Tauranga Harbour Link

Superior durability in a marine environment
INVITATION TO ATTEND

The Concrete Institute of Australia’s 25th Biennial Conference will be held in Perth, Western Australia, from Wednesday 12 October to Friday 14 October 2011. We are pleased to be able to announce some special features which will make this a great conference for attendees, participants, exhibitors and sponsors.

We have had a tremendous response to our call for abstracts, and anticipate that in addition to our invited speakers, we will have around 140 presentations. At least 10% of our authors are from overseas. Despite this strong response, we will be able to maintain a comfortable presentation time in four concurrent streams, for the benefit of both authors and audience. The Burswood Entertainment Complex provides us with three adjacent lecture rooms each of which holds more than 100 attendees, plus the Grand Ballroom, which will also be used for plenary sessions.

There has been a significant take up of sponsorships, following the lead of our Principal Sponsor, Boral, but there are still very good sponsorship packages available. The Burswood venue also provides excellent space integrated into the heart of the conference for trade display booths, a number of which have also already been taken up, but there are still plenty available.

Our various associated social events will culminate in our Gala Dinner on the Friday. The Concrete Institute Gala Dinner in Perth has become a very popular annual event, and in 2011, we plan to incorporate it into the Biennial Conference to make a really special night, which will feature the presentation of the Excellence Awards.

We look forward to your participation in what we are sure will be a popular and rewarding conference, whether as a sponsor, exhibitor, participant or attendee. And we especially look forward to welcoming you to Perth.

Joe Wyche
Chairman,
Conference Organising Committee

ORGANISING COMMITTEE

Anthea Airey
Graeme Burns
Ben Cosson
Matthew Flynn
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F: (08) 9389 1499
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VENUE

Concrete 2011 will be held at:
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With its advanced technical facilities and communication infrastructure as well as first class catering and service, the Burswood Entertainment Complex meets all the requirements needed for an enjoyable and productive working environment and is undoubtedly an ideal host venue for the 2011 Conference.

TRADE EXHIBITION

The Concrete 2011 Exhibition will be held in the Grand Ballroom at the Burswood Entertainment Complex. This exhibition space will provide the perfect platform for delegates to meet with organisations of particular interest to them. All catering breaks will be held amongst the exhibition to ensure ample time is available for meetings to be conducted so that industry partnerships can be enhanced into the future.

KEY DATES

Early Bird Closes
29 July 2011

Standard Registration available
29 July 2011 to 9 September 2011

Late Registration available
From 9 September 2011
Our Technical Program will be highlighted by an Invited Speaker plenary session on each conference day. The three speakers are:

**MARTIN CLARKE**

Martin Clarke is Chief Executive of British Precast Concrete Federation, which he joined in 2002. He graduated in Economics and Statistics from Exeter University, and took an MSc in Operational Research at Hull University before starting as a market analyst with ARC Concrete, a concrete pipe maker, in 1972. He became Marketing Manager in 1974 and stayed with the group for 18 years latterly as Business Development Director of quarrying, asphalt and concrete group ARC Aggregates, now Hanson. He worked on many global projects, especially in the USA, during a period of rapid expansion in the 1980s.

In 1990 he joined the British Cement Association as Market Development Director during a time of transition from the previous Cement & Concrete Association. He specialised in setting up focused development groups with other stakeholders. In 2002 he moved to British Precast as Chief Executive where he has driven their sustainability, safety and innovation programmes. He has been married to Lynne since 1973 and they have two children Cathy and Tom. His interests are photography, walking and rugby.

**LINDA FIGG**

Linda Figg is President/CEO and Director of Bridge Art for FIGG, an international firm that exclusively specializes in bridges. FIGG bridges have received over 320 awards for customers, recognizing economy, innovation, sustainability, and aesthetics, including three Presidential Awards through the National Endowment for the Arts. Linda presently serves as Chair of the board of directors of the Construction Industry Round Table. She was named as one of Engineering News Record’s Top 22 Newsmakers in 1998 and Concrete Construction magazine named Linda as one of the 13 most influential people in the concrete industry in 2007. In 2009 Linda was named to the Alabama Engineering Hall of Fame. Linda’s passion for creating environment friendly and functional bridge sculptures has led her to focus on improving the quality of life in communities with landmark bridges.

**PROF RAVINDRA GETTU**

Dr. Ravindra Gettu is a Professor of Civil Engineering at the Indian Institute of Technology Madras, Chennai since 2004. He has a Ph.D. degree in Structural Engineering from Northwestern University. After his post-graduate studies in the US, he moved, in 1990, to Spain and became the Director of the Structural Technology Laboratory in Barcelona. His areas of research have been fracture mechanics of concrete and rock, nonlinear behaviour of cement-based materials, high strength concrete, fibre reinforced concrete, self compacting concrete, and the effective use of chemical admixtures. In these and related areas, he has co-authored more than 300 publications. He is the Chairman of the Technical Activities Committee of RILEM, the International Union of Laboratories and Experts in Construction Materials. Structures and Systems based in France.
Directional investment

The first six months of 2011 has seen the Concrete Institute of Australia (CIA) implement major developments set out in the 2010-2012 Strategic Plan. The approval by members of the CIA’s new Constitution has been a major development. This initiative enables the institute to alleviate much of the red tape previously required and allows for future streamlining of operations. An article in this issue of Concrete in Australia provides more detail on what some of the new Constitution allows, however some key aspects are: the addition of a new young member portfolio position on Council, allowing for legal acceptance of electronic election processes and constitutional matters, and providing greater flexibility of national and state operations.

The institute has also launched its new website, the Knowledge Centre. This will allow for a platform to be created that will meet the needs of not only existing members in a more transparent manner, but will also better meet the needs of a younger demographic. It is also the intention to allow the website to serve as a means of being able to access technical material in a more efficient manner, and open up the door to provide additional services – such as the Standards Online and a potential joint American Concrete Institute/CIA membership – both of which feature as articles in this issue of Concrete in Australia.

The access to technical material has been one of the key benefits that members view as important from the CIA – as fed back by members through a recently issued membership survey. The Knowledge Centre also has close alignment with one of the key strategic projects that will be planned for the 2012-2014 period, whereby the institute will focus on increasing its range of intellectual property and capital.

Directional steps for the CIA through the period will be guided by member responses to the recent member survey. The survey, as you will read in this issue of Concrete in Australia, was designed to be brief to ensure that relevant key points are extracted and directive information attained.

As you will also read in the Projects section of this issue, the feedback from the two national seminars that have so far been delivered in 2011 has been very pleasing. The national seminars of Reinforcement Detailing and Shotcrete Essentials have provided attendees with valuable knowledge that has direct application in professional practices.

This issue

This issue of Concrete in Australia includes a feature on High Strength Concrete. It contains an interesting mix of technical papers on the topic, covering both theoretical and practical aspects.

Additionally, a feature news piece provides information relating to the institute’s third national seminar series on High Performance Concrete: Design and Applications, developed by Professor Priyan Mendis. This issue also includes three papers that are to be presented at Concrete 2011, which I would again encourage all members to attend in October 2011, in Perth, Western Australia.

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

4 President’s report

6 News

18 Projects

22 FEATURE TOPIC: HIGH STRENGTH CONCRETE

National Seminar Series

Adverse effects in high strength concrete when exposed to fire

Compressive behaviour of FRP-confined high strength concrete

High performance concrete in bridge decks – Opportunities for innovation

ALSO IN THIS ISSUE

Controlling water content of concrete for high durability applications

Extending the service life of concrete bridges – Corrosion monitoring of Lynch’s Bridge over Maribyrnong River

Self compacting concrete for superior marine durability and sustainability

CCAA Library

New members

The Tauranga Harbour Link duplicates an existing bridge to carry traffic to the Port of Tauranga in New Zealand. The construction used high durability, self-compacting concrete (SCC); as permitted by NZS 3101:2006. Recent international research had indicated that a hundred year design life could be achieved with 40 mm of concrete cover, but local New Zealand materials had not yet been tested. This paper reveals some of the testing, as well as the final outcomes, from this project. To read more, go to page 59.

PHOTO: GOLDEN BAY CEMENT

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www.concreteinstitute.com.au e x c e l l e n c e i n c o n c r e t e
Consider a changing climate in concrete design, report recommends

An analysis of potential climate change impacts on the deterioration of concrete infrastructure has been released by CSIRO. The report aims to allow engineering designers, asset managers and other professionals to understand the ramifications of a changing climate on concrete infrastructure. It also aims to provide guidance to assist appropriate adaptation responses at the design and maintenance stages.

The report’s lead author, CSIRO Climate Adaptation Flagship’s Dr Xiaoming Wang said: “Failure to consider the effects of climate change may compromise the safety of concrete structures but, overcompensating in our efforts to adapt for climate change may unnecessarily increase costs.”

Concrete deterioration is caused by a range of physical, mechanical and/or chemical factors. One of the major threats to the longevity of concrete structures is carbonation, which occurs when atmospheric CO₂ penetrates into the structure to expose steel reinforcements to corrosion. Corrosion caused by chloride penetration is another serious threat to concrete durability causing cracking, delamination, or spalling, especially in marine and coastal areas.

Chloride-induced and carbonation-induced corrosion of concrete infrastructure are directly affected by environmental factors such as temperature and humidity. Carbonation is also affected by the concentration of CO₂ in the atmosphere. All these factors

Concrete Institute of Australia

Office contact details

National and NSW Branch
Suite 401, Level 4, 53 Walker Street
North Sydney, NSW 2060
PO Box 1227, North Sydney 2059
Phone: 02 9736 2955
Fax: 02 9736 2639
Email: admin@concreteinstitute.com.au
Web: www.concreteinstitute.com.au

Queensland Branch
Suite 2, Level 2, 485 Ipswich Road
Annersley, QLD 4103
Phone: 07 3892 6668
Fax: 07 3892 5655
Email: qld@concreteinstitute.com.au

Victoria Branch
2nd Floor, 1 Hobson Street
South Yarra, VIC 3141
Phone: 03 9804 7834
Fax: 03 9827 6346
Email: vic@concreteinstitute.com.au

South Australia Branch
PO Box 559
Marden, SA 5070
Phone: 08 8362 1922
Fax: 08 8362 1822
Email: sa@concreteinstitute.com.au

Western Australia Branch
45 Ventnor Avenue
West Perth, WA 6005
Phone: 08 9389 4447
Fax: 08 9389 4451
Email: wa@concreteinstitute.com.au

Tasmania Branch
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Vol 37 No 2

NEWS

– temperature, humidity and CO₂ concentration – will vary as a result of increasing greenhouse gas emissions and climate change, CSIRO said.

Elevated CO₂ concentration accelerates carbonation and increases carbonation depth in concrete, the report states, increasing the likelihood that carbonation-induced reinforcement corrosion is initiated and resulting in structural damage of concrete structures.

Elevated temperature also accelerates carbonation, chloride penetration and corrosion rate of reinforcement, the report states. It surmises that lowered relative humidity may reduce or even stop carbonation and chloride penetration in areas where the current yearly average relative humidity is just above 40-50%. However increased humidity may result in carbonation and chloride penetration occurring in the regions where they are now negligible.

“Currently, the primary assumption in construction designs is that environmental conditions will be similar to those of the past.

“However, scientists and engineers from CSIRO, in collaboration with a colleague from the University of Newcastle, have shown that increased atmospheric CO₂ in addition to a changing climate – including ‘chronic’ factors like increasing CO₂ concentrations, temperatures and humidity, and ‘acute’ factors like extreme weather events – will alter environmental exposure of most concrete infrastructure over their relatively long lifetime.

“This means that concrete structures will generally deteriorate faster with major implications for the safety, serviceability and durability of infrastructure, particularly in warmer inland and coastal areas,” Wang said.

The report makes a number of recommendations on the design of new – and maintenance of existing – concrete infrastructure.

The authors write: “Decisions relating to infrastructure development, maintenance, replacement and refurbishment over the service lifecycle can have consequences for 30-200 years or more. Therefore such decisions and associated investments should take into account future climatic conditions.”

The report includes predictions that the arid zone area in central Australia, where no carbonation occurs (due to lack of moisture), will extend in the future. As temperatures rise there may be more carbonation around the New South Wales/Victoria border, and in a small area in the west of Western Australia.

Also, carbonation-induced corrosion initiation is likely to be higher around the boundary between the arid climatic zone in central Australia and the temperate climatic zone in the west, south and east of Australia.

“The main reason for this is the lower cover requirement in design for concrete structures in arid and temperate climatic zones, which are also away from coasts. Changes in carbonation-induced corrosion damage follow a similar geographic pattern,” the report states.

The risk of chloride-induced corrosion initiation of concrete structures along coasts increases only slightly, the report predicts, while in inland areas where temperatures increase and chloride and moisture are suitable for corrosion initiation, the risk of corrosion is likely to increase.

The report recommends that when carbonation-induced corrosion initiation has to be considered in the design of concrete structures, the effect of climate change impacts should at least be considered for structures at exposure A1 and A2.

The effect should also be considered for structures at exposure B1 that are designed for a service life of more than 60 years, especially in regions with a warm or tropical climate.

