Participation from the Institute's membership is what has kept, and continues to maintain, the Institute's kinship to a learned society. This level of participation allows services to be delivered to members that are intrinsically linked to the Institute's mission. The key objectives of the Institute and its strategic operations aid in the facilitation of growth of shared knowledge within the organisation. This in turn enables members to participate in an independent and yet collective manner in sharing their knowledge, expertise and experience. The Institute gains great benefit from encouraging active participation of members with relevant expertise on specific topics. Critical to maintaining and building the shared knowledge involves the constitution of working groups and committees that have a balanced level of participation, from suppliers, designers, consultants, contractors, the government sector and the research community.

A paper in this issue of Concrete In Australia, relating to a report from the activities of the Institute's Durability Committee, highlights the importance of this participation and how this provides positive outcomes for the industry in general. Over the past two years the Institute has seen the need to provide leadership in this important area and recognises the requisite to ensure that industry sectors are represented and that outcomes cover a broad spectrum of aspects associated with the wider concrete industry.

While it is important to maintain the participation of members within the Australian community, it is often just as important to keep abreast of work being undertaken within the international scene. In this issue, Frank Papworth, the Chair of the Institute's Durability Committee and deputy of the Australian National Member Group (ANMG) of fib provides an informative and comprehensive paper on the deliberations of the fib Commission 5 (C5) "Structural Service Life Aspects" meeting recently held at the Washington Congress. The information that Papworth has been able to bring back additionally serves as invaluable information to the activities within the six Task Groups of the Institute's Durability Committee.

This issue of Concrete In Australia also features a piece from the ANMG of fib, highlighting a two-day course that will be delivered in Brisbane and Sydney during November on Modern Concrete Technology. It is important to not only ensure that technical dissemination is provided to the Australian industry, but also to ensure that younger engineers and general practitioners are kept informed of necessary practical knowledge that is able to be directly applied to their work practices. This issue features a piece on the Institute's upcoming third National Seminar Series on Durability – Principles for Practitioners to be held in November.

The Institute's second National Seminar Series for 2010 on Serviceability – Design for Deflection and Crack Control held in July proved to be not only very popular, but very successful in providing attendees with a comprehensive program developed and delivered by Professor Ian Gilbert, a leading international expert on the topic.

This issue of Concrete In Australia additionally contains two very interesting technical papers on the feature topic related to local projects. Ian Godson provides a paper on cathodic protection to Swanson Dock West in Melbourne and Warren Green provides an informative paper on Durability assessment design and planning on the Port Botany Expansion Project.

Furthermore, this issue highlights the call for submissions for the 2011 Awards for Excellence Program. I would like to take this opportunity to encourage all interested members to submit an entry for this awards program.

Fred Andrews-Phaedonos
President, Concrete Institute of Australia
president@concreteinstitute.com.au
Contents

2 President’s report
4 Discussion
6 News
17 Projects
19 FEATURE TOPIC: Design for Durability – Principles for Practitioners
  Durability assessment, design and planning – Port Botany Expansion Project 22
  Report from the CIA Concrete Durability Committee 29
  fib and Durability 36
  Cathodic protection to Swanson Dock West in Melbourne 45
  Concrete Structure Ownership and Management: Part 1 54

ALSO IN THIS ISSUE
  CCAA Library 65
  New members 66

Pictured here is the recently completed Go Between Bridge in Brisbane’s inner city.

PHOTO: BRISBANE CITY COUNCIL
Dear Concrete in Australia,

The article by Wolfgang Merretz and Godfrey Smith “Achieving concrete cover in construction”, in the March 2010 issue raises some important points about the difficulties faced by contractors in meeting performance based durability parameters.

Here is an opportunity for the CIA Durability Committee to provide some much needed recommendations on the durability tests to be employed in the Australian industry. Hopefully precast concrete and ready mix concrete suppliers would then have the confidence to undertake durability testing to pre-empt the request for such data and provide the necessary correlations between diffusion tests for design purposes and routine quality control tests. The availability of the data in turn would provide reassurance to all parties and reduce the risk of disputes over compliance.

The authors appear to single out diffusion testing as the problem rather than the ASTM C1202 test. The paper by Wilfried Krieg “Rapid Chloride Permeability Testing – A critical review”, presented at the 2006 Bahrain Conference found that the ASTM C1202 coulomb did not correlate with chloride ion migration and merely reflected the electrical conductivity of the sample. The latter is influenced by numerous factors, not least the silica fume content. We had our own experience of the eccentricities of the test when asked to investigate non-compliant ASTM C1202 results at a facility in Qatar. The test results were non-compliant but no-one on site was certain why or what that meant for the large number of piles they represented. The problem was eventually tracked to poor dispersal of the silica fume. By a tortuous route we were able to translate this into a meaningful effect on durability. The use of a test with a more direct relationship to chloride migration would have saved the client much time and our modest fees.

Yours sincerely
Don Wimpenny

Figure: This graph showing rapid chloride permeability was extracted from Merretz W and Smith G, (2010), “Achieving concrete cover in construction”, Concrete in Australia, Vol 36 No 1, pp 43.
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Inner city bridge opened in Brisbane

Brisbane’s first inner city road bridge in nearly 40 years has been completed, connecting the northwest of Brisbane River (Milton) to South Brisbane and West End.

The Go Between Bridge was completed in June by the Hale Street Link Alliance. The alliance consisted of Seymour Whyte, Macmahon Constructions and Bouygues Travaux Publics, along with designers Hyder Consulting and the principal Brisbane City Council. Independent verifier for the project was Arup.

The design and construct contract was originally awarded in May 2007 and construction started in June 2008. Brisbane City Council said construction of the project cost $328 million, which was a revised cost following a saving of $42 million during construction. The original projected cost was $370 million.

It is a three span, twin concrete box girder structure built using balanced cantilever construction. The main bridge span is 117m long, supported by two river piers located 74m north and 80m south of the abutments on each river bank. Piles for the foundations of the bridge were bored 45m below the river bed.

The bridge consists of 94 concrete segments – each 5m wide. The first segment was poured in April 2009. By 3 December the final connection between the southern and northern deck had been poured. Approximately 1000t of reinforcing and tensioning steel and 5300m³ of concrete were used in construction.

Brisbane City Council said that in a Queensland first, the construction of the first concrete segment was poured in April 2009, and in July the new $328 million Go Between Bridge in Brisbane’s inner city was officially opened.

Concrete Institute of Australia

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the Go Between Bridge used a special concrete mix. The mix was designed to specifically ensure "maximum durability and constructability".

"The concrete – 50MPa concrete mix at 28 days – has a lower than usual quantity of cementitious material (380kg of cement and 100kg of fly ash) and low water/cement ratio (0.36), as well as a new generation admixture," a spokesperson for Brisbane City Council said. "As a result of the specially created mix, the Go Between Bridge has reduced its effective carbon footprint by using less cementitious material, a lower heat of hydration, and a very high early strength (25MPa at 15h), allowing the segments to be completed and the next one poured more quickly."

The concrete was also prepared differently, using the only wet batching plant in Brisbane to ensure the best quality. A wet batching plant fully mixes concrete, including water, in the plant rather than mixing in each truck.

Hale Street Link Alliance also used maturity meters to determine the compressive strength of concrete at an early age.

"The level of maturity determines the temperature of the concrete and from this, can determine the concrete strength. This has a number of advantages when compared to the standard cylinder methods. Results of concrete strength can be determined instantaneously onsite," the spokesperson said.

Brisbane City Council said it was a significant inner-city infrastructure project and a key component of its long-term transport plan. The bridge was deemed necessary to offer relief to existing bridge crossings, improve accessibility for motorists, public transport, pedestrians and cyclists.

Brisbane City Council Lord Mayor Campbell Newman said: "The bridge will relieve traffic congestion around the CBD and provide better access to some of Brisbane's most popular precincts."

For example, the bridge provides a link between the cultural hub of South Brisbane and West End and the sporting precinct around Suncorp Stadium.

The project includes a four-lane cross-river bridge as well as a two-lane viaduct on Coronation Drive over the Hale Street intersection on the north side, significant roadworks in the vicinity, dedicated pedestrian and cycle ways, toll collection systems and equipment, landscaping works and environmental works, and improvements and alterations to utility services.

The inner-city location and large peak hour traffic volumes presented the design and construction team with significant challenges. Innovative designs, detailed planning and traffic management, combined with extensive community consultation have helped minimise disruption.

Motorists using the new toll bridge will pay $1.50 until the end of the year and then $2.00 from 1 January to 30 June 2011. From 1 July 2011, it is expected that a toll of $2.35 will come into effect.

To celebrate the opening of the bridge, a sell-out crowd of 5000 enjoyed The Go Between Bridge Concert. Brisbane's latest musical export The John Steel Singers warmed up the crowd before surviving member in the band of the bridge's namesake, Robert Forster, took to the stage singing many of The Go-Betweens' hits. The Go Betweens are considered one of the Brisbane music scene's great exports.

Forster told ABC Triple J having a bridge named after his band was an unexpected honour.

"It's slightly surreal having a bridge named after a group because when you start a band, having a bridge named after you is not one of the things that comes to mind," he said.

Construction additives production facility opens

Construction admixtures are to be produced in a new facility in Melbourne after global chemical company WR Grace & Company (Grace) opened its new building in Epping.

The facility will provide areas for manufacturing operations, research and development activities and technical services. The products produced onsite will include concrete admixtures used in commercial, institutional and residential construction, and additives used in cement processing to improve energy efficiency and enhance the characteristics of the finished product.

The site also houses corporate administrative functions and warehousing for Grace's Materials Technologies product group. Grace said the facility is expected to enhance productivity by bringing employees together that had previously been located at separate offices within the same city.

This site is the second in less than a year the company has opened in Australia, following a facility in Rowville, Melbourne that supports pharmaceutical and biotechnology customers with chromatography products.
‘Lockable Dowel’
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The ‘Lockable Dowel’ allows initial shrinkage of the concrete to take place and is then locked in position with a mechanical plate and a controlled amount of epoxy resin. The locked dowel continues to transfer vertical load, but prevents further movement taking place.

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A NSW Concrete Institute Branch initiative – 2010 University & Industry Evening series

Wollongong University – March 2010
Newcastle University – August 2010
University of Technology, Sydney – September 2010

This joint initiative between the Concrete Institute of Australia and participating Universities showcases academic expertise involved in cutting edge industry research coupled with practising engineers, consultants, contractors and our industry leaders as they relate to various aspects of the broader concrete industry.

Each university has forged a distinctive identity amongst the Australian universities and is confidently building a national and international reputation for quality education and research. The seminars have been designed to provide great value to all involved in the various concrete market segments – those who wish to keep up with the latest, and emerging, developments with a focus on the next generation of engineers, our graduate and undergraduate students.

The event series provides a unique point of access for students, and those at the beginning of their careers, to some of the latest developments in concrete and design technology as presented by experts in the field using examples from their current practice.

On 17 March the University of Wollongong hosted the first in the NSW series with Professor Chris Cook, Dean of Engineering at Wollongong University opening the proceedings. Dr Muhammad Hadi from the University’s School of Civil Engineering and Tim McCarthy, Professor of Steel Structures and Design represented the university discussing two very different aspects of the concrete industry.

Hadi presented a review of current research in the use of Fibre-Reinforced Polymers for strengthening structures and how this relates to current practice. McCarthy reviewed the extensive work being carried out into how the structural design approach can result in more sustainable buildings and, contrary to popular belief, how sustainable design can equate to the lowest cost option.

Leigh Appleyard and Peter Geoghegan, from ACOR Appleyard Consultants, provided an industry perspective of current practice on the night. Appleyard gave an informative presentation and overview of the proposed changes to the Australian Standard BD-025, Residential slabs and footings.

Peter Geoghegan discussed the engineering aspects and technical excellence required in both precast and glass techniques that came together to

---

**EUROPE’S PROVEN SOLUTIONS FOR CONCRETE EQUIPMENT CLEANING PROBLEMS**

Hallweld Bennett has announced its appointment as Australian distributor for the range of concrete equipment cleaners produced by respected German company LEYCO CHEMISCHE LEYDE.

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Or visit: hallweld.com.au
make the “Berry Recreation Hall” one of the winners of the Concrete Institute of Australia 2009 Excellence Awards for Building Projects. This project has also been awarded the NSW 2008 RAIA Blacket Award for Public Architecture.

At the time of writing the second university and Industry Evening was due to be held on Thursday 5 August at the University of Newcastle. This seminar has been designed to provide a comprehensive investigation of a number of topics covered in the university curriculum with Professors Mark Masia and Rob Melchers presenting their current research projects into Structural Masonry and Steel Corrosion.

Industry representatives from the pre- and post-tensioned, precast and structural concrete areas will give presentations on their areas of expertise and encourage discussion with the audience.

Our third University & Industry Evening, which also promises to be an entertaining and informative event, will be held in September at the University of Technology, Sydney City Campus. Registration for all NSW Branch University & Industry Evening events is free of charge for all students. Access to presentations made on the night will be made available to all attendees after the event.

Full details of the event, online registration and payment can be found on the NSW seminars listings of the Institute’s website www.concreteinstitute.com.au.

Vinsi launch

Brad Dockrill and Warren Green, formerly of Izzat Consulting Engineers, have established a new business Vinsi Partners. It is an independent, consulting engineering firm, delivering specialist services in engineering (structural/civil), corrosion and asset control and durability assurance.

The company will operate Australia-wide in sectors including mining, maritime, water and wastewater, industrial, manufacturing, infrastructure, government authorities, architects and other consulting engineers.
The Concrete Institute of Australia was established to encourage the development of concrete technology and practice in Australia, and to foster improvements in the standard of concrete construction. To recognise the many significant contributions to these objectives and to publicise the many excellent examples of concrete structures erected in Australia each year, the Institute established a Biennial Awards Scheme in 1971. In 2011, this scheme will be expanded to include State Branch based Awards prior to the National Awards which will be presented at Concrete 2011 in October, 2011.

CATEGORIES
Awards will be made in the following categories:
Projects (includes Building Projects and Engineering Projects)
International Projects (must have a significant Australian content)
Technology (includes intellectual materials, physical materials, products for sale, and services)

AWARDS
Awards are made for significant contributions to the development of concrete technology and practice or to that of concrete construction as evidenced by:
- the effective use of concrete in a building or structure that reached substantial completion in the period from July 2009 to June 2011
- research publications, design innovations, material or plant improvements, educational or other activity.

Awards, consisting of citations to members of the team responsible for the achievement, will be presented at Institute State Branch functions in September 2011, and at the Biennial Conference to be held in Perth, Western Australia, from 26 to 28 October 2011, and will subsequently be given national and international publicity. A full colour poster display of all complying entries will be mounted at the Biennial Conference for its duration. A presentation of all National entries and winners will be given at the Conference Dinner to be held in October 2011. Entries will also feature on the Institute’s web site at www.concreteinstitute.com.au. A Commemorative Booklet containing all entries and winners will be distributed at the conclusion of the National Awards presentations, and copies will be available for all entrants.

THE KEVIN CAVANAGH MEDAL
In 1991 the Council of Concrete Institute of Australia established the Kevin Cavanagh Medal for Excellence in Concrete, which recognises an overall winner. This award will be judged from all National winners (excluding International Projects) in both projects and technology categories on the basis of being an outstanding contribution to the quality of concrete construction in Australia.

AWARD FOR ENVIRONMENTALLY SUSTAINABLE USE OF CONCRETE
In 2008, the Council of the Concrete Institute of Australia established the Award for Environmentally Sustainable Use of Concrete which recognises entries which demonstrate significant advances in environmental sustainability in concrete. This award will be judged from all relevant entries in both projects and technology categories (excluding International Projects). For this award, a recognised expert in sustainable development will be invited to assist the judging panel.

www.concreteinstitute.com.au
Precast facility in overdrive

A precast facility in Eagle Farm, Brisbane is in overdrive as it produces the 21,000 tunnel segments for the $4.8 billion Airport Link project.

At the time of writing the facility had manufactured more than 40% of the tunnel segments for the project. The facility, which employs up to 95 workers, had produced 9000 segments to date, equating to 900 rings or 1.7km of tunnel.

As the Airport Link’s tunnel boring machines (TBMs) tunnel under Brisbane, these segments will fit together to form rings creating the concrete lining of the Airport Link tunnel. Each ring has an internal diameter of 11.4m and is made up of nine segments plus a keystone which locks the ring into position.

Thiess John Holland project director Gordon Ralph explained the process involved the TBMs pushing forward 2m by hydraulic thrust cylinders while the 12.48m cutterhead rotates to cut through the ground. Once the 2m has been excavated the TBMs cease excavation and commence the installation of one full concrete ring, segment by segment. He said: “The segments, which weigh up to 8t each, will be automatically installed by the TBMs as excavation progresses using a massive vacuum erector.”

He said the precast facility delivering the segments was unique given the high quality of production required. “A vigorous regime of sampling, testing, measuring and inspecting is undertaken to ensure that each individual segment is compliant to the strict design criteria required for construction of the tunnels,” Ralph said.

The precast facility will manufacture ten different types of concrete products for the project. These include 436 box girders for the Airport Roundabout Upgrade, 800 T’roff girders for Airport Link, Northern Busway and Airport Roundabout bridges, concrete panels for noise barriers and retaining walls and architectural features for the Airport Link tunnel.

Local companies Neilsens, Wagners and Micodie have secured the major contracts for the supply of sands and aggregates, cements and delivery services respectively for Airport Link, Ralph said.

Emersteel, Idec and Hymix secured the top three major contracts for the supply of, respectively, mould fabricators, sheds and concrete.

BrisConnections CEO, Dr Ray Wilson said production of the concrete tunnel segments at the facility is expected to be complete by the end of January.

The Airport Link project is the largest infrastructure construction currently underway in Australia. The Airport Link and Northern Busway (Windsor to Kedron) are on schedule for completion by mid 2012, with the Airport Roundabout Upgrade set to open first next year.
The existing R&D Tax Concession and R&D Cash Offset is a critical, self-assessed and non-competitive program that supports innovation for Australian companies. Originally introduced in 1985, the underlying definition of what is “R&D” has been largely unchanged over the past 25 years.

However, if the federal government has its way, the current system will be reformed and replaced by a new R&D Tax Credit.

Current law
Current law provides a tax benefit equal to 7.5 cents in the dollar. For companies whose R&D spend is above their three year average expenditure, a benefit equal to 22.5 cents in the dollar may apply on the amount exceeding the three year average.

R&D activities that may be eligible include the creation of new knowledge or new or improved materials, products, processes, devices or services. The approach taken must be experimental and involve either innovation or high levels of technical risk. Both “core” and “support” activities may be eligible; the support activities must be “directly related” to the core activity, for example background research, pilot-scale/full-scale trials and prototyping.

Examples of potential R&D projects in the Concrete Industry include:
• Development of new concrete formulations with enhanced properties such as a lightweight, but strong, concrete that is formulated with a polyurethane additive.
• Experimental trialling and evaluation of the potential to use water based waste paint as an alternative to water in the production of concrete.
• Development of new design specifications and construction methodologies using “green concrete” or geopolymer technology as an alternative to traditional concrete.
• Design and development of a novel form work support table with inbuilt lifting apparatus.
• Development of new and improved construction methodologies for large scale polished concrete floors in high traffic commercial buildings.

Proposed changes to legislation
Behind the proposed R&D Bill is the government’s policy of redirecting R&D benefits to smaller companies and funding so-called “genuine R&D” as opposed to large-scale R&D that occurs in a quasi-production environment.

