimum recorded covers and is generally comparable to the mean cover over reinforcement, rendering it theoretically vulnerable to corrosion.

At the basement level, leakage of water into the Museum archive room through the below Sussex Street wall is a known problem for humidity control and document storage. In the interest of avoiding unnecessary disruption to Museum operations, destructive investigation was not undertaken in this area. However, the presence of water is clearly an undesirable risk for accelerating corrosion where the preconditions for reinforcement depassivation already exist.

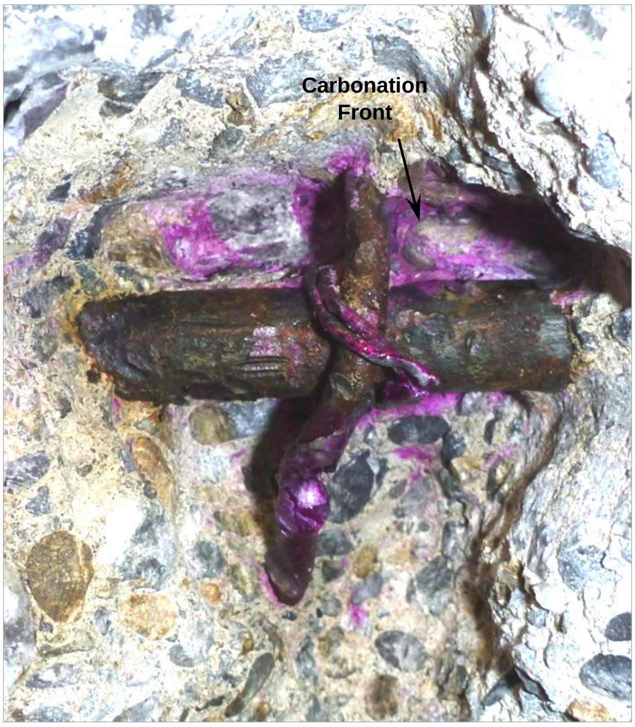


Figure 17. Exploratory breakout of riser beams in good condition showing main tension steel on bottom edge of beam (left) and stirrup in vertical face (right), both displaying superficial active corrosion. In both cases the concrete has been sprayed with a phenolphthalein indicator to determine carbonation depth. Note that the region of protective alkaline concrete signified by the purple staining commences substantially behind the bar. The hole in the right hand image needed to be drilled to 70 mm before uncarbonated concrete was encountered.

3.4 Carbonation & Chloride Measurements

The full suite of results from the instrumented site and laboratory testing undertaken to determine current and future corrosion risk are tabulated as Appendix A. For convenience of interpretation, the current progress of the carbonation front and extent of chloride ion ingress relative to the depth of concrete cover are illustrated graphically in Figure 18 (exterior test site locations) & Figure 19 (interior elements).

The chloride profiles (measured concentration vs. the midpoint of the sampling depth increment) are plotted in blue. The orange and red dashed horizontal lines on the plots indicate the UK Concrete Society's[1] suggested chloride concentration thresholds for various probabilities of the onset of corrosion and the grey and black dashed vertical lines represent the minimum and median reinforcement covers recorded at each site. Chloride contamination levels that lie above the risk threshold at or beyond the depth of protective concrete cover indicate an ongoing risk of reinforcement corrosion. The position of the carbonation front relative to the concrete cover over the reinforcement is illustrated by the position of the solid green vertical bar annotated with the label 'C.F.'. Where the carbonation front has progressed as far as, or beyond, the reinforcement it lies in alkali-depleted concrete and is consequently vulnerable to corrosion.