When carbonation-induced corrosion damage has to be taken into account in design, the effect of climate change impact should not be neglected for structures designed for more than 35 years for exposure A1 and 25 years for exposure A2.

For chloride-induced corrosion initiation and damage, climate change impacts should be considered for concrete structures designed for a service life of more than 50 years for exposure C2, and not be considered for structures of designed service life less than 100 years for exposure C1.

In general, the effect of climate change should be considered for the design of concrete structures for high environmental exposures, especially for those classified as C2.


The research was funded by the federal Department of Climate Change and Energy Efficiency (DCCEE) and the CSIRO Climate Adaptation Flagship.
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Launch of the Knowledge Centre

The Concrete Institute of Australia (the Institute) is pleased to launch its new website, aimed to serve as the Institute’s Knowledge Centre. The Knowledge Centre has been developed for several purposes, including:

- Providing a platform for further member benefits.
- Meeting the needs of and assisting to increase the younger membership demographic.
- Increasing and encouraging higher participatory levels of the membership.
- Providing greater transparency regarding all Institute activities and National Council portfolios.
- Providing greater efficiency for committee and business activities.
- Maintaining pace with technological advancements to match societal demands and sustain membership growth. The Knowledge Centre will provide for far greater transparency of Institute activities within National Council, State Committee’s and technical groups.

The response to the launch of the Knowledge Centre to date has been strong both in terms of existing Members and general industry interest.

The Institute will continue to utilise its new website as a platform to develop future initiatives that meet membership needs and attract further organisational growth.

Institute members are encouraged to login and browse the website and educational programs, view your local Branch and various committee activities and releases, provide feedback to committee’s where you feel it appropriate and make use of the Institute’s Resource Centre – that contains a wide range of technical material that will continually be expanded.

The Institute is currently in the process of developing some of these future initiatives. Feedback from the recently released membership survey will further assist in providing direction and priority for such initiatives. Such initiatives will include services such as:

- Increased technical material on the CIA library, contained within “Proceedings and Others”.
- Online “Mentoring Service” for Individual Young Members.
- Online “Media Library” contained within the Resource Centre.
- An area specific to research and development commercialisation opportunities.
- Increase in organisations listed on the Institute’s search portal.
- An online space for increased interaction with Corporate Members wishing to promote technical content of their product and services.
- A more user friendly online system for election of members to serve on National Council.
- Social networking ability through a medium that is most popular with the current membership.

A key aspect to the Knowledge Centre has been the ability to greater engage with younger members of the Institute and offer services, such as potential career placements, that attract student members who will help sustain the Concrete Institute of Australia over the longer term.

The new website also provides greater prominence for Concrete In Australia. Authors are now able to submit papers online, find clear instructions on submission details and information relating to the upcoming feature topics.

Members wishing to submit papers, project pieces or other information are encouraged to view the Concrete In Australia section via: http://www.concreteinstitute.com.au/Concrete-in-Australia.aspx.
What you CE is what you get!

BOSFA have introduced CE certification for the benefit of engineers and others interested in the use and specification of SFRC. Our aim is to provide a technically sound approach to the determination of SFRC performance.

EN14889-1 is currently the only performance based manufacturing standard for steel fibres, which means it requires manufacturers to declare the fibre dosage required to meet a standard performance level. Every pallet of product supplied to market has a CE label that details the manufacturing facility, fibre tensile strength, geometry and importantly this minimum dosage.

Proprietary design programs are often used by fibre suppliers with in some cases unverified design properties; the declared minimum dosage enables the engineer at design stage to quickly and easily compare the expected performance of different fibre types being offered and hence the reliability of the proposed designs.

HOW TO SPECIFY:
- Steel fibres must conform to EN14889-1, system 1, for structural use.
- A copy of the CE label and Certificate of Conformity to be supplied to the project engineer and ready-mix plant.

THIS PROVIDES:
- Engineers an independent means to compare expected fibre performance.
- Ready-mix plants the ability to introduce QC checks that the correct fibre is batched.
- Contractors the confidence that what’s been specified is supplied to site.

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Updates in concrete box culvert standards

By David Millar – executive director, Concrete Pipe Association of Australasia

The Australian Standard for small reinforced concrete box culverts AS 1597.1 was revised in 2010 for the first time since 1974.

The revision of the standard means that there are a number of significant changes and additions that will affect manufacturers, designers, specifiers and contractors.

One of the more significant changes to the new standard is to do with sizes. Table 2.5 of AS 1597.1 – 2010 provides 10 preferred internal dimensions for small box culvert units ranging from size class 300 mm span x 225 mm height up to 1200 mm span x 1200 mm height. This is a significant change as currently the range of sizes being manufactured throughout Australia is unregulated and uncontrolled.

A nominated range of preferred internal dimensions will create efficiency in design and selection of culverts for specifiers, and also in the production and manufacture of units by suppliers.

To highlight the benefits of the more streamlined internal dimensions for small box culverts the Concrete Pipe Association of Australasia (CPAA) recently released a Technical Note titled “Update – small box culverts and AS 1597.1” which is available from their web site www.cpaa.asn.au.

This same approach is already routinely used for precast concrete pipes. Table 4.2 of AS/NZS 4058 “Precast concrete pipe (pressure and non-pressure)” provides a good example of the efficiency and practicality of having nominated size classes for a concrete element that is subjected to hydraulic and structural design.

As a result, specifiers and manufacturers can expect a similar benefit with respect to efficiency in design, selection, production and installation of small box culvert units.

Other significant changes to the standard that are worth noting include:

- Performance test loads for small box culverts have increased. The Proving Load (previously known as proof load) has been increased from 90 kN to 112 kN and the Ultimate Load has increased from 135 kN to 202 kN.
- Concrete durability requirements have been updated to align with those found in AS 5100 and to reflect current design specifications. This includes:
  - Specification of durable concrete materials (e.g. aggregate durability, restriction on chemical content, use of blended cement).
  - Exposure classifications, concrete strength and cover to reinforcement.
  - Minimum curing requirements for various methods (e.g. time, maturity, concrete strength) have been updated to reflect current practice and requirements.

Further information on the changes to the standard can also be found in the CPAA Technical Note “Update – small box culverts and AS 1597.1”.

The latest version of AS 1597.1 – 2010 “Precast reinforced concrete box culverts – small” is now available.

However, with the introduction of any new standard a reasonable period of time is required to phase out previous manufacture and specification methods. The CPAA is urging specifiers, designers and manufacturers to work together to ensure a smooth transition to the latest requirements.
PURPOSE
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 12 to 14 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.
New Constitution for Concrete Institute of Australia

Graeme Burns, Chief Executive Officer

The membership of the Concrete Institute of Australia (the Institute) has strongly voted, in General Meeting, for the adoption of a new Constitution, with 87% of those voting in person or by proxy agreeing to the need for the proposed initiatives. Voting numbers were both representative and high – with over 20% of all eligible members electing to vote.

The new initiatives include:

- Bringing the Constitution in line with current Corporations law.
- The introduction of a new role on Council to represent the Institute on initiatives for younger members. This strategy is in line with the overall direction of developments for electronic communications and the Institute’s Knowledge Centre, which are focused on attracting a younger audience to the Institute’s membership.
- The ability to formally communicate with members electronically on Constitutional matters. This has allowed for the current Annual General Meeting to be efficiently called through electronic means, saving thousands of dollars in printing, processing and postage.
- Streamlining of day to day operations, by formal recognition of the Executive as the Board of Directors. This will further allow the Council to focus on the core responsibilities for determination and approval of the Institute’s strategic direction, policy determination and the oversight of operations of the company.
- Provision for flexibility of State and National operations to be governed by the Council, removing Constitutional restrictions; and
- The removal of automatic appointments to Council.

Finally, whilst it is recognised that positive support was received for the new Constitution it is also recognised that with a large base of members drawn from many segments of the industry not all members were in support of certain initiatives. The Institute would welcome feedback from members of any improvements to the Institute’s governance as we move forward with the new Constitution.

We look forward to the opportunities that the new Constitution provides for better servicing of members and more relevant governance provisions.

Conference specialising in concrete pavements

The Australian Society for Concrete Pavements (ASCP) will hold its first Concrete Pavements Conference in Sydney on Tuesday 2 August 2011.

ASCP national president Mark Hoskins said: “This conference represents a significant development for ASCP which commenced operations only just over three years ago.

In that time, ASCP has conducted at least four forums each year, facilitated specification reviews for both road and industrial pavements, established a branch in Queensland, and engaged in matters of training for the concrete road pavement industry. In 2011 we plan to establish a branch in Victoria and to be able to offer training for operatives in concrete road paving.”

The conference will focus on concrete road pavements, and will be addressed by many of the directors of the International Society for Concrete Pavements (ISCP) who are experts in the field. The conference topics are grouped into four categories, namely – Pavement Design, Pavement Materials, Sustainability and Pavement Performance.

Hoskins said: “This conference provides a unique opportunity for Australian concrete road pavement practitioners to receive current information from around the world at the one event.

We are also delighted to host the ISCP Board meeting in Australia and for the directors to be available to address our members.”

The ISCP speakers come from the US, South Africa, Chile and Belgium. The conference sessions will be followed by a dinner which will be addressed by the ISCP president Mark Snyder as well as EuPave representative Luc Rens who will provide details of Concrete Road Pavement activities from Europe.

Snyder said: “ISCP is delighted to accept the invitation from ASCP to come to Australia for its Board meeting and their inaugural Conference. This is the first time the ISCP Board has met in the Pacific region. Since ASCP commenced in 2008 we have enjoyed close relations with them, culminating in a Memorandum of Understanding between ISCP and ASCP in August 2009.”

Registrations for the Conference are now open, with substantial early registration fee discounts being offered. All details can be found on the ASCP web site – www.concretepavements.com.au. Opportunities are available for consultants, contractors, materials suppliers and equipment suppliers to sponsor and exhibit.

For more information contact Australian Society for Concrete Pavements executive director Kevin Abrams on 02 9918 2610 or by email: exec@concretepavements.com.au.
Concrete 2011 – Ensure you are part of it!

The Concrete Institute of Australia (the Institute) is pleased to inform industry that registrations are now open for the Concrete Institute of Australia’s 25th Biennial Conference, Concrete 2011, which will be held from 12-14 October, 2011, at the Burswood Entertainment Complex, Perth, Western Australia.

The Technical Program will be highlighted by an Invited Speaker plenary session on each conference day. The three speakers are:

- **Professor Ravindra Gettu, Indian Institute of Technology, Madras, India.** Professor Gettu is a widely recognised authority on concrete as a material, with interests in a variety of concrete performance characteristics, and also in aspects of sustainability.

- **Linda Figg, CEO, Figg Engineering Group, Tallahassee, Florida, US.** The Figg Group is a relatively small but highly influential consulting firm specialising in bridge engineering, with a portfolio of inspirational structures which have realised the company’s philosophy of “creating bridges as art”.

- **Martin Clarke, CEO, British Precast, Leicester, UK.** British Precast is a vibrant organisation which covers all aspects of precast, and in particular is very focused on sustainability issues and the role precast concrete will play in the future. Clarke will bring a British and European perspective to the conference.

Associated social events will culminate in our Gala Dinner on Friday 14 October. The Concrete Institute Gala Dinner in Perth has become a very popular annual event, and this year we plan to incorporate it into the Biennial Conference to make for a really special night, which will feature the presentation of the Excellence Awards.

We invite you to consider registering to attend the conference.

Early Bird Registrations – a saving of $200 per full registration – close on 29 July.

We would also appreciate you circulating this information to work colleagues.


For all information regarding the conference, including a download of the registration brochure, members are advised to visit the conference website: [www.concrete2011.com.au](http://www.concrete2011.com.au).

We look forward to your participation in what we are sure will be a popular and rewarding conference, whether as a sponsor, exhibitor, participant or attendee – and we especially look forward to welcoming you to Perth.

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Recognition of excellence

With 2011 being a year of the Biennial Conference, the Concrete Institute of Australia (the Institute), through its Recognition of Excellence Committee, is again excited about the opportunity to recognise areas of excellence within the concrete industry.

The Recognition of Excellence Committee is responsible for the identification of opportunities to openly recognise the achievements and contributions of members to the Institute and industry at large. Three key programs and areas of responsibility of the committee are:

- Biennial Awards for Excellence.
- National Engineering Bursary.
- Life and Honorary Membership.

The Institute is in the midst of ensuring a high level of recognition is provided in all of the above three areas.

2011 Awards for Excellence

The Institute is very pleased to announce to members that there has been an increased number of submissions for its 2011 Awards for Excellence program, and indeed the highest number of submissions of any prior Biennial year. To ensure that each submission receives the amount of recognition that is deserved, the Institute is judging entries on a state basis, with State Awards for Excellence being awarded and showcased during State events in September 2011.

Winners of state Awards for Excellence will be judged for National Awards which will be presented at the Institute’s Awards Dinner at its Biennial Conference in Perth on 14 October 2011.

The Institute encourages all members to remain informed of state-based awards events that will be promoted throughout the year via the respective state branch website.