The main provisions of the proposed R&D Tax Credit are:
• A two tier system:
  – Companies with turnover of $20 million or less will be able to access a credit to the value of 15 cents in the dollar of eligible R&D spend
  – Importantly, if these small companies are in tax losses, they can “cash out” the R&D benefit and the standard tax deduction, providing a cash refund of 45 cents in the dollar, uncapped
  – Companies or corporate groups with turnover above $20 million will be entitled to 10 cents in the dollar of eligible R&D spend.
• More relaxed restrictions on foreign ownership of results of R&D activities.
• A tighter definition of R&D activities. Eligible R&D will include activities that involve scientific method, seek new information and are technically challenging and require experimentation to resolve uncertainty.
• A tighter “dominant purpose test” regarding supporting activities – this will restrict the scope of “production related” trials and prototype R&D activity.
• A tighter registration process, giving AusIndustry greater powers to reject claims and modify applications prior to registration. This would result in a move away from the current self-assessment model.
• Introduction of more complex “feedstock” rules.

The government has stated that the new R&D credit should be revenue neutral. However, as a result of the tighter eligibility rules mentioned above, the proposed Bill...
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The Awards for Excellence is an essential program of the Concrete Institute of Australia and aids in promoting the values and expectations of Concrete Institute of Australia to its members and the broader industry. The awards present the ideal opportunity to highlight your accomplishments to your peers and the Institute's membership, which provides a message of encouragement to other members to aspire to achieve the same level of excellence.

The 2011 Awards for Excellence program will be expanded to include state branch awards. This provides an opportunity for each state to develop an event around which the state awards would be presented. Winners of State Branch Awards for Excellence will then be judged as a separate group for National Awards for Excellence to be presented at the National Ceremony during the 2011 Biennial Conference.

State branches of the Institute are Queensland, New South Wales, Victoria, Tasmania, South Australia and Western Australia. The Northern Territory is included in the South Australian Branch, and the Australian Capital Territory is included in the New South Wales Branch. Technology entries are typically of national relevance and will only be judged for National Awards. International Projects, by their very nature are not state related, and will only be judged for National Awards.

Further information on the Awards Program, submission dates and conditions of entry can be found on the Institute's website www.concreteinstitute.com.au.

The Institute strongly encourages our interested members to submit an entry to highlight to industry their achievements and be recognised for the valuable contribution they have made. All submissions for the Awards Program should be completed online, which also allows entrants to save their submissions and return at a later date via a secured login.

Awards are made for significant contributions to the development of concrete technology and practice or to that of concrete construction as evidenced by:

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- research publications, design innovations, material or plant improvements, educational or other activity.

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Update on Concrete Institute Technical Projects

Standards
A key activity within the Institute’s Standards portfolio in recent months has focused on representation on BD-010: Cement and the draft AS 3972 Portland and Blended Cements.

In recent correspondence, members would be aware that the Institute’s elected Council instructed the Institute’s Nominated Representative to Standards Australia’s Committee for Cement (BD-010) to cast a negative vote on the proposed draft AS 3972. The two key changes in the draft related to providing for an increase in the mineral additions (limestone) from the existing 5% to 10% and allowing the use of cement kiln dust (both changes relate to type GP cement). This decision to cast a negative vote was based on advice from the Institute’s Standards Committee relating to concerns on current information and data placed before the BD-010 Committee. These concerns arose relating to the potential concrete durability aspects associated with the proposed changes, together with limited local technical information. It was felt that prior to adopting the changes, that further tests on strength development, durability performance and volume changes are needed to establish effectiveness, assess potential side effects on design standards such as AS 3600 (Concrete Structures) and AS 5100 (Bridge Design), and consider implications on the total carbon footprint of concrete structures in the future.

A further meeting of the Standards Australia’s Committee was planned for 20 July at which time the result of public ballot votes was to be considered and alternative positions would be discussed in an attempt to reach consensus. If still unresolved, the Institute is willing to assist by providing a forum for debate amongst the broader industry and our diverse membership to help resolve this matter.

Education: Serviceability – Design for Deflection and Crack Control
At the time of writing, the second national seminar series Serviceability – Design for Deflection and Crack Control was soon to be delivered. The seminar was developed by Professor Ian Gilbert. The lectures and demonstrations were developed to provide essential knowledge on how to perform the necessary checks when designing concrete structures for the serviceability limit states.

Durability – Principles for Practitioners
The third National Seminar Series for 2010 will be delivered throughout November. The seminar is being developed by Frank Papworth, the Chairman of the Institute’s Durability Committee and expert in the field.

AS3600 – Regional Seminars
In May 2010, the AS3600 program that was developed in 2009 was delivered in both Hobart and Newcastle. Both these seminars were well attended with attendees providing positive feedback. It is the intention to continue to deliver key educational programs to regional centres to ensure that those in industry receive the valuable information and knowledge dissemination attained from Institute educational programs.

Publications
Most recent information relating to the status of the Institute’s publications portfolio is provided below:

Z 15 – Cracking in Concrete Floors and Paving
At the time of writing Members had recently received notification as to the availability of Z15 – Cracking in Concrete Floors and Paving for peer review. All peer review comments will be collated and the publication will be amended where required with peer review comments taken into account prior to final publication.

Z 7 – Durability Committee Publications
A detailed paper has been included in this issue of Concrete In Australia to provide members with information relating to the current activities of the six Task Groups that have been formed and charged with the responsibility of developing six respective publications.

For further information on these activities, contact Ben Cosson at the Institute’s national office on (02) 9736 2955.

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Modern concrete technology – two day course

The ANMG of fib is to deliver a comprehensive two day course on Modern concrete technology: The course will be held in Sydney (8-9 November) and in Brisbane (11-12 November). The course will be delivered by Dr Frank Dehn from Germany who will be complemented by local representation to provide local context to the course. Dehn has studied civil engineering at the University of Karlsruhe and received his PhD from the University of Leipzig. He presently works as a managing director of the Leipzig Institute for Materials Research and Testing (MFPA) and serves as a Professor for construction materials technology at the Institute for Mineralogy, Crystallography and Material Science at Leipzig University.

The ANMG of fib is looking forward to being able to disseminate international technical knowledge and expertise to the Australian industry. Delegates will receive comprehensive technical course materials and a Certificate on completion.

The ANMG has recently established a website which is currently being hosted on Concrete Institute of Australia’s website. The ANMG of fib encourages all interested members of Concrete Institute of Australia and those within industry to find out more about this course and register via www.concreteinstitute.com.au.

Course outline
The fib Modern concrete technology course will include:

TECHNOLOGICAL FUNDAMENTALS FOR MODERN TYPES OF CONCRETE
- Raw materials/ingredients (cement, aggregates, mixing water, additions, admixtures, fibres)
- Concrete mix design
- Production, processing and curing
- Particularities during hardening
- Temperature and strength development
- Microstructure
- Behaviour in compression and tension
- Time dependant properties (creep, shrinkage)
- Durability

HIGH-STRENGTH CONCRETE / HIGH-PERFORMANCE CONCRETE (in addition to the concrete technological fundamentals)
- Mechanical properties (strength, strain, young’s modulus etc.)
- Chemical properties (dissolving and expansive attack etc.)
- Physical properties
- Examples high-strength concrete / high-performance concrete.

SELF-COMPACTING CONCRETE (SCC)
- Rheology of cement paste
- Types and mix designs for SCC
- Mix ingredients
- Testing methods (fresh/hardened concrete)
- Hardened SCC properties
- Mixing, processing and curing
- Surface appearance
- Formwork pressure
- Examples SCC / fib TG 8.8 / TG 8.9.

FIBRE REINFORCED CONCRETE (FRC)
- Technology
- Fibre types / mix design
- Fresh FRC properties
- Hardened FRC properties
- Mixing, processing and curing
- Flowable FRC (Fibre Orientation)
- Effectiveness of fibres (durability, fire etc.)
- Examples FRC / fib TG 8.3.

ULTRA HIGH-PERFORMANCE CONCRETE (UHPC)
- Principles/granulometry
- Physical/chemical texture optimisation
- Selection of suitable ingredients
- Mixing, processing and curing
- Microstructural and chemical properties
- Examples UHPC / fib TG 8.6.

Australian National Members Group of fib – Course

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FEATURE TOPIC: DESIGN FOR DURABILITY – PRINCIPLES FOR PRACTITIONERS

National Seminar Series:
Design for durability – Principles for practitioners

The third program for 2010 in the Concrete Institute of Australia’s National Seminar Series will be a one day course on Design for durability – Principals for practitioners scheduled to be delivered across Australia throughout November. Topics for the 2010 National Seminar Series and the subsequent program development have arisen out of market research into member and industry needs, whereby a focus has been placed on the key topics where further information is being sought by industry and where knowledge gaps may lie.

The course is ideally suitable for materials suppliers, site engineers, design engineers, and owner representatives.

The Institute encourages all interested members and those in industry to register for this event via www.concreteinstitute.com.au.

Program outline

The National Seminar Series: Durability – Principles for practitioners will include information on the following:

DURABILITY DEFINED

This section will provide a background to durability that will help delegates understand the concepts that are incorporated in Durability Design including:

- Methods to undertaking durability design
- Risk Assessment
- Implementations of design through construction
- Service life condition control
- Issues arising from inadequate design and construction and how to handle them
- Deterioration mechanisms.

At the end of this session delegates will have a basic idea of the process of deterioration, the method of implementing durability design through the projects life cycle, and the key factors in respect of the owners input.

AUSTRALIAN STANDARDS, COMMENTARIES & REFERENCE DOCUMENTS

Here, the objective will be to highlight the durability requirements, conflicts, limitations to the requirements, and conventional methods of overcoming those limitations for each of the following Australian Standards.

- AS 3600 – Concrete Structures (CCAA TN 37)
- AS 5100 - Bridges
- AS 4200 – Marine Structures
- AS 2159 - Piling
- AS 3735 – Liquid Retaining Structures
- AS 3962 – Marinas
- AS 4048 – Concrete Pipes
- AS 3610 – Formwork.

AS 3600 will be the key standard as it contains a lot of information on exposure classifications and tolerances that is common. Reference to other standards will be to highlight differences to AS 3600.

At the end of this session delegates will have a good understanding of how to use the Australian Standards and some idea of overcoming limitations.

MATERIALS REQUIREMENTS

This section will review Codes and guidance notes that provide information on materials requirements that may affect the durability of the structure.

It will give delegates a background to code requirements for materials that affect the durability of a structure. It will also look at where code requirements might be improved on to better enhance durability.

INTERNATIONAL STANDARDS & REFERENCE DOCUMENTS

This section will outline the requirements and significance of differences between Australian and International Codes.

TESTS FOR DURABILITY

Tests to ensure durability occurs during four stages of construction. Common tests used during each of the stage will be reviewed as discussed below.

Testing of Raw Materials
- As required in Codes discussed in Section 3.

Trial Mix Tests
- Compressive strength
- Workability
- Density
- Permeability
- Chloride diffusion
- Temperature Rise.

Testing of Fresh Concrete(QA)
- Workability
- Air content.

Site Testing of Concrete(QA)
- Pre and post pour cover checks
- Compressive strength
- Capillary ingress tests
- Rapid Chloride Permeability
- Temperature.

At the end of this session delegates will have an insight into most tests that are used for durability assurance in Australia and understand their limitations and application. The course does not include testing undertaken during the service life or those used to evaluate problems during construction as this topic is a course in its own right.

DURABILITY SPECIFICATIONS

Many specifications for high risk and structures in severe exposures have specification requirements that go outside of the code requirement. Main Roads specifications from different states will be reviewed to indicate the reasons behind these deviations.

Examples of major project specifications will also be reviewed to indicate why the durability specifications went outside of the codes and the risks arising from doing so.

At the end of this lecture delegates will have a good understanding of what aspects may require special consideration and how it can be treated. More importantly they will understand the commercial risk of using non-standard solutions.

WORKSHOP

Delegates will be split into groups to undertake a preliminary durability design of common structures. The results will be presented to the group and a discussion held on each design as a means of highlighting the issues covered.
DURABILITY

Design for durability – a check list of challenges

By Don Wimpenny, Halcrow Group

When I joined my present employer in 1992 they were being pursued over durability problems with a large dry dock built 15 years previously in the Middle East. The cover, water/binder ratio and cement content used were consistent with the British Standards of the day. During this period there was a growing awareness of the importance of chloride induced corrosion, but admixtures were regarded with suspicion and only Portland cement (PC) and sulfate resistant Portland cement were readily available in the region. Despite care and diligence during construction and a nominal cover of 60 mm, widespread cracking and spalling quickly developed.

The aftermath of this concrete durability problem was still fresh in my mind when asked to prepare a durability plan for a tunnel scheme in Egypt. The scheme involved four 5.1 m diameter tunnels forming an irrigation syphon under the Suez canal. The ground was so saline that crystals would readily form in any standing groundwater. This first durability plan used deterioration models based on Fick’s second law of diffusion with crude adjustment for the effect of wetting and drying.

The specified mix used a blend with 70% slag (imported specially from Europe) to provide good resistance to chlorides and sulfates and selective use of coatings as an additional protective measure. Superplasticisers allowed the water/binder ratio to be kept to less than 0.35 despite the high ambient temperature. The concrete was tested using water absorption and permeability and oxygen diffusion. Corrosion ladders were also incorporated to allow long-term monitoring.

Fast forward fifteen years to 2004 and to the construction of a new deep water port in Morocco. The contractor’s design included a probabilistic durability design which allows a range of possible values for each assumption, for example cover values, in order to produce a reliability index similar to that used in structural design. The concrete mix was a triple blend of Portland cement, fly ash and silica fume. Chloride diffusion coefficients were measured during mix development and quality during construction was monitored using resistivity. The durability design included corrosion monitoring and cathodic protection.

These three projects illustrate the progress in both durability design methods and available materials over the past 30 years. In Europe, the design standards now include comprehensive exposure classes and recommendations for concrete for different design lives based on durability modeling (as well as some horse-trading). Australia through the Concrete Institute of Australia has embarked on a similar exercise and will face various challenges, some of which include:

- the design recommendations have a rational basis
- impact on designers, producers and laboratories has been considered
- durability tests during mix development are relevant to design method
- durability tests during construction are rapid and reliable
- specified limits for tests are appropriately strict but achievable and allows for the effect of binder type
- allows for difference in quality between standard samples and structure due to cracking, compaction and curing
- includes guidance on maintenance and monitoring.

Time will tell whether the new guidance and, more importantly, structures designed using it prove durable.

### Table 1. Three projects illustrate the progress in durability design methods over the past 30 years.

<table>
<thead>
<tr>
<th>Case</th>
<th>Cube Strength (MPa)</th>
<th>Binder content (kg/m³) and type</th>
<th>Free water/binder ratio</th>
<th>Admixture</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry Dock</td>
<td>30</td>
<td>390 (PC)</td>
<td>0.51</td>
<td>None</td>
</tr>
<tr>
<td>Suez Syphon</td>
<td>45</td>
<td>430 (70% slag)</td>
<td>0.34</td>
<td>Superplasticiser</td>
</tr>
<tr>
<td>Moroccan Port</td>
<td>75</td>
<td>480 (20% fly ash and 5% silica fume)</td>
<td>0.35</td>
<td>Superplasticiser and retarder</td>
</tr>
</tbody>
</table>

### Inhibiting corrosion

Grace Australia has introduced the next generation of corrosion inhibitor, DCI-N – an admixture system that interacts with the embedded steel in concrete to prevent chloride induced reinforcement corrosion. The product has been specified on the latest Port of Brisbane wharf project to enhance the durability and maintenance-free serviceable life of the asset.

Chlorides from seawater are a danger to reinforced concrete structures such as wharfs, piers and jetties. Protection of the embedded steel reinforcement is necessary to ensure the structure remains durable.

The company said that during the 90’s Grace Construction Products provided a corrosion inhibiting admixture, DCI-S to another of the Port of Brisbane’s wharfs.

The company said the use of DCI-S has maintained the integrity of the passivating layer covering the steel reinforcement thereby inhibiting corrosion reactions. Grace Australia has since continued to develop its corrosion inhibiting admixture technology.
BCRC and BAAM Consulting have joined forces to provide a comprehensive suite of services for their customers. The new company combines the practical engineering experience of BAAM Consulting in the building sector and the strategic durability focus of BCRC in infrastructure. The result is a comprehensive durability and remedial consultancy capable of serving owners of infrastructure, commercial, public, industrial, marine and residential facilities.

“The advanced inspection techniques, high level materials expertise and durability planning approaches of BCRC complement BAAM’s service perfectly,” said managing director of BAAM Consulting, Grahame Vile.

Bob Munn, managing principal of BCRC in NSW added: “The new company builds on the BCRC global network and experience in infrastructure, including international associates such as John Broomfield (international remedial expert), Dr Peter Farinha (cathodic protection) and Professor Phil Bamforth (international concrete durability expert). It increases our local capacity and experience in the building sector in Australia.”

Vile said: “We are able to exploit the strengths of our IT and practice management software to most effectively serve our growing client base.”

The merged company will also draw on greater technical experience of the durability and remedial specialists to provide expertise in concrete, timber, corrosion and coatings as well as non-destructive evaluation, Vile said.

He explained: “Now, we can offer a more seamless solution. Previously, we would have briefed external specialists and obtained client approval before continuing with a condition assessment or similar report.”

Marton Maroszeczy, foundation professor of construction innovation (UNSW 2003-2007) and concrete and building expert within the group, said the new enterprise also includes technical experts such as Frank Papworth (infrastructure durability planning), Bob Munn (concrete constructability and durability), Dr Zhen-Tian Chang (reinforcement corrosion) and Dr Tony Song (geopolymer concretes).

“We have built a team with exceptional depth in materials science, durability and remedial expertise. Papworth and Munn chair three of the key Standards Australia committees in the area,” Maroszeczy said.

The new business commenced operations from 1 June. The head office is based at North Gosford, while services will continue to be provided wherever clients need them.

Recently, BCRC has had involvement in durability consulting and compliance monitoring for the desalination plants in Perth and Gold Coast and is currently providing durability expertise for the independent verifier of the Port Botany expansion.
DURABILITY

Durability assessment, design and planning –
Port Botany Expansion Project

W Green, Partner, Vinsi Partners, Sydney
G Riordan, Team Leader – Civil Structures, Hyder Consulting, Sydney
G Richardson, Manager – Civil Engineering, Hyder Consulting, Sydney
A Marosszeky, Port Botany Design Manager, Baulderstone, Sydney

Abstract: The design life of the major assets and asset components of the Port Botany Container Terminal Expansion Project in Sydney is 100 years. Durability assessment, durability design and durability planning was an integral part of the delivery phase for the detailed design process to minimise the risks of long term deterioration of the structural assets and asset components. This paper details the durability approach undertaken, design durability risks identified and the durability design details adopted. Ternary blended (52% SL cement, 25% fly ash, 23% blast furnace slag) cement based S50 grade concretes were proposed for construction of the major assets and asset components. Design covers proposed were typically 75 mm.

1. INTRODUCTION

The Port Botany Expansion (PBE) Project comprises a new container terminal on the north-eastern shore of Botany Bay, approximately 12 km south of the Sydney CBD. The new terminal will be located between the existing port and the parallel runway at Sydney International Airport, extending approximately 550 m west and 1300 m north of the existing northern quay of Brotherson Dock container terminal and covering an area of approximately 63 ha. The project essentially involves design and construction of a berth structure and provision of ancillary structures associated with a port facility. Figure 1 is an overall general arrangement sketch of the project.

