# In Situ Assessments of Long-term Performance of Plain and Blended Cement Concretes

D. BAWEJA
Senior Experimental Scientist, CSIRO, Sydney
R.L. MUNN
Chief Civil Engineer, Boral Research, Sydney
H. ROPER
Associate Professor, School of Civil and Mining Engineering, University of Sydney
V. SIRIVIVATNANON
Principal Research Scientist, CSIRO, Sydney

**SUMMARY** Between 1987 and 1990, selected elements from ten individual structures from within four facilities were examined in order to assess the long-term performance of concretes used. The four facilities were the Prospect Nitrate Store, Northaven Retirement Village, Port Kembla Steelworks and Vales Point Power Station. Structural members examined encompassed slabs-on-grade, suspended slabs and wharf elements. Concretes forming these had been in service for between 10 and 28 years. Individual concretes forming the selected structures had binders of normal portland cement, portland cement with fly ash or ternary systems of fly ash, ground granulated blast furnace slag and portland cement. Slag aggregates were used in certain of the concretes studied. Interest focused on the long-term in-service durability of the concretes. Concrete properties were investigated and information on design and construction of the facilities was procured. Conclusions were drawn with respect to the performance of the concretes. Details relevant to future specification of fly ash, slag and ternary blend concretes were noted. It was found that durability problems affecting the structures were generally due to factors other than the binder types used.

## 1. INTRODUCTION

Despite the wealth of data available to engineers for design purposes and that available to cement chemists and technologists for materials selection, only relatively limited information is as yet available on the long-term performance of concrete materials and concrete structural elements under service conditions. Research was therefore conducted by collaborating organizations on a series of structures in order to gauge the performance of concretes designed to now superseded codes of practice. The work was specifically oriented towards assessing whether or not such structures would have remained durable if designed to comply with current standard requirements.

Much has been published on concrete durability loss through corrosion of reinforcement caused by chloride ion ingress and/or carbonation. Parrott and Killoh (1) observed on a 36 year old in situ column that carbonation ingress did not always result in the development of a well defined pH change front. This highlights some of the problems associated with the assessment of reinforced concrete durability based solely on methods of site carbonation measurement. Roper, Kirkby and Baweja (2) found that in a series of structures examined, there was no evidence to suggest that corrosion of reinforcement was more prominent in fly ash concrete than in equivalent AS 1315 Type A (plain) cement concrete. Crack development was however prevalent in all structural concretes considered in that study. Mehta (3) considered that the permeability of concrete was the most important factor for long-term durability and further indicated that higher strength concretes, generally greater than 40 MPa, showed low permeability. He further suggested that such concretes would generally contain cement together with either silica fume, fly ash or ground granulated blast furnace slag (slag). Takewaka and Mastumoto (4) report concretes having slag within their binder fractions to reduce permeability to chloride ion ingress in marine structures.

In the present work, ten separate sites from four facilities were chosen for study. Studies took place between 1987 and 1990. The structures were either slab-on-grade elements long exposed to aggressive service conditions, or wharf components in marine environments. Sites for study were selected largely on the basis of long service history, use of blended cement binders and available information on design and constructional aspects. The Prospect Nitrate Store slab was selected because it represented a typical late 1960's concrete in Sydney cast using a ternary blend binder including both slag and fly ash. The Northaven Old Peoples Home floor slabs were selected for study as they represented typical Sydney fly ash concretes cast in the late 1960's. Structures at the Port Kembla facility were of interest because the ternary blend and low heat cement concretes had been long exposed to marine service conditions. Structures at Vales Point Power Station were chosen because they represented some of the oldest fly ash concrete used in New South Wales. Structure selection at the Vales Point facility also enabled comparison studies between normal portland and fly ash concretes.

Ternary blend cement concretes typically contained fly ash and slag directly added to the mixes on site. In some cases, slag aggregates were also incorporated into mixes to go with the ternary blends. The fly ash concretes studied would have had fly ash materials batched at the mixing plant. Details of the structures studied and the concrete structural elements considered are presented in Table I.

| Site No. | Structure and Element Type | Service Condition** | Specified Grade (MPa) | Date of Completion | Coarse Agg. | Cement Content ($kg/m^3$) | Fly Ash Content ($kg/m^3$) | Slag Content ($kg/m^3$) | Water: Cement Ratio | Water: Binder Ratio |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Prospect Nitrate Store<br>(Slab-on-grade) | Industrial | 25 | 1977 | CRG* | 145 | 145 | 70 | (1.49) | (0.60) |
| 2 | Northaven Ret. Village [1]<br>(Suspended slab) | Under shelter | 20 | 1969 | Black Breccia | 293 | 69 | 0 | 0.67 | 0.54 |
| 3 | Northaven Ret. Village [2]<br>(Suspended slab) | Under shelter | 20 | 1969 | CRG* | 293 | 69 | 0 | 0.67 | 0.54 |
| 4 | Pt Kembla Wharf<br>Finished Products Berth | Salt spray | 21 | 1961 | Slag | (300) | 0 | 0 | (0.65) | (0.65) |
| 5 | Pt Kembla Wharf<br>Ore Unloading Berth | Salt spray | 21 | 1969 | 14 mm Slag | (300) | 0 | 0 | (0.65) | (0.65) |
| 6 | Pt Kembla Wharf<br>Raw Materials Berth | Salt spray | 21 | 1973 | Slag | 120 | 100 | 120 | 1.84 | 0.65 |
| 7 | Pt Kembla Wharf<br>Ro Ro Berth | Salt spray | 21 | 1973 | Slag | 120 | 100 | 120 | 1.84 | 0.65 |
| 8 | Coal Conveyor Support Slabs - Vales Point [1]<br>(Slab-on-grade) | Industrial | 20 | 1961 | Basalt | 256 | 46 | 0 | 0.71 | 0.60 |
| 9 | Coal Conveyor Support Slabs - Vales Point [2]<br>(Slab-on-grade) | Industrial | 20 | 1961 | Basalt | 247 | 62 | 0 | 0.75 | 0.60 |
| 10 | Boiler 2 Apron Slab - Vales Point<br>(Slab-on-grade) | Industrial | 20 | 1961 | Basalt | 313 | 0 | 0 | 0.65 | 0.65 |

* Crushed river gravel
** All sites are exterior environments
() Estimated value

**TABLE I** Site Information Summary - Structural Element, Service Condition, Design Concrete Grade, Construction Completion Date and Binder Components (After 6, 7, 9, 10, and 12)

## 2. DESCRIPTION, CONDITION AND INSPECTION METHODS OF STRUCTURES EXAMINED

### 2.1 Site 1 - Prospect Nitrate Store

**Description:** The structure investigated was a concrete slab-on-grade with a series of pedestals built adjacent to a nitrate storage facility. A schematic diagram of the site is given in Figure 1. The slab and pedestals acted as a foundation for a loading facility for nitrate materials used in quarry blasting applications. The slab carried heavy traffic loads through loading operations of multiple wheel trucks. Axle loads on these trucks would typically range between 8 to 10 tonnes. Top reinforcement used in the slab was square mesh with 8 mm diameter wires at 200 mm centres. The structure was built in 1977 and had 12 years of exposure to aggressive physical and chemical service conditions at the time of inspection. The exposure classification as defined by Australian Standard 3600 (5) for the concrete was assessed to comply with U specifications.

**Concrete As Used:** The concrete for the structure consisted of a ternary blend of cement, fly ash and slag. A combination of crushed river gravel and slag aggregates were incorporated into the mix. The fly ash source was from Munmorah Power Station and the slag was acquired from the Port Kembla Steelworks. Coarse aggregates and sands were sourced from Nepean River quarries.

**Assessment Methods Used:** A copper-copper sulphate half cell potential survey was carried out on the slab reinforcement. Cover meter checks were carried out and cores were extracted from various locations. Tests on core concrete included density, compressive strength, modulus of elasticity, pulse velocity, Brazilian tensile strength, absorption and carbonation depth. Core locations, half cell potential survey locations and abraded areas on the slab are shown in Figure 1.

**Observed Condition:** General observations on the structural elements of the concrete showed there to be evidence of scaling and general abrasion in areas of the slab where heavy truck traffic and exposure to nitrate material spills had occurred. The slab was found to be generally sound showing no evidence of reinforcement corrosion. The cover depth was found to be 40 mm, which in this case was assessed to be adequate for steel protection.