National Engineering Bursary

The judging panel for the National Engineering Bursary has recently made its recommendation on the successful recipient for the 2011 award. The award is made for excellence in thesis work on concrete and cement based products and processes.

The Institute thanks all those who submitted their thesis for the 2011 National Bursary and encourages all members engaged in PhD work to view the Bursary requirements to see if they may be eligible to make a submission for the 2013 Bursary – that will be open from November 2011.


The recipient of the 2011 Bursary will be announced during the Awards Ceremony at Concrete 2011.

Life and Honorary Membership

The Council of the Institute makes Awards for Life and Honorary membership to recognise people who have made significant contributions.

These awards carry significant prestige. The Nominating Committee is now in the process of reviewing all nominations before making its recommendations to council.

The achievements and contributions which all nominees have made to the industry are extremely impressive and the Institute is honoured to be able to bestow such prestigious awards on truly deserving recipients.


New venue for International Symposium

The International Symposium on High Performance Concrete and the NZ Concrete Industry Conference which were to be held in Christchurch will now be in Rotorua.

Both events will be held at the Energy Event Centres on the same dates as originally scheduled – Concrete Conference – 8 August; Symposium – 9-11 August.

An outstanding response to the call for papers – more than 250 abstracts from 44 countries – augurs well for attendance numbers at what will be the largest international conference ever hosted by the New Zealand Concrete Society. The 9th International Symposium on High Performance Concrete – Design, Verification and Utilisation is expected to attract up to 300 international delegates and will feature six keynote speakers and 11 invited speakers.

It will be preceded by a one-day NZ Concrete Conference.

Conference Organising Committee chairman Michael Khrapko said the conference gives New Zealanders in the concrete industry a unique opportunity to meet leading world experts in the field of high-performance concrete and to learn where concrete technology is heading.

The keynote speakers – all professors – and their areas of expertise are:

- Pierre-Claude Atiein, University of Sherbrooke, Canada – use of high and ultra-high performance concretes and industrial byproducts in concrete
- Michael Collins, University of Toronto, Canada – improving design for reinforced and prestressed concrete structures
• Mitsutaka Hayakawa, Tokyo Polytechnic University, Japan – mixing method to obtain high-quality concrete; building materials and construction methods
• Olafur Wallevik, Innovation Center Iceland, Reykjavik University – made first BML Viscometer, developed other measuring instruments
• Joost Walraven, TU Delft, Holland – new types of high-strength, self-compacting, high-performance fibre-reinforced concrete, former FIB president
• Francis Young, University of Illinois, Urbana-Champaign – inorganic chemistry, advanced cement-based materials.

Symposium delegates will have the option of attending Concrete Conference 2011.

The symposium is to be hosted by NZCS with support from:
• FIB (Federation Internationale du Béton)
• RILEM
• JCI (Japan Concrete Institute)
• ACI (American Concrete Institute)
• JSCE (Japan Society of Civil Engineers), and
• Concrete Institute of Australia.

Joint membership initiative

Progress has been made on the development of a Joint Membership opportunity between the Concrete Institute of Australia (CIA) and the American Concrete Institute (ACI). An initial proposal was made by the CIA around 12 months ago. The proposal has been favourably dealt with by the ACI’s Membership Committee and has received enthusiastic support from the ACI’s Board. The CIA’s CEO and Secretary/Treasurer recently met with the Executive team of ACI in the US and reached agreement.

A draft Memorandum of Understanding has now been progressed and is awaiting finalisation.

The Joint Membership will be available through subscription to all Individual Members (Individual, Individual Young, Retired and Student Members) and will include full access to the ACI’s online services and Knowledge Centre.

Full details will be provided when the agreement has been finalised.

This initiative will provide a significant additional service for CIA members – and is directly correlated with the strategic objectives of raising the CIA’s positioning and profile and strengthening relationships.
Standards
Recent activities of the Standards Committees with which the Concrete Institute of Australia (the Institute) has an active involvement, include:

BD-002: Concrete Structures
There was a general agreement that within two years of the publication of AS 3972-2010 General purpose and blended cements, the allowable proportions of mineral additions within general purpose cement would be reviewed to allow for the optimum level proven by additional testing.

Standards Australia wrote to organisations represented on the BD-010 Committee inviting them to take part in a working group to continue the discussion and reach agreement on the optimum amount of mineral additions in general purpose cement.

The Institute looks forward to being able to contribute to the working group and participate in continuing discussions, designing and reviewing results of testing programs established.

BD-066: Tiltup Construction
A meeting of BD-066 was held during April 2011, whereby the committee was in the initial stages of reviewing and updating key sections of AS 3850. The Institute’s reference group of BD-066 provided the nominated representative Simon Hughes with comments that were able to be brought into discussion at the meeting. The Institute will continue to have significant input into the revision of the standard and looks forward to keeping members informed of key progress.

BD-010 Working Group:
A BD-010 Working Group, a group whereby Nominating Organisations of BD-010 are continuing the discussion and

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implementing a research/testing program to reach agreement on the optimum amount of mineral additions in general purpose cement, met for the second time during March 2011. Institute representative and University of Technology Sydney professor Abhi Ray, with significant experience with research and testing programs, is providing input into the group meetings on behalf of the Institute. Currently the Stage 1 cement testing program is well under way and a second stage tentative concrete testing program has been developed.

**BD-090: Bridge Design**

Following a call for nominations to the Institute’s member base, the Institute’s Council has approved and appointed Linda Lee, from GHD, as the Institute’s nominated representative. After due consideration of her technical qualifications Lee was appointed to serve on the committee and be prepared to independently represent the views of the Institute and abide by the Governance Guidelines for representation.

**Education**

**Reinforcement detailing – Getting it right**

The national seminar series on *Detailing – Getting it right*, developed and delivered by David Beal, John Woodside and Steve Freeman and presented to the eastern states throughout March 2011 was a major success. Attendees at the seminar were provided with extremely comprehensive course materials that are able to serve as reference material suitable for practising professionals in the concrete industry. At the time of writing the seminar series was soon to travel to Perth and Adelaide, where again registrations have reached capacity. The Institute would also like to thank Scott Munter, executive director of the Steel Reinforcement Institute of Australia for providing chairmanship at each of the seminars.

The seminar reached capacity at all locations proving the popularity and emphasising the importance and current knowledge gaps of the topic. An extremely pleasing aspect of the registrants to date has been the large number of younger engineers and the broad cross section of all sectors involved within the supply chain. The Institute is proud to be able to engage all sectors of industry, disseminate valuable information to younger less experienced engineers, highlight knowledge gaps and add to the collective industry knowledge base – helping to reduce risk and liability, increase project efficiencies and assist correct professional practices occurring.

A survey was issued to registrants post seminar to obtain feedback, predominantly on:

- Comprehensiveness of the technical material provided.
- Improvement in awareness and understanding.

**Fig 1.** For the technical material provided, please indicate the comprehensiveness of the information provided. (5 - Highly comprehensive, 0 - Not comprehensive at all)

**Fig 2.** Did the seminar improve your awareness and understanding of shotcrete? (5 - Highly improved awareness and understanding, 0 - Did not improve)

**Fig 3.** Did the seminar provide information that you will be able to apply in practice? (5 - Highly comprehensive, 0 - Not comprehensive at all)
The ability to apply what they have learnt.

Overall the feedback of the survey was very positive. As always, feedback from the survey has provided valuable comments that the Institute takes on board to continually improve the educational seminars that are delivered.

Responses to a selection of the survey questions asked are indicated on the previous page.

**Shotcrete Essentials: What you need to know**

At the time of writing the national seminar series *Shotcrete Essentials – What you need to know*, had just concluded its final seminar of the series. The Institute is again pleased to advise that the seminar reached a broad spectrum of industry sectors. The presenters wealth of experience and knowledge within their respective fields provided for informative and practical information that can be directly applied in practice.

Key feedback from registrants has been praise for the degree of the presenters’ respective knowledge together with feedback stating that the program structure provided very good coverage of all aspects associated with shotcrete.

The Institute is very pleased that the outcome of these seminar series will provide all parties involved with projects involving the use of shotcrete with a broad overview of associated issues. The issues are relevant through both contracting and design lenses.

The seminar also provided information relating to appropriate use, latest technology to improve the material properties and applications and the latest in the way of research and appropriate testing.

Registrant feedback in relation to the course meeting their expectations, comprehensiveness, practical application and increased understanding and awareness again yielded very pleasing results as indicated below.
High Performance Concrete: Design and Applications – State of the Art and Current Practice

The Institute will soon commence promotion of the third National Seminar for 2011 which will be delivered during late August and features as a news piece in this issue of the journal.

AS 3600 Design Guide; delivered during November 2011

The Institute will deliver a one day course to designers in industry that will cover comprehensive design examples in accordance to AS 3600 – 2009. Professor Ian Gilbert will be working on the Course with other industry leaders including the Institute's Nominated Representative on BD-002, Gil Brock, and Professor Stephen Foster.

Publications

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

Z 15 – Cracking in Concrete Slabs on Ground and Pavements

The Institute has just finished the first print run of Cracking in Concrete Slabs on Ground and Pavements. Institute Members can download a pdf copy of the publication via the Institute's Resource Centre on its website. Institute members are referred to the news item on this publication in this issue of Concrete In Australia.

Geopolymer – Recommended Practice

The Institute's committee developing a Recommended Practice on Geopolymer Concrete is in the final stages of review, prior to releasing the draft publication to members for peer review. It is anticipated that the peer review will commence during June 2011 and the Institute encourages all members interested in the topic to provide review comments to the committee.

Institute's Membership Questionnaire Feedback

The Institute has recently released to membership a questionnaire, as part of a broader Membership campaign, aimed at collecting usable data that will assist with the direction of future initiatives. The campaign utilised a multi-channel approach making use of both direct and electronic direct mail – with a pleasing response rate to date in excess of 20%. This is not only exceptionally high of typical campaigns, but more importantly will help ensure that the direction of Institute activities is truly reflective member needs.

The campaign has been further developed as a segmented approach – whereby members of the Institute have been asked to provide responses based upon their current demographic and level of participation within the Institute.

Two particular segments have included student and young individual members, whereby the Institute is keenly interested in providing services to the younger market that will help sustain the organisation over the longer term. The questionnaires were designed to be brief and ensure that relevant key points are extracted and directive information attained.

At the conclusion of the campaign, a detailed report, providing analysis and future directional plans will be made available to members.

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

Reinforcement Detailing Handbook

This is the fourth edition of the “Reinforced Concrete Detailing Manual”, updated and republished in 2010. Detailing of reinforcement is the interface between the actual design of the concrete structure and what is to be constructed. 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.

Shotcreting in Australia

This document has been written as a guide to the use of shotcrete in Australia. It is based on established practice within the Australian context and is targeted toward designers, specifiers, owners, suppliers, contractors and other end users of shotcrete. This guide is the second edition of this document, updated and prepared by the Australian Shotcrete Society, a special interest group within the Australasian Tunneling Society.

To purchase copies of these manuals please contact the Concrete Institute of Australia National Office on phone 02 9736 2955 or email admin@concreteinstitute.com.au

www.concreteinstitute.com.au e x c e l l e n c e i n c o n c r e t e
National Seminar Series: High Performance Concrete: Design and Applications

The third program for 2011 in the Institute’s National Seminar Series will be a one day course on High Performance Concrete: Design and Applications – State of the Art and Current Practice scheduled to be delivered across Australia throughout August 2011.

A leading professional on the topic, Professor Priyan Mendis, from Melbourne University has developed and will deliver this one-day program. Mendis’ vast theoretical knowledge combined with his many years of advanced practical application, on some of the most interesting global building projects, will provide for a very informative set of lectures together with an extremely comprehensive set of course materials.

This course is predominantly targeted towards structural engineers of all age demographics and levels of experience. The course program covers the full gamut of necessary practical design considerations together with knowledge that will be able to be immediately applied to professional practice.

This learning is essential for all designers – as it covers all major relevant aspects to an important and rapidly growing area of concrete design – the next generation of concrete. The program has been structured in a manner to ensure maximum benefit to attendees; not only covering code provisions but also detailing alternative design approaches, design and limit state criteria, lessons to learn from typical problems and common mistakes.

Other key aspects of the course include information relating to; critical actions, design of tall buildings, sustainability requirements and future developments – both locally and globally. Full program details and registration can be found on the Institute’s website http://www.concreteinstitute.com.au/Events.aspx.

Together with Engineers Australia – our supporting partner, the Institute aims to deliver a thorough program that addresses all the necessary considerations regarding high performance concrete. Members of Engineers Australia attend the seminars at the discount member rates.

The National Seminar Series: High Performance Concrete: Design and Applications – State of the Art and Current Practice will include a detailed overview of:

- High Performance Concrete.
- What AS 3600 covers and what it does not cover.
- HPC in other standards:
  - Bridge design code;
  - Piling code;
  - Maritime structures code;
  - Precast concrete codes;
  - Major International developments;
  - Other.
- Compressive strength and other performance and design criteria.
- Applications – examples including buildings, pavements and precast applications.

It will also feature a session providing information relating to columns, walls, axial shortening in tall buildings, the role of shrinkage and creep effects and transmission of axial loads through slabs.