The Baulderstone Hornibrook Jan De Nul Consortium (BHJD) was awarded the contract for the design and construction of the PBE works. Design sub-contractor was Hyder Consulting in association with Scott Wilson and Golder Associates. Specialist durability consultancy to Hyder was undertaken by Izzat Consulting Engineers.

The Sydney Ports Corporation (SPC) Project Deed for the works required that durability had to be addressed throughout the design and construction of all assets and asset components, and be reflected in the project plans and the maintenance manual.

Durability plans (checklists) were included in design reports. A Durability Report was also produced which represented the team’s consolidated design durability assessment and durability design. The durability plans and design durability assessments demonstrated how the designs, proposed materials and construction, would achieve the durability objectives for the principal structural assets and asset components over their respective specified design life.

2. DURABILITY STRATEGY: DELIVERY PHASES

2.1 Durability philosophy

The protective measures adopted for assets and asset components within the Port Botany Expansion project depended on the risk of deterioration, the cost of preventative measures, the feasibility and cost of remedial actions and ongoing preventative maintenance. These were balanced to arrive at the best whole-of-life cost and optimised value for money.

Durability design objectives contained in the Durability Report and durability plans were translated to the construction process. The outcomes of the Durability plans and Addenda during construction will be incorporated into the Maintenance manual for the structures. A continuous link in durability objectives is thereby established between design, construction and maintenance.

2.2 Detailed design phase durability methodology

The durability assessment, review and planning was an integral part of the delivery phase for the detailed design process. Inputs into the process included:

• concept, preliminary, detailed and final durability assessment and planning;
• material data;
• environmental data; and
• design life requirements.

Outputs to the process included:

• durability checklists and durability plans in design
component design reports;
• input into construction drawings;
• input into specifications;
• input into risk assessment;
• inspection and monitoring requirements of the Maintenance Manual.

2.3 Construction phase durability methodology

Implementation of the durability plan and procedures is an integral part of the delivery phase construction.

Inputs to the process included:
• delivery phase – detailed design outputs;
• construction work method statements;
• material datasheets; and
• test certificates.

Outputs from the process included:
• as constructed drawings;
• trial test results;
• material testing results;
• material datasheets;
• quality assurance records;
• addenda to the durability report;
• inspection and monitoring requirements;
• inspection and monitoring schedule;
• maintenance schedules;
• remedial methods (if required);
• maintenance strategy;
• commissioning records; and
• Maintenance manual.

Following construction, Addenda to the Durability Report and Durability Plans will be issued containing any non-conformances and the durability impacts thereof plus any recommendations made and rectification work undertaken.

The Maintenance Manual that will be prepared for the project will incorporate the outcomes of the durability report, durability plans/checklists and addenda and will suitably address inspection and monitoring requirements during service life.

3. DESIGN LIFE REQUIREMENTS

A critical factor in durability planning is the design life criteria determined for assets and asset components within a project. Sydney Ports, through its planning processes and specification for the project, nominated the minimum design life requirements and provided definition of the meanings.

"Design Life" for the project means the period over which an asset or asset component must perform its intended function without replacement, refurbishment or significant maintenance. In the case of maritime assets and asset components, significant maintenance would include works that would result in interruption to the operations of:

• Port Botany, including commercial shipping and tug operations; or
• the new boat ramp facility.

The design life of the major assets and asset components are provided in Table 1.

In addition, the Sydney Ports Project Scope and Technical Requirements (PSTR) indicated that where an asset component has a design life less than the design life for the asset of which...
the asset component forms a part then, the asset component and asset must be designed for ease of replacement, refurbishment and significant maintenance of the asset component after the expiry of the asset component’s design life.

4. ENVIRONMENTAL CLASSIFICATION

4.1 Micro-Environments

The structures are located in a marine environment. An environmental assessment identified four dominant micro-environments in which assets and asset components will be constructed. Table 2 summarises the four micro-environments and includes the characteristics of each.

4.2 Groundwater and soil analysis

Information regarding contaminants that are deleterious to concrete, steel and plastic asset components in the soil and groundwater was determined by Golder Associates. Groundwater sampling and testing was conducted at groundwater monitoring wells because of their accessibility, proximity to proposed utilities and port components, and to achieve optimal coverage across the site. Soil samples were collected at near surface and at selected depth intervals from test excavations.

4.3 Atmospheric pollutants

In terms of the Zone 3 and Zone 4 micro-environments in Table 2, it was known that atmospheric gases other than carbon dioxide may be present (e.g. sulphurous oxides, nitrous oxides, carbon monoxide) that become mildly acidic when combined with water/ water vapour. Secondary pollutants that are formed by atmospheric reactions of primary pollutants (e.g. ozone, acid rain) may also be present. Emissions from marine vessels, motor vehicles, rail, aircraft and nearby industries as well as atmospheric particulates and aerosols may also provide pollutants. Ultra-violet and other forms of radiation also needed to be considered.

---

Table 1. Minimum design life for major Assets and Asset Components.

<table>
<thead>
<tr>
<th>Asset</th>
<th>Minimum design life</th>
<th>Asset component</th>
<th>Minimum Design Life</th>
</tr>
</thead>
<tbody>
<tr>
<td>New terminal area container berth structures and tug berth structures, including: a. permanent fixings cast into concrete structures for attachment of asset components to berth structures including fenders, fender chains, bollards, ladders, deck-mounted navigation aids</td>
<td>100 years</td>
<td>Impressed current cathodic protection systems for structural steelwork or for steel embedded in concrete</td>
<td>50 years</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Bollards</td>
<td>50 years</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Edge protection to quay crane rail rebates and cable slots</td>
<td>40 years</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Lids, frames and edge protection to quay crane cable pits, shore power supply pits, ships’ water pits and spare pits</td>
<td>40 years</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Container berth structure fenders, including fender panels and fender chains</td>
<td>30 years</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Tug berth structure fenders</td>
<td>15 years</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Ladders</td>
<td>30 years</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Petrolatum tape wrap systems or other types of encapsulation systems for the purposes of corrosion protection</td>
<td>30 years</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Protective coating systems for the purposes of corrosion protection</td>
<td>15 years</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Bollard fixing bolts</td>
<td>30 years</td>
</tr>
<tr>
<td>Boat ramp structure</td>
<td>50 years</td>
<td>Bearings</td>
<td>75 years</td>
</tr>
<tr>
<td>Reclamation edge structures</td>
<td>100 years</td>
<td>Reinforced concrete barriers and parapets</td>
<td>50 years</td>
</tr>
<tr>
<td>Bridge structures, including: a. approach slabs b. expansion joint systems c. bridge deck waterproofing under deck wearing surface</td>
<td>100 years</td>
<td>Steel barriers, guard railings</td>
<td>40 years</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Deck wearing surface</td>
<td>15 years</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Impact attenuators for bridge piers</td>
<td>25 years</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Bollards</td>
<td>30 years</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Joint sealants</td>
<td>15 years</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Footpaths and shared paths</td>
<td>20 years</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Pavement linemarking</td>
<td>5 years</td>
</tr>
<tr>
<td>Inaccessible parts of drainage systems, sewerage systems, potable water supply systems, electrical systems and communications systems, including culverts, pipes, fittings, cables and conduits</td>
<td>100 years</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 2. Environmental micro-environments and characteristics.

<table>
<thead>
<tr>
<th>Environment (exposure category)</th>
<th>Location and description</th>
<th>Characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone 1 – Continuously immersed or buried</td>
<td>Element of asset or asset component that is either: • continuously immersed in seawater below -1.0m Chart Datum (CD); or • buried below +3.0m CD.</td>
<td>• Hydrostatic pressure; • chlorides, sulphates and other natural minerals found within seawater or groundwater; • soil contaminants at varying concentrations; • flow of groundwater; • microbiological activity (potential); • aggressive carbon dioxide contents; • potential for stray current corrosion.</td>
</tr>
<tr>
<td>Zone 2 – High Risk (tidal/splash)</td>
<td>Element of asset or asset component with surfaces subject to tidal action or periodic splashing, including surfaces located at • between -1.0m and +3.0m CD; or • between +3.0m CD and +4.0m CD and exposed to seawater splash within 4m of a continuous wave-reflective face including rear walls and transverse beams under berth decks; or • between +3.0m CD and +4.0m CD and exposed to seawater splash within 1m of a discrete wave-reflective or turbulence-inducing element, including deck soffits and the sides and soffits of beams in proximity to piles.</td>
<td>• Chlorides, sulphate and other natural minerals found in seawater; • wetting and drying effects; • capillary sorption effects; • wave impingement.</td>
</tr>
<tr>
<td>Zone 3 – Medium Risk</td>
<td>Element of asset or asset component with surfaces located above +3.0m CD and not subject to seawater splash.</td>
<td>• Airborne salts; • atmospheric gases (predominantly CO₂); • pollutants; • humidity; • rainfall; • airborne particulates.</td>
</tr>
<tr>
<td>Zone 4 – Low Risk</td>
<td>Element of asset or asset component with surfaces that are located: • above +4.0m CD; or • above +3.0m CD and more than 3m landward of the HAT seawater interface with a structure; or • buried above +3.0m CD.</td>
<td>• Atmospheric gases (predominantly CO₂); • pollutants; • humidity; • rainfall; • airborne salts; • airborne particulates.</td>
</tr>
</tbody>
</table>

regarding such contaminants and factors was gleaned from the Environmental Impact Statement (EIS).

5. DURABILITY DESIGN AND ASSESSMENT

5.1 General design requirements

The Sydney Ports PSTR required that the durability design must consider and take into account the impacts of all agents and processes arising from the environment at the location of and surrounding the assets and asset components including:

- stormwater and potable water supply, including the ionic species and any organisms present in the water;
- groundwater and seawater, including the ionic species and organisms present in the water;
- sulphur compounds or chlorides present in soil or fill materials;
- microbiological organisms present in soil or fill materials;
- relative humidity;
- atmospheric gases;
- atmospheric particulates and aerosols;
- emissions from marine vessels, motor vehicles, rail, aircraft and nearby industries;
- ultra-violet and any forms of radiation;
- heat;
- wetting and drying cycles;
- expansion and contraction cycles; and
- other pollutants or agents deleterious to construction materials which are present in the water, the air, the soils or fill materials or as a result of container terminal operations.

Deterioration mechanisms that needed to be considered and addressed for concrete elements within assets and asset components included, as a minimum:
DURABILITY

5.2 Durability risk assessment findings

Given the marine environment and micro-environments, as expected, chloride ion induced reinforcement corrosion was the principal risk to major concrete asset components. The risk of chloride ion induced reinforcement corrosion at cracks, joints and areas of local imperfections of the concrete was determined to be high and durability management measures were proposed.

Groundwater and soil chemical analysis and testing at various site locations indicated the risk of long term deterioration of concrete arising from chemical attack (i.e. sulphates, acids, aggressive carbon dioxide, magnesium, and ammonium) and microbiological attack was low to buried asset components.

Carbonation induced corrosion of reinforcement was determined to be a low risk to atmospherically exposed concrete components. Carbonation modelling was utilised to confirm that the design covers and concrete grades proposed would not present a durability risk for an asset component design life.

5.3 Chloride diffusion model

In broad terms, the mitigation strategy against the chloride ion induced reinforcement corrosion risk was to design and construct the concrete and cover to reinforcement to meet each respective specified design life. Chloride diffusion modelling was utilised to determine design covers and establish concrete mix...
The Sydney Ports PSTR required that all reinforced concrete elements of assets and asset components must be defined to comply with the following corrosion initiation periods (T0):

• where the reinforced concrete element is inaccessible for maintenance during its Design Life, the minimum value of T0 must be the Design Life of the asset or the asset component; and

• where a reinforced concrete element is inaccessible for maintenance during its Design Life:
  
  i) the minimum value of T0 must be 70 years plus the full period between the time of commencement of exposure of the concrete element to surface chlorides and the Date of Construction Completion where the Design Life of the asset or asset component is 100 years; and
  
  ii) the minimum value of T0 must be 70% of the Design Life of the asset or asset component plus the full period between the time of commencement of exposure of the concrete element to surface chlorides and the Date of Construction Completion where the Design Life of the asset or asset component is less than 100 years.

The corrosion initiation period (T0), is defined as the time between the commencement of exposure of the concrete elements to surface chlorides and the initiation of corrosion in the reinforcement in the concrete elements and is expressed in years.

The chloride diffusion model that was stipulated to be used comprised the following equation:

\[
\frac{c_{x,t}}{c_s} = 1 - e^{-\left(\frac{x}{2D\left(\frac{t}{m}\right)^m\left(t_0 + \frac{m}{t}\right)^m}\right)}
\]

where:

• \(c_{x,t}\) is the chloride concentration (% by weight of concrete) at depth \(x\) (mm) and time \(t\) (seconds);

• \(x\) (mm) is the depth measured from the external surface of a concrete element in an asset or asset component to a point in the concrete element;

• \(t\) (seconds) is the time to initiation of corrosion measured from the time of commencement of exposure of the concrete element to surface chlorides.

• \(c_s\) is the surface chloride concentration (% by weight of concrete) for the Zone (concrete exposure category) and the location of the element of the asset or asset component and must be no less than the values given in Table 3;

• \(D\) (m²/s) is the diffusion coefficient from the NT Build 443 method;

• \(t_0\) (seconds) is the age of the trial mix at the time of testing which must be 56 days converted to seconds;

• \(m\) is the age factor for supplementary cementitious materials given in Table 4; and

• \(t_1\) (seconds) is the age of the trial mix at the commencement of exposure to surface chlorides.

The surface chloride concentration for concrete in the various Zones/Exposure Categories (in Table 2) were then to be no less than the minimum concentrations listed in Table 3. Where a concrete element extended across more than one Zone, the chloride diffusion model and design was to use the minimum surface chloride concentration for each Zone in which part of the element of the asset or asset component is located.

The Sydney Ports PSTR required that the age factor “m” in the chloride diffusion model must be based on the presence or otherwise of supplementary cementitious materials (SCM) using, as a maximum, the values in Table 4.

The Sydney Ports PSTR also required that the threshold values of \(c_{x,t}\) at the initiation of corrosion at the outer face of the reinforcement or prestressing steel must be those given in Table 5.

### 5.4 Carbonation modelling

To assess the risk of reinforcement corrosion due to carbonation, the spreadsheet model, CARBUFF, was used to predict times to corrosion initiation at design covers.

The predictive model requires knowledge of the concrete strength grade, chemistry of the Portland cement (PC) component, in particular the C3A content, the quantity of PC in the mix, and the nature and quantity of other cementing components. The period of curing (expressed as equivalent days of wet curing) and the exposure conditions (average temperature, CO₂ concentration and average relative humidity) must also be defined.

As the PBE Project is also within 2 km of a petrochemical refinery (Caltex refinery) and Sydney Airport, above-ground concrete structures may be exposed to acidic aerosols arising from emissions of CO, NO₂ and SO₂ compounds attributable to refinery activities.

The normal atmospheric CO₂ gas concentration of 0.038% was assumed together with an anticipated elevated concentration of 0.076%. The elevated concentration was derived from a review of the Environmental Impact Statement (EIS) whereby EIS modelling indicated that the predicted NO₂ and SO₂ air concentrations will effectively double during the future 100 year operation of the new terminal.

### 5.5 Concrete mixes and design concrete covers

Chloride diffusion modelling, carbonation modelling and durability assessment of other deterioration mechanisms lead to the development of concrete mixes and design covers for the different exposure zones and the different asset components. Table 6 provides a summary of concrete mix details and design covers for the major assets.

### 6. PROPRIETARY CIVIL ITEMS DURABILITY

Concrete drainage pipes and pits, services pipes and pits and precast concrete headwall asset components are proprietary products. The required design life for proprietary items was demonstrated by following the manufacturers’ recommendations for similar asset components of a high standard and quality in similar exposure environments.

### 7. DURABILITY DESIGN OF NON-CONCRETE MATERIALS

Durability design of the major concrete asset components forms the basis of this paper. Materials other than concrete are also
part of the project, including steel, stainless steel and aluminium elements, non-metallic pipes and tanks, geotextile materials and revetment armour and under layers. These materials also required durability review, design and planning.

8. CONCLUSIONS
Durability assessment, durability design and durability planning was an integral part of the delivery phase for the detailed design process to minimise the risks of long-term deterioration of the structural assets and asset components within the Port Botany Container Terminal Expansion Project. Sydney Ports set the design life criteria for assets and asset components within the project. Major assets and asset components have a design life of 100 years.

An environmental assessment identified four dominant micro-environments in which assets and asset components would be constructed.

Chloride ion induced reinforcement corrosion was determined as the principal risk to major concrete asset components. Chloride diffusion modelling, carbonation modelling and durability assessment of other deterioration mechanisms lead to the development of concrete mixes and design covers for the different exposure zones and the different asset components.

Blended cement based S50 grade concretes were found to be necessary for major assets and asset components.

Design covers for tidal/splash and immersed/buried exposure zones were 75 mm. Atmospheric exposure design covers were typically 65 mm.

9. ACKNOWLEDGEMENTS
The authors would like to thank the directors of the relevant organisations for permission to publish this paper and acknowledge the contributions of other project team members to the design approaches and solutions discussed herein.

REFERENCES
1. NORDTEST, Concrete, hardened: Accelerated chloride penetration (NT Build 443), 1995, Espoo.

<table>
<thead>
<tr>
<th>Environment (Exposure Category)</th>
<th>Example asset components</th>
<th>Concrete strength grade $f_c$ (MPa)</th>
<th>Binder combination</th>
<th>Cement content (kg/m³)</th>
<th>w/c ratio</th>
<th>Design cover (mm)</th>
<th>Design 56 day drying shrinkage (μS)</th>
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<tbody>
<tr>
<td>Zone 1 (immersed or buried)</td>
<td>Counterfort Base Landward Crane Beam Bridge Piles Bridge Piers &amp; Abutments Amenities Building Slabs &amp; Beams</td>
<td>50</td>
<td>52% SL, 25% Fly ash, 23% Blast furnace slag</td>
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<td>Counterfort Wall Blockwork Units Cope Beam Terminal Access Bridge – Piers &amp; Abutments Retaining Walls Headwalls</td>
<td>50</td>
<td>52% SL, 25% Fly ash, 23% Blast furnace slag</td>
<td>600</td>
<td>0.38</td>
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<td>As for Zone 2</td>
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<tr>
<td>Zone 4 (atmospheric)</td>
<td>Pedestrian Bridge Deck Units Concrete Roof Concrete Columns</td>
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<td>35% SL, 65% Blast furnace slag</td>
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DURABILITY

Report from the CIA Concrete Durability Committee

1. INTRODUCTION

The Concrete Institute of Australia’s Concrete Durability Committee first met in December, 2008 and quickly agreed that AS 3600 requirements for durability had changed little since it was introduced in 1989 and even back then there was a strong call that they did not meet industry needs. The committee also agreed that for concrete in aggressive environments, the requirements in various codes (AS 2159, AS 3600, AS 3735, AS 4997, AS 5100.5) were sometimes conflicting and un-conservative, and generally limited in range (exposures and solutions).

The committee felt the lack of inclusion of Supplementary Cementitious Materials (SCM’s) and water: cementitious ratio were major issues with AS 3600 and wrote to the standards committee (BD10) advising of this in March, 2009. Unfortunately these comments were too late for consideration for the 2009 revision of AS 3600.

Although the committee was aware of complaints about AS 3600’s limitations going back to 1990 they were uncertain what the construction industry wanted by way of durability requirements. It was decided to hold workshops around Australia in June, 2009 to outline the issues, to find out how industry currently handled code limitations and to determine what industry would like to see developed. The results of these workshops were analysed and presented at the CIA biennial conference in October, 2009.