### 2.3 Sites 2 and 3 - Northaven Retirement Village

**Description:** During the latter part of 1969, a series of concrete slabs were cast as part of the construction of an unenclosed car park of a motel building. The building changed ownership in the mid 1970's from being a motel to serving its current use as the Northaven Retirement Village complex. In all, there were four slabs cast for this structure, a slab-on-grade, a flat plate forming the first level and two beam and slab systems forming the upper two levels. Long-term deflections of the Level 1 reinforced concrete flat plate

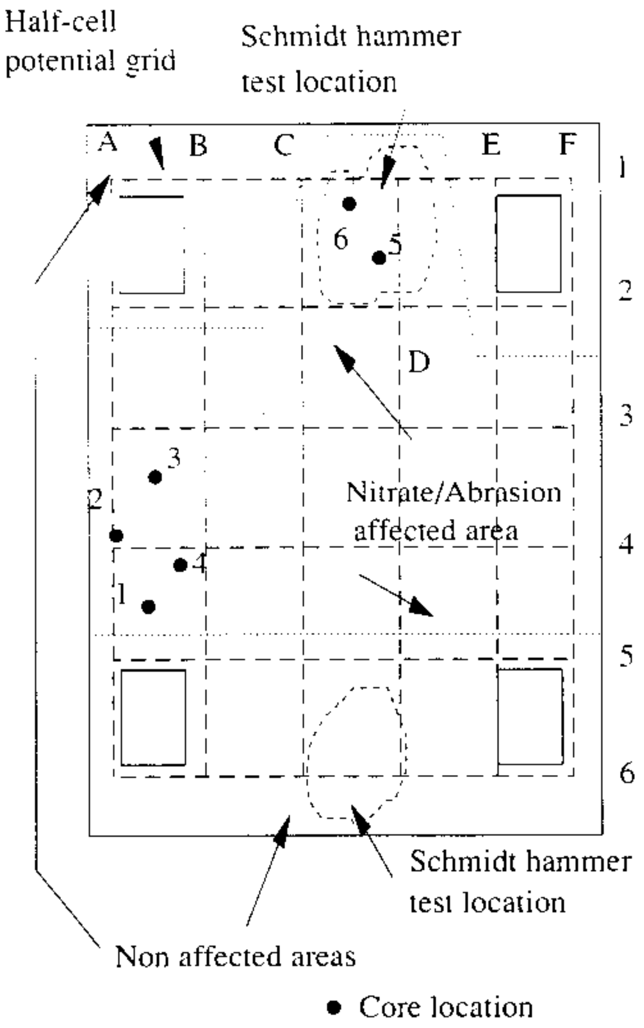

Figure 1. Prospect Nitrate Store General Layout of Site (Half cell grid interval - 500mm)

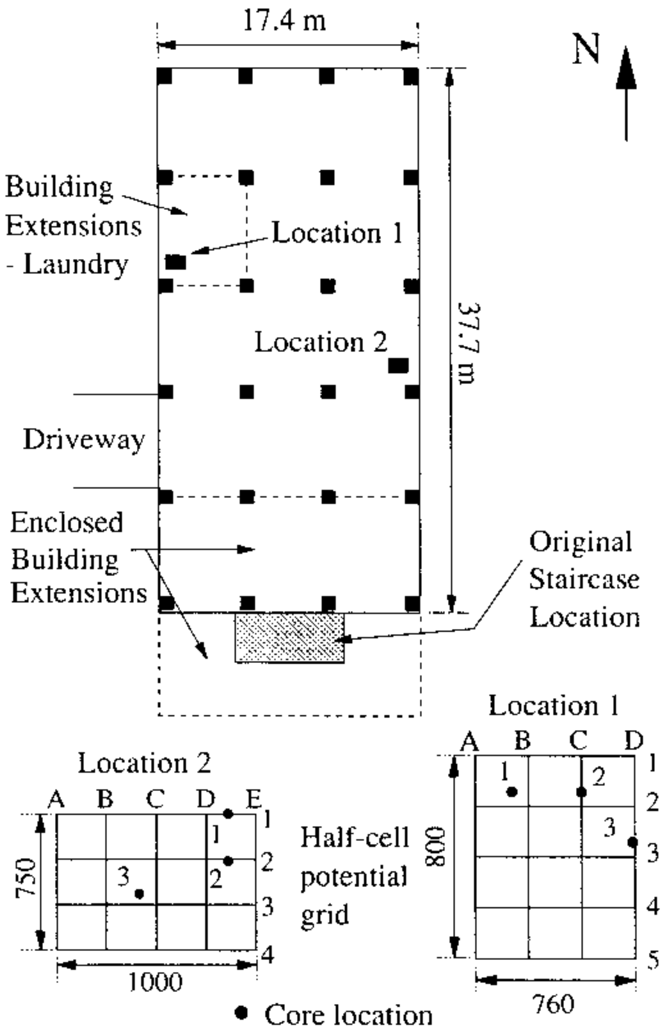

Figure 2. Northaven Retirement Village General Layout of Site

| Shrinkage/ Creep (µstrain) | Data | Standard | Specimens | Age (Days) |
|---|---|---|---|---|
| $\varepsilon_{sh}$ | 620 | ASTM C157-64T | Average of 3 | 365 |
| $\varepsilon_{cp}$ | 1060 | ASTM C512-66T | Average of 2 | 583 |
| $\varepsilon_{sh}$ | 564 | ASTM C512-66T | Average of 2 | 583 |
| $\varepsilon_i$* | 850 | ASTM C512-66T | Average of 2 | 583 |
| **Concrete Properties** | **Data (MPa)** | **Standard** | **Specimens** | **Age (Days)** |
| $E_c$ | 22,205 | ASTM C469-65 | Average of 2 | 28 |
| Compressive Strength | 24 | ASA 105-1968 | Average of 3** | 28 |

* Initial elastic strain

** Core dimensions 75 mm diameter x 150 mm

TABLE II - Original Test Results from Studies Carried out on Northaven Retirement Village Concrete, 1969 (6) were monitored and reported in order to provide data on serviceability of these member types (6)(7).

Alterations were carried out on the structure after the change in ownership in the mid 1970's and work included the enclosing of Level 1 of the building. This meant that the Level 1 slab, which was the focus of the original study, could no longer be accessed. It was decided to focus investigations on two separate locations on the ground floor slab. A similar concrete would have been used for the ground level and Level 1 slab elements. Northaven Retirement Village Location 1 (Site 2) was at the eastern edge of the ground floor slab and Location 2 (Site 3) was at the western edge. Scanning electron microscopy work carried out on concrete sampled from the investigated element confirmed the use of fly ash. Cover meter readings and cores extracted showed reinforcement to be 12 mm diameter at 320 mm centres north-south and 230 mm centres east-west. Heiman and Taylor (6) noted that cold worked deformed bars had been used for the reinforcement in the Level 1 slab. The AS 3600 exposure classification for the concrete would conform to A2 specifications.

**Concrete As Used:** The slabs studied were documented to have had fly ash from Wangi Power Station included in the concrete. Black breccia and crushed river gravel aggregates were used. The average compressive strength at 28 days for a set of cylinders taken from the original concrete was 24 MPa and the concrete slump was reported to be between 75 and 90 mm. Data reproduced from an original study by Heiman and Taylor (6) on concrete properties are presented in Table II. Shrinkage values for the concrete determined in this study appeared to be high.

**Assessment Methods Used:** Similar assessment methods to those detailed for the Prospect Nitrate Store element were used on these sites. The general layout of the site showing the specific areas of study, half cell potential grids and core locations is shown in Figure 2.

**Observed Condition:** The general condition of the slab was satisfactory in terms of in-service performance. No major faults were found at the time of inspection. The cover to reinforcement as determined from the cores was typically 50 mm although this was found to vary over the core locations. In field studies conducted on the building in 1984, cracking both parallel to reinforcement and diagonal to reinforcement was observed on the Level 2, 3, 4 and 5 slabs. Corrosion of reinforcement was observed in confined areas of the Level 5 slab and some of the beam elements. Durability defects on superstructure slab, beam and wall elements have been documented in a University of Sydney report (8).