New limit states in construction will be discussed, providing a practical overview of the need for speed in construction, design implications, early age effects, and sustainability issues.

A session on tall buildings and other major infrastructure will provide detail on designing for critical actions such as wind, fire and earthquake. Also, designing for blast and impact loading and common problems and mistakes will be discussed.

A workshop session will include case studies and interactive dialogue with delegates, providing practical guidance while also detailing developments on the next generation of concrete.
Adverse effects in high strength concrete when exposed to fire

Jay Sanjayan – Professor of Concrete Structures, Centre for Sustainable Infrastructure, Swinburne University of Technology, Hawthorn Campus, Victoria

**Summary:** High strength concrete is often chosen when high strength, low permeability and/or high elastic modulus are required. However, it is less known that high strength concrete suffers high risk of spalling in a fire. Spalling of concrete is loss of concrete section due to concrete falling off the fire exposed surface. Risk can be minimised by optimisation of concrete mixture and testing samples of concrete. Certain aggregates and silica fumes are known to exacerbate the spalling, while polypropylene at certain dosages is effective in reducing spalling. There are three known failure modes by which spalling of concrete can occur in fire:

1. Moisture clog spalling due to the steam pressure build up in the pore system of concrete;
2. Spalling due to restrained thermal dilatation; and
3. Thermal incompatibilities between cement paste and aggregates.

Spalling occurs at the early stages of the fire (about 30 minutes into the fire). Therefore, it has been known to hamper rescue efforts. In the first 30 minutes, temperature in hydrocarbon fire rises at much higher rate than the standard fire; therefore, causes higher steam pressures and thermal gradients with increased spalling risk. The Australian Concrete Structures code does not provide any guidance on spalling of concrete. Hence, Australian engineers normally resort to Eurocode for guidance. However, Eurocode design provisions for spalling are not applicable for hydrocarbon fire exposures or unreinforced concrete.

There are simple test methods developed and presented in the literature for spalling. The tests are designed to expose the specimens to rapid temperature rise similar to standard or hydrocarbon fires. Since the risk of spalling is highly influenced by the type of concrete and aggregates used, these tests, if performed as part of the mixture trials, will help identify concretes which are not suitable for this type of construction. Failure of concrete slabs may occur due to thermal warping of the concrete due to large thermal gradients. The failure may occur by two modes:

(a) Buckling of the slabs, particularly in longitudinal direction, due to large restraining compressive forces. This occurs when the slab exposed to fire expands while the remaining lengths of the slab in lower temperatures provide restraint.

(b) If buckling does not occur, continued exposure to fire can cause large deflections. These large deflections cause the compressive forces to subside, however tensile membrane forces develop. Concrete suffers chemical and physical damage at high temperature, with permanent loss of strength and stiffness. For example, a 60% loss occurs after exposure to 600°C temperature. This permanently reduces the load carrying capacity of the structural members.

**1.0 SPALLING OF HIGH STRENGTH CONCRETE IN FIRE**

**1.1 Introduction**

Generally, concrete is regarded as a fire resistant construction material, especially when compared to the alternatives such as steel and timber. However, high strength concrete is susceptible to a phenomenon termed *spalling* in fire (Crozier and Sanjayan, 1999a). Spalling of concrete in fire is dislodgement of small pieces of concrete (~50 mm) popping out from the surface of the concrete, sometimes explosive in nature (Figure 1). It is possible that the entire concrete can be lost layer by layer due to the spalling process. For example, on 18 November 1996, a fire on a shuttle transporting trucks destroyed parts of the south tunnel of the railroad tunnel connecting England with France (the "Chunnel"). The fire caused severe damage to concrete tunnel rings owing to the spalling of concrete. Figure 2 shows a typical damage pattern in reinforced concrete tunnel rings as observed after the Chunnel Fire (Ulm et al, 1999b). The figure illustrates the role of reinforcement in preventing complete collapse of the tunnel ring.

A fire in the Great Belt tunnel in Denmark in 1995 also caused severe spalling of concrete tunnel rings (Hertz, 2003). Spalling occurs between 15 to 30 minutes after the commencement of fire (Sanjayan and Stocks, 1993; Crozier and Sanjayan, 2000) – a critical period for fire control and escape. Of the 18 columns tested by Ali et al (2001), most of them

![Figure 1. Spalling of concrete tunnel lining in fire.](image)
explosively spalled, and only one column did not exhibit spalling. Figure 3 shows a high strength column after a fire test. Figure 4 shows fire induced damage due to a tanker-truck crash at the highway bridge at the I-65/I-59 interchange in Birmingham, Alabama (Yanko 2004). Judging from the condition of the steel beams, it is clear the fire temperatures did not reach the very high values anticipated in a hydrocarbon fire, possibly due to the open nature of the environment. However, spalling of concrete in columns is evident; the spalling has predominantly occurred in the cover concrete.

1.2 Failure modes of spalling

Spalling is widely believed to be caused by the steam pressure build-up in the pores of concrete in fire, termed moisture clog spalling, first proposed by Shorter and Harmathy (1961). However, a number of failure modes have been identified by researchers to fully describe the observed spalling in fire. They are classified into three as follows:

- Spalling due to restrained thermal dilatation: described by Bazant (1997) and adopted by Ulm et al (1999a) and Nechnech et al (2002).

1.2.1 Failure mode 1: Moisture clog spalling

Moisture clog spalling (Shorter and Harmathy, 1961) occurs due to steam pressure build-up in the pores of the concrete. The moisture residing in small pores of concrete expands as steam when heated, building up pressure in the pore matrix, which takes time to be released through the pores of concrete. In a fire with rapid temperature rise (e.g. a hydrocarbon fire), the pressure may not have enough time to be released, and when the bursting pressure exceeds the tensile strength of concrete, the spalling (sometimes explosive) occurs. Figure 5 shows the steam pressure profile in concrete exposed to fire. The peak pressure occurs at a distance from the surface, where the fracture is likely to occur.
1.2.2 Failure mode 2: Spalling due to restrained thermal dilatation

Spalling due to restrained thermal dilatation was identified by Bazant (1997) and later adopted by Ulm et al (1999a) and Nechnech et al (2002). This failure mode considers that the spalling results from restrained thermal dilatation close to the heated surface, which leads to compressive stresses parallel to the heated surface.

These compressive stresses are released by brittle fracture of concrete, i.e., spalling. Due to the volume expansion of a growing crack, and the slowness of release of additional water into the crack, the pressure in the crack must rapidly decay after the crack begins to open. As a result, the pore pressure can play only a secondary role as far as the growth of a larger crack is concerned. The pore pressure may affect the onset of instability in the form of explosive thermal spalling (Figure 6). Figure 7 shows a slab after a fire test exposing only the middle region of the slab to fire, and illustrates this failure mode (Hertz, 2003).

1.2.3 Failure mode 3: Thermal incompatibilities between cement paste and aggregates

When subjected to increasing temperature, the cement paste initially expands and when it is heated beyond about 300°C, it starts to rapidly contract. Figure 8 shows the behaviour of four different cement pastes reported in the literature. Due to thermal gradients in concrete, parts which are still under 300°C would be experiencing expansion while the other parts which are more than 300°C would be experiencing contraction. This competition between simultaneous expansion and contraction damages the concrete matrix. This behaviour depends on the type of cement binders used. With increasing temperatures, most types of aggregates undergo expansion. Aggregates typically occupy about 60
to 80% of the total volume of concrete. Therefore, they have a very important effect on changes in volume of the concrete exposed to elevated temperatures. The thermal incompatibilities arising from expanding aggregates and contracting cement pastes can lead to spalling of concrete (Figure 9). This failure mode is believed to be responsible for explosive spalling observed in many dry laboratory specimens.

The types of aggregates and their corresponding expansions are shown in Figure 10.

The most important factor affecting the expansion of aggregates is the mineralogical composition. For instance, the thermal expansion of aggregates containing quartz (SiO₂) is affected by polymorphic inversion of quartz, which occurs around 570°C. This causes a significant increase in expansion resulting in spalling of concrete. Generally, aggregates containing high silica contents are vulnerable to high expansions. Aggregates with low expansions have low spalling risk in concrete. The thermal characteristics and mineralogical compositions of the aggregates are important characteristics when assessing spalling risk.

### 1.3 Factors affecting spalling

#### 1.3.1 Rate of temperature rise

Risk of spalling is significantly raised when the rate of temperature rise is rapid (Copier, 1979). All the test results reported in the literature on concrete spalling were carried out to Standard Fire (or cellulotic fire) specified by national standards (e.g. AS 1530 and ISO 834). The rate of temperature rise of this fire is low, e.g. 349°C in the first 1 minute. In a hydrocarbon fire, the rate is twice this, e.g. 743°C in the first 1 minute. Figure 11 shows the nominal temperature versus time curves for standard and hydrocarbon fires recommended in Eurocode 1 (Part 1-2, 2001). The thermal gradients and steam pressures generated in a hydrocarbon fire would be significantly higher than in standard fire conditions.

#### 1.3.2 Strength of concrete

High strength concrete (strength > 50 MPa) is particularly vulnerable to spalling in fire because of its low permeability as compared to low strength concrete. Therefore, high strength concretes are more vulnerable to steam pressure build up (Failure Mode 1). Further, high strength concretes are stiffer and more brittle than normal strength concretes. Therefore, high strength concretes are less able to accommodate the thermal incompatibilities described in Failure Modes 2 and 3 leading to a higher risk of spalling.

It is very common in concrete constructions where the actual strength used is far higher than the design strength due to a variety of reasons, mostly to achieve early strengths to speed up the construction time. It would not be uncommon to find 70 MPa to 80 MPa concrete in a structure where the characteristic design strength is 40 MPa. Therefore, for spalling considerations, the mean strength of the actual concrete mixes rather than characteristic strength should be used.

#### 1.3.3 Moisture content of concrete

The higher the moisture condition of the concrete, the greater is the risk of spalling. High strength concretes tend to retain moisture more than normal concretes due to their low permeability. Based on the test data available in the literature, it can be concluded that if the moisture content of concrete at the time of fire is greater than 2.5% (by mass), the risk of spalling is significantly raised (CIRIA, 1987). Concrete at the time of completion of construction is likely to contain about 6% moisture by mass. When exposed to a typical indoor environment, concrete slowly dries with time and can reach an equilibrium moisture content of about 2.5% after about five years of construction (Selih, 1996). The equilibrium moisture contents of concrete inside a tunnel, and the length of time it would take to reach this level are not known.

#### 1.3.4 Compressive stresses

Concretes under compressive stresses are more prone to spalling than ones that are previously cracked from flexural stresses (Crozier and Sanjayan, 2000). The restrained concrete slabs can develop compressive stresses which can increase the
risk of spalling. Further, high strength concretes generally are subjected to higher compressive stresses.

1.3.5 Type of cement used for concrete

The Australian Standard permits Portland cements (e.g. Type GP) to contain up to 5% mineral additions, such as limestone. Cements containing more than 5% supplementary cementitious materials, such as slag or fly ash are classified as blended cements (Type GB).

It is widely accepted in the literature that cements containing silica fume (a mineral addition to cement) have an increased risk of spalling (Sanjayan and Stocks, 1993). Eurocode 2 (6.2(1)) specifies that any concrete containing more than 6% silica fume in cement requires one of the four additional measures prescribed in Section 6.2, Item (2). However, these additional measures are based on the assumption that the fire exposure is not hydrocarbon fire and the concrete contains reinforcement with suitably small cover.

The reason silica fume has adverse consequences in fire is because it significantly reduces the permeability of concrete and also increases the brittleness. Both are not helpful in fire.

1.3.6 Reinforcement in concrete

Reinforcement in concrete has a significant role in limiting the spalling of concrete to the cover regions. Typically, cover to reinforcement of less than 50 mm is recommended to reduce the risk of spalling. Reinforcement carries the tensile stresses induced by the thermal incompatibilities described in Failure modes 2 and 3.

1.3.7 Use of polypropylene fibres

Studies (Breitenbucker, 1996; Bentz, 2000) have shown that the use of polypropylene fibres in the concrete mix reduces the tendency for spalling. This is because the polypropylene fibres melt during the fire, thus increasing the internal voids of the concrete and decreasing the vapour pressure build-up within the concrete.

2.0 Global member failure in fire

Large inward deflections of the concrete slabs may be induced by thermal warping. Thermal warping is due to the curvature in the concrete slab due to thermal gradients. In a fire, the underside of the concrete slab will experience high temperatures while the top layers will remain at lower temperatures. Thermal warping of a flat slab supported by columns exposed to a fire from below is shown in Figure 12 (Moss et al, 2006) and Figure 13 (Bailey, 2004). Figure 12 is a result of a numerical simulation using a special purpose computer program SAFIR (Franssen et al. 2002) for a standard fire exposure. An experimental verification of this phenomenon is shown in Figure 14. These deflections and thermal gradients would be larger in a hydrocarbon fire where the rate of temperature rise of the fire is higher compared to standard fire.