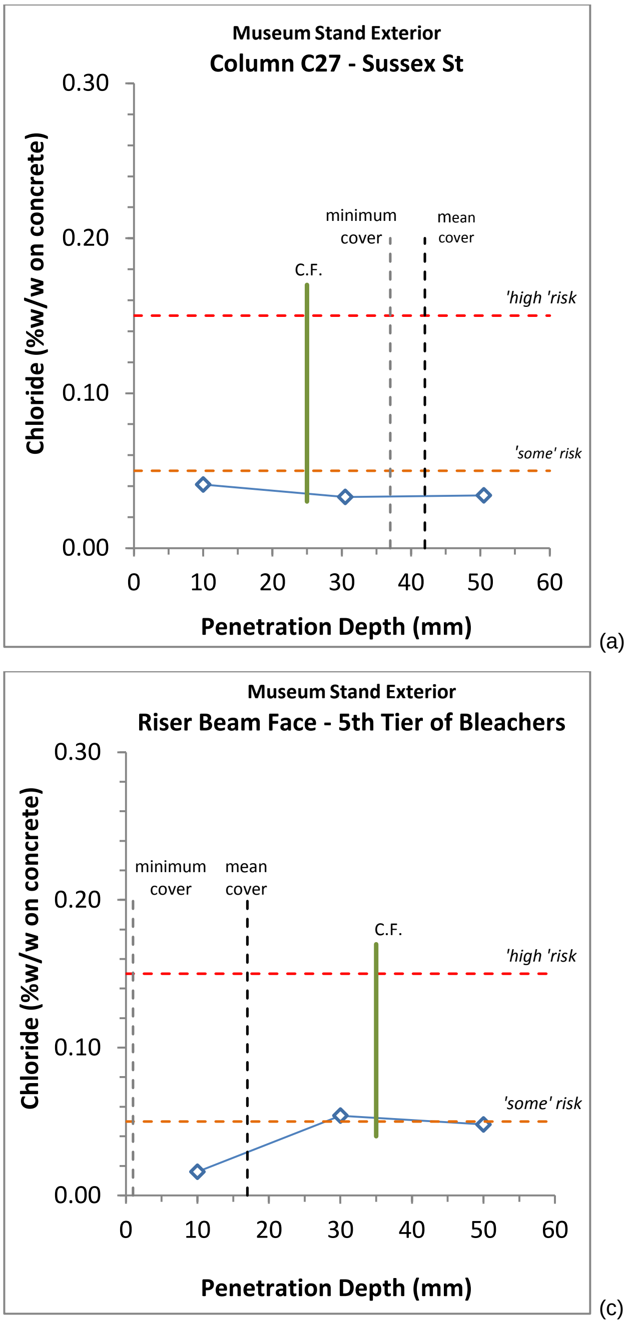


Figure 18. Chloride contamination profiles, depth of reinforcement cover and position of the carbonation front for selected elements on the exterior façades of the Museum Stand.

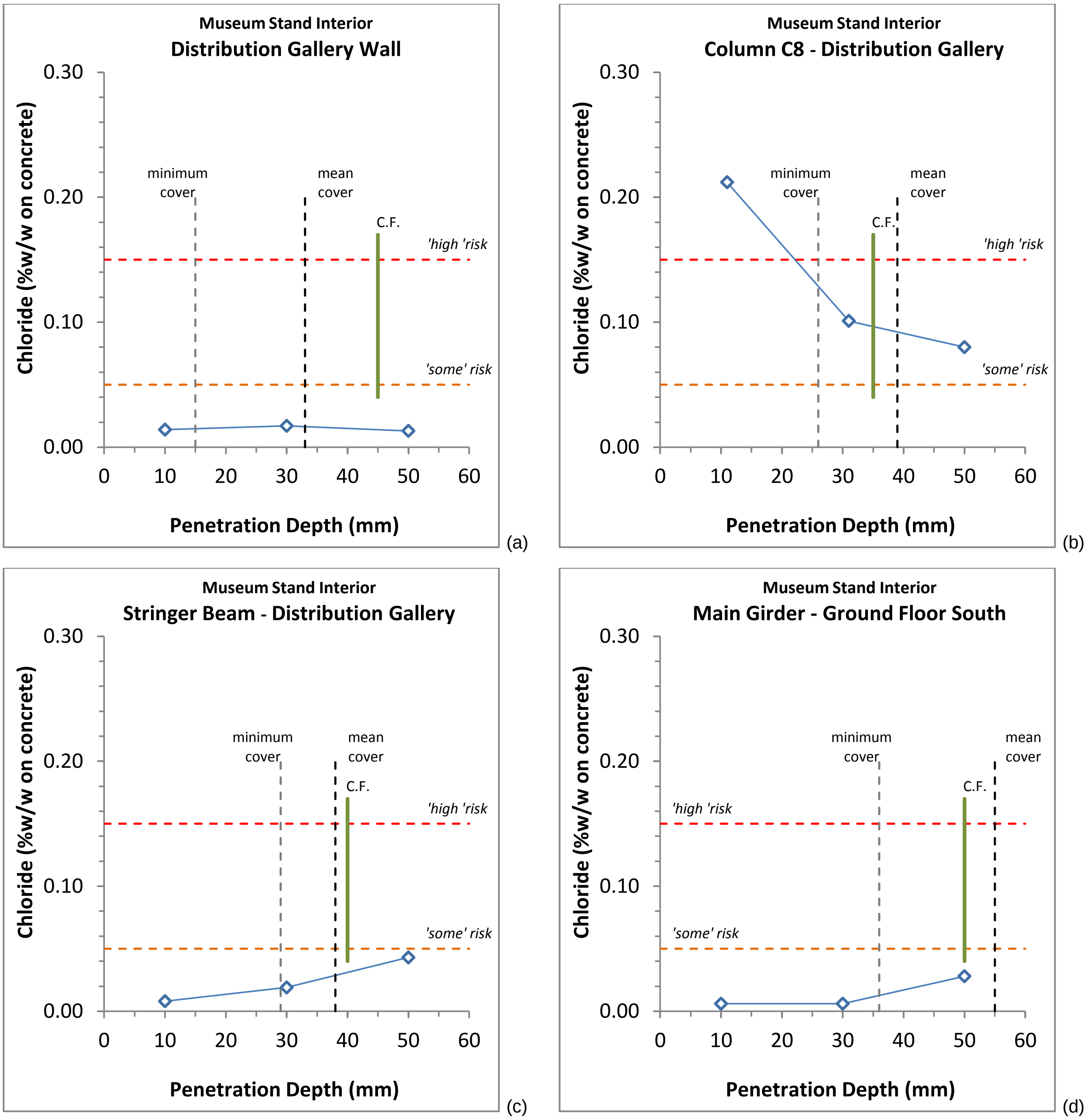


Figure 19. Chloride contamination profiles, depth of reinforcement cover and position of the carbonation front for selected elements within the interior of the Museum Stand.

Two trends are apparent from the corrosion risk plots:

- Carbonation of the concrete is well-advanced. The measured penetration front has typically progressed beyond the depth of the reinforcement with the minimum cover recorded in the vicinity of the test site. In many cases, the carbonation of the concrete is sufficiently developed to threaten depassivation of bars with average or greater covers, explaining the preponderance of broken out bars in sound concrete displaying some level of active corrosion.