At that biennial conference a large majority voted for a unified, comprehensive concrete durability code to be developed. Recognising development of such a document was a major exercise the Durability Committee was expanded and six Task Groups formed (Figure 1) in April, 2010 to tackle various aspects. Each task group has been provided a general scope of work and has been asked to consider tackling the work in two phases:

1. produce a Current Practice Note (CPN) giving key guidance on major issues relatively quickly
2. in the long-term produce a Recommended Practice in the form of an industry standard.

2. PLANNING TG1

At a structures conception the asset owner must decide on its required condition and reliability through its life. They must actively determine the requirements for design, construction, maintenance and intervention methods over the design life, i.e. 50+ years, in advance. To make these decisions the asset owner needs a good grasp of what the decisions mean in terms of cash flow and asset value. The durability committee recognised that there were no structure or implementation methodologies for the conception stage and that it is a critical phase in regards durability. Performance improvement decisions can be implemented at a low cost relative to the life cycle cost, however the lack of decisions can lead to poor performance which radically increases the life cycle cost and reduces the asset value. Hence, it was considered that a precise guide specifically written in terms an asset owner would understand was needed and that a more detailed support document for engineers to lead owners was needed. This ties in closely with the 210 page fib bulletin 44 Concrete structure management: Guide to ownership and good practice and associated short version BRE’s Guide for owners.

Figure 1. Members of the Durability Committee and Task Groups.

Durability Committee

Frank Papworth Daksh Baweja
Ben Cousin David Mahaffey
Rodney Paul Fred Andrews-Phaedonos
Tony Thomas Warren Green
Wolf Merretz Frank Collins
Shengjian Zhou Ian Gilbert

TG1 Planning
Rodney Paul
Paul van Bergen
Warren Green
Frank Collins
Miles Dacre
Allister Paul
Peter Finlay
Graham Vile

TG2 Design
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James Aldred
Miles Dacre
Don Wimpenny
Linda Lee
Xiaoming Wang
Michael Moore
Peter Trinder

TG3 Construction Practice
Tony Thomas
Wolf Merretz
Gary Jackson
Rob Landorf
Luigi Mazzarolo
Craig Whitaker
Graeme Hastie

TG4 Modelling
Shengjian Zhou
Frank Collins
Chris Weale
Liam Holloway
Ben Saba
Peter Trinder
Tom Benn

TG5 Cracks & Crack Control
Ian Gilbert
Don Wimpenny
Chris Edwards
Declan Barrett
Doug Jenkins
Allister Paul
Peter Gabor
Gwian Chadbourn
Gill Brock

TG6 Performance Testing
David Mahaffey
Daksh Baweja
Warren Green
Andrew Peck
Ibn Dumfyn
Reuben Barnes

Figure 1. Members of the Durability Committee and Task Groups.
Topics for TG1 to consider are:

- Methods for considering life cycle costing over 20, 50, 100 and 300 years
- Usage of “Replacement Intervals” within specified “Design lives”
- Serviceability requirements and impact on design (Figure 2)
- Specifying risk (input on consequence and allowable risk, affect on likelihood)
- Materials significance on reliability (e.g., HPC, stainless steel, coating)
- Designing with redundancy (e.g., coating even if not needed) for high reliability
- Value of Durability Plans, how to specify for them and ensure implementation
- Durability consultants role in design, construction and maintenance
- Safety in design and its impact on durability
- Cracking significance and specifying what is wanted/needed (approach complementary to TG5)
- Waterproofing systems approach for different applications and performance
- Inspection and monitoring significance and specifying requirements in D&C and other contract forms
- Visual inspection, non-destructive test and monitoring techniques for different structures
- “Birth Certificates” giving concept, design, construction and planned maintenance information

3. DESIGN – DEEMED TO COMPLY REQUIREMENTS TG2

Workshops showed that contractors and suppliers had a strong preference for standards to provide deemed to comply requirements wherever possible. A major concern was the proliferation of competing performance tests and their perceived efficacy and the related contract issues arising. By contrast, designers are forced to use modelling and support it by durability related performance specifications as codes are sometimes inadequate and solutions provided are limited. TG4 and TG6 will consider modelling and performance tests while TG2 is to focus on provision of unified and comprehensive deemed to comply requirements.

The workshops also supported the introduction of additional exposure zones to cover situations conspicuously absent in current Australian Codes (Figure 3) and more design options including a wider range of materials (Figure 4).

Documents to be produced will provide a simple approach to specifying durability that will be safe throughout Australia. A significant question will be whether to provide requirements for the worst set of circumstance (materials, exposures, and risk) in Australia or to provide adjustments for the different variables.
in different locations. What is clear is that Deemed to Comply requirements for aggressive exposures will probably need to be increased. The question then is how these requirements should be checked to ensure they are safe. With limited long-term exposure data for new materials it is likely that modelling will have to be used. A close look at the approaches taken in Europe to check German and UK codes would be appropriate but this may have to wait for the Recommended Practice as fib’s TG 5.8 (see separate article this issue) is not to be produced until 2011.

The first challenge for the TG2 will be to work out limited unified requirements (i.e. applicable to all structure types, specific design lives and various risk levels) based on strength, cement type, w/c ratio, cover and curing. Exposures zones will also need to be considered as those in current codes are not rationalised and leave many exposures unaccounted for. In the first instance exposures are likely to be limited to atmospheric chlorides, immersion in seawater and brine, atmospheric carbon dioxide, sulphates and acids.

In the longer term the following could be considered:
- more varied combinations of all the durability factors
- industrial exposures
- allowance for various levels of maintenance, implementation of performance testing (i.e. increasing reliability), compaction methods and concrete flow, and level of quality assurance
- provision of detailing requirements
- development of a model specifications.

4. CONSTRUCTION TG3

The Workshops found that AS 1379 adequately covers most aspects of concrete supply but there was little to limit how contractors control durability through construction. It was particularly felt that the construction industry needs a greater understanding of how their work affects durability and better guidance on training. It was also felt that contractual implications to the contractor, designer and owner needed to be clarified for performance and prescriptive specifications.

More specific guidance is required and topics for consideration by TG3 include:

**Atmospheric**
- B1 Low atmospheric chloride – Unlikely to be used
- B2 Moderate atmospheric chloride
- C1 Seawater spray
- C2 Splash - Vertical Surfaces
- C3 Splash – Horizontal Surface
- C4 Capillary rise

**Underwater**
- D1 Immersed quiescent
- D2 Immersed turbulent
- D3 Air one side, immersed the other
- D4 Cyclic long wet, long dry

**Concrete Supply**
- Content of concrete supply Method Statements
- Outline materials issues in Australia (water, cement, aggregate)
- Assessment of plants and transport vehicles
- Batching and mixing concrete assessment
- Control of supply logistics (temperature, travel time, weather, slump loss)
- Significance of bleed, bleed control and use of bleed measurement results (Figure 6)
- Trial mix specification and management
- Self compacting concrete specification
- Testing and test reporting (water content, slump, air, bleed, strength, material results)
- Construction
- Training of concrete gang and supervisors
- Supervision requirements on-site
- Placing, compaction, finishing and curing Method Statements
- Cover significance, spacers, pre and post pour cover checks
- Strength development, maturity, matched curing, stripping and curing
- Early age restraint cracks and the contractors influence
- Blistering, delaminating and how it is caused
- Finishing issues
- Formwork removal issues
- Penalties.

Of significant concern to suppliers was the introduction of

**Pre-pour planning**
- Review of design detailing (buildability)
- Liaison between supplier, contractor and designer,
- Concrete set up onsite
- Slump loss, transport time and delivered temperature issues
- Method statements, inspection and test plans, pre-pour check lists
- Method of delivering performance test requirements.
the large range of performance tests. TG6 will cover technical aspects of performance tests while TG3 will consider contractual implications.

5. MODELLING TG4
There are many forms of deterioration and durability is a very broad subject. There are codes on models that cover chloride ingress and carbonation. Data on other deterioration mechanisms, such as acid and sulphate attack, aggressive carbon dioxide and leaching attack, magnesium and ammonium induced attack are also available. The target group for these documents is experts in durability who will understand the limitations of the models being used and will be able to judge if the output is reasonable.

The Durability Committee found that use of modelling was widespread amongst Durability consultants in Australia and all used similar models for carbonation and chloride ingress. The committee had no great issues with the models set down in fib 34 (see separate article this issue on fib) and TG4 has been asked to consider adoption of these. However, there was significant difference to input values to the models and unification is a prime consideration. Two approaches to modelling were considered:

- partial factor safety approach, and
- full reliability analysis (Figure 7).

In the partial safety factor approach there is no general agreement between the Australian experts on input values to use for the different variables or partial safety factors. It is also clear that for some variables there are no agreed measurement methods and wide variations in standard values used. Differences in chloride activation levels, chloride diffusion reduction with time and surface chloride level selected by different experts can all make for significant differences in calculated design life for the same project. The lack of agreement in regards this basic design tool leads to confusion and frustration in the industry and a key objective will be for a CPN that confirms the equations to use, details standard input values (e.g. surface chloride levels) and gives an indication on how mix design relates to performance values.

Reliability analysis offers the potential to remove uncertainty over input values by replacing partial safety factors with probability distributions. There has been even less discussion in Australia on these distributions but as Europe is already using reliability analysis as a key durability tool it seems appropriate that TG4 moves on to provide a unified approach to reliability analysis but this will probably only occur at the stage of producing a Recommended Practice.

This task group is to consider all major durability models, and not just those relating to carbonation and chloride ingress.

6. CRACKS AND CRACK CONTROL TG5
While the durability workshops in June, 2009 covered cracking, the CIA held a further lecture series on cracks and crack control around Australia in March, 2010 and this has given further input into TG5.

The first consideration for TG5 is to resolve how to handle crack width design. Australian Standards do not have crack width requirements but only consider crack control as a function of steel stress and various detailing requirements. This is thought to reflect the problem of enforcing a specified crack width requirement. There is no standard method for calculating crack width. Crack widths vary during the day and the width can open or close over a longer period. However, controlling the crack width is the only basis for determining whether cracks are acceptable from a durability perspective and the steel stress approach has been shown to be un-conservative on some occasions when using high performance concrete.

TG5 will need to consider various approaches in resolving the issues. In Europe, it is a design requirement to check that the reinforcement is adequate to control crack widths to the limits set for different exposures but attainment of this crack width is not translated to a contractual requirement. The engineer can check crack widths and make a decision as to whether some form of intervention is required based on a detailed assessment of possible damage arising, whether that be functional or aesthetic. As a simplified design approach, deemed to comply maximum steel stresses could be given based on defined limits and would depend on such factors as cement type, cement content, element thickness and climate. The full crack width design would then only need to

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Figure 5. Guidance on construction to avoid durability problems is a key focus of TG3.
Concrete in Australia Vol 36 No 3

Figure 6. Bleed Testing and its Application to Durability Design will be one of the aspects considered by TG3.

Bleed Volume

Elapsed Time (min)

Figure 7. Use of full reliability analysis to assess probability of corrosion activation.

Calculation of mean carbonation depth $x_f(t)$

$$x_f(t) = \sqrt{\frac{2}{k_e} k_c k_t \left( R_{acc,0}^{-1} + c_t \right) C_s \sqrt{W(t)}}$$

- $k_e$: environmental function
- $k_c$: execution transfer parameter
- $k_t$: regression parameter (test method)
- $R_{acc,0}^{-1}$: inverse effective carbonation resistance
- $c_t$: error term
- $C_s$: $CO_2$-concentration of the ambient environment
- $W(t)$: weather function
- $t$: time
be used in exceptional cases.
A design approach is given in CIRIA C660 (2007) but limitations on the methods have been raised in several papers 4, 5 and these will need to be addressed. However, TG5 may adopt the C660 6 approach for crack width design with a commentary on general limitations and consideration for Australian materials and climates.
TG5 will also need to provide advice on the effect of load induced strains on the ultimate crack width, joint design and plastic crack issues.

7. PERFORMANCE TESTING TG6

Performance testing is a relatively new requirement and has not yet found its way into codes but is increasingly finding its way into specifications. While performance specifications could be a way of allowing contractors and suppliers to more fully use their experience to provide the lowest cost design and construction solution inappropriate specifications raise significant contractual problems. A three stage approach may be necessary:
1. Develop a range of performance test methods that the industry agrees to adopt.
2. Specify project performance testing using the designated methods but without contractual requirements on performance values. The Durability Consultant can then assess whether the performance achieved meets the design requirements. This will also provide the industry an opportunity to refine its testing technique and to gain important data (repeatability and reproducibility) on test method and concrete variance. The relationship between field performance and test values will need to be assessed.
3. Develop performance specifications that are based on sound industry accepted tests that have been fully evaluated for use in design and evaluation of concrete mixes, quality control during construction and evaluation of insitu performance of concrete.
TG6 has been given the following list for consideration:
• Statistical analysis of results – Number of tests required and how to apply to design, and quality assurance
• Cover – Review the accuracy, methods, application and specify approach
• Chloride diffusion
  – NT Build 443 as test method for mix trials and its limitations
  – NT Build 492 as a rapid test method, how it relates to NT Build 443 and use as a QC tool.
• Rapid Chloride Permeability – ASTM C1202 limitations and use as a QC tool after establishing a value for the mix
• Capillary rise (laboratory) – Review ASTM, TEL, BS and Ho Sorptivity/Absorption tests and VPV test and detail preferred method(s) in relation to assessing the sorption component of the chloride ingress model and general quality (penetrability) assessment of concrete (Figure 8)
• Capillary rise (field) – Review ISAT and RILEM methods and provide method
• Resistivity – Define appropriate test for laboratory assessment of cylinders and indicator of general quality assessment of concrete
• Carbonation rate – Specify fib Bulletin 34 method
• Sulphate resistance
  – Specify AS sulphate resistance test for cements

Figure 8. TG6 will consider the methodology, application and veracity of capillary rise tests, such as AS 1012.21 shown here.
– Consider CSIRO sulphate resistance test for concrete
• Alkali silica reactivity – Review existing tests and specify approach
• Water permeability – Review existing Australian tests and specify which one is recommended as standard
• Consider any other durability test methods (e.g. leaching, chemical attack, etc).

It is likely that only some of these will be addressed in the initial CPN while others are left to the later Recommended Practice, possibly with monitoring methods and non-destructive tests.

8. CONCLUSION

The task groups met in June, 2010 for the first time. The task groups are expected to review the scopes and further develop them as a first step. These will then be reviewed by the Durability Committee to ensure each groups work ties in with the other groups. It can be seen from the above that each group has a major job with limited resources. Hence, it is not expected that the Current Practice notes will start to become available until January, 2011 and Recommended Practices are unlikely to be complete before mid-2012.

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Reinforcement Detailing Handbook

A complete revision of the original first published in 1975.

The 2007 edition takes into account changes to relevant standards, design practice and developments in the choice of available reinforcement types. Available via the Institute’s web site or through Standards Australia/SAI Global.

The basic requirements of good reinforced concrete detailing are clarity and conciseness.

Unfortunately, there has been a steady deterioration in the quality and quantity of drawings supplied for reinforced concrete over the last twenty years. The net result of poor quality drawings is increased costs in the material supply and construction sectors and unacceptable levels of dispute.

The aim of this manual is to guide designers, draftsmen and other professionals toward a uniform method of communicating the design intention to the construction team so that confusion cannot arise from the misinterpretation of the drawings.

www.concreteinstitute.com.au  excellence in concrete
This article provides my interpretation of durability events at the fib congress in Washington DC in June. The article is in two parts. Part 1 outlines discussion and papers presented at the two day fib Commission 5 (C5) “Structural Service Life Aspects” meeting and Part 2 provides a brief review of the papers presented at the four days of conference following commission and congress meetings.

For those that don’t know fib is the “International Structural Concrete Institute”. Australia’s involvement has been through the Concrete Institute of Australia (CIA) for several years but in 2009 the fib Australian National Member Group (ANMG) was formed by companies seeing the significant knowledge based benefits. Details of the fib ANMG and the member benefits can be found at: www.concreteinstitute.com.au/view.php?page_id=127. Now is a good time to join the ANMG as membership subscription requests have now gone out.

1. COMMISSION 5 MEETINGS

1.1 fib Commission 5 overview

fib sets up “Commissions” to consider areas of general interest and durability is of such interest that C5 has been established with a broad remit, i.e., “to provide rational procedures to obtain an optimal performance of concrete structures in service and to ensure that sustainability, whole-life cost and associated through-life perspectives are taken into account as part of the process by which experience gained from practice is fed back to the design, execution, maintenance and rehabilitation stages.”

Hence C5 is very relevant to CIA’s Durability Committee as it oversees preparation of various documents (see article this issue) rationalising durability requirements for Australian concrete structures. fib Commissions typically work by establishing Task Groups to consider specific aspects and prepare reports (Bulletins). C5 has six current task groups and has previously produced six Bulletins (Table 1).

C5 also provides support to fib’s Special Activity Group 7 (SAG7) “Assessment and interventions upon existing structures”. SAG7 is preparing documents to support a CEN standard on the same topic and will provide the “how, when and why” of building and structures condition management literally from conception to the grave. C5 and SAG7 meetings were held concurrently on 27-28 June with one morning of combined presentations. The results of all this effort will be a comprehensive, intimately woven set of guides to building and structures management through their design life.

Amongst all this a presentation was given on SHRP 2. SHRP 1 was a major US federally funded program bringing researchers from all over the world to the US to develop bridge deterioration solutions. The areas related to C5 were NDT, Specification, 100 year life design and serviceability limit states.

C5 has also had significant input into fib’s “Model Code” (fib Bulletins 55 and 56) which is closely aligned to ISO’s 16204 “Durability – Service Life Design of Concrete Structures” and fib’s structural concrete textbook (fib Bulletins 51 to 54). The Australian National Member Group (ANMG) have received copies of the Bulletins as part of their membership.

1.2 TG 5.8 – Condition control and condition assessment of reinforced concrete structures

The TG5.8 report was circulated to C5 for final review by mid-July. The report provides a methodology for determining when a structure should be assessed in service. The report provides a sophisticated reliability based approach that would be a vast improvement on the current subjective system of determining when to inspect structures. Two examples of the methods application are included for Serviceability Limit State (SLS), i.e. for structures subject to carbonation and chlorides, and one method for the ULS for prestressed concrete.

The conflict of designing for steel stress rather than crack width was discussed, i.e. crack width design leads to problems

<table>
<thead>
<tr>
<th>No</th>
<th>Task Groups</th>
<th>Title</th>
<th>Bulletins</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.8</td>
<td>Condition Control and Assessment of Reinforced Concrete Structures Exposed of Corrosive Environments</td>
<td>Management, maintenance and strengthening of concrete structures</td>
<td>17</td>
</tr>
<tr>
<td>5.9</td>
<td>Model specification for repairs and intervention</td>
<td>Monitoring and safety evaluation of existing concrete structures</td>
<td>22</td>
</tr>
<tr>
<td>5.10</td>
<td>Birth and re-birth certificates and through life management aspects</td>
<td>Durability of post-tensioning tendons</td>
<td>33</td>
</tr>
<tr>
<td>5.11</td>
<td>Calibrations of Code deemed to satisfy provisions for durability</td>
<td>Model Code for Service Life Design</td>
<td>34</td>
</tr>
<tr>
<td>5.13</td>
<td>Operational documents to support service life design</td>
<td>Concrete structure management: Guide to ownership and good practice</td>
<td>44</td>
</tr>
</tbody>
</table>
of measurement and enforcement on site while steel stress limits may not always adequately restrict crack widths. Steiner Helland pointed out that the issue can be resolved by requiring design for crack widths but not specifying limited crack widths in the contract. He agreed to compile a draft list of points for consideration on the relationship between cracks and durability.