Figure 15 illustrates a possible longitudinal warping mode of concrete slab exposed to fire. During the early stages of the fire (about 20 to 30 minutes after commencement), large compressive forces are likely to develop due to restraining from the unheated lengths of the tunnel, which remain at lower temperatures outside the fire region.
The compressive forces combined with warping deflections may cause $P-\Delta$ effects large enough to cause buckling failure of the concrete slab. If this critical phase is passed and buckling failure is avoided, then the compressive forces will subside but further increases in warping deflections will develop. The concrete slab is then likely to develop tensile membrane action at later stages of the fire, as found in concrete floors exposed to fire (Figures 12 and 13).

### 3.0 Residual capacity after fire

Due to chemical and physical damage of concrete at high temperatures, concrete suffers a loss of strength and stiffness. A guideline for loss of strength at high temperatures provided by AS 3600 (2001) Concrete Structures Standard is shown in Figure 16. This figure along with other figures on elastic modulus were removed in the AS 3600 (2009) since the modern concretes are no longer comprised of cement, aggregates and water but variety of supplementary cementitious materials and admixtures that it is very difficult to have a general chart that represent all concretes. Further, the validity of these figures for high strength concretes is uncertain. Also shown in the same figure are the test results of residual strength of concrete after exposure to elevated temperatures, obtained from Crozier et al. (1998) for high strength concretes. As can be seen, the damage to concrete at elevated temperatures is permanent.

After a fire, the concrete members will have loss of section due to spalling and loss of strength in the remaining concrete. Before returning to service of the structure, the load capacities should satisfy full load factors, rather than the reduced load factors applied during a fire. The residual load capacity of the structure may not be sufficient to carry the load with full load factors. In this case, a reconstruction or retrofitting of the structure should be considered.

The normal practice of repairing fire damaged concrete structures is to remove the visibly damaged portions and restore them with new concrete. However, little attention is given to the long-term performance of fire exposed concrete which is not removed from the structure. Mendes et al (2009, 2011) addressed this issue. Ordinary Portland cement (OPC) pastes, when exposed to a critical temperature of 400°C, undergo complete breakdown. This behaviour is attributed to the dehydration of Ca(OH)$_2$, followed by the expansive rehydration of CaO. This process is confirmed by partial replacement of the OPC binder with slag, which had a beneficial effect in the mechanical properties of the paste after exposure to high temperatures, as slag significantly reduces the amount of available Ca(OH)$_2$ in the cement paste. Mendes et al (2009, 2011) studied the long-term (after the exposure event) effect of CaO rehydration in the cement pastes.

After one year the ongoing effect of the CaO rehydration was severe in the Portland cement paste. The slag blended cements were not affected by rehydration.

### 4.0 TEST METHODS

As described in this report, the behaviour of concrete at high temperatures depends on a large number of factors such as types of aggregates, concrete mixture proportions and types of binder (cement) used. Two types of tests can be performed to assess the spalling risk:

- Relatively simple tests on small size specimens can be used to optimise the concrete mixture proportions;
More elaborate tests on larger scale test specimens can be performed to verify the spalling tendency of the concrete members a fire.

4.1 Small scale tests

4.1.1 Test using standard cylinder size specimens

Using standard cylinder size specimens of 150 mm diameter and 300 mm height, the tests can be performed to test the failure modes (1) and (3) (i.e. (1) moisture clog spalling and (3) thermal incompatibilities between cement paste and aggregates). The test involves conditioning the cylinder to the required moisture condition. Since the specimens are relatively small, accelerated controlled drying can be used to achieve the desired moisture content in a matter days. One face of the test specimen is then rapidly exposed to 800°C temperature. This is achieved by using a preheated oven to about 1000°C. This method provides a rapid temperature rise on one surface of the concrete similar to a hydrocarbon fire. The method of testing is illustrated in Figure 17. More details of the test method are outlined in a paper by Zhao and Sanjayan (2010). Since this test is relatively simple to perform, it can be used to optimise the concrete mixture parameters and polypropylene fibre dosages to reduce the spalling risk.

4.1.2 Test using small slabs

Similar to the concrete cylinder test described in the section above, concrete slabs can also be exposed to rapid temperature rise. In this test, a slab of the required thickness is made and conditioned to achieve the required moisture condition. Thicker slabs will require a longer time to reach the required moisture condition. The slab specimen is then exposed to 800°C by exposing to a preheated oven at 1000°C. In this test, the middle region of 150 mm diameter is exposed to high temperature while the rest of the slab remains at lower temperatures (Figure 7). This method is a test for failure mode (2) spalling due to restrained thermal dilatation.

5.0 CONCLUSION

There are three major design considerations in high strength concrete structural members as follows:

- **Spalling of concrete**: Spalling is the dislodgement of concrete pieces from the fire-exposed concrete surface. Spalling will cause loss of section. Excessive spalling may lead to structural failure.
- **Global member failure of concrete slabs**: The slab may buckle due to compressive forces generated by restraint to thermal expansion. Thermal warping in fire can cause excessive deflection which may lead to premature failure.
- **Residual capacity after fire**: The concrete members which is not spalled-off during a fire will sustain permanent damage. The residual strength of the concrete may not be sufficient to provide the required structural capacity for long term service. The assessment of the structure should also consider long-term progressive deterioration of the concrete after fire.

REFERENCES


Philleo, R., (1958) Some physical properties of concrete at high temperatures, Journal of the American Concrete Institute, 29/54(10), 857-64.


Compressive behaviour of FRP-confined high strength concrete

Togay Ozbakkaloglu, Thomas Vincent
School of Civil, Environmental and Mining Engineering, The University of Adelaide, Australia

1.0 INTRODUCTION
The use of high strength concrete (HSC) in building and bridge construction has increased over the last two decades. HSC offers significantly better structural engineering properties compared with conventional normal strength concrete (NSC), and forms an attractive alternative to other construction materials.

On the other hand, the use of HSC in seismically active regions poses a major concern because of the inherent brittle nature of the material. It is well established that lateral confinement of concrete with fibre reinforced polymer (FRP) composites can lead to a significant increase in both the compressive strength and ultimate strain of concrete. In recent years, a large number of studies have been conducted on the compressive behaviour of FRP-confined concrete. However, most of these studies have focused NSC with strengths lower than 60 MPa and the research on the compressive behaviour of FRP-confined HSC has so far been very limited with only a few tests reported in literature 1-5.

This paper presents the results of an experimental investigation on the behaviour of FRP-confined NSC and HSC under concentric compression. A group of FRP-confined concrete cylinders were tested to investigate the influence of the concrete strength and type of fibres used as confinement on the compressive behaviour of FRP-confined concrete.

2.0 EXPERIMENTAL PROGRAM

2.1 Test specimens
A total of 16 FRP-confined concrete cylinders with 152 mm diameter and 305 mm height were manufactured and tested under concentric compression. The test parameters included the concrete compressive strength (i.e. 30 MPa, 60 MPa and 90 MPa), type of fibres used to confine concrete (i.e. carbon and aramid fibres) and amount of confinement (i.e. one to six layers).

The cylinders were prepared using three different concrete mixtures with target compressive strengths of 30 MPa, 60 MPa and 90 MPa. Four of the cylinders were cast with 30 MPa concrete, another four with 60 MPa concrete and the remaining cylinders were cast with 90 MPa concrete. The 30 MPa and 60 MPa cylinders were confined by carbon FRP (CFRP), whereas the 90 MPa cylinders were confined by either CFRP or aramid FRP (AFRP). The properties of the unidirectional fibre sheets used to manufacture the confinement reinforcement are shown in Table 1. These fibre properties were supplied by the manufacturer.

2.2 Instrumentation and testing
Axial deformations of the specimens were measured with four linear variable displacement transducers (LVDTs), which were placed at 90° intervals around the specimen between the loading and supporting steel plates of the test machine. These deformation readings were used in the calculation of the average axial strains along the height of the columns. Prior to testing, all columns were capped at both ends to ensure uniform distribution of the applied axial load. The columns were tested under axial compression using a 5000 kN capacity universal testing machine. The test data were recorded using an automated data acquisition system.

3.0 EXPERIMENTAL RESULTS
Experimentally recorded stress-strain relationships of the test specimens are shown in Figure 1. The figure illustrates the influence of important confinement parameters on the compressive behaviour of FRP-confined concrete, which are discussed in the following sections.

3.1 Influence of concrete strength
In designing the FRP tubes, the performance criterion was chosen to be the normalised confinement pressure \( \left( \frac{f_l}{f'_c} \right) \), which is the ratio of the maximum confinement pressure \( f_l \) to the unconfined concrete strength \( f'_c \). Assuming a uniform confinement pressure distribution, the maximum confinement pressure in a circular section can be calculated by

\[
\frac{f_l}{D} = 2 \frac{f_c}{f'_c}
\]

where \( f_c \) is the total thickness, \( f_c \) is the ultimate tensile strength of the fibres, and \( D \) is the diameter of the cylinder. It is well understood that the performance of confined concrete...
is directly related to the applied confinement pressure and it improves with increased pressure.

Furthermore, it is also known that high-strength concretes require more confinement than normal-strength concretes because the confinement demand increases almost proportionally with concrete strength.

Therefore, to allow a meaningful comparison, FRP-confined cylinders with different concrete strengths were designed to have similar normalised confinement pressures ($f/f'_c$). This was accomplished by adjusting the number of FRP layers, thereby changing the total fibre thickness ($t_f$) of each tube.

Accordingly, 30 MPa, 60 MPa and 90 MPa specimens were designed to have one, two and three layers of CFRP, respectively for the first comparative group, and two, four, and six layers of CFRP, respectively for the second group.

Figure 2 illustrates the influence of the concrete strength on the compressive behaviour of FRP-confined concrete cylinders. The axial stresses shown in Figure 2 are normalised with respect to the unconfined concrete strengths ($f'_c$) to exclusively study the effect of concrete strength on confinement effectiveness.

It should be noted that the test-day strengths of the concretes were slightly different (i.e. 35 MPa, 65 MPa and 100 MPa) than their design strengths; this resulted in reasonably close but not identical $f/f'_c$ ratios for the specimens. Figure 2 illustrates that concrete strength influences two characteristics of the stress-strain curve.

First, high-strength concretes tend to exhibit softening behaviour in the form of a loss in compressive capacity after the initial peak. Subsequently, this loss is recovered as confinement stresses increase.

Second, and not completely independent from the first observation, the strength enhancement ratio ($f'_c/f'_c$) tends to decrease as concrete strength increases. Nevertheless, the results of this study show that sufficiently confined HSC can exhibit highly ductile behaviour; however, the amount of confinement plays an important role in the compressive behaviour of FRP-confined HSC.

3.2 Influence of fibre type

Figure 3 shows the influence of the confinement material properties on the axial stress-strain behaviour of FRP-confined concrete cylinders. The cylinders shown in Figure 3 were confined either with 5-layer CFRP or with 4-layer AFRP, with two specimens cast for each confinement type. The maximum confinement pressures provided by the tubes ($f_c$), as calculated from Eq.1 using the fibre properties shown in Table 1, were

![Figure 1. Stress-strain curves of FRP-confined NSC and HSC cylinders.](image)
almost equal for all four specimens.

The unconfined concrete strengths of all the specimens were approximately 100 MPa by the time of testing. Figure 3 shows that all specimens have developed very similar ultimate compressive strengths. On the other hand, the figure indicates that there are differences in the ultimate axial strains of AFRP- and CFRP-confined specimens, with AFRP-confined specimens developing significantly higher ultimate strains. The significant correlation between the ultimate tensile strain of fibres and the ultimate axial strain of FRP-confined concrete is evident from this comparison.

4.0 CONCLUSIONS

An experimental study on the compressive behaviour of FRP-confined normal and high strength concrete has been described. Based on the results and discussions reported in this paper, the following conclusions can be drawn.

When sufficiently confined with FRP, HSC can exhibit highly ductile behaviour. However, for the same normalised confinement pressures, strength enhancement observed in FRP-confined HSC is lower than those seen in FRP-confined NSC.

The amount of confinement required to sufficiently confine HSC is significantly higher than that required for normal strength concrete.

The mechanical properties of the fibres used as confinement reinforcement influences the compressive behaviour of FRP-confined concrete. A clear correlation exists between the ultimate rupture strain of fibres and the ultimate axial strain of FRP-confined concrete.

REFERENCES


High performance concrete in bridge decks – Opportunities for innovation

Doug Jenkins – Principal, Interactive Design Services

1. INTRODUCTION
The use of high performance concrete offers advantages in durability, ease of placement, and reduced creep and shrinkage, as well as increased compressive, shear and tensile strength. Offsetting these advantages are potentially reduced ductility and fire resistance, and increased unit cost. High strength concrete is increasingly used in concrete bridge decks internationally, especially in North America, Northern Europe and Japan. This paper examines the international use of high performance concrete in bridge decks, and reviews the potential use of higher strength grades in Australia, both within the limits set by the current bridge code AS 5100 (2004), and for the higher strengths covered by the latest concrete structures code AS 3600 (2009).