### 2.3 Sites 4 to 7 - Port Kembla Wharves

**Description:** During the period between 1961 and 1973, a series of wharf structures were constructed at the Port Kembla Steelworks. These facilities provide a range of services encompassing loading and unloading operations of raw materials for steel production and transfer of finished products. Four individual wharves were selected for study, the Finished Products Berth, the Ore Unloading Berth, the Raw Materials Berth and the Roll On Roll Off (Ro Ro) Berth (Sites 4 to 7 respectively).

The main vertical reinforcement used in the sea face of the wharves was 28.5 mm in diameter placed at 200 mm centres. The top working surface of the wharves had 28.5 mm diameter bars running perpendicular to the sea face at 200 mm centres and 25 mm diameter bars running parallel to the sea face at 300 mm centres. Cores were extracted from the wharf top working surface immediately adjacent to the sea face. Cover to main reinforcement was specified to be 75 mm. The cover specification did not apply to ligature reinforcement or embedments. The selected structures were assessed to conform to AS 3600 exposure classification C specifications.

**Concrete As Used:** The characteristic strength of the concrete was specified to be 21 MPa. Of the four wharves selected for this study, the Finished Products Berth and the Ore Unloading Berth were constructed using low heat cement binders. The Raw Materials Berth and the Ro Ro Berth were constructed using ternary fly ash, slag and portland cement binders. Slag aggregates were used in the concretes in all these structures. The Finished Products Berth (Site 4) was constructed in 1961 and the Ore Unloading Berth (Site 5) was constructed in 1969. The Raw Materials Berth (Site 6) and the Ro Ro Berth (Site 7) were constructed in 1973 (Table 1). The ternary blend cement mixes had an estimated 120 kg/m3 of Type A cement, 100 kg/m3 of fly ash from Munmorah Power Station, 120 kg/m3 of ground granulated blast furnace slag from Port Kembla steelworks, 1,100 kg/m3 of air cooled slag coarse aggregate and 800 kg/m3 of sand (9). A water:binder ratio of around 0.65 would have been used for the concrete. A similar binder content and water:binder ratio would have been used for the concretes for Sites 4 and 5.

**Assessment Methods Used:** Similar assessment methods to those detailed for the previous structures were used on these sites. Half cell potential grids were typically at 450 mm centres. A schematic diagram of the wharf locations at the Port Kembla Steelworks is given in Figure 3.

**Observed Condition:** A report on the in situ performance of Sites 4 to 7 was published in 1981 (10). In that study, it was observed that the wharf face of Site 4 was found to have extensive horizontal cracking over reinforcing bars and edge spalling of concrete. It was noted that the water-line concrete and the wharf soffit appeared sound. Defects possibly due to sulphate attack were observed in some areas. The same report states that the Site 7 ternary blend concrete, which was eight years old at the time, exhibited a much improved surface finish and an almost total absence of cracking. Some discolouration on the concrete resulting from residual iron in the slag was observed.

During recent inspections in 1989, it was found that cover to reinforcement at some locations at the sea face of Sites 4 and 5 was much lower than that specified. Inspections of Site 4 after 28 years indicated significant corrosion problems and concrete delamination on the sea face of the wharf. Despite these problems, the structure was in use at the time of

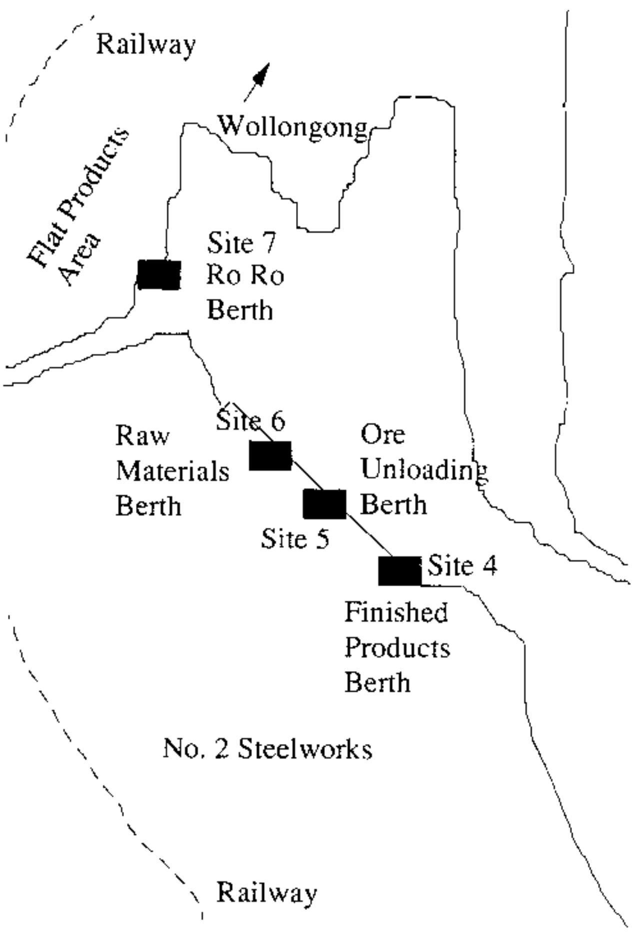

**Figure 3.** Port Kembla Wharves
General Layout of Site Locations

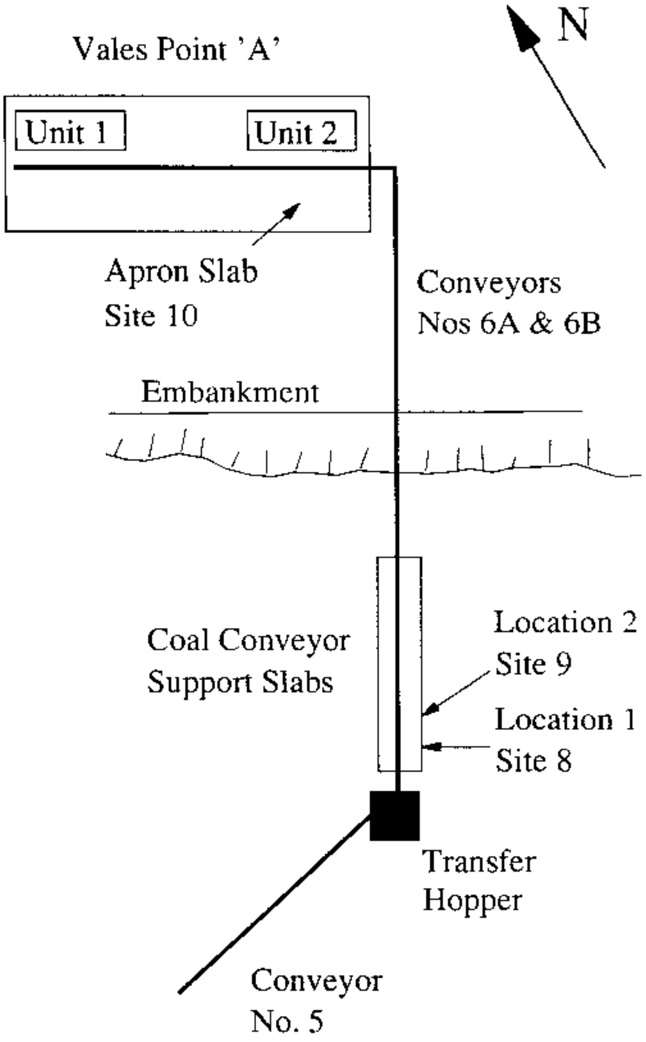

**Figure 4.** Coal Conveyor Support Slabs
Vales Point Power Station General Layout

inspection. Site 5 having served for 20 years was found to have similar problems to those of Site 4. This structure was again, nevertheless, in use at the time of inspection. Sites 6 and 7 were found to be performing satisfactorily after 18 years of service, showing only minor problems in the areas investigated.

### 2.4 Sites 8, 9 and 10 - Vales Point A Power Station

**Description:** In 1961, during the construction of the Vales Point 'A' Power Station, a series of slabs supporting the main coal feed conveyor to the power station was cast using fly ash concretes. The placement of these slabs was part of an experimental programme carried out at Vales Point Power Station on fly ash concretes. The original programme included both laboratory studies and field tests in the form of the installation of the support slabs (8)(11)(12). The laboratory test programme included a study of the strength development characteristics of plain cement and fly ash concretes.