- The chloride profiles display the erratic levels and absence of a clearly defined reduction in concentration with depth that are typically observed to result from a cast-in source, such as use of a marine aggregate, rather than environmental exposure. The chloride levels at all depths from the surface are elevated above normal baseline values. Particularly telling in this regard is the profile for Column C8 [Figure 19(ii)], which is not exposed to the elements but possesses chloride levels ordinarily associated with structures in aggressive marine environments.

Extensive carbonation is consistent with the low concrete strengths reported in section 3.1 and the general environment in which the Museum Stand exists. While carbonation is frequently modelled as a diffusive process, evidence suggests that the principal material factor controlling the rate is actually the chemical buffering capacity of concrete.[7] This buffering ability is directly proportional to the quantity of cement used in the mix design and thus would be expected to be minimal in weak concrete.

Furthermore, the initial step of the carbonation mechanism involves dissolution of CO_2 into the capillary pore water of the concrete to form bicarbonate anions, which in-turn react with the calcium hydroxide and calcium silicate hydrate gel that form the cement binder Consequently, the moisture content of the concrete is also highly influential to the level of carbonation developed: If the concrete is fully saturated, its permeability to CO_2 is hindered but, conversely, at low levels of R.H. (relative humidity), little bicarbonate anion can be formed.

The maximum carbonation rate therefore occurs when the internal relative humidity of the concrete is in the range of 50 – 70%. Drying of concrete to equilibrium with this moisture content tends to be favoured by exposure to dry interior environments or where exterior concrete is protected by vapour-permeable decorative finishes such as plaster and paint.

The chloride contamination of the concrete is generally at levels that would be considered tolerable, particular where the concrete remains dry. However in combination with the advanced carbonation, the susceptibility to corrosion may be higher than the conventional risk thresholds plotted on Figure 18 & Figure 19 indicate: It is generally accepted that the corrosion risk is controlled by both the chloride and hydroxide ion concentrations within the pore fluid of concrete, with mild steel reinforcement vulnerable under conditions where $[Cl^-]/[OH^-] > 0.6$.[8]

Due to the absence of a convenient measurement technique and the fact that hydroxide ion concentration is, to a first approximation, constant in uncarbonated concrete, corrosion risk thresholds are ordinarily formulated in terms of chlorides alone. By definition, carbonated concrete is depleted of hydroxide ions, thus increasing the $[Cl^-]/[OH^-]$ ratio and hence the intrinsic corrosion risk at any given level of chloride contamination. The situation is rarely encountered since environments conducive to environmental chloride ingress are seldom also favourable for carbonation. The carbonation reaction also decomposes the C_3A (tri-calcium aluminate) phase in cement that ordinarily immobilises a certain fraction of the chloride ions, liberating them to take participate in corrosion reactions.

3.5 Structural Steelwork

3.5.1 Main Roof Columns

The Museum Stand canopy and allied structural steelwork was replaced in 1984. The roof is supported along the Eastern elevation by four lattice work columns. Each column is constructed from four vertical angle section steel corners braced on all sides by welding on diagonal flat plates using a 6 mm radius fillet weld. These columns were inspected with a cherry picker to ascertain the integrity of the welds and, as far as could be observed, the connection of the columns into the roof trusses.

All of the welds were observed to be structurally sound, with no visual evidence of cracks developing. The metal work on the columns has been hot-dip galvanised and is predominantly free of corrosion. However, the condition of the organic coating protecting the galvanising is poor, especially on the two outer columns. Blisters and peeling failures are developing on the arriss edges of the steel where pinholes through the coating are likely and the abrupt changes in direction place most demand on coating flexibility. Figure 20 illustrates a typical example of coating failure on the columns.

In most cases, the material underlying the blisters is 'white rust' (i.e. soluble hydrated zinc hydroxides), as shown in Figure 21. This suggests penetration of chlorides from marine aerosols through the coating defects, but also indicates that the galvanising is continuing to act as a sacrificial anode to prevent corrosion of the structural steel (and welds). However without the protection of an intact organic coating, the galvanic layer will be rapidly consumed, allowing corrosion to penetrate through to the base steel. In the worst cases, this has already occurred, as witnessed by the presence of underlying red rust (ferrous corrosion) when the blisters are chipped off with a hammer (Figure 22).