The use of high performance concrete in bridges has been driven by durability problems associated with reinforced concrete structures subject to de-icing salts and freeze-thaw conditions. In Australia, typical exposure conditions are not so aggressive and use of high performance concrete for durability reasons has been limited to particularly aggressive conditions. Use of concrete with high early age strength has however become common in the manufacture of precast bridge girders for operational reasons. This paper focuses on the direct economic benefits of the use of high strength concrete, and discusses the requirements of Australian codes and specifications relating to high strength concrete. Case studies are presented illustrating the potential saving in materials from the use of higher strength grades.

2. WHAT IS HIGH PERFORMANCE CONCRETE?
The term “High Performance Concrete” (HPC) is in danger of becoming a marketing phrase with little meaning. It has been observed that in the US all concrete suppliers claim that their product is high performance concrete because no one wants to buy low performance concrete. Nonetheless the term can sensibly be applied to concrete that has specific superior properties, to satisfy specified performance requirements. The US Federal Highways Administration defines HPC as 1:

“A high performance concrete is a concrete in which certain characteristics are developed for a particular application and environments. Examples of characteristics that may be considered critical for an application are:

• Ease of placement.
• Compaction without segregation.
• Early-age strength.
• Long term mechanical properties.
• Permeability.
• Durability.
• Heat of hydration.
• Toughness.
• Volume stability.
• Long life in severe environments.”

High Strength Concrete (HSC) in the Australian context is here defined as concrete with a characteristic 28 day cylinder strength greater than 50 MPa, and up to 100 MPa. Concretes of still higher strength are now available but are outside the scope of this paper due to their special design requirements and higher cost. Other improved properties associated with HSC include improved shear and tensile strength, high modulus of elasticity, high early age strength, and reduced creep deformation. Less desirable properties that may be associated with HSC include reduced ductility, reduced fire resistance, and greater susceptibility to early age cracking.

3. INTERNATIONAL USE OF HIGH PERFORMANCE CONCRETE IN BRIDGES
3.1 General
HPC was used in Japan as early as 1940 3 and has been used more widely for particular applications for over thirty years, with the first International Conference on Utilization of High Strength Concrete being held in Stavanger, Norway, in 1987. The most recent of these conferences, the eighth, was held in Tokyo in October 2008, and the next, now named The 9th International Symposium on High Performance Concrete, is due to be held in Rotorua in August this year. Early developments were centred in northern Europe and focussed on applications in longer span bridges as well as high rise buildings and offshore structures, with more general use becoming mandatory in some countries by the early 1990s. These developments were summarised in “High-Performance Concretes, a State-of-Art Report (1989-1994)” 3, which remains a valuable summary of HPC technology.

Over the past 10 years the use of HPC in short to medium span bridges has been actively promoted by government agencies in the US and Canada, both for improved durability and efficiency in the use of materials. North American information resources on HPC available on the internet include:

• “Bridge Views” 1 – http://www.cement.org/bridges/br_newsletter.asp

Concrete in Australia Vol 37 No 2
3.2 Japan
A concrete with a compressive strength of over 100 MPa was developed by Dr T Yoshida of the University of Tokyo in 1940. This material was mainly used for precast concrete tunnel linings in the 1940s. These tunnels continue in service, including in undersea conditions, and show no problems after 70 years without maintenance.

Three high strength concrete (HSC) bridges built for Japan National Railway in 1973 are of historical importance. The reasons for utilising HSC were to lower the dead load, to reduce deflection, vibration and noise, and to reduce maintenance costs. After nearly 40 years of service, the bridges representing the first generation of HSC bridges worldwide have performed according to all the expectations.

The durability of concrete structures became a major topic of interest in Japan in the early 1980s. Combined with shortages of skilled construction workers, this led to the development of self-compacting concrete, starting in 1986. By the early 1990s, a concrete that did not require vibration to achieve full compaction had been developed. As of 2000, the annual amount of self-compacting concrete used in Japan was about 400,000 m³.

3.3 Scandinavia
In Norway the combination of harsh climatic conditions, a long coastline with many structures subject to chloride attack, and the development of concrete offshore drilling platforms in the North Sea led to the early adoption of HPC. For instance, in 1989 the Norwegian Roads Administration introduced a requirement for a water-binder ratio of less than 0.40, combined with the use of silica fume on all infrastructure projects. In the same year, concrete with a characteristic cube strength of 105 MPa was introduced in the Norwegian concrete design code. Lightweight aggregates have been used in many Norwegian structures, particularly balanced cantilever structures. Characteristic strengths are in the range 55 to 70 MPa, with densities in the range 1900 to 1950 kg/m³.

The development of HPC in Denmark and Sweden was driven by the construction of the massive Great Belt and Oresund Link bridge projects, with construction starting in 1988 and 1989. The concrete had to meet high performance requirements, however, the term “high performance concrete” (HPC) is not used in Denmark. Requirements can be high or low, but performance can only be “yes” or “no”. Therefore, per the Danish definition, there is no such thing as HPC. Nevertheless, in reality, concrete for the Link would be described as HPC according to US terminology.

3.4 France
The first use of the term high performance concrete (HPC) in France goes back to 1983 and specifically to the building of a bridge at Melun under the impetus of LCPC and SETRA (Research Agency and Bridge Department of the French Highways Administration, respectively). The use of HPC in France has been mainly in the bridge, rather than the...
building sector. The reasons for this are firstly because high rise building is dominated by steel construction in France, and secondly because of the partnership between bridge owners and the concrete industry, leading to the formation of a joint government/industry group to advance the use of HPC, called BHP 2000. Concrete with characteristic strengths in the range 70 to 80 MPa are now common in France, with significant progress being made in modifying codes and standards to address the use of HPC.

According to Virlogeux, “The development of high performance concrete is one of the major trends in recent years for concrete construction. High performance concrete and not only high strength concrete because the increased compactness is a major advantage for the long term durability of concrete structures.” Virlogeux sees the main benefit for standard and medium span bridges as being in increased durability, because “engineers cannot take a large advantage … from an increased strength” however this opinion is not supported by recent studies on precast girder bridges carried out in the US.

3.5 North America

The use of high strength concrete has a history of over 30 years in the US, and over the last fifteen years the use of HPC in bridges has been actively encouraged by owner organisations in partnership with industry groups.

The AASHTO “Task force on Strategic Highway Research Program (SHRP) Implementation” developed and instituted the Lead State Program in 1996. Seven “high pay off” SHRP technologies (including HPC) had been identified in 1987, and a strategy based on “Lead States” was implemented to encourage the utilisation of these technologies. The team members represented industry, FHWA, and eight states. Their mission was to promote the implementation of HPC technology for use in pavements and bridges and to share knowledge, benefits, and challenges with the states and their customers. An FHWA implementation survey published in March 2004 reported that 44 out of the 50 US states had used HPC in specifications in the last 10 years and that the great majority had made changes to curing requirements and specified concrete strengths to allow the efficient use of HPC.

In 1999 the National Council Bridge Council (NCBC) and FHWA came to a cooperative agreement intended to develop and implement means to enhance the use and quality of concrete materials and bridge systems. The three key objectives were:

- Identify needs related to HPC practices and procedures in relation to bridge design and construction.
- Develop new and improved HPC practices and procedures related to concrete construction.
- Implement technology transfer, training, and outreach activities on new and improved HPC practices and procedures; and develop partnership opportunities and joint efforts between Federal, State, and local governments, academia, and the private sector.

“HPC Bridge Views”, a bimonthly newsletter on implementation of HPC usage and associated technical issues, was the first product of this agreement. This publication continued up to issue 45, and all issues are available for free download on the internet.

In Canada, extreme climatic conditions and problems with durability led to the conclusion that the impenetrability of concrete cover was of paramount importance, and the development of HPC as the key element to achieve this aim. A network of centres of excellence on HPC, funded under the Federal Government “Centres of Excellence Programme”, commenced in 1990. In 1994 the network became known as Concrete Canada, and by 2000 network researchers had published over 400 papers. The majority of HPC bridges constructed in Canada to date have had concrete strengths in the range 50 to 60 MPa, however the Canadian Standards Association (CSA) concrete code (A23.3 – 94) covers compressive strengths up to 80 MPa, and the Canadian Highway Bridge Design Code (CSA S6, 2000) has a compressive stress limit of 85 MPa, unless otherwise approved.

The Cement Association of Canada’s review of HPC Structures in Canada provides further detailed information on Canadian structures and code requirements.
Table 1. Maximum effect girder compressive strength, after Kahn and Saber 34.

<table>
<thead>
<tr>
<th>Spacing, m</th>
<th>0.6 in diameter strands</th>
<th>0.5 in diameter strands</th>
</tr>
</thead>
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<tr>
<td></td>
<td>3.4</td>
<td>2.7</td>
</tr>
<tr>
<td>Section</td>
<td></td>
<td></td>
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<tr>
<td>AASHTO</td>
<td>90</td>
<td>83</td>
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<tr>
<td>Type I</td>
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<td></td>
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<tr>
<td>AASHTO</td>
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<td>90</td>
</tr>
<tr>
<td>Type II</td>
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<tr>
<td>AASHTO</td>
<td>83</td>
<td>83</td>
</tr>
<tr>
<td>Type III</td>
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<tr>
<td>Type IV</td>
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<td>NU1350</td>
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<td>76</td>
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</table>

4. USE OF HIGH PERFORMANCE CONCRETE IN AUSTRALIA

The use of high strength concrete in Australia has been led by the building industry where competition in the high-rise building sector has led to the use of concrete with strengths of 100 MPa and higher for highly loaded columns. Until the introduction of the Australian Standard Bridge Code (AS 5100) 15 in 2004 the maximum strength of concrete in bridges was limited to 50 MPa, and use of HPC in bridges has been mainly limited to structures in particularly aggressive environments. An example of the use of HPC for durability reasons is the Sorrell Causeway Bridge in Tasmania, where high performance concrete with a low w/c ratio and high slag and silica flume content was used to minimise shrinkage and reduce the ability of surface chlorides to diffuse 16.

Whilst many documents relevant to the specification, production and use of HPC, particularly with reference to concrete durability, have been produced by organisations such as the Concrete Institute of Australia 17-22, and Austroads 23-26, there has been no activity in HPC research and development. The use of 15.2 mm strand was most effective when girder strengths exceeded 55 MPa (34).

However, there has been little published research examining the economic benefits of the use of higher concrete strengths in Australian Bridges, and there is no national coordination or implementation programme for High Performance Concrete.

5. ECONOMICS OF HIGH STRENGTH CONCRETE

A number of studies of the economics of using concrete with higher compressive strengths in precast pretensioned bridge girders and in-situ bridge decks have been published in the US 33-36. These studies were broadly in agreement; the main conclusions being:

- Beam sections that have a large bottom flange are efficient for HPC applications 33.
- The most significant property is compressive strength at transfer. Allowable tension at service has a minor impact 33, 35.
- For AASHTO beam sections, maximum spans were increased between 20% and 45% when the concrete strength was increased from 41 to 96 MPa and when strand diameter was increased from 12.7 mm to 15.2 mm 34.
- Use of 15.2 mm strand was most effective when girder strengths exceeded 55 MPa (34).
- With AASHTO Type 1 to Type IV girders, using 15.2 mm strand, concrete strengths greater than 83 to 90 MPa did not significantly increase maximum span lengths 34. See Figure 1 and Table 1.
- The strength of the composite deck had little influence on the maximum span of high strength girders 34.
- The availability of HPC allows designs with longer spans, fewer girder lines, and shallower girder sections, depending on the parameters of the project 35.
- Maximum useful concrete strengths with I and bulb-T girders are in the range 62 to 69 MPa with 12.7 mm strand and up to about 83 MPa with 15.2 mm strand. With U beams with a wide bottom flange and three rows of strands strengths up to 97 MPa are beneficial 36.

These recommendations are reviewed in the Australian Context in Sections 6 and 7.

6. AS 5100 PROVISIONS FOR HIGH STRENGTH CONCRETE

With the introduction of AS 5100 the Australian Bridge Design Code was extended to cover the low range of high strength concrete (compressive strength up to 65 MPa) for the first time, and maintained compatibility with the Concrete Structures Code, AS 3600 – 2001. In addition to the characteristic compressive strength requirements the following clauses are relevant to the design of high strength concrete bridges:

"1.5.1 Provided that the requirements of Section 2 are met, this Standard shall not be interpreted so as to prevent the use of materials or methods of design, or construction not specifically referred to herein."

This clause has been retained from the previous bridge code, and allows the use of higher strength concrete than that covered by the Code, subject to approval of the client and adequate justification of design parameters.

"2.5.2 The maximum concrete compressive stress under the fatigue design loading specified in AS 5100.2 shall be limited to the smaller of 0.45 f'c and 18 MPa."

This clause appears to place an unrealistically low compressive stress limit on high strength concrete in prestressed structures subject to...
fatigue loading. In practice the compressive stress in service in composite structures is only likely to exceed 18 MPa under fatigue loading in structures with concrete at the upper end of the HSC range. Nonetheless it is desirable that this requirement should be reviewed to cover higher strength grades.

"6.1.1 (b)(ii) \( f'_c \) may be: … (ii) determined statistically from flexural strength tests carried out in accordance with AS 1012.11.”

Increased tensile strength provides some increased bending capacity with no increase in prestressing force, which may be further enhanced if testing shows higher tensile strength than that given by the code.