The potential of slower autogenous healing at cracks using SCM’s was raised but Carola Edvardsen strongly felt this was not an issue. In her PhD thesis she found that crack scaling was a function of pressure, concrete thickness and crack width. She developed a formulae for rate of crack healing and will provide additional information to C5 from her thesis.

Steiner Helland presented the latest draft of ISO 16204 that is now to be reviewed by the national delegates for “final discussion, public enquiry and voting.” A draft of the ISO standard was circulated at the meeting for comments.

Bulletins 55 and 56, fib’s Model Code 2010 (MC-2010), were tabled and those receiving bulletins were asked to provide feedback on durability sections.

Helland noted that in 2001 the ISO TC-71 committee asked fib to work upon a model code (i.e. 16204) dealing with service life design of concrete structures. In response to this fib set up C5 TG 5.6 to prepare the approach and they obtained fib General Assembly approval for the SLS design method in 2006. ISO TC-71 has accepted fib’s approach as the basis for the ISO standard. The fib model code is written as a parallel supplementing set of provisions to the ISO and European standards and ranges from “basis of design” to “maintenance” through to “decommissioning”.

Helland stated that there were some differences in opinion within fib SAG-5 on the suitability of existing probability models, with input parameters derived from accelerated testing on young specimens, as the basis for design of new structures. His opinion is that modelling should, and must, be the backbone for Serviceability Limit State Design, but MC-2010 needs to be mindful not to mislead the reader to apply it where the uncertainties in the “partial safety factor method” of modelling. This is particularly an issue for chloride ingress modelling but was less certain about it for chloride ingress modelling.

Helland noted that regardless of that the codes included the methodology for full probability analysis on the basis that if reliability analysis was considered unsuitable the models could still be used for the partial safety factor approach.

For many years probability analysis has been considered as the “holy grail” of durability analysis as it held the promise of resolving the uncertainties in the “partial safety factor method” of modelling. Hence, I have put to Gehlen and Helland the assessment shown in Figure 1.2.

Durability design is based on an expected condition distribution (distribution A). The targeted mean condition is set sufficiently high such that with the assumed distribution the number of failures at the 95% confidence level will not exceed the allowable. In reality the industry designs and constructs to try and ensure that the specified value for each property that affects the condition is sufficiently exceeded that there is a low risk of failure, e.g. the actual characteristic strength may significantly exceed the specified minimum characteristic strength. This results in the condition from general construction exceeding the required performance (distribution B). Unfortunately on some occasions the construction quality control is poor and the condition ends up lower than that required (distribution C).

At some point after construction the actual condition is measured by testing and a fourth distribution is obtained (distribution D). The measured condition distribution (distribution D) is different to the actual distribution (distribution B) due to the precision and accuracy of testing. It is important that testing has appropriate precision and accuracy or the structures reliability could be significantly misjudged (Δx).

Reliability is the extent to which the minimum (e.g. 5% failure) acceptable condition is met. This is shown as points A, B, C and D. A small reduction in average condition relative to design, together with the expected higher variability, results in a large reduction in reliability. Conversely a small increase in condition, with the expected reduced variability, results in a significant increase in reliability. This indicates that reliability is very sensitive to small changes in condition. As condition assessment, as judged by chloride ingress for example, is very sensitive to small changes in performance the combined sensitivities makes reliability a difficult tool to use for design.
1.3 TG 5.9 Model technical specifications for repair and interventions

I attended the small group meeting for TG 5.9. The original plan was to take a Norwegian document detailing methods of repair as a basis for a fib Bulletin. However, it seems that the resultant document was not accepted as it was not sufficiently analytical. A new approach was discussed that was to consider each repair type and list the fundamental reasons for failures and then provide calculation methods to assess if proposed methods of repair would be prone to these failure mechanisms. This method was subsequently put to C5 where it was accepted. TG 5.9 now intend to produce an example related to one repair type for C5 approval before extending the principle to all methods.

1.4 TG 5.10 Birth and Re-birth Certificates and through life management

There was some discussion on the use of the term “Birth Certificates”. It was felt that this was originally intended to mean the initial documentation surrounding a structure’s service life design but was now being used to be the collective information throughout the structure’s life.

Two presentations were made to the group showing examples of breaking structures down into components to give an inventory and then how to handle that through the design life.

1.5 TG 5.11 Calibration of deemed to satisfy provisions for durability; and, TG 5.13 operational documents to support service life design

The main discussion centred on the overlap between these two groups and how they would move forward. This will be a main topic of discussion between the two groups in November. However some background to the April meeting of TG 5.13 was given:

This meeting developed the scope of the task group. It was decided that the main aim was to deal with reinforcement corrosion through the structure’s life design but was now being used to be the collective information throughout the structure’s life.

There was discussion on serviceability limit states and whether reinforcement activation was a limit state. It was considered that any point of integrity concern could be set as a serviceability limit state. This could be related to “t₀” (activation), “t₁” (cracking/ spalling) or collapse (Figure 1.3) and would depend on the structure and consequence of reaching the serviceability limit state.

It was agreed that chloride activation threshold was best considered as a probability distribution.

During the C5 meeting many other aspects were raised where guidance was required.

1.6 SAG 7 Assessments and intervention upon existing structures

Commission 5 and SAG 7 joined together for the primary presentations associated with SAG7. There were five presentations. All focused on reliability assessment of existing structures, and most on shear aspects.

Ane de Boer from the Netherlands explained that increasingly aged structures, increasing traffic and asphaltic load and change in design and safety philosophy has led to an urgent need to review the methods of assessment of existing bridge structures in the Netherlands. This had been spurred on by, for example the Montreal Bridge collapse in 2006, where brittle cracks were apparent but not understood and failure occurred. In the Netherlands, an inventory assessment of all structures has been undertaken and a non-linear finite element reliability approach to assessing shear cracks developed. Supporting laboratory structural testing program and insitu bridge load test programs were outlined.

Gerit Dieteren also outlined concerns about shear in view of the change to building codes, heavier traffic and the lack of shear steel in flat slab decks. He focused on materials properties to be used in design. Tensile strengths were found to be lower than predicted for compressive strength formulae so laboratory shear tests were undertaken to better define the relationship and samples extracted from old structures to measure the relationship with time. They developed an approach for rapid assessment of structures reliability followed by core extraction and detailed analysis for structures of questionable reliability.

Walraven continued the theme of reliability analysis of existing bridges in Netherlands and materials issues with shear assessment. He reported on:

• bleed affected weakened areas under bars affecting shear capacity

• investigation of sustained load capacity differences between an old beam sawn from an old structure and a new beam that replicated it
• the significance of load positions for failure potential
• the significance of compressive membrane action
• proof load testing that showed the E-modulus of the concrete but could not be used to predict reliability as the ultimate strength of the concrete was unknown
• extensive tests (Figure 1.4) on strength of cores and reliability analysis to show that six cores from an old structure was optimal for assessing the compressive strength and could be used with load testing to predict reliability.

Another paper reported target values for the reliability index at different ages, gave the logic for reduction over time (e.g. reduction from 2.9 for 1 year to 1.5 for 50 years for SLS) and how to incorporate additional information into the analysis.

The final presentation stressed the importance of assessing the age of the structure and putting that into the context of engineering at the time to give an indication of the load patterns considered, standards and construction methods applied.

1.7 SHRP program
Dr Atorod Azizinamini, a principal investigator for SHRP 2 gave a presentation on their work. Relevant SHRP topics included accelerated bridge construction, non-destructive testing, specification, service life design and serviceability limit states. His presentation focused on a project to improve “existing and new systems, subsystems and components that historically limit the service life of bridges”. Their guide on design for life is in 16 chapters dealing with various elements.

The initial part of the project focused on looking at reported environmental, traffic, material, construction and maintenance problems. They then considered available design philosophies for each and what issues related to them. This will ultimately lead to recommended strategies for design. The process of investigation of bearings was given in detail as an example.

1.8 CIA’s Durability Committee
I presented an outline of the work of CIA’s Durability Committee (see separate article this issue) and received some feedback from delegates.

2. REVIEW OF DURABILITY PAPERS PRESENTED AT FIB’S THIRD INTERNATIONAL CONGRESS

This article provides an assessment of the papers from FIB’s third International Congress in Washington in June. More detailed analysis of the papers, including reference to the original data, could well lead to modification of the views given. Hence, readers should not consider the content as a final position, but more of an introduction to the papers. The papers have been grouped by topic to allow consideration of all papers on an issue together. There were other public presentations at the conference by the FIB commissions but as no papers were included these are not reported here.

2.1 General durability design
Georgescu et al reported on implementation of a “Durability Class” system as given in Eurocodes. The difficulty is seen as establishing the durability class from various exposure classes. However once established it is seen as giving clear requirements for strength, cement type, cover, crack widths, placing and curing. The paper is useful as it gives examples of the application of the European durability classes.

Cussigh et al detailed the approach taken to give a 120 year design life on Greece’s Rion-Antirion Bridge (Figure 2.1). The bridge is located in the Gulf of Corinth and is the longest multi-span cable stayed bridge in the world with a continuous span of 2.2 km. The splash zones of the superstructure had a minimum cover requirement of 75 mm (85 mm nominal) and strength of 60 MPa while the atmospheric zones of the superstructure had covers of 50 mm and strength of 50 MPa. This significant variation is consistent with the reduction in surface chloride level with height. The cement was a local 60% slag blend and all concrete had a w/c maximum of 0.4. Quality Assurance acceptance criteria comprised water penetration ≤ 20 mm and rapid chloride permeability ≤ 1000 Coulombs substrate and ≤ 2000 Coulombs superstructure as established from trial mix results.

Assessment of the trial mix was based on chloride binding, oxygen permeability, chloride diffusion, capillary ingress, porosity and resistivity results. Long term performance is to be assessed using destructive tests on a sacrificial wall built specifically to avoid the need for destructive testing of the bridge. The systems addressed for this structure are relevant to CIA’s Durability Committee and all of its Task Groups (TG), although other tests may be more appropriate in Australia, and is an example of the appropriate level of durability design and ongoing assessment for major structures.

Liu et al outlines the building of durable structures in “one of the worlds harshest climates”, the Arabian Gulf. Problems...
five to 15 years after construction in many structures due to the combination of high ambient temperatures (<52°C), wide ranging humidity’s (20-100%), high and saline (4.7-6.1 g/L sulphate; 2.5-5.7% chloride) ground water levels, and high wind blown chlorides are reviewed in the context of international design codes and local durability provisions. The review of international codes is a useful summary but the references to local requirements (e.g. sulphate resisting cements, low permeability concrete, appropriate cover, un-reactive aggregates, proper curing) are not startling.

Skarabis et al presented papers on the effect of curing and fly ash on pavement quality. They examined the quality of exposed aggregate concrete, and curing before and after exposing aggregate using capillary uptake and freeze thaw. It was found that both curing cycles were important. Interestingly fly ash had not been used before in Germany. It was also found that concrete quality using fly ash could be suitable but by using the Torrent Air Permeability Test (Figure 2.2), a common non-destructive quality test in Europe, the concrete quality was found to be highly dependent on curing.

McKenna presented two papers. In the first he found that for fly ash, slag and OPC (GP in Australia) cement concrete up to 40 MPa the insitu w/c ratio was 0.1 higher at the formed surface than the bulk, leading to a reduction in concrete quality in the critical cover zone. This is not a new finding and, combined with the elimination of blow holes, is a key selling point for permeable formwork. The second paper compared the use of permeable formwork to the use of coating to reduce the risk of carbonation for typical moderate strength concrete used in buildings. He indicates that the permeable formwork may provide a lower whole of life cost as coatings must typically be maintained at 10 year intervals. However, he does not compare the cost of an improved concrete quality to give the level of protection required and this is likely to be the lowest cost option unless zero bug holes is an aesthetic advantage.

Takaya considered the often ignored part of the design life model, the propagation stage. Although the paper gives some useful insights into the affect of cover thickness and reinforcement diameter and spacing results do not align with practice and hence the paper is more of academic importance.

Beushausen et al outline the implementation of the South African Durability Index (DI) approach to precast road barriers from three suppliers. Various tests are included in the DI system but in this case “Oxygen Permeability Index” (OPI) was used as the criteria as it was considered to relate to the prime deterioration mechanism of carbonation. For the project 1% of panels were tested and performance related to acceptance, level of penalty and rejection. The pragmatic commercial assessment of performance data is something that CIA’s TG3 on construction and TG6 on performance will need to grapple with and the South African approach is worth serious consideration but in the context of tests appropriate to Australia.

2.2 Conservation of concrete structures

fib Commission 5 has a significant focus on conservation of structures as the industry has determined that appropriate attention is required throughout the structures life. This is leading to greater guidance in this area becoming available and is something that could be usefully taken on board for Australian structures.

Mathews (chairmen fib Commission 5) et al report, as authors, on Chapter 9 of fib Model Code 2010 (available as fib Bulletin 55 and 56) deals with issues concerning conservation (see paper in this issue) and is significant as it is “consistent with the concepts and overall philosophy” of MC2010 that “introduces a new integrated life cycle perspective” including the service life concepts in fib Bulletin 34 “Model code for service life design of concrete structures”. fib 34 is already a key document being considered by CIA’s TG4 on modelling. The paper provides background to proactive, reactive and non-active strategies for conservation including management flow charts that may be useful to CIA TG1 dealing with planning. However, it also highlights the importance of condition evaluation and remedial intervention through the life of the structure that CIA’s durability committee may need to take on board. It also outlines the meaning of “Birth Certificate” and “Re-birth Certificates”. The former is a formalisation of the package of drawings, calculations, durability plan, NCR’s and maintenance plan that Australian structures have. The re-birth certificate adds the same sort of information following repair.

Zwicky reporting on the draft Swiss code SIA 269 “Basis of Conservation of Structures” notes that “conservation” is the term adopted to cover all activities (Figure 2.3) to ensure existence of a structure. He estimates that 40% of the US$2300 billion replacement cost of Switzerland’s infrastructure has been spent on conservation and that although “Switzerland is built” this percentage will increase. This underpins an increasing need for codes on conservation rather than codes for new structures. The paper gives a comprehensive review of the concept behind the code and as it is generally accepted that conservation is increasingly important from sustainability and national competitiveness perspectives it suggests this is an area CIA’s Durability Committee should consider in more detail.

2.3 Test methods

Considering the extent of debate in Australia on performance...
tests it was surprising that there was not more discussion on the relative merits of different tests. Only two papers.

Leng et al compare ASTM C1202 and NT Build 492 rapid chloride ingress methods with water penetration to Chinese Standard GB50082-85 for fly ash concrete with strengths of 20-80 MPa. The results from various tests seem to show similar sensitivities to change in performance from 20-30 MPa, greater sensitivity for water permeability from 30-50 MPa and greater sensitivity for rapid chloride tests from 50-80 MPa. The authors note that "standard" quality criteria give conflicting assessments for water and chloride permeability due to the chemical nature of the concrete and suggest water permeability testing may give a better differentiation of lower strength concrete and chloride permeability may be more suitable for higher strength concrete. The results are not particularly useful from an Australian perspective as the tests may be more suitable for higher strength concrete. The results from various tests seem to show similar chloride ingress methods with water penetration to Chinese relative merits of different tests. Only two papers.

Krishnaswami et al report on chloride penetration of 40 MPa and 50 MPa concrete with fly ash (FA), silica fume (CSF) and rice husk ash (RHA). Test methods employed included ASTM C1202 rapid chloride permeability test (RCPT), non-steady state diffusion (NSSD) assessment of the chloride penetration in a RCPT test (Tang Laping method), and steady state diffusion (SSD) assessment using the Nerst Planck equation. The authors limit the analysis of results to showing that all tests show SCM's improve concrete performance but this belies the usefulness of the analysis in showing that RCPT tests might be able to extended to give significantly more information. Another interesting paper for CIA's TG6 to consider.

2.4 Ultra high performance concrete (UHPC)

The German Research Foundation has funded a concerted effort in UHPC and hence many of the papers on this topic came from them. Overall it was clear that this group is making major steps forward in this area and the German sessions were a very valuable contribution to the conference.

Moeser reported on three group projects on ultra high strength concrete. These projects considered:

- reduction in Portland Cement by use of slag, fly ash, quartz flour and silica fume and the implications of super-plasticiser type and dosage. They found that strengths in excess of 200 MPa could still be achieved using slag and fly ash if appropriate fineness was used. Fine fly ash could not directly replace silica fume but strengths of over 200 MPa were achieved without silica fume.
- use of internal curing using super absorbent polymers (SAP) to reduce autogenous shrinkage, which is considered the primary cause of early age cracking when using low w/c. It was found that at early ages SAP use led to an increase in fine pores size while large pores were unaffected but in the long term there was a pronounced reduction in pore volume and densification of the mix.
- the influence of silica fume, slag and heat treatment on microstructure development. It showed that in CSH development microstructure are quite different to normal concrete.

Muller focused on durability and specifically freeze thaw, cyclic climatic damage, chemical attack and influence of cracking. They found that microcracks had no influence on durability (Figure 2.4) and UHPC had a resistance to freeze thaw damage. However, the resistance was reduced when the microstructure development was impeded at an early age and steel fibres in the mix corroded. Muller reported that the acid resistance of UHPC was high but Papworth noted in the discussion that this performance was only 30% better than normal concrete and as normal concrete had poor resistance to acid the UHPC performance was not outstanding. Muller also reported that the cracking propensity of UHPC was high unless countermeasures, such as SAP or shrinkage reducing admixtures, were taken but also showed that fine cracks quickly self healed, at least in humid environments.

2.5 Self compacting concrete

Trezos et al compared the rate of reduction in water flow through 40 MPa (approx) self compacting concrete (SCC) made with Cem II and normal slump, 20-30 MPa concrete after air and water curing. The paper is as interesting in its use of water permeability testing, and for assessment of the importance of curing as it is for its assessment of SCC as the difference in strengths (and w/c) of the two sets of concrete (Figure 2.5) make it difficult to gauge the significance of the high flow aspect.

Bermejo compared the durability performance (pore structure, EN 12390-8 water permeability, accelerated carbonation and
ASTM C1543 chloride diffusion) of 25 MPa SCC (slump flow 700-750 mm) made with fly ash (FA) and limestone (LP). Although seven day strengths for both concretes were around 30 MPa, 90 day strengths were 62 MPa for the FA concrete (w/c = 0.5) and 40 MPa for the LP concrete (w/c = 0.55). The level of durability performance for the FA concrete was superior in all respects to the LP concrete. The authors put this down to the use of fly ash rather than limestone and do not try to separate the effect of cement system from the effect of w/c ratio. It is also difficult to determine how suitable the FA SCC would be at lower strengths as the strength achieved was high.

2.6 Chemical attack
Breitenbucher reported on acid and sulphate attack due to oxidation of sulphide materials, something of high relevance to many parts of Australia. However the aeration ensured that sulphide oxidation occurred and although pH started at 7, ultimately it dropped to 2-4 and sulphate concentrates ended at 8,300 to 40,000 mg/l.

Although the rates of deterioration cannot be applied directly to real applications, due to the artificial aeration, the analysis does give an insight into the mechanism of deterioration (e.g. sulphate leading acid attack at depth) and the relative performance of materials (e.g. recommendation not to use high slag rather than Portland limestone cement in iron disulfidic soils).