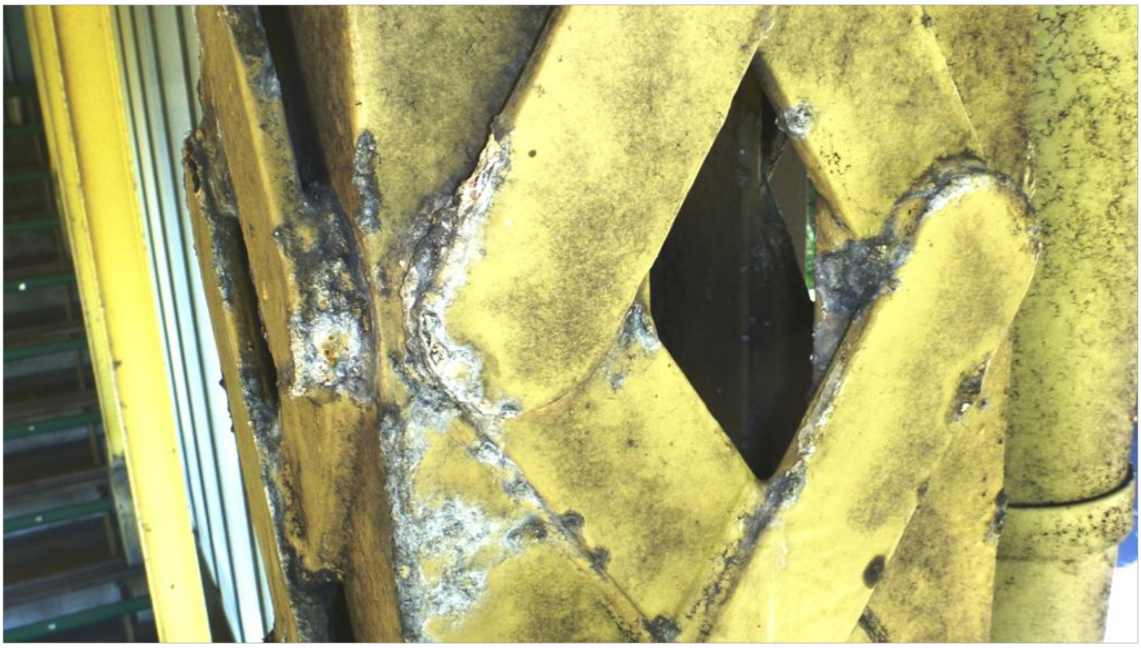
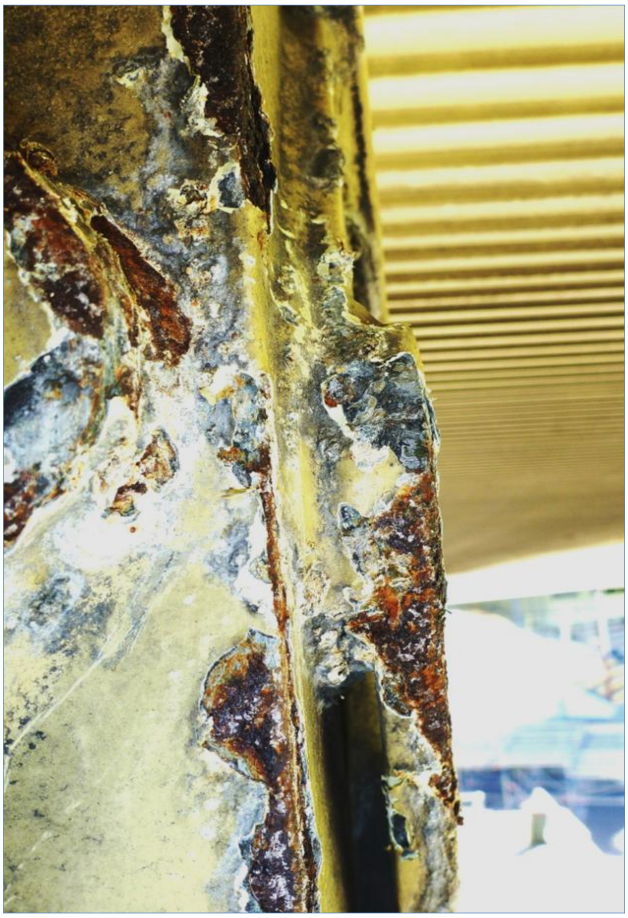


Figure 20. Typical blistering to the protective coating on the edges of the steel work on the main roof support columns (Northern end column shown).

Figure 21. 'White rust' under paint blister, indicating rapid consumption of the hot-dip galvanising that protects the underlying weld and base steel against corrosion.



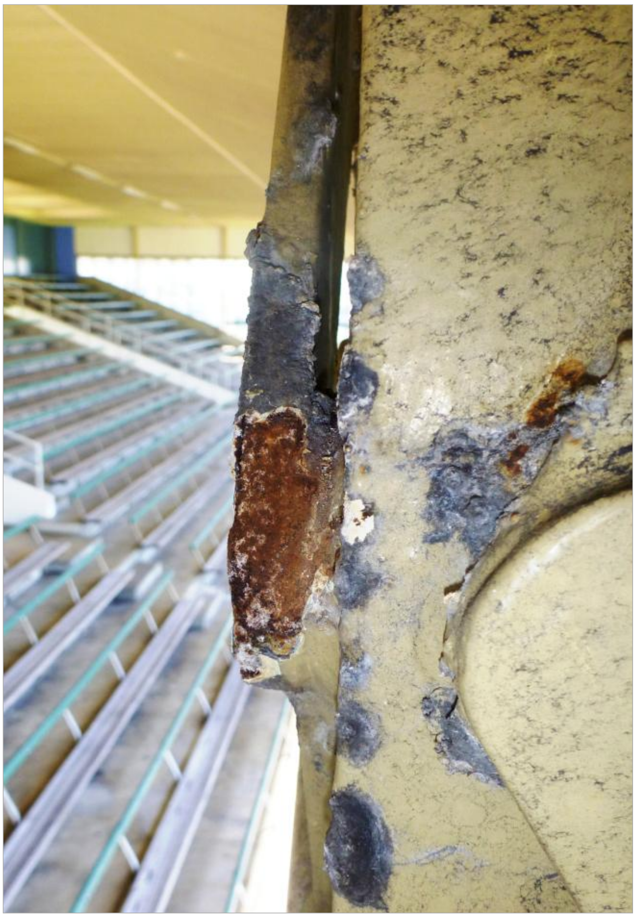


Figure 22 Corrosion of base steel observed after chipping away coating in more heavily deteriorating areas.