"6.1.7. (a) The basic shrinkage strain of concrete may be: …

(iii) determined by tests in accordance with As 1012.13

6.1.8.1 The basic creep factor of concrete … shall be:

(b) determined from measurements on similar local concrete; or

(c) determined by tests in accordance with As 1012.16

6.1.8.2 The design creep factor \((q_{cc})\) of concrete shall be determined from the basic creep factor \((q_{cc.b})\) by any accepted mathematical model for creep behaviour, calibrated such that \(q_{cc.b}\) is also predicted by the chosen model.

6.4.3.3 The loss of prestress due to creep of the concrete shall be calculated from an analysis of the creep strains in the concrete. …”

High strength concrete has significantly less long term creep deflection than lower strength grades, and may have lower long term shrinkage, but the default shrinkage strain given in AS 5100 does not vary with concrete strength, and the creep factor is constant for strengths over 50 MPa. It may therefore be worthwhile to use measured creep and shrinkage parameters, and to carry out a detailed creep and shrinkage analysis, to reduce design prestress losses. It should be recognised that if concrete compressive stresses have been
designed close to the allowable limits overall long term strain may be increased when high strength concrete is used, especially if maximum allowable compressive stresses are applied at transfer.

7. CASE STUDIES

The following case studies illustrate the potential for either reducing girder spacing or reducing girder depth by using concrete with characteristic compressive strength in the range 65 to 100 MPa with standard Super-T open top pretensioned girders. Maximum span lengths have been calculated for standard Super-T girders, and the optimum design has been investigated for a typical three lane overbridge with 28.5 m simply supported span with M1600 loading, placed to produce the most severe loading effects on an exterior girder.

The maximum span length achievable with varying levels of prestress and concrete grade was found for the girders listed in Table 2. For each level of prestress the minimum concrete grade was used that satisfied the stress requirements at transfer. For the larger girders (Types 4 and 5), 65 MPa concrete was adequate for the maximum level of prestress that can be achieved by placing 15.2 mm diameter strand on a 50 mm grid within the bottom flange. To investigate the effect of higher levels of prestress a standard Type 4 girder was modified as follows:

- Increase bottom flange width by 200 mm to allow 20 strands in each layer (Type 4A).
- Increase bottom flange depth by 50 mm to allow one additional layer (Type 4B).
- Increase bottom flange depth by 100 mm to allow two additional layers (Type 4C).

The following parameters were assumed:

- Compressive strength at transfer = 0.7 f’ c.
- Steam curing applied (hence strand relaxation applied at time of transfer).
- Strand stressed to 80% specified tensile strength.
- Creep, shrinkage, and temperature stresses in accordance with AS 5100.
- Insitu concrete 40 MPa, 160 mm thick in all cases.
- Assumed girder spacing = 2.7 m.

The results of the analysis are shown in Figures 3 and 4. The main features are:

- Increasing the concrete grade to 65 MPa increased the maximum span of each girder type by from 13% to 14%. Grade 80 concrete increased the span capacity of Type 1 and 2 girders by 21% to 23% over Grade 50 concrete, but for Type 3, 4 and 5 girders the available strand locations were already filled with Grade 65 concrete, and an increase to 80 MPa gave little further benefit.
- Modifying the Type 4 section to allow more prestress to be applied increased the maximum span, compared with a standard Type 4 section with grade 50 concrete, by 38%, 23%, and 30% for type 4A, 4B, and 4C sections respectively.

All available strand locations were filled with Grade 80 concrete in the modified girders, and little additional capacity would be gained by using Grade 100 concrete or higher.

For the typical bridge studied, five Type 4 girders with 34 strands would have been required with Grade 50 concrete. Use of Grade 65 concrete would allow the use of Type 3 girders with 44 strands, and Grade 80 concrete would allow the use of modified Type 2 girders with 56 strands.

Alternatively, maintaining Type 4 girders and increasing the girder spacing, Grade 65 concrete would provide adequate capacity with four girders with 42 strands, and Grade 80 concrete would allow the use of three modified Type 4 girders with 64 strands.

The study shows that significant savings in concrete quantities and/or construction depth are achievable by using Grade 65 concrete with standard girders, or Grade 80 concrete with modified girders. To achieve significant savings from Grade 100 or higher strength concrete would need more substantial changes to the beam cross section and method of construction.

8. CONCLUSIONS

The history of the introduction of HPC in bridges in Europe and North America shows a clear correlation between the extent to which government and bridge owner organisations have actively promoted the use of HPC and the penetration into the bridge market. In all cases the superior durability of dense high strength concrete has been the original motivation for the active support of HPC, but in many cases, particularly in North America, the use of higher strength grades has been found to give a significant direct economic benefit in

### Table 2. Super-T section properties.

<table>
<thead>
<tr>
<th>Type</th>
<th>Depth mm</th>
<th>Precast</th>
<th>Section Properties</th>
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<td></td>
<td></td>
<td>A mm²</td>
<td>I mm⁴</td>
<td>Yc mm²</td>
<td>A mm²</td>
<td>I mm⁴</td>
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<td>1</td>
<td>750</td>
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<td>3,280E+10</td>
<td>389</td>
<td>887,584</td>
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<tr>
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reduced materials and transportation cost and/or reduced bridge construction depth. Studies in the US have found the optimum strength grade using existing standard bridge beams to be in the range 60 to 90 MPa, and these strengths have been found to be achievable in practice provided due attention is paid to the special requirements of HPC. Optimisation of beams to allow higher prestress forces may result in concrete of still higher strengths proving economical.

In Australia code restrictions on design strength have resulted in HPC only being specified where a particularly aggressive environment demands special attention be paid to durability. Case studies using typical Australian precast bridge girders and composite bridge construction show the potential for similar savings to those found in the North American studies. The following actions are recommended to encourage the greater use of high strength, high performance concrete in Australia:

- 65 MPa to be considered the standard concrete grade for use in precast pretensioned bridge girders and post tensioned bridge decks.
- The use of 80 to 100 MPa concrete to be considered where significant benefit can be shown.
- AS 5100 to be revised to allow strength grades up to 100 MPa as soon as possible.
- Optimisation of standard Super-T bridge girders for higher strength grades to be investigated.
- Investigation of higher strength grades for bridge deck slabs, using membrane action to achieve greater spans and/or reduced slab depth.
- Integrated action by government and industry bodies to ensure education of designers, precasters, and contractors in the requirements for producing high quality high strength concrete bridges.

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1.0 INTRODUCTION

When an Australian concrete supplier produces concrete in accordance with AS 1379 \(^1\) it is intended that the supplier measures batch weights of materials to a level of accuracy as required under section 4.2.1.2 of that standard. This allows for control of water by either of the following methods:

(a) By control of slump within the standard limits. This assumes that a mix has been assessed and the water content is directly related to the slump of the design mix but is silent on control of mix SSD batch weight control.

(b) By control of “water/cement” ratio (“water/binder” or “W/B” ratio). This standard allows for a variation of ± 10% of this ratio. Given the binder content can vary by up to ± 2.5% or more under the requirements of this standard then this means the water content can effectively only vary by approximately ± 7.5% to stay in compliance.

In the case of normal class concrete the Australian premixed concrete industry have utilised method 1.0 a) above and this has been seen as a reasonable approach provided routine review of concrete constituents, strengths and density are carried out to ensure limited variance.

Over many years the Australian premixed concrete industry have been requested to guarantee W/B ratio for special class concrete mixes on specific projects where there is a specified requirement for increased frequency of measuring sand and aggregate moisture content and provide the results of measurements as part of a quality plan. There are a number of methods to assess the aggregate moisture, adjust mixes for it and report the information to clients. With the introduction of concrete mixes containing high range water reducing admixtures, many concrete specifiers have become suspicious of a concrete mixes actual water content and its less well defined relationship to slump when containing a high range water reducing (HRWR) admixture. In these cases specifications requiring more reliance on water measurement at the concrete plant including moisture contents contained in the constituent materials are being applied.

One of the impacts of specifying higher durability concrete, where high range water reducing admixtures are more commonly used, is that in this type of concrete water control is critical to the concrete durability performance.

With this background it can be seen that a renewed focus needs to go into accurate measurement of mix water content.

2.0 CURRENT PROCESSES AND TESTS FOR WATER CONTROL OF PREMIXED CONCRETE

The water content or total water of concrete can be assessed by either measuring the water in plastic concrete by testing a sample or can be calculated by measuring the moisture content of each constituent in the concrete mix. The latter methods are more useful to the concrete supplier as this allows accurate corrections of batch masses for moisture content relative to the SSD condition prior to batching and thereby a more efficient and effective production process. Current practice methods used for either approach are discussed in the following sections.

2.1 Assessment of the water content of plastic concrete

Two key approaches are taken to this type of assessment:

(a) Measurement in the mixer.

(b) Measurement of a sample of concrete.

Accurate measurement of the water content of a concrete batch either requires an adequate sample size to be assessed or the averaging of multiple smaller samples. In the case of 2.1 a) above the measurement in a mixer is more commonly carried out by using a microwave device attached to the surface of the concrete mixer or placed in the concrete during mixing. The manufacturers of such devices report a reading accuracy of ± 0.3% which is approximately ± 7 l/m³ for normal weight concrete. A paper by Hu & Zhang \(^2\) provides data that suggests that this type of device may be less accurate if sufficient mixing time, adequate coverage of the sensor and a significant number of samples are not used (and averaged once complete mixing has occurred).

In the case of 2.1 b) above there are also a number of methods available. The accuracy of these type of methods depend on the sampling method and sample size used. One such method is used by the Victorian road authority VicRoads and is given the test method number RC 251.01 \(^3\). This method assesses a 1.5 kg sample of plastic concrete and the author recommends reporting accuracy to 0.1% of dry mass (approx ±2 kg/m³ water content).
In a paper by F Andrews-Phaedonos, details of research into this method recommend an accuracy of ± 5% of the total water value for concrete with W/B ratio under 0.50 (which is typically the case for higher durability concrete) and this would suggest a likely accuracy of approximately ± 10 l/m³.

Another method proposed by at least one equipment manufacturer is to place a standard volume of compacted concrete into a container that forms part of the device and use microwave to measure the water content of the contained concrete. Care is needed in the use of these devices as the sample size of concrete appears to be quite low and reported accuracy of ± 0.3% of total concrete (± 7 l/m³) has not been achieved in experiments carried out on concrete under the authors direction.

These methods could serve as useful means of assessing compliance of a batch of concrete if the results are proven to be sufficiently accurate but are of limited use in the concrete production process unless combined with a suitable assessment method for concrete constituents which is used to maintain target total water and saturated surface dry batch masses of concrete constituents.

2.2 Assessment of the water content of concrete constituents

There are a number of methods available to assess the water content of concrete constituents (generally coarse and fine aggregates). Examples include:

(a) In plant sampling and drying by microwave oven, laboratory oven or other suitable means of drying in combination with accurate wet and dry mass determinations.
(b) In plant sampling and indirect measurement by moisture flasks or absorbent/gas pressure devices.
(c) Hand held, calibrated resistance or microwave measuring devices with a significant number of samples taken to estimate moisture value.
(d) Plant installed, calibrated microwave probes in constituent holding bins.

Sampling of materials at a concrete plant and accurate methods of measuring moisture content either by laboratory techniques such as AS 1289.2.1.1, AS 1289.2.1.4 or AS 1289.2.1.6 are the most accurate means of determination as per 2.2 a) above. These methods will provide a reportable accuracy to 0.1% which, given an accurate measurement of aggregate absorbed water, will provide aggregate contained water to ± 2 l/m³. A potential shortcoming of this method is that by the time a result is processed it is possible the aggregate or other constituent has changed moisture due to rain or drying or may in fact have been used to manufacture concrete.

Also, sufficient samples need to be taken to accurately determine the average moisture of a specific lot or load of aggregate. To ensure that these methods are meaningful a very careful quality plan will need to be set up by the concrete supplier to ensure that tested concrete aggregates are not used before relevant moisture content test data is available to assist in control of batch masses and total water content.

The use of indirect methods noted in 2.2 b) above for assessing the moisture content of an aggregate such as a Speedy Meter volume flask methods introduce a further level of uncertainty of the measured value of moisture content and at best will be within ± 1% (contained water to ± 18 l/m³).

An alternative indirect method of measurement of aggregate moisture noted in 2.2 c) is using a hand held resistance or microwave device. The accuracy of these devices are reported as ±0,3% (contained water to ± 7 l/m³) but even getting close to this accuracy is dependant on regular, accurate calibration of the device for each individual material being tested and a careful procedure for sampling and compaction of a sufficient volume of sample for testing.

The speed of this method overcomes some of the concerns relating to 2.2 a) noted above but a reasonable number of samples need to be taken per load of aggregate or lot and moisture content averaged to ensure accuracy as suggested above for 2.2 a) as well.

The ideal state is that noted in 2.2 d) using a system where plant installed measuring probes continuously and accurately monitor the moisture contents of all aggregate and sand with an interface of this data to the plant batch system for water correction and batch mass correction.