Inspections focused on three sites within which the selected elements were all slabs-on-grade. The Coal Conveyor Support Slabs (Sites 8 and 9) were cast using fly ash concretes. The Site 8 concrete binder had fly ash used as a 15% replacement for cement. The Site 9 concrete binder had fly ash used as a 20% replacement for cement. Site 10 (the Boiler 2 Apron Slab) was selected to represent as closely as possible a control plain cement concrete used under similar service conditions. Concrete sections in the form of slab extensions were cast immediately adjacent to the Sites 8 and 9 slabs at various points in order to facilitate later coring and study. The design thickness of these slabs was 150 mm and top surface cover to reinforcement was specified to be 50 mm. Site 10 concrete was specified as being 175 mm thick with the same cover as for Sites 8 and 9. Reinforcement for the three sites was specified as square mesh with 6 mm diameter wires at 150 mm centres. Exposure classifications for all pavements as set down by AS 3600 would correspond to A2 specifications. A general layout of the sites is shown in Figure 4.

**Concrete As Used:** The two fly ash concretes used had 15% mass replacement of cement with fly ash and the other having 20% mass replacement of cement with fly ash (Sites 8 and 9 respectively). The original laboratory study on the concretes tested showed one year strengths of the 15% fly ash replacement concrete to be similar to the plain cement concrete. The concrete strengths for the 20% fly ash replacement mix were however typically around 5 MPa lower than the plain cement concrete at one year (10).

**Assessment Methods Used:** Similar assessment methods to those detailed for the previous structures were used on these sites. Details of site locations, half cell potential grids and core locations are presented in Figures 5 and 6.

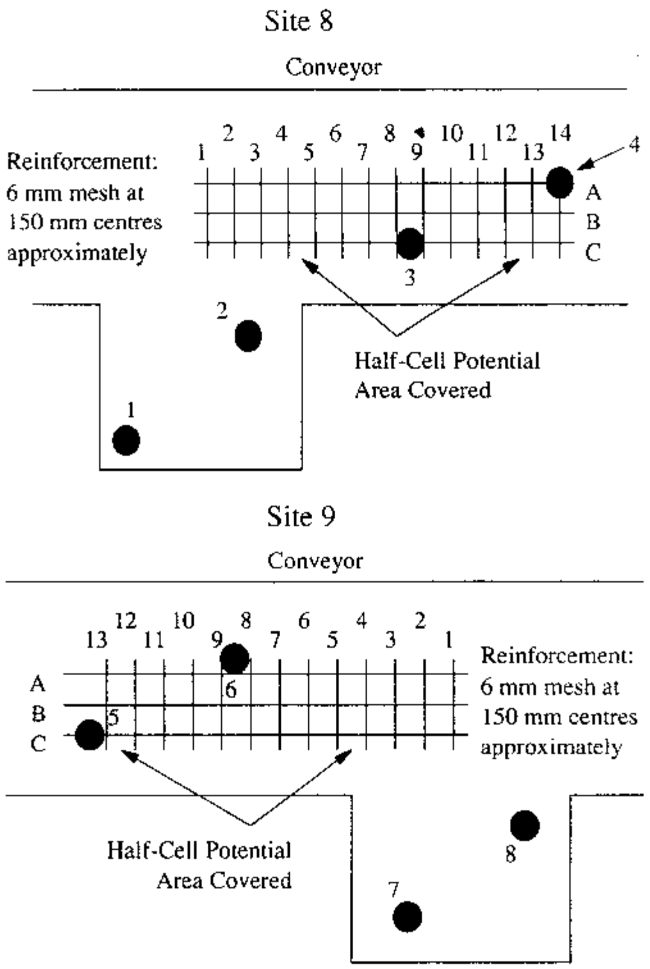

**Figure 5.** Coal Conveyor Support Slabs
Vales Point Power Station Sites 8 & 9

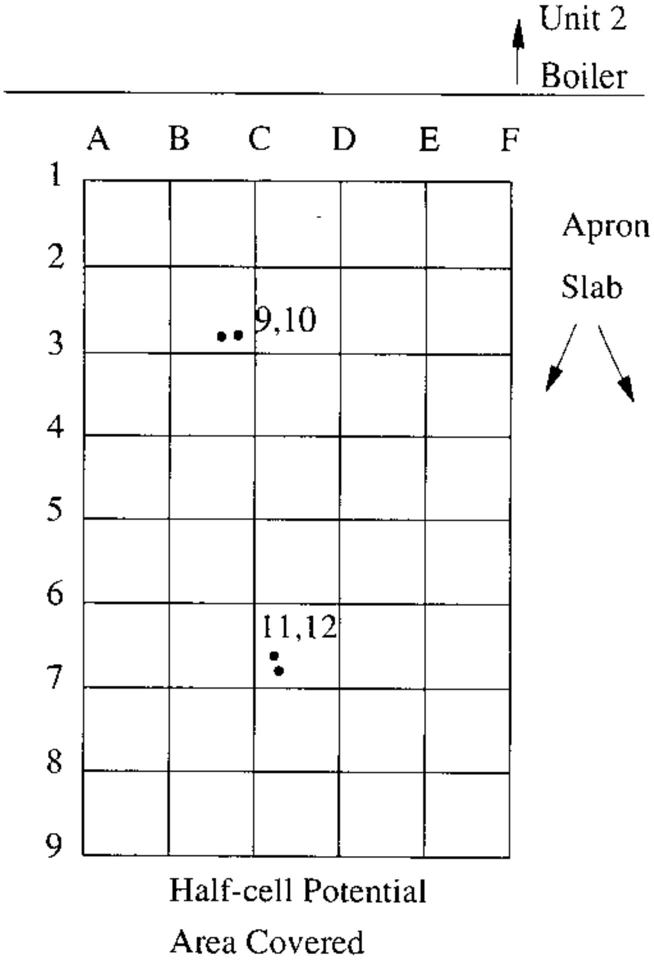

**Figure 6.** Boiler 2 Apron Slab, Vales Point Power Station
Site 10 (Half cell grid interval - 300mm)

**Observed Condition:** Minor cracking both parallel and diagonal to reinforcement was found on Sites 8 and 9 concretes. Reinforcement was found to be located around 60 mm from the bottom of the slab at the core locations on Sites 8 and 9. Observations taken on extracted cores showed concrete at the bottom of the slabs at Sites 8 and 9 to be poorly compacted in some areas with large voids present. No visual evidence of reinforcement corrosion was found during inspections of the structural elements. Site 10 concrete was found to be performing well and had a top surface cover to reinforcement of around 100 mm.

## 3. DATA ON STRUCTURE CONCRETES

Core samples extracted from the structures were investigated in the laboratory. Data was obtained on modulus of elasticity, compressive strength, Schmidt hammer, Brazilian tensile strength, concrete density, ultrasonic pulse velocity (carried out under ambient laboratory conditions) and absorption. A summary of results is presented in Table III. The calculated modulus of elasticity data were determined using procedures set down in AS 3600 Section 6.1.2.

Correlations between Schmidt hammer test results and core strengths were poor, due, in part, to the different aggregates used in the mixes from the different structures. Correlations between Schmidt hammer and carbonation data were however good, with a logarithmic function making the relationship significant at the 95% confidence level (Figure 7). The observed relationship is probably explicable, in part, by the fact that both factors reflect properties of the surface concrete. The degree to which concretes would carbonate would be governed primarily by their permeability to CO2. Low values of carbonation would indicate a low permeability to CO2. Such low permeabilities have correlated well with higher rebound characteristics as shown by the Schmidt hammer results. Concrete core strengths determined after varying field durations were considerably higher than the originally specified concrete strengths. This covered all binder types. The only exception was the Site 4 concrete where the ratio of core strength to design strength was approximately 1.5.