The bases of the lattice columns are embedded in concrete pedestals atop columns extending from the main structure of the Stand. It is generally recommended that ferrous or galvanised metals protruding from concrete in this fashion are provided with an epoxy or similar protective coating over the first 400 mm of the embedded length. This measure guards against corrosion cells forming in response to the sudden change in electrochemical potential that occurs at the surface of the concrete.

Such protection has not been provided to the lattice columns and, as a result, some pitting corrosion has developed on the embedded steel angles. Investigation of the southern column, which appears to be the worst affected, suggests the pitting is confined to a band about 10 mm deep immediately below the surface of the pedestal (Figure 23). The pits are up to 2 mm deep; the angle itself is formed from 10 mm thick steel.

Figure 23. Narrow band of pitting corrosion present on column angles where they are embedded in a concrete pedestal. The arrowed pit is approximately 2 mm deep.

3.5.2 Wall End Columns

The secondary roof support columns along the north and south elevation sloped concrete end walls are formed from 178x89x22 mm RSJs. These are bolted through a welded foot plate into the inclined tops of concrete columns. All of these columns (3 per wall) are badly spalled in the immediate vicinity of this connection; Figure 24 shows an example. Note that the nut on the embedded masonry fastener is very badly corroded and also that the column is only bolted through the 'upslope' side of the foot plate shown (refer to Figure 25 for a view of the same pedestal from the seating tier below).

Exploratory breakout of these spalls reveals they result from corrosion of the remnants of the original RSJ roof supports that were replaced in 1984, and their connection to the top of the concrete column. Figure 25 demonstrates some of the corroding steel work contributing to the column spalling. It appears that the first RSJ has been cut off flush with the top of the concrete column and the new RSJ installed over the top of it in a bed of dry-pack mortar. No attempt has been made to waterproof the top of the column and corrosion of the redundant embedded steel has occured.

Figure 24. Characteristic example of spalling to concrete column pedestal at connection with secondary roof support column. The bolt and nut fixing the column are heavily corroded.

Figure 25. Exploratory breakout of the pedestal showing remnants of the original RSJ and connection to the structural concrete column. Note that the new RSJ is not bolted on this side.

3.6 Roof Soffit Lining

The roof soffit ceiling lining is in poor condition. On the exposed eaves, particularly those extending outward from the north and south walls, the protective coating has failed completely and perforations through the underlying steel substrate have developing (Figure 26). The heads of the fasteners are heavily corroded in places and have limited remaining fixing strength.

Figure 26. Corrosion of roof soffit lining – southern eave.

Access was not available to inspect the main roof cladding. It is likely to be in better condition than the soffit lining because it has the advantage of rain washing to remove dirt and corrosive contaminants. However, the roof is now 28 years old. This is approaching twice the minimum mandatory life for a non-structural component of the building envelope under the NZ Building Code[9] and it is reasonable to assume the residual service life of the cladding is limited.

4 Estimation of Residual Life

Note: The following discussion pertains to the reinforced concrete elements of the structure. It is assumed that the foreseeable residual life of the structural steelwork is solely reliant on an adequate level of ongoing maintenance to protective coatings.

The Museum Stand was constructed in 1924 and was therefore approximately 88 years old at the time of inspection. The original design life is unknown, but NZS 3101:2006[6] provides a good basis for estimating the potential life of concrete structures. For different exposure environments, NZS 3101 defines the requirements for concrete cover depth, concrete compressive strength, and cementitious binder type for specified intended lives of 50 and 100 years. In current practice, a design life of 100 years is commonly used for infrastructural assets.

A naïve interpretation of NZS 3101 might therefore suggest that the Stand has a maximum of approximately 12 years of residual life. However the Standard's design solutions are intended to be reasonably conservative, i.e. they were developed using relatively aggressive cases for each exposure environment. They also consider service life to end shortly after conditions permit corrosion of the reinforcement to commence, rather than the later time when structural damage has occurred and the asset is no longer fit for purpose. Furthermore, the solutions are predicated on a requirement for carrying out routine maintenance only and discount the possibility of reconstruction or major renovation.

A fairer estimation of residual life therefore needs to be made by considering the durability issues identified in this report in combination with the current age and physical condition of the structure.

Reinforcing steel in concrete is ordinarily surrounded by a tightly-adhered protective oxide film that is created by the highly alkaline nature of Portland cement. While this protective oxide film remains intact, the reinforcement is in a passive state and does not corrode. However, the integrity of the oxide film can be depassivated (disrupted) by chloride ions, or by a reduction in concrete alkalinity through carbonation.

Once these necessary preconditions for depassivation are established, corrosion will commence in the presence of oxygen and water. Moisture itself is not detrimental to reinforced concrete per se but, following depassivation, it is an important practical control on the corrosion rate and hence the speed at which structurally significant loss of section on the reinforcement and concrete spalling develop.

Reinforced concrete deterioration thus proceed by two steps, involving *initiation* of corrosion by carbonation or chloride ingress followed by *propagation* of damage of increasing severity as the corrosion progresses. Because of these distinct phases, deterioration is often visualised by the simple model presented in Figure 27. Note that time axis on this diagram is not intended as quantitative: there is no expectation that the propagation period is of equivalent or longer duration than the initiation period and it can frequently prove to be very much shorter.

The reinforcement on nearly all of the concrete elements investigated on the Museum Stand is vulnerable to carbonation and, due to the apparent use of marine aggregate, the corrosion risk is heightened by hotspots of elevated chloride contamination.