Ganpule studied the effect of aggressive ground contamination of fresh concrete piles by considering the ultimate damage to the piles. The paper is of interest because it highlights the shortcomings in just considering pH, chlorides and sulphates. Like these basic inputs Ryzner and Langelier Indexes were also found not to correlate with pile condition, principally because they did not include factors such as dissolved oxygen and water velocity. Ganpule suggest the use of Biological Oxygen Demand (BOD) and Chemical Oxygen Demand (COD) as tools available to most laboratories for better assessment of ground aggressivity and outlines some experimental work to validate this. The idea certainly has some merit but further development is required before it could be considered for general application.

Messad et al provide details of the effect of C3A content, w/c ratio and binder type on external sulphate attack using an accelerated concrete test, designed so that the actual mechanisms that occur in the field are replicated. Chloride diffusion tests were also undertaken and the paper gives a methodology for combining results to make a performance assessment (Figure 2.6). This
combination approach to tests is appropriate for consideration by CIA's TG6 dealing with performance tests.

2.7 Cracks

Zanotti presents a finite element analysis of cracking risk to indicate the effect of restraint, reinforcement ratio, geometry, mix design, and construction. The paper provides a good guide as to important factors to minimise crack widths but these are not new and are all accounted for in CIRIA C660's guide to early age cracking. CIA committee TG5 on cracking will be providing guidance on where to use different methods of crack analysis.

Demilecamps reports on a French program to experimentally check crack calculations methods given in codes for combined live load and early age shrinkage strains for walls on a restraining base. The paper reports on the test set up only as no data is yet available. However it is interesting to see that structural testing to validate commonly accepted design methods for cracks is only just being undertaken. The combination of load induced and early age strains is a significant matter for CIA's TG5.

Zhu considers the performance of insitu pours to join precast units. He outlines the durability and cracking resistance of materials for pours aimed at giving structural connections at one and seven days. The paper gives an overview of the test methods used to assess materials and criteria that could be adopted for this type of construction. The freeze thaw and chloride penetration tests may not be relevant to Australia but the bond and shrinkage requirements may be worth considering for this type of construction.

Holicky notes that load induced crack widths have “considerable scatter and significant vagueness” and considers serviceability requirements using reliability analysis of Eurocode empirical design approaches. From his analysis of slabs he concludes that the allowable crack width in Eurocodes is orientated to the mean crack width that may be expected and that a factor needs to be included in the design to determine the characteristic width. A reliability index of 1.5 to 3.5 is suggested depending on the ratio of cost of providing better crack control reinforcement compared to the cost of fixing unacceptable cracking. The paper gives an excellent insight into the technical and practical issues of crack width design and should be useful to the CIA TG5.

2.8 Reinforcement

Sistonen et al reported on accelerated corrosion tests and mechanisms of corrosion of hot dipped galvanised (HDG) bars in carbonated, chloride contaminated and cracked concrete. The paper provides activation potentials for HDG bars and resistivity values that relate to corrosion rate. It also notes there was no correlation of corrosion rate with crack widths but does not give the range of widths tested. It shows the chloride activation is 1-1.5 wt % cement for HDG bars and concludes this could increase the life of structures in chloride environments by three to five times. However, this is a theoretical assessment based on an assumed activation for normal bars of 0.3-0.4 wt % cement and this may be a significant under estimate for high performance concrete. For carbonated concrete, the time to initiation is not reduced but the galvanising provides sacrificial protection that Sistonen suggests could double the life of the structure. The basis of this is not clear and as activation time is highly dependent on cover and concrete quality the galvanising effect would be more realistically assessed as an extension to the propagation phase.

Mercall et al report on the corrosion of low-alloyed (X2CrNi2), carbon and stainless (AISI 2205 & AISI 304) steels in test slabs constructed with concrete at a w/c ratio of 0.7 (20 MPa) and monitored using Galvapulse corrosion rates, macrocell c.m.f., electrical potentials and a newly developed electrical resistance (ER) probes. Cycles of one week ponding of sodium sulphate and two weeks atmospheric exposure commenced at an early age. The results showed that the electrical potentials for all steel types dropped quickly due to high chlorides at steel depth due to the method of exposure and mix. However, only the black steel sustained a high corrosion rate. The stainless steels maintained a low corrosion rate throughout while the alloy steel had a corrosion rate half that of carbon steel. ER probes results were consistent with other test results. Lounis et al investigated the chloride induced corrosion resistance of stainless and chromium steel. The paper will be of particular interest to CIA TG4 considering modelling as it uses reliability modelling of chloride diffusion to show that cover is the most influential factor and concrete quality the second most influential factor in durability. It also considers the impact of chloride threshold to deduce the high significance of the corrosion resistance of stainless steel in durability design. Finally the paper showed that the improved strength and ductility of some stainless steel could lead to a reduction in the reinforcement required.

2.9 Post tensioning

It may seem that there was a disproportionate focus on post tensioning but it should be remembered that this was a joint conference with the US Prestressed Concrete Association, the US affiliate of fib, stemming from fib's identity as fib (prestressing institute) before merger with CEB. In general, the papers will be useful to CIA's TG2 when it deals with deemed to comply provisions of stressing cables.

Kamotani et al reviewed the reliability of post tensioning in severe chloride environments. They found that the safety index for the Ultimate Limit State is high initially but degrades over time. Whether it reaches the threshold specified in ISO 13822 within the design life depends on design factors including cover and the initial safety index. I doubt whether the conclusions can be applied in general as it does provide a methodology for assessing the risk of failure of prestressing tendons.

Nojima et al found that corrosion at small grout voids in post tensioning ducts only occurred if the void was water filled and that corrosion became significant if the water contained salts. This indicates that ducts and anchorages that seal out water would be beneficial in marine environments at least.

Wu et al also reported on tendon corrosion but their paper related to monitoring of corrosion passivation using a “Grout Void Sensor” developed by the Swiss Society for Corrosion Protection for VSL. The paper outlines the technical basis for the electro chemical method and provides details of a full scale demonstration. Wu concludes that the method can be used to detect areas not passivated where voids occur due to bleed but evaluation methods need some further development. The system may prove useful in Australia where it is necessary to reduce risk associated with poor grouting and could be considered by CIA’s TG6 at some stage.

Nurnberger considers the pitting, crevice and fatigue corrosion...
resistance of high strength stainless steel prestressing strand (European Standard 1.4301; 1.4401; 1.4436 and 1.4439) in high chloride environments. He concludes that 1.4401 provides satisfactory resistance for critical structures but 1.4301 does not. He also concludes that 1.4436 and 1.4439 steels provide higher corrosion resistance than 1.4301 but their higher cost is not justified. Where readily available only specific consideration will tell whether use of 1.4401 will provide cost effective relative to other measures that could be used to provide the risk level required.

Krauser considered protection of prestressing tendons in precast segmental bridge construction as required in fiber bulletins, particularly Bulletin 33. Six classes of aggressivity, including causes of corrosion, are defined to three levels of protection that can be provided (high, medium and low). The paper outlines concrete quality and cover; waterproofing; drainage; expansion, construction and segment joint details and cracking as the key inputs into three levels of protection and is a good starting reference in assessing prestress protection requirements.

3. CONCLUSION
The research projects were extensive and diverse showing that durability is still a major topic internationally. Many of the papers will provide useful input into the CIA’s Durability Committee and its Task Groups. Anyone wanting further information on any paper please contact F.Papworth@brc.com.au. There is also significant information available from the conference on fire damage/inspection, fibers bridge design and other topics.

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Cathodic protection to Swanson Dock West in Melbourne

Ian Godson, Ian Godson & Associates

The Swanson Dock is Melbourne’s main container port with two wharves, East and West, both approximately 1km-long (figure 2). Both wharves were scheduled for repair and strengthening coinciding with a major port deepening project to accommodate larger vessels.

Swanson Dock West (figure 3) was constructed in stages from 1974 to 1988, with the berths differing in construction type and condition. Cathodic Protection (CP) based repair was selected by the Port of Melbourne and the resulting CP systems to protect the elements of this structure are the subject of this paper.

The paper outlines the stages of the development of the CP systems from the commencement of design and trials in 2007, installation through 2008 to commissioning in early 2009.

There are three major CP systems in the Swanson Dock West project:

1. Tidally effected transverse beams in berths 3 and 4, utilising internal anodes in zones to protect the tidal and atmospheric beam sections
2. Fender beam sections in berths 1 and 2, using a titanium ribbon based CP system.
3. Water anode system of 3500A to protect the tidal concrete beams and the submerged steel sheet piling and piles.

The stages of the design of the concrete CP systems are reviewed for both the transverse beams and fender beam CP with particular focus on how the trial CP systems illustrated required modifications to the design prior to large scale installation.

The water anode CP incorporated a major departure from traditional design, with the resultant system being far more efficient and economical over the 30 year design life. The transformer rectifiers (T/R) are rack-based switch mode units placed in airconditioned control rooms at either end of the wharf (1km apart). The T/R’s output high voltage, low current DC (maximum 200V, 2A) through 2.5mm2 distribution cables (up to 600m in length) to a series of “multiplier” units adjacent to the water anodes. The multiplier converts the low current, high voltage to the required high current, low voltage (maximum 9V, 45A) for the titanium cylindrical anodes. The conversion at the multiplier takes place at a set conversion ratio (approximately 25:1) allowing full control of the current from the T/R control rooms at the end of the wharves. The systems efficiency stems from the reduced losses in the distribution cabling and the high efficiency T/R and multiplier units.

The design, trials, installation and commissioning of all the CP systems are illustrated during the phases of this large project.

1. TRANSVERSE BEAM CP DESIGN

The 44 transverse beams requiring CP protection are in berth 3 and 4, between bents 192 to 235. The beams (figure 1) are 13m-long and 1m-wide and the rear 9m of the beam underside is approximately 200mm above mean high water level. The front 4m of the beam drops 600mm in elevation (figure 1 and 4) so the base of the beam is tidally affected, being approximately 400mm below the mean high water level.

The tidal effect on the beam is a major design factor with ribbon anodes in slots not favoured due to the likely poor durability of the concrete caused by acid attack on the mortar. A design utilising internal anodes was preferred and adopted for the project. While the water anode CP system can assist protection to the tidal beams, the design and trial of the beam protection was completed without consideration of the beneficial water anode current.

The basis of the beam CP design, completed in accordance with the Australian Standard 1 was to provide a design current density of 20mA/m2 reinforcing steel at a maximum anode current density of 110mA/m2 anode area. The reinforcement density varied considerably over the various sections of the beam, from relatively light reinforcement in the rear beam (section D and E) to very heavy reinforcement in the front beam section (section C and A) (figure 6).

Early design calculations considered the use of internal anodes of various configurations and outputs as illustrated by the calculations below for the rear section of beam (section D and E). Long anodes (600mm) were an option, to be installed from one side of the beam. Further options of 400mm long anodes of various outputs from both sides of the beam were also considered.

A low risk design strategy was adopted, utilising short (400mm) internal anodes installed from both sides of the beams at relatively close spacing. The benefits of this conservative design are an expected low applied voltage, an even level of protection and a low anode current density resulting in the long life of the anode (75 years plus). The design was to be proven by a site trial to be undertaken on one beam.

Due to the tidal influence, the design allowed for an additional zone in the lower beams at the front section of the wharf.

The preliminary design using the 400mm-long anodes of 5mA output is summarised for the various sections of the beam in Table 1.

The layout of the preliminary design is illustrated in drawing SK3 (part shown below in figure 7).
DURABILITY

Figure 1. Cross section of wharf showing 13m-long beam with front beam section tidally affected.

Table 1. Beam section current and anode spacing.

<table>
<thead>
<tr>
<th>Section</th>
<th>Location</th>
<th>Length</th>
<th>Current (mA)</th>
<th>No. 5mA Anodes</th>
<th>Spacing (mm)</th>
<th>Zone</th>
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<tbody>
<tr>
<td>A</td>
<td>Below fender</td>
<td>0.67</td>
<td>34</td>
<td>5</td>
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<tr>
<td>B-lower</td>
<td>Front beam</td>
<td>1.83</td>
<td>106</td>
<td>17</td>
<td>230</td>
<td>2</td>
</tr>
<tr>
<td>B-upper</td>
<td>Front beam</td>
<td>1.83</td>
<td>57</td>
<td>8</td>
<td>300</td>
<td>1</td>
</tr>
<tr>
<td>C-lower</td>
<td>Middle front</td>
<td>2.5</td>
<td>144</td>
<td>22</td>
<td>230</td>
<td>2</td>
</tr>
<tr>
<td>C-upper</td>
<td>Middle front</td>
<td>2.5</td>
<td>85</td>
<td>12</td>
<td>300</td>
<td>1</td>
</tr>
<tr>
<td>D and E</td>
<td>Rear Beam</td>
<td>8.0</td>
<td>262</td>
<td>49</td>
<td>300</td>
<td>1</td>
</tr>
</tbody>
</table>

Trial of anode layout

The anode layout, based on the theoretical anode spacing in Table 1, using 400mm-long internal anodes (5mA capacity), was installed in a typical beam in December, 2007.

The monitoring of the trial consisted of eight silver/silver chloride reference cells installed in the concrete at specific locations. In addition to the reference cells, an equipotential survey was completed at a 250mm grid on the side and underside of the beam before the trial was energised. The beam was marked so that the identical locations could be re-tested with the equipotential survey at set times during the trial (figures 9 and 10).

Following recording of all base readings, the system was energised at 14:00 on 11 December, 2007 and allowed to polarise for three days before the instant off readings were taken (current interruption device utilised) and the system allowed to depolarise for approximately 24h.

The results of the trial were excellent (in excess of 100mV depolarisation) in all locations except at the front of the wharf, section A, where the implanted references and equipotential surveys indicated under protection.

The underperformance of the internal anode system in Section A below the fender beam was partly due to the very heavy reinforcement and the difficulty the site team encountered during the installation. A modified anode installation technique was developed, drilling and installing additional anodes from the front end of the beam. With the additional anodes installed, the trial was re-commissioned and allowed to polarise prior to completing depolarisation testing on 5 February, 2007. These tests indicated that the revised anode arrangement was successful and the anode arrangement was finalised. The trial process had taken over nine
weeks to complete and included three modifications of anode arrangement and three separate depolarisation tests.

The final arrangement of the CP system had five beams grouped to form a region, with two electrical zones per region. The 44 beams were grouped into nine regions (eight regions of five beams and one region of four beams) with a total of 18 electrical zone outputs from the transformer rectifier.

Eight Ag/AgCl reference electrodes were installed per region, giving a total of 72 references. The layout had two zones per beam, with the Upper Zone 1 covering the complete length of the beam. The Lower Zone 2 extended for the tidally effected front section of the beam (approximately 4.25m).

The transverse beam CP system was commissioned in January, 2009, and has successfully polarised the reinforcement achieving the 100mV decay criterion and the absolute criterion in some locations in the tidal section of the beams.

2. FENDER BEAM CP DESIGN

The fender beam in berths 1 and 2, approximately 500m-long, was heavily chloride contaminated and cracking and spalling. The fender is approximately 0.5m above the high tide level. A new ship fender system was to be installed and the beam was required to be strengthened to support the anticipated loads. The fender beam sections were to be protected by CP with the installation incorporated into the strengthening reconstruction of the beam. Due to financial restrictions, the fender beam was to be strengthened and CP protected only in 30 locations 3.5m in length, with the CP to protect to a height of 0.5m from the base of the beam.

The initial design utilised titanium ribbon anode installed on insulated spacers, with the titanium anode quantities calculated on a conservative basis of 20mA/m² steel area. The resultant anode layout adopted three longitudinally placed anodes on the base of the beam and two on each face of the beam, each anode running the full 3.5m length of the beam section (figure 8).

The first of the 30 fender beam sections was treated as a trial section (figures 11 and 12). The section was spray repaired, concrete cured for 28 days prior to the commissioning of the CP system. The monitoring was completed with two implanted reference cells, one placed at the contractual protection level of 0.5m from the base.

In addition, an external equipotential mapping was conducted at fixed points on a 250mm grid (figure 13).

The natural potentials were taken at the grid points as detailed above (and the reference locations) and the trial fender beam section was initially energised on 21 September, 2007, at the maximum design current of 132mA over a three day period. At this time the instant off potentials were measured at each grid point and the system allowed to depolarise for 24h. The depolarised potentials were again taken at the grid points and references. The 24h depolarisation results are shown in Table 2.

A review of the results indicated that the trial system had not met depolarisation criterion (100mV) with the average depolarisation approximately 55mV, with the implanted references only achieving 35mV and 64mV respectively.

The results were clearly unsatisfactory and the trial was re-energised on 1 October, 2007, at the same design current, but this time for an extended 10 day period. Again the instant off and 24h depolarised potentials were recorded at each of the grid positions (and implanted references). It was observed that the 24h depolarisation was well below the negative shift from the native...
DURABILITY

Figure 6. Early design considered the use of internal anodes of various configurations and outputs as illustrated by the calculations for the rear section of beam (section D and E). The design adopted the conservative approach of 400mm-long, 5mA output anodes from both sides of the beam at a spacing of 300mm.

Figure 7. Trial layout of internal anodes in transverse beam.
potentials, so the system was allowed to depolarise for a total of 72h with the depolarised potentials again recorded at the grid points. The results are detailed in Table 3.

The results are discussed below.

- The 24h depolarisation saw only 40% of the grid points meet the 100mV decay criterion, with 58% of the points greater than 90mV decay.
- The negative shift from the native potentials was very good, with over 97% of the grid points achieving a greater than 150mV shift.
- Approximately 50% of the grid points achieved the absolute potential criterion, (< -720mV Ag/AgCl). The extremities of

Figure 8. Initial design of the CP had two ribbon anodes on each side and three on base, to protect the 3.5m beam section from the base to a height of 0.5m.

Figure 9. The front section of the trial beam during anode installation. The grid reference on the beams was utilised for the external equipotential survey.

Figure 10. A battery powered temporary T/R was utilised for the trial CP.

Figure 11. Initial design had two side ribbon anodes longitudinally on each side of beam (three on base).

Figure 12. The ends of the 3.5m section, with reference electrode placed at the 0.5m height from the base.
the system (grid A,B,H,I and 8) did not reach the criterion.  
- The 72h depolarisation had over 90% of the grid points  
  achieve the 100mV decay criterion, but generally not at the  
  extremities.

It was clear that the rate of depolarisation was very slow in the  
reconstructed beam concrete, likely due to the very dense dry  
process gunite repair concrete and the fact that the repair was  
still under two months old. However, even adopting the 72h  
depolarisation period, there was approximately 10% of the grid  
points and one of the implanted references not achieving the  
100mV decay criterion. It was also noted that the location of the  
under protected areas were at the extremities of the CP system,  
at the end of the 3.5m beam section and the upper extremities  
(0.5m up from the base).

It was concluded that there was insufficient current to fully  
protect the reinforcement at the extremities of the fender beam  
section and that additional anode should be provided. The solution  
was relatively straightforward with an additional anode provided  
on each side of the beam, 500mm up from the base, and an  
additional transverse anode added at the ends of the 3.5m beam  
section (figure 14).

The modified design was installed in all 30 reconstructed fender  
sections, with five sections grouped to form a zone, with six zones  
for the total system (figure 15). The zone outputs and cables  
were sized for future expansion of the CP system to protect the  
remainder of the fender beam, with the overall system having a  
capacity of 30A.

The fender beam CP was commissioned in January, 2009 and  
has successfully polarised the reinforcement achieving the 100mV  
decay criterion.

3. WATER ANODE CP SYSTEM

The water anode system was required to protect all of the  
submerged elements of the structure, including the tidal concrete  
beams, over 1km of steel sheet piling and approximately 1000  
steel piles. The design was completed in accordance with the  
Australian Standard AS 2832.3 2005 and the calculated current  
requirement was 3500A. The major requirement of the clients brief  
was that no monitoring or high maintenance parts of the system  
should be placed along the berths, with access for maintenance  
and monitoring only at the ends of the wharf.