Concrete modulus of elasticity testing as determined from cores taken from the selected structural elements was carried out. A series of strain gauges 67 mm in length were placed vertically on sides of cores so as to provide confirmation of test results. These data were compared to those calculated using procedures set down in AS 3600 Section 6.1.2 (Table III and Figure 8). Results showed that the site concrete values generally fitted within the limits of the calculated values with the exception of Sites 2, 8 and 10 which were slightly outside the limits. In the case of Site 2, the low modulus result is consistent with the known properties of the black breccia coarse aggregates used in the concrete (13).

| Site No. | Concrete Density (SSD) (kg/m$^3$) | Core Comp. Str. (MPa) | Average Modulus of Elast. (x 10$^4$ MPa) | Calculated Modulus of Elast. (x 10$^4$ MPa) | Ultrasonic Pulse Velocity (m/sec) | Schmidt Hammer Reading | Tensile Strength** (MPa) | Brazilian Tensile Strength (MPa) | Absorption (%) |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 2374 | 49* | 3.05 | 3.55 | 4260 | 31 | 2.00 | 7.29 | 5.05 |
| 2 | 2305 | 55 | 2.80 | 3.76 | 4230 | 31 | 1.79 | 3.32 | 7.14 |
| 3 | 2341 | 40 | 2.80 | 3.21 | 4390 | 30 | 1.79 | 4.15 | 5.35 |
| 4 | 2182 | 31 | 2.05 | 2.80 | 4220 | 28 | 1.83 | 3.95 | 7.90 |
| 5 | 2422 | 47 | 3.75 | 3.48 | 4720 | 25 | 1.83 | 2.35 | 5.63 |
| 6 | 2201 | 51 | 3.45 | 3.59 | 4480 | 30 | 1.83 | 4.50 | 7.77 |
| 7 | 2317 | 46 | 3.50 | 3.44 | 4480 | 26 | 1.83 | 5.14 | 6.87 |
| 8 | 2511 | 45 | 2.85 | 3.37 | 4320 | 33 | 1.79 | 5.33 | 4.78 |
| 9 | 2486 | 52 | 3.50 | 3.64 | 4450 | 33 | 1.79 | 4.69 | |
| 10 | 2551 | 52 | 3.10 | 3.65 | 4510 | 33 | 1.79 | 4.97 | |

* Two cores were tested. A core from the nitrate/abrasion affected area resulted in 39 MPa. A non affected core resulted in 49 MPa (Figure 1).

** Characteristic principal tensile strength - Calculated using AS 3600 Section 6.1.1.3 with specified design grade data (Table 1).

**TABLE III** - Data on Structure Concrete Properties

Predictions of the characteristic principle tensile strength of the concretes as dictated by AS 3600 Section 6.1.1.3 were compared with Brazilian tensile strength data determined on cores from the site concretes (Table III). This analysis was carried out to determine if the design tensile strength values under-predicted the site determined values for the site concretes considered. The calculation of the characteristic principle tensile strength was based on the specified 28 day design strengths of the concretes. It can be observed that the predicted values always considerably underestimated the site observed figures. These two data sets showed poor correlations when compared statistically. The ratio of Brazilian tensile strength to characteristic principle tensile strength for all selected concretes was found to range between 1.3 and 3.7 with a mean ratio value of around 2.5, consistent with the different concrete types studied.