On this basis, the structure as whole can be considered to have reached at least the second serviceability limit state at T_2 identified on Figure 27: The duration of the initiation period has clearly been exceeded and corrosion is established on the reinforcement, albeit not necessarily at a uniformly high rate.

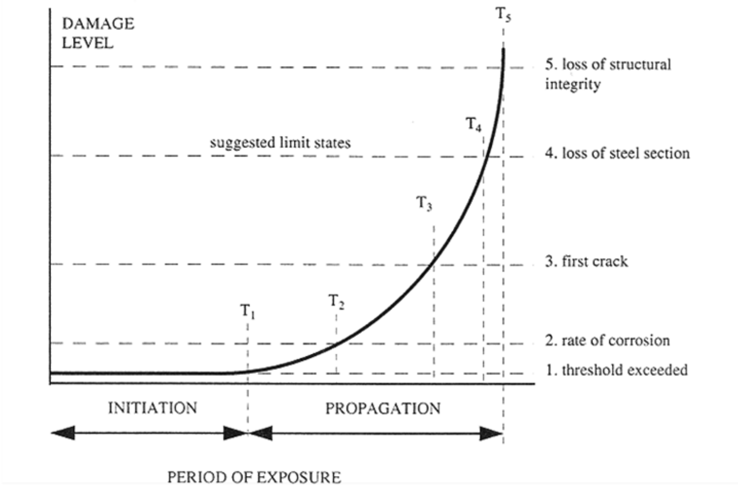


Figure 27. Simple reinforcement corrosion deterioration model and suggested serviceability limit states.[10]

There are some elements, notably the window surrounds on the Sussex Street façade and the riser beams for the seating terraces, where the propagation period is extremely well-advanced with loss of steel section and, in severe cases, structural integrity, equating to a late period T_4 or T_5 limit state. However, all the significant deterioration observed is explainable in terms of specific construction defects combined with environmental conditions that are not universal to the structure. For example, the columns, stringer beams and main girders are less vulnerable and appear to be in generally good condition.

It is difficult to predict how quickly these key structural elements may progress through the propagation stage to a point where they are unfit for purpose because of the detailed information that would be necessary to fully characterise the corrosion process. Even if a particular corrosion rate is assumed, the time to a certain level of damage will be highly variable due to the influence of factors such as localised concrete quality, element geometries and restraints on cracking.

The main structural elements in the Stand largely exist within a protected interior environment. In combination with carbonation as the key driver of reinforcement depassivation this is beneficial because it implies a relatively lengthy propagation period. Experimental evidence suggests that corrosion rates in carbonated concrete are at a peak in situations when the internal humidity within the concrete is around 90 – 95% R.H. In drier conditions corrosion is reduced, to the point of being effectively suppressed with rates of less than 0.002 $\mu A/cm^2$ where the humidity is less than 80%, as shown in Figure 28. It is important to note however that the R.H. experienced by the reinforcing steel is not simply related to atmospheric conditions and that water run-off, condensation, solar heat gain, thermal mass of the concrete and other factors may generate quite different conditions within the concrete to the ambient environment.

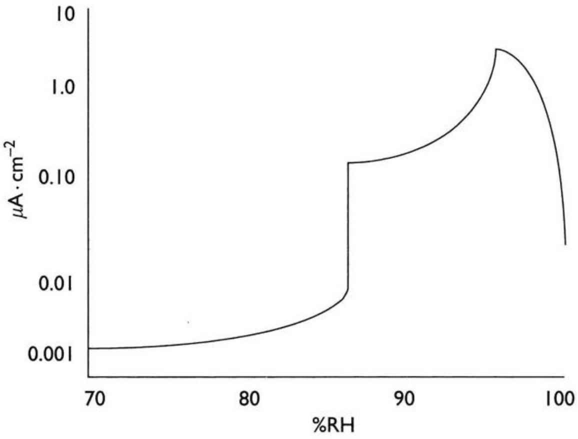


Figure 28. Variation of corrosion rate in carbonated concrete as a function of internal relative humidity.[8]

Some caution needs to be applied to a literal interpretation of Figure 28 to the Museum Stand because of the potential and largely unquantifiable contribution of the cast-in chloride contamination: Corrosion rates of chloride-contaminated concrete can be three orders of magnitude greater than that of carbonated concrete at 80% R.H., partly as a result of the deliquescent nature of the salt. However, on the basis of current condition, the assumption that existing defects are repaired, and that a strategy is adopted to maintain the structure in a dry condition, a residual life for the Museum Stand on the order of 25 to 30 years does not seem unreasonable.

5 Conclusions & Recommendations for Future Action

The Museum Stand is 88 years old and is in a state commensurate with its relatively advanced age. The concrete used in construction is not of high quality, with a mean measured compressive strength of 24 MPa, and there are some clear construction deficiencies with regard to consolidation during placement and achieved covers. Carbonation of the concrete is well-established and it is probable that near-systemic reinforcement depassivation exists throughout the structure, with a consequent risk of reinforcement corrosion wherever moisture is available. This threat is elevated by variable levels of cast-in chloride contamination introduced through the use of a marine-sourced aggregate in the concrete mix.