Faced with cable lengths of up to 600m from the ends of the  
wharf to the central sections, traditional CP systems passing  
current to the water anodes at typical voltages of 10V-15V would  
require massive cable sizes to minimise the voltage drop over the  
large distances. Initial calculations suggested approximately 40t of

![Figure 13. Grid arrangement for fender CP Trial.](image)

<table>
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<th>Trial 1 – 3 Day Polarisation</th>
<th>Reference 1</th>
<th>Volts</th>
<th>mA</th>
<th>Reference 2</th>
<th>Volts</th>
<th>mA</th>
<th>24 HOUR DECAY</th>
<th>25/9</th>
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Table 3. Results from Trial 2 after 10 days of polarisation, including 24h decay, shift from native, absolute criterion (<720mV) and 72h decay.

Trial 2, After 10 days Polarisation

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<th>Test Locations</th>
<th>Instant off</th>
<th>+4hr OFF</th>
<th>+24hr OFF</th>
<th>24 HOUR DECAY 12/10 9.00</th>
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<td>364 mV</td>
<td>352 mV</td>
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<tr>
<td>Reference 2</td>
<td>774 mV</td>
<td>680 mV</td>
<td>632 mV</td>
<td></td>
</tr>
<tr>
<td>Volts</td>
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Test Locations

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Trial 2, After 10 days Polarisation

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Test Locations

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Trial 2, After 10 days Polarisation

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Test Locations

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DURABILITY

Trial 2, After 10 days Polarisation

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Figure 14. Additional anode is placed at the extremities of the beam section as shown.

Figure 15. The overall fender CP system incorporated six zones, with five strengthened CP sections in each. Overall system capacity was 30A, with provision for expansion to the remaining fender sections.

copper cabling would be required for the project. Apart from the large cost of cable supply, the installation of the cabling was to be a major challenge. In addition, the expected power losses would have resulted in a power efficiency of the system of less than 25%.

The system developed for the project, known as the “Multiplier System” reticulates the current as high voltage DC to the “multiplier unit” placed adjacent to the water anode. The multiplier unit converts the high voltage/low current to low voltage/high current at a fixed multiplication rate of 25 for this project. For example, a transformer current of 1.8A at 200V DC would be
converted to 45A at 8V at the multiplier and anode. Because of the fixed multiplication rate, the output at the anode is always known, being 25 times the T/R output. The multiplier unit is housed in a totally sealed (IP66) enclosure and is never to be opened, with replacement multiplier units fitted in the event of failure.

A trial of the system arrangement incorporating multipliers with up to 500m of small diameter cable and 200V DC T/R’s was successfully undertaken at the early stage of the project, providing confidence and allowing the system development to continue.

The transformer rectifiers were rack-based switch mode units placed in air conditioned control rooms at either end of the wharf (figure 16). The T/R’s output is passed through a 2.5mm² distribution cable pair to one of 77 multiplier units adjacent to the water anode (figure 17). The water anodes were 25mm diameter MMO cylindrical anodes 1.2m-long, rated at 45A for a 30 year design life. The systems efficiency stems from the reduced losses in the distribution cabling and the high efficiency T/R and multiplier units.

The major advantage of the multiplier system for this project included the following:

• Large efficiency improvement compared with traditional system (85% : 25%)
• Power cost saving of over $1 million over life of system
• Over 35t of copper cabling reduction
• Individual control over current output to each anode
• Control system monitoring and maintenance all from air conditioned control rooms at either end of wharf
• Significant reduction in installation cost.

The multiplier water anode system was commissioned in January, 2009 and has successfully polarised the steel piles and sheet pile to an average of −950mV (Ag/AgCl).

4. CONCLUSION

The Swanson Dock West Cathodic Protection project incorporates three distinct CP systems, namely:

1. Transverse beam CP (concrete)
2. Fender beam CP (concrete)
3. Water anode CP system (steel and concrete)

In each case, a trial of the initial design was carried out which resulted in important changes to the final system design.

In the case of the concrete systems several important aspects of the design were highlighted by the trial works, namely:

• External half cell mapping is useful in trials to add to the effective reference locations
• Beware of reduced protection levels at the extremities of the CP systems
• Beware of slow depolarisation in repaired elements (and wet areas)
• Consider the installation difficulties in heavily reinforced and tidal areas
• The adequate trialling of CP systems can take a significant time and may require modifications and repeated commissioning and monitoring.

The design of the water anode CP system illustrates that there are always alternative methods and systems available to the designer to solve particular problems. Innovation is therefore encouraged on the condition that there is sufficient time and resources available to fully develop and prove the system.

REFERENCES

1. Australian Standard. AS 2832.5 2008 Cathodic Protection of Metals Part 5: Steel in Concrete Structures

ACKNOWLEDGEMENTS

The Swanson Dock West CP project was a design/construct Cathodic Protection and repair project completed by Freyssinet (Australia). Ian Godson & Associates completed the CP design, installation supervision and commissioning for Freyssinet.
Concrete Structure Ownership and Management: Part 1

S L Matthews, J Jacobs, I Stipanovic Oslakovic and D J Cleland

Abstract: With correct design, specification and construction concrete structures provide high performing durable assets with long service lives. Owners can maximise the benefits to be gained from concrete structures, whilst minimizing through-life cost and sustainability impacts, by taking a through-life perspective upon the design, specification and management of their structures; rather than simply focusing upon first-cost. This two-part paper, the second of which will be published in the next issue of Concrete in Australia magazine, provides an overview of the advice given in the fib Guide to Good Practice entitled Concrete Structure Management – Guide to Ownership & Good Practice (fib Bulletin 44). Summarised guidance has been published in a concise single document, namely BRE Digest 510 entitled Concrete Structure Management – The Owners’ Guide to Good Practice 1. Part 1 of the paper deals with general issues associated with concrete structure ownership giving owners an insight into their responsibilities and obligations, what they should do and seek to achieve in the context of concrete structure management, together with information upon the potential deterioration mechanisms and the merits of adopting proactive versus reactive structure management. Part 2 of the paper 2 is to be published in the next edition of the magazine.

1. INTRODUCTION

1.1 Stages in the life of an asset: Sustainability impacts and through-life cost

Society now expects owners to take a greater interest in the sustainability impacts and through-life cost of their assets. These issues include construction and operational costs as well as the delivery of required functional performance for a pre-defined life-span. In the past, construction and operational costs have often been viewed in isolation, resourced out of different budgets, with different budget holders and management teams. Figure 1 illustrates stages in the development and through-life use of an asset.

Actions of the CIA Concrete Durability Committee

Task Group 1 of CIA’s Concrete Durability Committee (see separate article in this issue) is focused on durability planning and is developing separate guidance documents for engineers and asset owners. The need was openly discussed and endorsed at CIA Durability Workshops in the major Australian capital cities in June, 2009 and the CIA National Conference Technical Forum in September, 2009.

Why the focus on durability planning?

Apart from the obvious fact that it is far cheaper to correct durability issues at the design stage than once the structure is built, there can be a reality disconnect as early as the concept stage. Structural design and formal design by structural engineers are accepted by all parties. Structural inadequacy or failure of civil and building structures is typically not a concern.

Durability design is expected by all parties but formal design by durability engineers is rarely a specified requirement. The disconnect is the common informal expectation that someone completes the durability design within the design process and in the absence of a named person the structural engineer must have completed the task. This is not a reasonable obligation for the structural engineer who is not durability trained. An alternative view is the Australian Standards take full account of durability such that structural design being acceptable equates to acceptable durability design. Experience shows this is not always the case.

Is premature deterioration or unacceptable maintenance a present day problem? Yes, in some corrosively aggressive environments, where materials selection is inadequate or building techniques are inadequate. Additionally, durability design is different with variable Specification design life far greater than AS 3600 and other Australian Standards with recent examples in Australia of 50, 100, 150 and 300 years.

Where does TG1 start to develop the CIA guideline documents? An obvious approach is to review relevant international documents. The 210 page fib Bulletin 44 (Concrete structure management: Guide to ownership and good practice in 210 pages) and associated short version BRE Concrete Structure Management Owner’s guide to good practice (Digest 510 in 16 pages) provides one useful source for CIA review. A key approach is to engage the asset owners more formally in the durability considerations affecting their structures, taking account of the whole life cycle, via these publications.

Technical papers by Stuart Matthews and other fib colleagues “Concrete Structure Ownership and Management Parts 1 and 2” provide an overview of fib Bulletin 44 and BRE Digest 510. CIA has obtained Matthews’ approval to publish these papers as an Australian introduction to this important topic. Industry comments on requirements for durability planning guidelines, including thoughts on the relevance to fib Bulletin 44 and BRE Digest 510 to Australia should be submitted to CIA.
A variety of design and management strategies can be adopted for assets throughout their functional life. But decisions on strategy are frequently complicated because other circumstances (use, energy costs, exposure, legislation, safety rules, etc) may change over time meaning that existing facilities may need to be adapted to meet new requirements. Ideally, from concept stage, the professional team and the owner need to be able to work together to address the various technical and process matters that may arise relating to design, construction, maintenance and end of life issues.

However there are implications for the owner post construction in terms of tasks and responsibilities (steps 2-5 – Figure 1); where current intervention options may be limited by the decisions made during design and construction phases. Rehabilitation works can be more complex than new construction and there are many factors to be considered before making choices.

1.2 Illustration of initial cost versus through-life operational costs

Design and through-life management strategies for assets can be summarised somewhat crudely by two conflicting ideologies, namely:

- Buy cheap (low initial cost) and pay more later via higher through-life operational cost.
- Pay more initially (higher initial cost), but gain through a reduced through-life operational cost.

Figure 2 shows cumulative costs and illustrates the importance of early decisions. It illustrates the value of committing significant pre-construction funding to the acquisition of adequate knowledge about the through-life performance of a structure and its component materials and the value of controlling and verifying construction processes in an effective manner. This is necessary in order that the (post-construction) behaviour of the structure during operation and use will conform to the required performance levels. The difference between the red and blue lines symbolises the additional investment or saving being made. Where the red line is above the blue line this implies that extra expenditure is being incurred during planning and stages of construction. Later when the red line is below the blue line, this symbolises that a through-life return is being gained on the additional investment made earlier.

Two marine piers constructed at Progreso in Yucatán, Mexico in the very aggressive marine environment of the Gulf of Mexico provide a practical illustration of the balance between first-cost and through-life costs.

The first pier was built in the 1940s and the designers foresaw that the concrete used in construction would be of low quality. However stainless steel reinforcement was specified, which incurred higher construction costs (the red line in Figure 2) than if normal carbon steel had been used (the blue line in Figure 2). The 1940s pier has performed very well for over 60 years and there has been little deterioration or need for maintenance works – see Figure 3. This demonstrates that satisfactory service life performance can be achieved and that repairs are not inevitable in properly designed and adequately maintained structures.

A neighbouring pier was built some 30 years later in the 1970s. In this instance the structure was reinforced with cheaper carbon steel – which has a much lower resistance to corrosion than stainless steel in the aggressive marine environment. Today the neighbouring pier is totally destroyed, with the only evidence of this newer neighbouring pier being the founding piles sticking out of the sea (see the yellow dashed ellipse in Figure 3A).

This outcome might be summarised by saying that by the application of appropriate knowledge and expertise, the designers of the original pier were able to utilise low performance concrete to produce a high performance concrete structure. Clearly spending
slightly more on the construction of the 1940s pier has paid significant dividends in terms of reducing whole-life costs by minimising expenditure required upon maintenance and repairs. However, the neighbouring pier has to be considered to be an example of a low performance concrete structure. In this case buying cheap (lower first / direct cost) has resulted in a need to pay much more in the later stages by way of higher through-life operational costs, as the neighbouring pier would need to be rebuilt.

Clearly it is best to get construction "right first time" in order to minimise problems with durability. In many instances in-service durability problems could have been avoided if things had been done just that little bit better during the design, specification and construction processes. Of course, this requires an initial investment of time and money to allow the issues to be thought through properly, to utilise the available professional experience, to make any necessary investigations and to take appropriate care.

Life cycle cost analysis (LCCA) provides a tool for predicting the costs of through-life ownership and can assist in making decisions which consider the whole-life cost of the structure, potentially enabling a better and informed judgement to be made on an appropriate balance between first-cost and whole-life cost. Similar considerations can be made in respect of the environmental impact and carbon burden associated with the structure.

The service life of a concrete structure and its associated through-life costs and environmental impacts are largely determined by the achievement of appropriately specified durability related requirements, which need to be set down in the project specification and effectively applied in the design and construction. A long service life requires a satisfactory combination of material and workmanship factors. fib Bulletin 44 ¹ indicates how the project specification can be used in conjunction with the execution standards (EN 13670 ¹ and ISO 22966 ²) and associated quality plans to provide the owner and his professional team with a means of defining and achieving the standard of long-term performance required from the structure.

1.3 Proactive versus reactive structure management

Through-life management strategies generally involve a balance between works for regular (preventive) maintenance and other responsive (reactive) interventions for repair / other purposes. However many structures are managed using primarily reactive approaches. Proper and planned regular inspection, monitoring and assessment processes will give owners an understanding of the necessity and urgency of any interventions required upon a structure, and include different possible options for the future management of the structure.

Overall such actions lead to lower maintenance costs, reduced energy consumption, less pollution (including noise), as well as to improvements in health and safety. Benefits can also arise from advances in intervention processes. Because social, economic and environmental benefits arise from enhancing the performance and reliability of civil infrastructure, preventive maintenance interventions can be part of a sustainable management strategy.

Concrete is a reliable and relatively cheap material (hence commonly used for structures), but it is not maintenance free, especially in aggressive exposure environments. As time has progressed our understanding of the mechanisms underlying the behaviour of concrete and its performance in service has improved. To make good decisions owners need to know the maintenance requirements and the likely cost over the envisaged service life of the asset. Recent guidance for the design of structures, such as fib Bulletin 34 ³, has addressed the issue of service life (also see Table 1).

The main attributes contributing to a satisfactory through-life asset management strategy are:

Table 1. Possible service life requirements for concrete structures (after ISO 2394 & EN1990) ⁴, ⁵.

<table>
<thead>
<tr>
<th>Class</th>
<th>Notional design service life (years)</th>
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<tr>
<td>1</td>
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<td>2</td>
<td>25</td>
<td>Replaceable structural parts</td>
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<td>3</td>
<td>50</td>
<td>Building structures and other common structures</td>
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<tr>
<td>4</td>
<td>100</td>
<td>Monumental building structures, bridges and other civil engineering structures</td>
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</table>
Attribute 1. Consideration of through life performance (and adaptability) as an integral part of the design and construction concept, materials specification and the quality of execution of the structure; with these elements contributing to the preparation of the structure’s Birth Certificate 1, 3, 6.

Attribute 2. Monitoring of through-life performance after construction to assess the overall condition of the structure, or parts thereof, via an agreed structural assessment methodology 1, 3, 6, 9, 10.

Attribute 3. Know and understand the details of any preventive or remedial interventions, including material specifications, application techniques and appropriate criteria used to evaluate the subsequent performance of such interventions; with these elements contributing to the preparation of the structure’s Re-birth Certificate 1, 3.

Attribute 4. Monitoring of through-life performance post intervention using some form of agreed performance indicator 1, 3 to assess the overall condition of the structure and specifically the performance of any preventive or remedial interventions.

2. THE OWNERS’ REQUIREMENTS AND OBLIGATIONS

Society expects designers of structures to fulfil a number of primary requirements which may include:

- Building with appropriate levels of safety against global hazards such as earthquake, hurricane, flood, tsunami and landslide, to avoid collapse or failure of the structure. This also applies to local hazards such as falling debris arising from spalled concrete.
- Preserving the functionality of the structure by minimising or avoiding interruption to the services provided to society or the owner by the structure.
- Ensuring that an appropriate service life is chosen for the structure at the time of design and that design, materials, construction and maintenance strategies meet this criterion.
- Ensuring that the structure has acceptable (limited) health and environmental impacts during its construction, subsequent use and maintenance.
- Satisfying minimum aesthetic requirements for the structure.
- Building to permit easy repair, demolition or change of use at the end of the initial use period.

The use of performance specifications, rather than prescriptive specifications, may imply that:

- Owners need to be able to define and clearly specify reasonable performance requirements for the structure and be sufficiently confident that these will be met.
- The supporting professional team should be able to turn performance requirements into design strategies.
- Contractors need to be aware of and use appropriate materials and processes to achieve the required performance levels.

Special consideration may be needed for certain types of structures, such as those that:

- Have minimal structural redundancy – such as some forms of grandstand.
- Attract large numbers of people.
- Are particularly tall.
- Exist in an aggressive environment.
- Are outside the scope of verified code methodologies, or use innovative materials / design.
- Were designed to now outdated parts of codes that are now recognised as being not sufficiently conservative (eg in terms of shear strength issues or concrete cover in carpark structures).

Although owners and operators of structures will generally have duties under law, the exact nature and extent of these duties will vary from one country to another. It should also be borne in mind that different elements of a structure may be subjected to different local environments and loadings and thus may perform differently.

ISO 2394: General Principles on Reliability for Structures’ details assumptions that are made when a structure is designed about the way it will be constructed and then subsequent maintained. Eurocode EN 1990: Basis of Structural Design sets out some slightly different but critical assumptions applicable to all structures designed within its remit. These include requirements for:

- Adequate supervision and quality control during construction.
- Use of construction materials and products that comply with general Eurocode stipulations and standards.
- Use of the structure in accordance with the design assumptions.
- Adequate maintenance, so that no deterioration will occur in service that will reduce structural capacity.

Currently, in many situations, these requirements are not being met. Positive actions by the owner / operator to address these requirements might include:

- Specific provision for undertaking inspections and maintenance works.
- Consideration of the sensitivity of the structure to the effects of deterioration.
- Measures to achieve resistance to the envisaged deterioration processes, as far as that may be possible.

For example, EN 1990 identifies that a suitable quality management system is required. Other current codes commonly contain similar, but less emphatic, assumptions. Almost all current design codes, including the structural Eurocodes, make no allowance for the effects of deterioration during the lifespan of the structure. Thus to maintain their intended performance appropriate through-life management, care and maintenance must be provided to existing structures and buildings. An appropriate budgetary allowance (per year) for maintenance works / through-life care expenditure would be between 1.5% and 2% of the cost of construction.

Owners have responsibilities in respect of the supervision, inspection and checking of the structural integrity of existing structures. The issues to be addressed in conjunction with the owner’s professional team include:

- Specifying clear requirements for the minimum documentation to be supplied with the structure.
- Understanding the conceptual and overall structural design aspects of the building / structure.
- Understanding the potential risks associated with the particular type of structure (e.g. long span roof).
**DURABILITY**

![Diagram of Owner requirements](image)

Figure 4. Illustrative owner requirements defined in terms of components of sustainable construction.

- Assessing the current condition of the structure.
- Understanding the implications of changes which have occurred throughout the life of the structure, as well as any new build additions and alterations.
- Defining an appropriate inspection regime, together with cycles and inspection / monitoring requirements.
- Recommendations on how inspections / monitoring should be undertaken.
- Requirements for the professional standing and technical background of the inspector / structure assessor.
- Recommendations for assessment of innovative structures and structural elements.