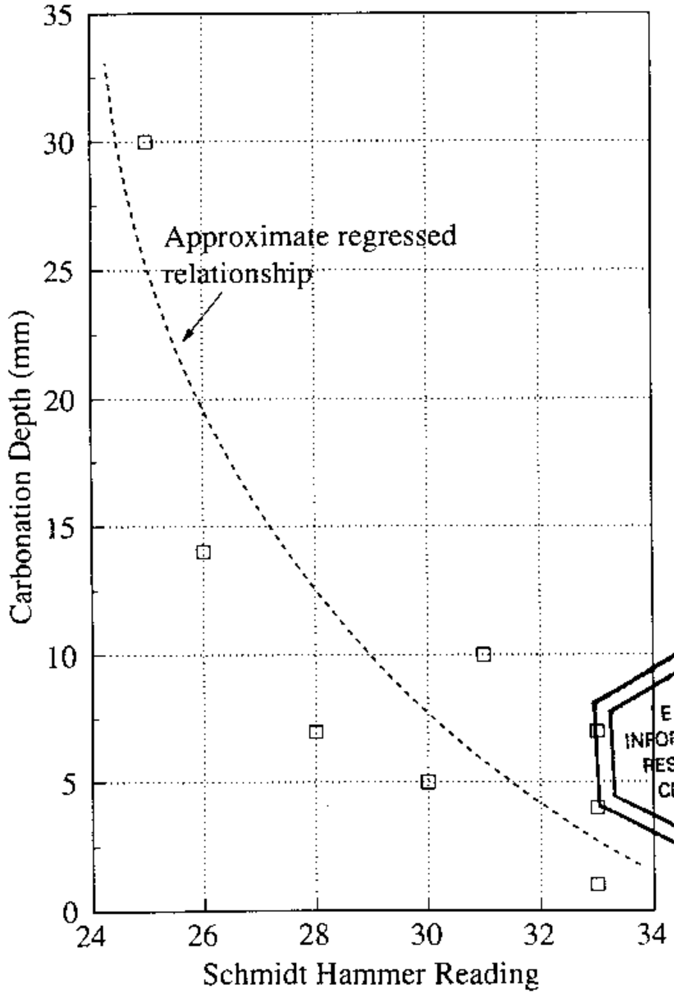

**Figure 7.** Relationship Between Carbonation Depth of Concrete and Schmidt Hammer Reading

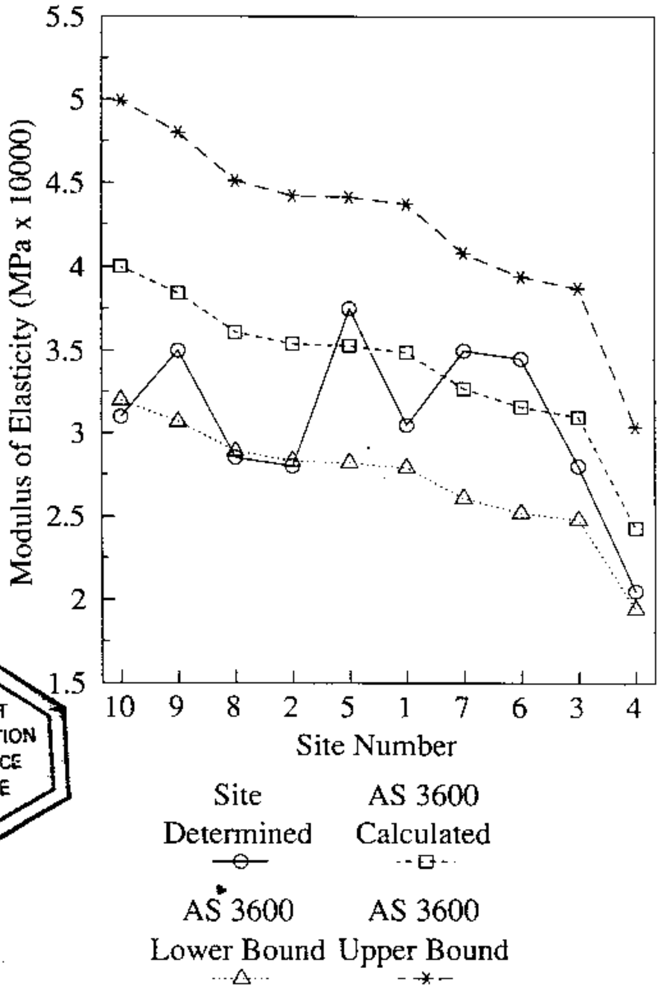

**Figure 8.** Comparison of Modulus of Elasticity from Site Concretes and AS 3600 Calculated Design Values

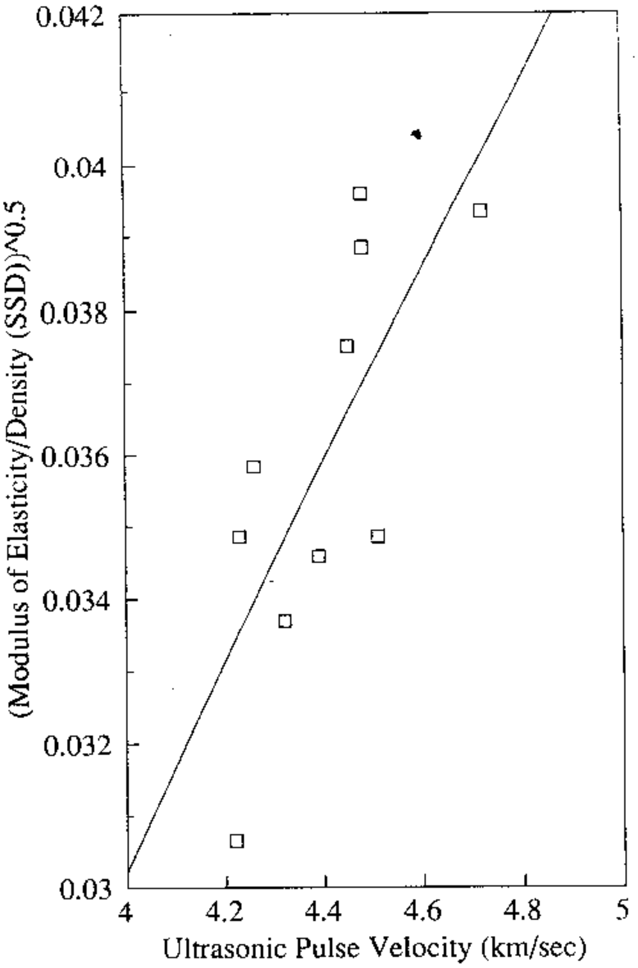

**Figure 9.** Relationship Between Ultrasonic Pulse Velocity Data and ASTM C-597-83

The relationship between the ultrasonic pulse velocity values determined on these concretes and values calculated by ASTM C597 (14) procedures is shown in Figure 9. ASTM C597 states that for a solid material, the pulse velocity is proportional to the root of the ratio of modulus of elasticity and density. Statistically, a correlation coefficient ($R^2$) of 0.56 was found for the concretes tested. This value was found to be significant at the 95% confidence level even with the wide range of concrete types considered in this study.

## 4. LONG-TERM DURABILITY

### 4.1 General Data Collection Rationale

Chloride ion concentrations, carbonation depth, general durability defects and reinforcement corrosion activity were recorded for all site structural elements considered. Concrete drillings taken in situ on the sites were analysed for acid soluble chloride ion content for depth ranges up to 60 mm below the surface. Copper-copper sulphate half cell potential data were acquired from selected areas adjacent to the core locations on each of the sites. Carbonation depths were measured by phenolphthalein spray on fracture surfaces of cores taken from the site concretes. The general condition of the structural elements was recorded.

### 4.2 Chloride Ion Concentrations and Half Cell Potentials

Figure 10 shows details of concrete chloride ion concentration variations with depth from surface for a pertinent selection of sites. Data for Sites 1 to 5 and 7 are presented. Site 6 data was not included as the concrete was found to be similar to Site

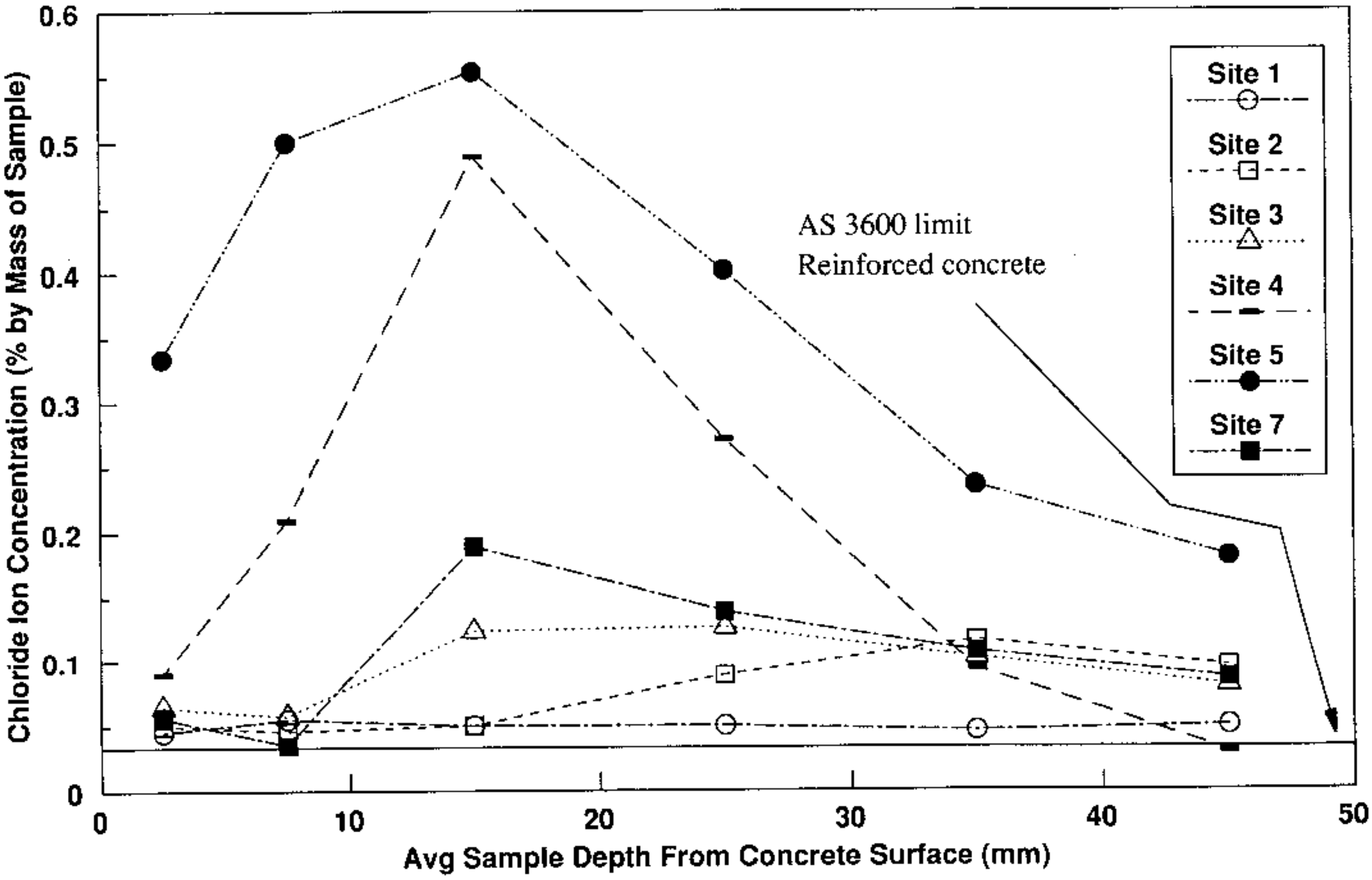

**Figure 10.** Variation of Chloride Ion Concentration With Depth

7. Chloride ion concentrations for concretes from Sites 8 to 10 were found to be generally low and were thus not included.

For Sites 1 to 3, airborne salts would be the most likely major source of chlorides. Chloride ion concentrations on these concretes were found to be in excess of the limit specified in AS 3600. For Sites 2 and 3, the data showed peak chloride ion concentrations to be between 20 mm and 40 mm below the concrete surface. Half cell potential data acquired from Sites 1, 2 and 3 are presented in Table IV. For Site 1, values of potentials ranged between -75 and -266 mV. According to ASTM C876 specifications (15), this would represent a 90% probability of no active corrosion. Half cell potential data on Sites 2 and 3 ranged between -5 and -160 mV, suggesting again a low probability of the presence of active corrosion. No corrosion activity was found during inspections of Sites 1, 2 and 3.

For Sites 4, 5 and 7, all chloride ion concentrations determined were found to be well above the limit set down in AS 3600 for reinforced concrete. Chloride ion concentrations were found to peak at a depth around 15 mm below the concrete surface. Data showed that chloride ion concentrations for Sites 4 and 5 were generally significantly higher than for Site 7 with Site 5 showing the highest chloride ion concentration values at all depths tested. Half cell potential data acquired from Sites 4, 5, 6 and 7 are presented in Table V. For Site 4, potentials were found to range between -356 and -453 mV suggesting a high probability of active reinforcement corrosion. Half cell potential data obtained on Site 5 concrete ranged between -487 and -605 mV, again suggesting a high probability of active corrosion. Site 6 half cell potential data ranged between -28 and -370 mV (low probability of active corrosion). An area of higher negative potential values was observed on this structure suggesting the possibility of localised reinforcement or ligature corrosion (Table V). For Site 7, half cell potentials were found to range between -169 and -492 mV. Again, a row of higher negative potential values was observed directly over an embedded metal pipe located close to the concrete surface. The data suggested that active reinforcement corrosion may have been present in localised areas.