Despite these negative characteristics much of the concrete, including the columns, main girders and stringer beams are in good condition and do not display obvious cracking or spalls that would compromise their functional performance. This is attributed to the majority of these elements being located within the protected interior of the stand, where predominantly dry conditions effectively suppress the rate of reinforcement corrosion, irrespective of the actual passivation state of the bars. Serious deterioration of the structure is primarily restricted to the Sussex Street façade where water leakage through corroding metal window frames, in combination with poor covers and compaction defects, has produced severely compromised reinforcement and significant concrete spalling. The riser beams that form the tiered seating also show numerous spalling defects, both on the exposed exterior and interior faces, as a result of low covers. However, this deterioration is largely a cosmetic issue and an ongoing maintenance nuisance, rather than a threat to the integrity of the structure.

The Museum Stand roof and supporting structural steelwork were reconstructed in 1984 but are again in need of major maintenance. In particular, the roof lining and fasteners are badly corroding and need replacement and this is also likely to be true of the roof cladding. The protective coating on the steel lattice columns supporting the front of the roof is in poor condition and requires remediation. It is blistering in locations where pin-holes and other defects have allowed salt-laden aerosols to penetrate. This is resulting in rapid dissolution of the galvanising and consequent corrosion of the base steel and welded joints in the worst-affected areas. Also in need of addressing are the structural deficiencies created by the deteriorating connection between the secondary roof support beams and the concrete columns on which they terminate.

On the basis of the observed concrete condition, age and current deterioration, it is suggested that the Museum Stand has a residual life of approximately 25 to 30 years, if appropriate maintenance is carried out. To achieve this life, the following immediate remedial work is recommended. Note that this list is presented as guidance regarding the likely quantum of work required; it is not intended to be a definitive scope or specification.

- Remove the window frames from the Sussex Street elevation. Reconstruct the walls between existing columns as necessary to ensure that all steel showing loss of section is replaced, and recast in new concrete.

- Excavate the Sussex Street elevation below grade and install a tanking membrane, or other suitable waterproofing system, to eliminate leakage through the concrete wall to the interior of the Stand.

- Repair any spalls to the other elevations using conventional concrete patch repair techniques. Remove any areas of unsound or drummy exterior plaster and reinstate.

- Reconstruct the kerb beam of the northern stand access stairs using conventional patch repair.

- Once repairs of any spalling or unsound plaster are completed, waterproof the exterior of the Stand (i.e. the currently plastered and painted surfaces). This should be achieved by application of a high quality vapour-permeable elastomeric coating, specifically formulated for crack accommodation in concrete and masonry structures, with a finished dry film thickness in excess of 400 μm.

- Address spalls to the riser beams on both the exterior tiered seating surface and underside in the interior of the Stand using conventional patch repairs techniques.

- Apply an octyl triethoxy silane-based impregnant to the exterior unpainted surfaces of the tiered seating to provide a hydrophobic (water-repellent) surface. The product selected should contain >75% active ingredient, formulated as an aqueous thixotropic cream, and comply with the requirements for a class 4 impregnant under APAS Specification AP-S0168 or be of equivalent quality.

- Replace the metal roof cladding and soffit linings.

- Address the coating deficiencies on the roof columns. In outline, this necessitates grit blasting of red-rusted and blistered areas to remove salt-contaminated corrosion product, patch priming the cleaned areas with a 50 μm thick build of >85% w/w zinc-impregnated epoxy, and then over-coating with a 250 μm epoxy barrier, prior to application of any decorative finish coat. Areas of the existing protective coating with low build thickness, pin-holing, holidays or other defects should also be over-coated with the barrier epoxy. Note that the nature of the existing coating system will need to be confirmed prior to commencing this remediation to avoid potential compatibility issues.

- Breakout the bases of the lattice work columns, where they are embedded into concrete pedestals, blast clean and epoxy coat the steel for 400 mm below the concrete surface to inhibit further corrosion, and reconstruct the pedestals.

- Reconstruct the tops of the concrete columns on the north and south elevation wall ends and provide a structurally-sound detail for connection of the secondary roof columns.

With regard to maintenance of the concrete, the emphasis is placed upon preserving it in a sufficiently dry state to minimise the corrosion rate of the reinforcement, hence the preference for a high quality external coating. The sole exception to this is the exposed exterior surfaces of the bleachers where, for functional reasons, any coating would need to be trafficable (i.e. sufficiently abrasion resistant to withstand the wear associated with being walked upon). Suitable heavy fibrereinforced membranes run the risk of failing to bond adequately to the concrete allowing water to accumulate beneath them. This is a very dangerous situation because severe and rapid pitting of the reinforcement is then able to occur without any tell-tale cracking or spalling of the concrete because the normally expansive corrosion product stays in solution. For this reason the silane treatment recommended is strongly preferred. It also has the advantage that it does not change the surface appearance of the concrete and no further maintenance is required beyond reapplication at approximately 5 – 10 year intervals to preserve its function.

The most cost-effective and technically sound repair strategy for spalling concrete is considered to be conventional best practice-patch repair combined with regular monitoring of the structure to allow timely repair of newly developing defects. Patch repair involves the removal of unsound concrete at visibly obvious locations, such as spalls and cracks, to a depth of at least 20 mm behind the reinforcement to eliminate carbonated or chloride-contaminated concrete. After replacing or treating the steel as appropriate to the level of corrosion damage, the concrete is reinstated by hand-application, casting or spraying, depending on the volume and geometry of the element being repaired. Best practice for conventional patch repairs indicates hydro-demolition for removal of the concrete, Portland cement-based repair materials (from reputable manufacturers) to restore alkalinity and the provision of sacrificial zinc anodes to guard against incipient anode corrosion cells developing adjacent to the newly-completed repairs.