In the European system, essential requirements for performance in service are defined by the European Construction Products Directive (CPD). The two of the greatest relevance and importance to asset management are (1) mechanical resistance and stability and (2) safety in case of fire. It is therefore critical to spell out the responsibilities held by the different parties (owner, engineer, repair specialist …) involved in the overall assessment, maintenance and repair processes for a concrete structure. For example:

- The owner is responsible for the development and implementation of a life-care plan.
- The owner should state requirements for the structure after intervention, such as:
  - The required remaining service life and the corresponding performance criteria.
  - The need for upgrading of the structure if necessary.
  - An equal or improved aesthetic appearance.
- The engineer (with others in the professional team) will assess the quality, the present condition and the estimated remaining service life assuming there is no maintenance or repair intervention, but a defined future use of the structure. The potential effects of envisaged future climate change may be considered.

- The engineer (with others in the professional team) will evaluate the technical and economical ability of alternative interventions to provide the required short and long term performance as well as assessing their potential influence on the safety and serviceability of the structure.
- The owner must select a preferred solution compatible with the business / service goals of the organisation.

Several of the owner’s requirements illustrated in Figure 4 may conflict and these need to be addressed and balanced carefully. The requirements will vary for each structure and need to be clearly understood. In some cases legal obligations, such as these related to dangerous substances, may override the owner’s preferred requirements. To improve the quality achieved at all levels in the operation and maintenance of concrete structures, owners have to establish the basis for assessing both the potential benefits and the results of the remediation actions. The technical information is unlikely to resolve business related assessments related to the owner’s particular activities.

3. IMPLICATIONS OF CONCRETE STRUCTURE DETERIORATION

Deterioration affects all structures and materials to varying degrees and at varying speed. In order to better understand the aging of their asset, owners need to establish knowledge on potential deterioration mechanisms.

Deterioration of concrete structures may arise for a number of reasons including:

- Mistakes in design or during construction.
- Lack of maintenance and underestimation of its importance.
- Environmental aggressivity and actions upon the structure.
- Ageing processes.
- Increased loading or changes in exposure conditions.

Although a concrete structure may look to be in a poor condition, its ultimate strength may not be significantly impaired. Chemical
Reactions associated with deterioration generally progress slowly when little moisture is present. Thus appropriate technical expertise and knowledge of the particular circumstances is required to make an informed assessment of structural capacity and to understand the related implications.

Figure 5 shows schematically a number of stages in the development of deterioration of a concrete structure due to reinforcement corrosion. This primarily portrays the effects of carbonation induced corrosion and some aspects of that due to the influence of chlorides. However, this portrayal excludes effects such as pitting corrosion and the dissolution of steel which may occur in certain circumstances (such as high levels of chloride, long periods of exposure or saturated concrete conditions) and would probably not result in cracking of the concrete. The stages portrayed in Figure 5 are:

- Depassivation of the steel reinforcement due to external environment influences. At this point corrosion is possible, but there is no observable or actual physical damage.
- The development of corrosion products, occupying a volume significantly larger than the original steel. The induced tension in the concrete cover leads to cracking. At this point there is observable damage and possibly rust staining, but typically no significant loss of structural strength of the affected members.
- Widening cracks facilitates the penetration of moisture and aggressive substances into the concrete, promoting a faster rate of corrosion eventually leading to spalling of cover concrete. At this point there is probably a small reduction in strength, as well as observable damage (i.e. cracks, spalling of concrete).

- Corrosion progresses at an increased rate, resulting in more cracking and spalling of concrete cover, and increasing loss of cross-sectional area of the embedded steel reinforcement. At this point there is observable damage (i.e. widespread and severe cracks and spalling of concrete), plus structural implications such as loss of section to reinforcing bars and a reduction in the bond / anchorage of reinforcement. This leads to a significant reduction in strength of the affected members which may eventually lead to their collapse. This situation requires urgent intervention.

The deterioration mechanisms causing durability problems in reinforced concrete structures are often grouped as follows.

- Corrosion of reinforcement, causes of which include:
  - Atmospheric carbon dioxide carbonating the concrete and depassivating the steel
  - The presence of significant levels of chloride ions in the concrete

- Deterioration of the concrete itself, causes of which include:
  - Freeze and thaw effects.
  - External chemical attack caused by acids, sulfates, nitrates etc.
  - Internal deleterious chemical reactions such as

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#1 Depassivation is the state where, as a result of the loss of the surface protective oxide film (see carbonation footnote), the embedded steel reinforcement is able (to start) to spontaneously corrode.

#2 Carbonation occurs when (acidic) atmospheric carbon dioxide reacts with the alkaline pore water solution in the concrete. The associated reduction in the pH of the concrete pore solution creates a situation where the surface protective oxide film on the embedded steel reinforcement becomes unstable and breaks, allowing corrosion to occur. When concrete is carbonated it no longer protects the steel reinforcement against corrosion.
leaching, alkali-aggregate reactions (AAR) and alkali-silicate reactions (ASR), delayed ettringite formation (DEF) etc.

- Bacterial action which can occur in some conditions (sewers).

- Physical external damage to the structure, potentially caused by actions such as:
  - Overloading
  - Fire
  - Impacts, accidents and malicious attack
  - An earthquake / seismic event
  - Abrasion and erosion.

Table 2 presents an overview of some of the above cited deterioration mechanisms and their potential occurrence in some types of structure or components of structures.

The nature and the progress of deterioration may vary across the structure. It is important to detect deterioration processes at an early stage, and to determine their causes. This allows the owner to make appropriate decisions and act in a timely manner.

Generally damage to a concrete structure is the result of a deterioration process acting over a period of time, probably over years and possibly over decades. Only some causes of physical damage, such as earthquake, impact or overloading, may result in the sudden occurrence of severe damage. Once damage has occurred further deterioration and damage may progress at a faster rate (refer Figure 5).

Most deterioration processes cause cracking of concrete. Small cracks may thus be an early indication of future damage. An intervention at this stage will generally provide reasonable results for modest (low cost) concrete repair and protection efforts. Unfortunately the first signs of deterioration are often neglected and cracking may progress to spalling at an increasing rate. This may create a hazard for users and passers-by. Ultimately, deterioration may affect the structural capacity of the affected members as noted above.

At this stage, in addition to the repair of the damage, replacement of reinforcement or strengthening may be necessary. In some cases, demolition or replacement of parts of the structure or the whole structure may be the only feasible course of action.

The management of concrete structures could be improved by:

- Early intervention, before damage is visible
- Proactive monitoring and maintenance in support of this approach.

Table 2. Overview of some deterioration mechanisms for some types of structure.

<table>
<thead>
<tr>
<th>Type of structure</th>
<th>Corrosion of steel</th>
<th>Deterioration of concrete</th>
<th>Physical damage</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>CO₂-induced</td>
<td>Chloride induced</td>
<td>Freeze/thaw</td>
</tr>
<tr>
<td>Above ground buildings</td>
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<td>■</td>
<td>■</td>
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<td>Industrial floors</td>
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<td>Sewage plants</td>
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<td>Tanks and pipes</td>
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KEY: ■ Commonly affected by deterioration mechanism ■ Sometimes affected by deterioration mechanism ■■ Infrequently affected by deterioration mechanism
Correct diagnosis of the problem and mechanism(s) causing the deterioration.

Using effective intervention systems for preventive and remedial treatments.

4. ENGINEERING ASPECTS OF STRUCTURE MANAGEMENT

There are two main divisions in the approaches to selecting technical treatments as part of the process of managing a structure; these are the reactive and the proactive approaches.

- The reactive approach to intervention works is typically triggered by the occurrence of readily observable damage to a structure, such as the existence of cracking or spalling of concrete. Works are then initiated to make a repair in order to slow the rate of deterioration and extend the service life of the structure.

- The proactive approach to maintenance / intervention works involves taking action earlier to extend the period of initiation, thereby delaying the onset of corrosion. In the case of corrosion, proactive treatments might include the early application of a coating to the surface of the concrete or perhaps the provision of a cathodic prevention system.

The implications of some aspects of the reactive and proactive approaches upon the through life functionality of the structure, disruption to users or the service provided and upon cost are illustrated in Figure 6. It indicates that the proactive approach to structure management can lead to savings in overall cost, shorter periods of disruption to services etc. portrayed by the dashed ‘columns’ in the upper part of the figure.

Proactive approaches to structure management means adopting proactive methods for gathering information about the condition of the structure and it will not be satisfactory to rely solely upon visual inspection procedures.

The proactive approach can be applied both to individual structures and to groups of structures in a common ownership, such as highway and railway structures. In these cases a proactive management system should help to prevent unforeseen service disruptions and functional / safety failures. Such a system will include formal procedures for organising and undertaking various operational activities such as inspection, assessment of structural and non-structural components, maintenance, remedial works and protection interventions.

Critical aspects of planning and implementing proactive through-life care of a structure are the service life period for which the structure is designed (the design service life) and the associated maintenance plan put in place to support this.

The durability of the structure in its environment needs to be such that it remains fit for use throughout its design service life. This requirement can be achieved in various ways, including:

- By designing suitable protective or mitigating systems to reduce the risk of premature failure.
- By using materials that, with appropriate maintenance, will not degenerate during the design service life.
- By over-sizing the components of a structure that will experience deterioration to compensate for the anticipated deterioration occurring during the design service life.
- By choosing a shorter lifetime for the structural elements but making provision for them to be replaced one or more times during the design life.

The above need to be undertaken in combination with appropriate inspection and investigation activities (eg. surveys, testing etc) at fixed or condition dependant intervals and with appropriate maintenance activities. The inspection and investigations activities can also take place on an ad-hoc basis.

The assessment of a structure is a complex interaction between various information sets, including:
DURABILITY

- Structural, environmental and service data.
- Data from existing documents which might include asset management systems and related software.
- Data from visual inspection (from formal surveys or obtained during maintenance work).
- Test data from in-situ and laboratory investigations.
- Test data and experience of the performance of potential remedial actions and interventions.
- Planning of assessment activities, including gathering of information about the history of the structure, a first site visit, programming of activities, preparation of a proposal and related contractual tasks.
- Routine inspection, consisting of visual inspection, basic testing, reporting and simple condition evaluation and the planning of a detailed investigation, if this is necessary.
- Detailed investigations: intrusive examination (eg core samples), special testing of materials and monitoring of service environments. Monitoring may involve one-off site measurements of various parameters or possibly the use of installed / embedded instrumentation, and analysis and
forecasting of deterioration phenomena for assessment of safety, durability and prediction of progress of corrosion or other deterioration phenomena.

• Special tests and investigations: structure response testing, measurement of actions acting on the structure.

• Deterioration assessment based on routine inspections and detailed investigations:
  – For concrete: categorisation of degraded areas on the structure.
  – For reinforcement and prestressing: degree and rate of corrosion, remaining prestressing forces, etc.

• Structural assessment and evaluation based on special tests and investigations: actual carrying capacity and estimation of safety level, prediction of the remaining service life, adequacy rating, etc.

The main activities and the overall process of undertaking structural inspection, testing, assessment and evaluation are illustrated in the flow chart presented as Figure 7.

Before any tests are commissioned, the influence of the test results on subsequent decisions should be established. If decisions would not change, testing is not helpful. Detailed information on the specific test procedures can be found in various publications including fib Bulletins 17 and 22, in International Standard ISO 13822 and also in guidance prepared by The Institution of Structural Engineers.

In general, the choice of appropriate concrete preventive or remedial intervention method is a complex task and will depend on a number of factors, such as:

• Requirements defined by the owner. (ie What the owner wants).
• Protection or remedial intervention assumptions (ie How the structure is to be looked after).
• Experience and success of using the different available preventive or remedial intervention techniques.

In the European system, EN 1504-Part 9 outlines six options that an owner may consider in response to deterioration of a

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**Table 3. Deterioration processes and possible remediation actions (after Part 9 or EN 1504)**

<table>
<thead>
<tr>
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<th>Observation</th>
<th>Cause of defects</th>
<th>Principle of remedial actions</th>
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<td>• Delamination</td>
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<td>• Disintegration of the matrix</td>
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<td>• Uniform corrosion</td>
<td>Carbonation of concrete</td>
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<td>• Pitting corrosion</td>
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<td>Control of anodic areas (CA)</td>
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<td></td>
<td>• Stress corrosion</td>
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<td></td>
<td>• Cracking</td>
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<td>Corrosive contaminants</td>
<td>Cathodic control (CC)</td>
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<td>Cathodic protection (CP)</td>
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<td>Control of anodic areas (CA)</td>
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<td>Preserving or restoring passivity (RP)</td>
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<td>Stray electric currents</td>
<td>Increasing resistivity (IR)</td>
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Concrete in Australia Vol 36 No 3 63
DURABILITY

Concrete structure. Equivalent ISO standards are being developed by ISO Committee TC71SC7. The six options detailed in EN 1504-Part 9 are:

- Postpone the repair-work, but implement regular inspection and/or monitoring of the degradation process. This may lead to a restriction on how the structure is used.
- Re-evaluate the structural load capacity or other critical performance characteristic.
- Make efforts to avoid further degradation.
- Undertake structural strengthening/restoration and aesthetic improvement works.
- Reconstruct selected components or the complete structure.
- Demolition.

The six options detailed in EN 1504-Part 9 are shown pictorially in Figure 8.

EN 1504-Part 9 also suggests links between the deterioration processes described in Section 4 and possible preventive or repair interventions; these links are shown in Table 3.

Critical factors that influence the selection of protection or repair options include:

- The importance of correct diagnosis.
- The nature of the problems to be overcome.
- The time to when intervention will be required and/or the required residual life.
- Whether structural or non-structural issues are to be addressed.
- Load take-up and sharing mechanisms active within the structure.
- The required service life of the repair materials and repair methods.
- Maintenance or intervention works needed and how these might be undertaken.

The following steps are involved:

- Design and specification of required works.
- Whether the intervention works are to be structural or non-structural.
- The selection of appropriate materials.
- Consideration of associated non-engineering/non-technical issues which may influence the selection of an appropriate intervention or structural management option (eg. operational or business constraints).
- Selection of structure management and intervention options. Planning and timing/phasing of intervention activities involves consideration of various factors including:
  - Execution of works.
  - Supervision of all processes including design, specification and execution.
  - Verification of the effectiveness of a remedial or preventive intervention and the quality achievable.
  - Monitoring of performance.
  - Evaluate results achieved by intervention and plan future actions.
  - Maintenance or post-intervention life-care actions.
  - Costs (may be elements other than costs for engineering and technical issues).

By giving each of these factors a certain score (how good) and weight (how important), a methodology for selecting a repair method can be developed. This approach, although still under development, can be helpful in various situations, such as when establishing priorities for the management of a number of assets.

5. CONCLUDING REMARKS

Part 1 of the paper has considered the general issues associated with concrete structure ownership and has given owners an insight into their responsibilities and obligations, what they should do and seek to achieve in the context of concrete structure management, together with what the potential deterioration mechanisms might be and the merits of adopting proactive versus reactive structure management. A description has been given of the stages in the life of an asset, noting the issues of sustainability impacts and through-life cost, the potential trade-off between initial costs and overall through-life costs, the roles of proactive versus reactive structure management and a number of potential deterioration mechanisms to be avoided.

Part 2 of the paper, to be published in the next issue, provides more specific guidance on the overall process steps involved in the through-life management of a concrete structure, the actions to be taken, illustrating these by means of an application example and considers some frequently asked questions.

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3. Matthews et al, Concrete structure ownership and management: Part 2, Structural Concrete
6. fib Bulletin 34, Model code for service life design, fib, Lausanne, Switzerland, 2006.
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ACKNOWLEDGEMENTS

The authors acknowledge the valuable contributions of fib Commission 5 members to fib Bulletin 44, Concrete structure management – Guide to ownership & good practice, which provided the basis for this publication.
CCAA Briefing 11: Sustainable concrete materials
CCAA, St Leonards, NSW, 2010
Throughout history, the use of concrete as a building material has contributed significantly to the built environment. Enduring examples of various forms of concrete can be found as far back as the early Egyptian civilisation. Significant building remnants still exist from the Roman civilisation, which used concretes made from naturally occurring volcanic ash pozzolans, mixed with water, sand and stone. In the modern era, the properties of concrete were refined in the late 1800s, with the introduction of a patented manufacturing process for portland cement. Such a ubiquitous form of construction has a significant impact on sustainability – a concept that needs to be clearly understood. Environmental responsibility is certainly part of it, but any form of construction must also be socially beneficial and economically viable. To be truly sustainable a building material must address all three of these criteria. When considered within this framework, it becomes clear that concrete construction delivers a very strong sustainability performance. Available online at http://www.ccaa.com.au/publications/pdf/Briefing11.pdf.

CCAA Briefing 12: Thermal mass benefits for housing
CCAA, St Leonards, NSW, 2010
Thermal mass is the ability of a material to absorb and store heat. Concrete's high thermal mass, as part of an integrated passive solar design approach, can significantly reduce heating and cooling energy requirements and the associated greenhouse gas emissions. It makes economic sense for householders to invest in a thermally comfortable home that will provide cost savings for the rest of its life. This Briefing demonstrates that concrete is the responsible choice for energy efficient and sustainable homes. Available online at http://www.ccaa.com.au/publications/pdf/Briefing12.pdf.

CCAA Briefing 13: Sustainable concrete buildings
CCAA, St Leonards, NSW, 2010
The design and construction of sustainable buildings involve striking a sensible balance of social, environmental and economic considerations. Concrete, in its many forms, is a versatile building material that can provide many sustainable benefits by virtue of its economic value, thermal mass, durability, fire resistance, acoustic performance, adaptability and recyclability. Choosing the appropriate method of concrete construction for the particular type of building will ensure these benefits are utilised to deliver the most sustainable outcome. Available online at http://www.ccaa.com.au/publications/pdf/Briefing13.pdf.

CCAA Briefing 14: Sustainable concrete infrastructure
CCAA, St Leonards, NSW, 2010
Infrastructure must be based on a sensible balance of social, environmental and economic considerations, and is essential to the development of modern civilisation. Road, rail, airport, utilities and port facilities provide for the necessary transportation of people, goods and communications that is the life blood of modern society. Infrastructure provides the essential supply of potable water, the removal and treatment of waste and sewage; it enables the provision of health, education and law-and-order. This Briefing provides asset owners, designers and specifiers with information on how concrete meets the performance demands of infrastructure. Available online at http://www.ccaa.com.au/publications/pdf/Briefing14.pdf.

Concrete: the responsible choice
CCAA, St Leonards, NSW, 2010
The World Commission on Environment and Development (WCED) defines sustainable development as development that meets the needs of the present without compromising the ability of future generations to meet their own needs. Sustainable development can be conceptually broken into three constituent parts – social, environmental and economic. “Social” sustainability refers to the quality of life of individuals and their communities; “environmental” references the management and preservation of our air, water, land and ecosystems; and “economic” the level of prosperity for organisations and individuals. “Sustainability” is achieved where all three of these parts overlap. The cement, concrete and aggregate industries are continually striving to ensure their processes and practices are consistent with the principles of sustainable development. Available online at http://www.ccaa.com.au/publications/pdf/SustainabilityBrochure.pdf.

Recent advances in concrete technology and sustainability issues: Proceedings Tenth ACI International Conference, Seville, Spain October 2009. SP-261
P. GUPTA, T.C. HOLLAND, AND V.M. MALHOTRA
American Concrete Institute, 2009
Accession number : 08A04372
This publication contains the proceedings from the Tenth ACI International Conference on Recent Advances in Concrete Technology. The 21 papers include Durability of ultra-high-performance concrete; Shrinkage reducing effect of a combination of internal curing and shrinkage compensating agents on high performance concrete; and Geopolymer concrete-sustainable cementless concrete.
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