After 28 years of service, significant corrosion problems were evident with the Site 4 concrete. Problems with regard to cracking over reinforcing bars were noted both in the current study and earlier (9). The structure was providing its full designed function despite the observed problems at the time of inspection. Studies on Site 5 concrete confirmed the presence of active reinforcement corrosion after 20 years of service. Chloride ion concentrations within Sites 4 and 5 concretes were found to be high. The present fendering system for incoming and outgoing vessels could also, in part, have contributed to the observed problems by allowing some impact loadings between vessels and wharf concrete. Concrete for Sites 6 and 7 appeared to be performing satisfactorily in the areas inspected. Site 7 concrete showed some surface discolouration consistent with earlier reports (9).

Half cell potential data acquired from Sites 8, 9 and 10 are presented in Table VI. Values of potentials were found to be more negative than -385 mV for Site 8 concrete and more negative than -400 mV for Site 9. It was found that the reinforcement was located around 60 mm from the bottom

| Grid Row Ref. | Half Cell Potentials at Column Reference (mV - All Values Negative) | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Site 1 | | | | | | Site 2 | | | | Site 3 | | | | |
| | A | B | C | D | E | F | A | B | C | D | A | B | C | D | E |
| 1 | 205 | 185 | 218 | 158 | 176 | 110 | 100 | 90 | 100 | 100 | 60 | 40 | 40 | 30 | 10 |
| 2 | 75 | 183 | 212 | 214 | 204 | 161 | 80 | 90 | 100 | 160 | 60 | 60 | 25 | 40 | 20 |
| 3 | 129 | 173 | 208 | 218 | 221 | 181 | 80 | 70 | 60 | 70 | 60 | 10 | 20 | 5 | 20 |
| 4 | 159 | 201 | 218 | 220 | 220 | 192 | 70 | 70 | 80 | 70 | 60 | 25 | 30 | 10 | - |
| 5 | 164 | 204 | 214 | 212 | 207 | 187 | 60 | 60 | 60 | 70 | - | - | - | - | - |
| 6 | 266 | 166 | 177 | 189 | 201 | 181 | - | - | - | - | - | - | - | - | - |

**TABLE IV** - Half Cell Potential Data - Sites 1, 2 and 3 (Figures 1 and 2)

| Grid Row Ref. | Half Cell Potentials at Row Reference (mV - All Values Negative) | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Site 4 | | | | | | Site 5 | | | | | Site 6 | | | | Site 7 | | | | |
| | A | B | C | D | E | F | A | B | C | D | E | A | B | C | D | A | B | C | D | E |
| 1 | 420 | 449 | 446 | 453 | 440 | 226 | 605 | 572 | 580 | 594 | 602 | 28 | 37 | 93 | 246 | - | - | 242 | 291 | 375 |
| 2 | 398 | 449 | 414 | 425 | 425 | 407 | 560 | 523 | 536 | 539 | 620 | 218 | 68 | 81 | 204 | - | - | 200 | 326 | 325 |
| 3 | 402 | 407 | 415 | 421 | 420 | 394 | 538 | 499 | 490 | 501 | 524 | 121 | 118 | 130 | 370 | 188 | 224 | 271 | 484 | 392 |
| 4 | 402 | 409 | 400 | 424 | 418 | 401 | 537 | 505 | - | 496 | 513 | 130 | 172 | 182 | 343 | 264 | 320 | 322 | 492 | 211 |
| 5 | 408 | 413 | 404 | 425 | 418 | 402 | 538 | 498 | 487 | 498 | 519 | 123 | 103 | 153 | 319 | 169 | 218 | 280 | 439 | 194 |
| 6 | - | 394 | 409 | 405 | 400 | 378 | 543 | 515 | 515 | 507 | 503 | 109 | 106 | 145 | 300 | - | - | 200 | 468 | 197 |
| 7 | - | - | 356 | 364 | 377 | 393 | - | - | - | - | - | 80 | 82 | 149 | 363 | - | - | 182 | 480 | 158 |

**TABLE V** - Half Cell Potential Data - Sites 4, 5, 6 and 7 (Figure 4)
(Shaded area represents embedded concrete pipe location)

| Grid | Half Cell Potentials at Grid Reference (mV - All Values Negative) | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Ref | Site 8 | | | Site 9 | | | Site 10 | | | | | |
| | A | B | C* | A | B | C | A | B | C | D | E | F |
| 1 | 455 | 425 | 385 | 512 | 635 | 644 | 38 | 87 | 117 | 133 | 83 | 102 |
| 2 | - | - | - | - | - | - | 40 | 83 | 72 | 91 | 46 | 55 |
| 3 | 451 | 449 | 421 | 500 | 614 | 648 | 62 | 71 | 77 | 84 | 35 | 34 |
| 4 | - | - | - | - | - | - | 45 | 57 | 58 | 87 | 72 | 46 |
| 5 | 452 | 452 | 451 | 486 | 628 | 644 | 79 | 75 | 81 | 101 | 80 | 45 |
| 6 | - | - | - | | - | - | 90 | 82 | 84 | 101 | 83 | 64 |
| 7 | 471 | 471 | 441 | 468 | 590 | 653 | 111 | 105 | 93 | | 111 | 93 |
| 8 | - | - | - | - | - | - | 79 | 71 | 75 | 117 | 117 | 120 |
| 9 | 459 | 473 | 455 | 455 | 471 | 608 | 97 | 89 | 77 | 127 | 123 | 135 |
| 11 | 485 | 475 | 425 | 443 | 450 | 583 | - | - | - | - | - | - |
| 12 | 483 | 483 | 460 | 433 | 452 | 585 | - | - | - | - | - | - |
| 13 | 476 | 476 | 460 | 440 | 454 | 584 | - | - | - | - | - | - |

**TABLE VI** - Half Cell Potential Data - Sites 8, 9 and 10 (Figures 5 and 6)

surface of the slab at the core locations. Site 10 half cell potential data ranged between -34 and -135 mV, suggesting a low probability of active corrosion. Differences in slab design between Site 10 and Sites 8 and 9 have been detailed earlier. Reinforcement was found to be at a depth of around 80 mm from the top slab surface. After 28 years, concretes for Sites 8 and 9 continue to provide their designed function despite the observed corrosion potentials. Site 10 concrete is performing well after 28 years.

### 4.3 Carbonation Characteristics

Average carbonation depth data for all site concretes are presented in Table VII. Noteworthy features of the data are that for all the concretes studied, the maximum average carbonation depth was 30 mm. This value was obtained for a low heat cement concrete with a 20 year service life (Site 5). The lowest value of average carbonation depth was 1 mm for a fly ash concrete with a 28 year service life (Vales Point, Site 9). Carbonation depths on all concretes studied at Vales Point Power Station (fly ash concretes and normal portland cement concrete) were less than 7 mm. These low values would be explicable, in part, as result of the exposure conditions of the concrete and the concrete mix designs. Carbonation data did not correlate well with core strength data. The poor correlations observed between concrete strength and carbonation could be due, in part, to the difference in exposure conditions of the structures, their ages and variations in mix designs. Reasons for the high carbonation values found for Site 5 when compared to Site 4 could be due, in part, to Site 5 concrete being generally drier and having less exposure to chlorides.

## 5. DISCUSSION

Information on the structural elements and concretes studied is presented in Table VII. Exposure classification information as defined by "AS 3600 Design for Durability" procedures has been summarised along with original specified concrete strength data and representative concrete strengths as determined from extracted cores (AS 3600). Structural element age, reinforcement half cell potentials and carbonation depth are also presented.