Installation of an impressed current cathodic protection system in response to the systemic reinforcement depassivation has been considered and rejected as both unnecessary and impractical. Aside from potential technical issues concerning the electrical resistivity of carbonated and sometimes badly consolidated concrete, and the possibly limited electrical conductivity of reinforcement, this is a costly remediation that requires commitment to ongoing monitoring at a moderate level of technical sophistication to remain effective. Moreover, it should again be emphasised that the major structural elements of the Stand, although theoretical vulnerable to corrosion, are not currently showing signs of distress.

Similar considerations (i.e. cost, complexity and limited potential benefit) rule out electrochemical re-alkalinisation or chloride extraction as viable remedial options.

6 References

[1] U.K. Concrete Society. 1984. *Repair of Concrete Damaged by Reinforcement Corrosion*. Technical Report 27. Camberley, Surrey, United Kingdom.

[2] Rilem Recommendation CPC-18. 1998. 'Measurement of hardened concrete carbonation depth'. *Materials & Structures* 21 no 6, pp 453 – 455, November 1998

[3] Standards New Zealand. 1986. NZS 3112: Part 2 *Tests Relating to the Determination of Strength of Concrete*. Wellington.

[4] ASTM International. 2010. ASTM C42 / C42M - 10a *Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete*. Philadelphia.

[5] Concrete Institute of Australia. 2002. *The Evaluation of Concrete Strength by Testing Cores*. Recommended Practice Z11. Crow's Nest, NSW, Australia.

[6] Standards New Zealand. NZS 3101:2006 *Concrete Structures Standard: Part 1 – The Design of Concrete Structures & Part 2 – Commentary*. Wellington, New Zealand.

[7] U.K. Concrete Society. 2004. *Enhanced Reinforced Concrete Durability. Guidance on Selecting Measures for minimising the Risk of Corrosion of Reinforcement in Concrete*. Technical Report 61. Camberley, Surrey, United Kingdom.

[8] Broomfield, J.P. 2007. *Corrosion of Steel in Concrete: Understanding, Investigation & Repair*. 2nd Edition. Taylor & Francis, United Kingdom.

[9] Department of Building and Housing. 2011. Compliance Document for the New Zealand Building Code. Clause B2 – Durability. Wellington, New Zealand.

[10] Bamforth, P. 1999. 'A New Approach to Durability Design Using Risk Analysis', in *Report of the Seventh Annual Durability by Intent Worksho*p. Building Research Establishment, Watford, United Kingdom.

Appendix A: Summary of Instrumented Durability Test Results

Sampling Location	Depth of Concrete Cover[1] (mm)			Carbonation Depth	Chloride Ingress	
	Mean	Minimum	Maximum	Maximum (mm)	Depth from Surface (mm)	Concentration (%w/w on concrete)
Sussex Street Elevation **Column C27**	42	37	56	25	0 – 20 20 – 41 41 – 60	0.041 0.033 0.034
Sussex Street Elevation **Wall panel above window at Bay 2**	48	35	64	47	Not determined	
North Elevation **Wall 1.5 m above ground level**	49	32	63	50	0 – 22 22 – 40 40 – 60	0.075 0.036 0.028
Bleachers (5th Tier) **Front Face of Riser Beam**	17	1	44	35	0 – 20 20 – 40 40 – 60	0.016 0.054 0.048
North Access Stairs **Spalling Kerb Beam**	25	20[2]	80[3]	10	0 – 22 22 – 40 40 – 62	0.027 0.031 0.026

[1] Includes thickness of plaster render where present on exterior surfaces; [2] Measured from beam soffit; [3] Measured from vertical face.

Appendix A (continued).

<table>
<tr><th rowspan="2">Sampling Location</th><th colspan="3">Depth of Concrete Cover (mm)</th><th>Carbonation Depth</th><th colspan="2">Chloride Ingress</th></tr>
<tr><th>Mean</th><th>Minimum</th><th>Maximum</th><th>Maximum (mm)</th><th>Depth from Surface (mm)</th><th>Concentration (%w/w on concrete)</th></tr>
<tr><td>Distribution Gallery Interior Wall
Bay 6</td><td>33</td><td>15</td><td>39</td><td>45</td><td>0 – 20
20 – 40
40 – 60</td><td>0.014
0.017
0.013</td></tr>
<tr><td>Interior Column
Column C8; Distribution Gallery</td><td>39</td><td>26</td><td>48</td><td>35</td><td>0 – 22
22 – 40
40 – 60</td><td>0.212
0.101
0.080</td></tr>
<tr><td>Stringer Beam
1st beam north of stairwell; Distribution Gallery</td><td>38</td><td>29</td><td>69</td><td>40</td><td>0 – 20
20 – 40
40 – 60</td><td>0.008
0.019
0.043</td></tr>
<tr><td>Main Girder
Ground floor; South end</td><td>55</td><td>36</td><td>72</td><td>50</td><td>0 – 20
20 – 40
40 – 60</td><td>0.006
0.006
0.028</td></tr>
<tr><td>Riser Beam Soffit
Ground floor; South end</td><td>20</td><td>15</td><td>27</td><td>30</td><td colspan="2">Not determined</td></tr>
<tr><td>Riser Beam Interior Face
Ground floor; South end</td><td>33</td><td>15</td><td>61</td><td>70</td><td colspan="2">Not determined</td></tr>
</table>

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