Specifications for concrete strengths following AS 3600 guide-lines have increased for these structure types when compared to those nominated in codes of practice over 12 years ago (Table I and VII). None of the concretes considered in this study would comply with the current AS 3600 guide-lines for specified concrete strength. Conversely, current AS 3600 recommendations for cover for Sites 4 to 10 are lower than those originally specified. Recommendations in AS 3600 to increase the specified concrete strength requirements for structures of the sort considered in this study are generally positive. However, strength specifications in AS 3600 for marine concretes could preclude the use of some blended cement systems which may be particularly applicable to sea-water exposures.

Originally specified concrete strengths, although complying with earlier codes of practice, appeared to be unduly low when considering the loading and exposure conditions of the structural elements and current standard requirements. In many cases, a structural element needs to function for long periods sometimes in excess of 50 years. Concrete structural elements, in particular those of heavy engineering structures, tend to receive minimal performance diagnosis or repair in

| Site | AS 3600 Exposure | | | Spec. | Site | Cover to Reo | | Age | Half Cell Pot. | | Carbonation | Major |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| No. | Env. | Grade (MPa) | Cover (mm) | Str. (MPa) | Str. (MPa)[a] | Spec. (mm) | Meas. (mm) | (yrs) | Min | Max | Depth (mm) | Durability Problem |
| 1 | U[b] | - | - | - | 54 | - | 40 | 12 | 75 | 256 | 10 | Abrasion |
| 2 | A2 | 25 | 30 | 20 | 61 | - | 50 | 20 | 60 | 160 | 10 | - |
| 3 | A2 | 25 | 30 | 20 | 44 | - | 50 | 20 | 5 | 60 | 5 | - |
| 4 | C | 50 | 50 | 21 | 34 | 75 | 40 | 28 | 356 | 453 | 7 | Corrosion |
| 5 | C | 50 | 50 | 21 | 52 | 75 | 60 | 20 | 487 | 605 | 30 | Corrosion |
| 6 | C | 50 | 50 | 21 | 56 | 75 | 60 | 18 | 28 | 370 | 5 | Minor Corrosion |
| 7 | C | 50 | 50 | 21 | 51 | 75 | 60 | 18 | 169 | 492 | 14 | Minor Corrosion |
| 8 | A2 | 25 | 30 | 20 | 49 | 50 | 60[c] | 28 | 385 | 485 | 4 | Minor Corrosion |
| 9 | A2 | 25 | 30 | 20 | 57 | 50 | 60[c] | 28 | 433 | 635 | 1 | Minor Corrosion |
| 10 | A2 | 25 | 30 | 20 | 57 | 50 | 80 | 28 | 34 | 135 | 7 | - |

[a] Site determined strength - Core strength x 1.1 (AS 3600 Section 21.4.3).
[b] No recommendations provided by AS 3600.
[c] Cover estimate measured from bottom face of slab.

**TABLE VII** - Summary of Site Concrete Performance

their service lives due, in part, to a low priority on aesthetics. Such considerations need to be paramount in design if such structural element types are to remain functionally efficient for long periods.

Cover to reinforcement on the wharf structures was specified to be 75 mm. Corrosion problems were observed on some locations on these structures where covers were estimated to be between 60 to 75 mm. Current AS 3600 recommendations for cover to reinforcement for these structure types would be 50 mm and concrete strength recommendations would be 50 MPa. Although the AS 3600 strength specifications for marine structures have increased, the permissible cover specification decrease from 75 mm to 50 mm for large marine structural elements seems to be at odds with experience. For large concrete marine structural elements, the new AS 3600 specifications may or may not ensure long-term durability.

It appears unlikely that carbonation itself would have been the major cause of any of the corrosion problems observed on site as the covers to reinforcement were generally high. For the structural elements considered in this study, the carbonation front had not advanced to depths greater than an average 30 mm at ages between 12 and 28 years. For the blended cement concretes studied, this front had reached a maximum of 14 mm average depth for an 18 year old concrete. Upon comparing the achieved cover with the carbonation depth measured (Table VII), it can clearly be observed that carbonation has not reached the depth of the embedded reinforcement in any of these structures. The lower portland cement contents particularly in the ternary blend have not appeared to influence carbonation characteristics significantly in the structures examined.

Half cell potential data appeared to correlate well with observed corrosion occurrence based on ASTM C876 guidelines. Chloride ion concentration data for concretes also correlated well with observed corrosion. The low heat cement concretes used in Sites 4 and 5 were found to have chloride ion concentrations which were significantly higher than the limit specified for reinforced concrete as a material in AS 3600. These concretes were 28 and 20 years old respectively and were found to be undergoing active corrosion. The 16 year old ternary blend concrete used in Site 7 showed a lower maximum chloride ion concentration value when compared to concretes from Sites 4 and 5. Chloride ion concentration values for Site 7 were nevertheless again higher than the limit specified in AS 3600 for concrete material and it is likely that some reinforcement corrosion was taking place in localised areas. Sites 8 and 9 concretes had high probabilities of undergoing active corrosion despite the fact that both carbonation and chloride ion concentrations within the concrete were low. This was due to misplaced reinforcement located 60 mm above the bottom surface of the slab in badly compacted concrete.

## 6. CONCLUSIONS

For the structures examined in this study, it can be concluded that all ternary blend concretes and fly ash concretes were performing adequately. The major factor influencing concrete performance adversely appeared to be the low initial specified strengths. This factor was particularly pronounced in the case of the wharf structures. Although the initial specified strengths for the structures would have complied to earlier codes of practice, they would be low by current standards particularly for the wharf structures considered.

Chloride ion concentration profiles within the first 50 mm of the surfaces of the wharf structures and the slab-on-grade structures considered showed these to peak at around 20 mm below the concrete surface. This could have implications on reinforcement corrosion if bars were located in these regions. Carbonation depths on plain and blended cement concretes tested were generally low for the structures selected. Calculated modulus of elasticity values based on AS 3600 guide-lines correlated well with the site determined values. Half-cell potential data obtained for the structures appeared to conform to the guide-lines given in ASTM C876.

For large marine structures of the sort considered in this study, although specified concrete strengths have increased, minimum cover to reinforcement requirements appear to have decreased. It is still unclear whether or not these specifications will ensure long-term durability for such structures in the light of the reduced cover specification. It is unlikely that carbonation itself would have been the cause of any of the corrosion problems observed in the structures examined due partly to the covers achieved on site. Active reinforcement corrosion was recorded in Site 4 and 5 concrete. Chloride ion concentrations within these concretes were found to be high.

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D. BAWEJA

Daksh Baweja, B.E. (Hons.), M.Eng.Sc., M.I.E. Aust, is a Senior Experimental Scientist with the CSIRO Division of Building, Construction and Engineering in Sydney, Australia. He has a Masters Degree from the University of Sydney and is conducting Research towards a doctorate in the field of corrosion of steel reinforcement in concrete. He has carried out research in the fields of utilization of mineral additives and concrete durability in Australia, UK and USA. He is actively involved on technical committees of the Ash Development Association of Australia, the Australasian Slag Association and the Standards Association of Australia.

R.L. MUNN

Robert Munn, B.Sc., B.E.(Hons.), M.I.E. Aust, currently holds the position of Chief Engineer with Boral Resources, a unit of the Boral Limited Group of Companies. In this role, he provides advice to a number of Boral subsidiaries and is responsible for the Boral Research Laboratory. His experience includes management, technical and sales responsibility in construction materials and particularly ready mixed concrete both in Australia and overseas. He is actively involved with the Standards Association of Australia, the Concrete Institute of Australia and the National Ready Mixed Concrete Association technical committees.

H. ROPER

Harold Roper, B.Sc., B.Sc. (Hons.), M.Eng.Sc., Ph.D., is an Associate Professor in the School of Civil and Mining Engineering, The University of Sydney. He has teaching, research and consulting interests in the field of materials for use in reinforced and prestressed concrete structures. He has thirty years experience in cement and concrete technology and is the author of over seventy research papers on the properties of aggregates, cements and concretes.

V. SIRIVIVATNANON

Vute Sirivivatnanon, B.E., Ph.D., is a Principal Research Scientist at the CSIRO Division of Building, Construction and Engineering at North Ryde, Sydney. He is a Project Leader in the Construction Materials Program specialising in Concrete Technology. His research interests cover corrosion of steel reinforcement in concrete, use of supplementary cementitious materials, quality assurance of concrete cover and selection of sandstone for restoration work.

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