Unfortunately this state still proves hard to achieve due to measurement system accuracy issues. The accuracy of plant installed moisture content systems depend on the level of variability of the materials in respects other than moisture content (minor contaminants and particle size distribution for example), the location and maintenance of probes, the number of readings used and averaged to provide a result and the method of maintaining calibration of probes. Suppliers of such systems are continuing to improve their products and their accuracy.

2.3 Overview of the assessment of the water content of concrete

From sections 2.1 and 2.2 it can be seen that at best we may be able to assess concrete constituent water within ± 7l/m³ unless an intensive program of rapid direct moisture content measurement is applied to constituents of a concrete mix (given other factors noted above in relation to 2.2 d) that impact current systems this will be closer to ± 10 l/m³ at best).

If a concrete supplier can cover aggregates to prevent the impacts of rain or drying and aggregates are brought into the concrete in identified lots with the moisture content of each lot assessed quickly as per methods applicable to 2.2 a) above and mixed corrected for changes in moisture then it is feasible to batch concrete with total water within ± 2 l/m³. The reality is that these conditions are rarely capable of being met and so control of total water within ± 2 l/m³ is highly unlikely.

In addition to the above there is also the accuracy of water added at the concrete plant. In plants where weigh systems are available for water measurement the calibration accuracy will generally be accurate within 0.5% of the target value and so within ± 0.7 l/m³. In plants using volumetric batching of water the accuracy is lower and this is reflected in AS 1379 which suggests an accuracy of this device as ± 2% of the target value and so within ± 2.5 l/m³. While, added water accuracy is a consideration for accuracy it is not as significant an issue as that of assessing the water contained in constituent materials.

Concrete in Australia Vol 37 No 2 43
2.4 Indicators of concrete mix total water

As noted in the introduction AS 1379 \(^1\) control of slump measured to AS 1012.3.1 \(^{10}\) is one of the options available for supply of concrete at a consistent water content. The reasons for this are best described by a review of the relationship between total water content and slump of a specific concrete mix tested under the same conditions (timing and temperature) and without use of varying admixture doses.

Figure 1 shows a fairly common relationship between mix total water and laboratory measured slump for a specific concrete mix. AS 1379 \(^1\) Section 5.2.3 details acceptable tolerances on slump. Comparison of the tolerances in this section of AS 1379 \(^1\) to Figure 1 will show the tolerances relate to a variation in water content of approximately ± 4 l/m\(^3\) at any particular target slump (e.g. ± 15 mm at a target slump of 80 mm). This amount will vary a little from mix design to mix design but for mixes containing a maximum sized aggregate of between 10 mm and 20 mm will be reasonable estimate.

This appears to be a greater level of accuracy than many of the aggregate moisture measurement methods discussed in section 2.2 above.

The value of the slump of a specific batch of concrete with fixed water and admixture content is both time and temperature dependant. This “slump loss” occurs in all concrete as noted
by Tuthill and as he also noted is cement source and type dependant. Both Tuthill and Meyer attribute the slump loss to early hydration of the cement. The author also notes from experience that slump loss of a single batch produces slump values equivalent to a constant loss of water over time for the first hour or since a concrete batch started mixing (i.e. when water was blended with the cement in this load). The rate of the apparent initial loss in water is dependant on cement type, cement content, concrete temperature and admixtures used.

Given this it becomes apparent that slump is only a really useful quality control measure for water content if the concrete slump is always assessed at the same time after batching and the concrete always delivered at the same temperature. An alternative is to develop admixture systems that prevent slump loss at least for a period that allows the concrete to be assessed at the point of delivery (this is typically 25 minutes to 60 minutes in most cases).

Another alternative is to develop a set of slump loss curves over time for the specific design concrete mix at different concrete temperatures over the range supplied so that a “slump at batching” can be estimated and controlled within expected limits to control water variations. Figure 2 is the authors demonstration of how such a set of curves may look for a mix with two water contents at upper and lower acceptable limits for different concrete temperatures over time from mixing.

As an example of the use of a graph such as given in Figure 2 a concrete mix is supplied at a temperature of 20°C and assessed for slump at 35 minutes then the acceptable slump range would be 75 mm to 100 mm in this case. There are measurement uncertainties associated with the slump test and this does diminish the value of the above methods of control and these uncertainties are built into the slump tolerances in AS 1379 section 5.2.3. If applying this method a specifier will need to be mindful of these tolerances.

The plastic density of a concrete mix is also influenced significantly by water and air content variations. While solid mix constituents do vary marginally in density over the longer period, these variations have far less impact than water and air content where material sources are carefully controlled and normal batch accuracy limits are applied.

As concrete air content can be measured with a reasonable level of accuracy and is reported to the nearest 0.1% (1 l/m3) using test method AS 1012.4.1 it is possible to use the plastic density of concrete measured in accordance with method AS 1012.5 and corrected to “Mass per unit volume of air free concrete” by the method detailed in AS 1379 section A.3.6 (except that “βAc” is set to zero as this is measuring concrete density and not the mortar fraction) as a means of indirectly controlling water content.

Test method AS 1012.5 recommends a reporting accuracy to the nearest 10 kg/m3 and it can be seen that this will have an impact on the accuracy of water control. In Figure 3 the author proposes a relationship between total water content and the “air free unit mass of concrete” in a mixture of concrete solid ingredients which are otherwise held constant.

From Figure 3 it can be seen that assuming a range of accuracy of ± 10 kg/m3 for the “air free unit mass of concrete” suggests an associated total water range of ± 7 l/m3.

Concrete compressive strength is influenced by water content amongst other factors. The relationship between W/B ratio and compressive strength has been researched by many authors. This author has generally found that the general relationship between W/B ratio and 28 day compressive strength follows a relationship that varies with binder type in actual value but assuming a W/B ratio of 0.40 has a 28 day strength at 100% then the portion of the relationship in the W/B ratio range from 0.30 to 0.50 can typically be described by Figure 4.

Using Figure 4 it can be seen that, for example, if a concrete

![Air Free Unit Mass of Concrete Vs. Water Content](image-url)
A review of this concern and proposed method of managing it needs to be part of a concrete supplier’s ongoing inspection and test plan.

3.0 STATISTICAL APPROACH TO WATER CONTROL

From the above it can be seen that there is a level of uncertainty on the actual water content of any batch of concrete and even with the best processes this uncertainty can be significant. While there are methods that directly or indirectly assess water content no one method can be said to provide accuracy within 4 l/m³.

A specifier wishing to be assured that a concrete has a W/B ratio less than a specified value will need to ensure that a supplier designs a mix with a lower target W/B ratio than the maximum proposed.

The value of the lower target W/B ratio will need to come from an understanding of the variance arising from batching and measuring process as well as the level of confidence required to ensure that the maximum W/B ratio is not exceeded. This level of certainty depends on the risk of failure to the consumer and is something that should be included in the project specification (e.g. W/B ratio should not be exceeded at a 95% confidence level).

The process for specifying and assessing compliance can be developed using the concepts used for assessing concrete compressive strength provided the level of consumer risk is known and acceptable. By way of a worked example the author will demonstrate this process and in the next section discuss a quality plan for concrete supply where the highest level of water control may be required.

As an example the author proposed that a specifier requires a concrete mix with a W/B ratio to be less than 0.40 in order to assure certain concrete durability measures are achieved as previously assessed from trial mixes. One concrete supplier in this example has a quality plan that assures total water within ±10 l/m³ of estimated water content at a 90% confidence (two sided confidence level). The supplier also assures that the binder content is within ± 10 kg/m³ at a 90% confidence (two sided confidence level) on the basis of load sizes are in excess of 2 m³.
After applying basic statistical analysis the supplier will determine that for a 95% confidence level that the W/B ratio is under 0.40 the target W/B will need to be less than or equal to 0.376 and likewise for a 99% confidence level that the W/B ratio is under 0.40 the target W/B will need to be less than or equal to 0.366.

For a given sampling program a supplier or specifier may be able to verify conformance using a similar methodology to that applied to compressive strength in AS 1379 1 except that in this case the “characteristic” value of W/B ratio becomes an upper bound rather than lower bound. Standard deviations on W/B ratio can be estimated from production and test data or from analysis of variation for individual parts of the process as above.

4.0 A PROPOSED QUALITY CONTROL METHODOLOGY FOR HIGHER ACCURACY WATER MEASUREMENT

A suitable quality plan for control of water for a project requiring a high durability concrete supply may need several levels of independent assessment to reduce the risk of exceeding a design maximum W/B ratio. The options that the author recommends are:

(a) Measurement of concrete constituent moisture contents and inclusion in the batch process.
(b) Accurate monitoring of the moisture content of all constituents and tracking of any water additions during the manufacture and delivery process.
(c) Check of concrete slump at the start of delivery on site and prior to any variable dosage of HRWR Admixture – possibly use slump loss chart developed for product to improve accuracy of the assessment.
(d) Measure concrete “Air free unit mass” on site and compare to trial mix target and acceptable range.
(e) Use an onsite test to check of concrete water content (such as VicRoads RC 251.01 3).
(f) Cast compressive strength specimens for 28 day strength compliance as well as for check for uniformity through control of mean strength and standard deviation of statistically viable sampling numbers.

Each of these measures is useful and will help to verify that the concrete mix is supplied within a range of total water content as discussed in section 2 above. In reality assessment methods (c) to (f) above should just be verifying the concrete plant batching process and accuracy of checks made in the production process such as noted in (a) and (b) above but not limited to this.

If three of the above test methods are selected to estimate the total water content of a batch and each method has a similar variance (or standard deviation) associated with measurement then it may be possible to gain a more accurate fix on the total water of a batch by combining the tested total water estimates for a batch. This process is demonstrated in the following.

**Example:**

A batch of concrete has been assessed by measuring the moisture content of constituents (course aggregate, fine aggregate and additive including moisture) at the batch plant to have a total water content of 176 l/m³. The maximum allowable water in this concrete mix is 180 l/m³. Three site tests have been used to assess the water content of the concrete as delivered and the data from these are used to determine the confidence level that the maximum water content has not been exceeded. Table 1 details the relevant test data and calculations for this assessment:

In this analysis it has been assumed that the estimated values of water content for the three test methods can be combined and estimated standard deviation derived from combining the standard deviations of large populations of the three test methods data.

5.0 A PLAN FOR ASSESSMENT OF THE TOTAL WATER OF PREMIXED CONCRETE

Where maximum accuracy of measuring water content is required to warrant that the W/B ratio of a concrete mix is not exceeding a design value then the author recommends a supplier and site quality plan is developed with the following elements:

(a) The specifier provides a requirement for maximum W/B ratio in a statistically meaningful way, e.g. maximum W/B of 0.40 at a 95% confidence level.
(b) The concrete supplier must provide a suitable plan for control of the moisture of constituent materials so that significant variations are limited.
(c) The concrete supplier must have a system of measuring constituent material moisture content at the concrete plant and recording water additions at any point in the
The concrete supplier will ensure that sufficient testing measures are carried out to provide adequate confidence of mix water content at delivery to site so that the concrete supplier’s water control performance can be assessed against the specification requirements. Selection of such methods should be based on those with the highest accuracy in estimating a given batch of concrete water content as reflected in the standard deviation or variance associated with the difference between actual and test measured water content. The methods proposed here are based on the those listed in 2.1 and 2.4 above and could include:

(i) Slump test assessed according to a slump vs. time and temperature chart developed for the mix.
(ii) The use of test method RC 251.01 ¹.
(iii) Assessment of air free unit mass against design value
(iv) The assessment of a statistically viable population of 28 day compressive strength tests for mean strength and standard deviation (suggest 30 test results as a reasonable population to assess standard deviation).
(v) The results of site tests will need to be available to the concrete supplier as soon as possible so that any variation from expected values can be reviewed and corrective action put in place.
(vi) If any batch of concrete has site tests indicating that the maximum water content may be exceeded at the agreed confidence level then the concrete may be rejected and the supplier called on to take immediate corrective action.

6.0 CONCLUSIONS

From Section 2 it can be seen that measurement of concrete mix total water to accuracy within 10 l/m³ is unlikely with currently available methodology. This may present an issue for specifier’s who need to assure that W/B ratio can be maintained within tighter limits and may require the specification of an un-acceptably low target mix W/B ratio to guarantee such a maximum in some cases.

The author has proposed the elements of a quality plan and assessment method that is designed to provide an agreed confidence level that concrete mix total water has been maintained below a specified maximum limit. This is based on using two or more site measurements of the delivered concrete properties to verify concrete total water content and combining the data to assess a maximum expected water content based on a selected confidence level.

While not necessary for all concrete this level of control may be necessary where concrete is specified for to achieve agreed higher performance requirements and more commonly a high durability requirement. It is clear that the additional inspection and testing proposed in the quality plan outline above will come at some additional cost but the specifier may assess that this is justified in certain applications.

ACKNOWLEDGEMENT

Test data used in this paper has come from a number of sources, some from published works that have been referenced and some from unpublished sources. The author would like to acknowledge Boral Construction Materials for access to some of its previously unpublished data.

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Extending the service life of concrete bridges –
Corrosion monitoring of Lynch’s Bridge over
Maribyrnong River

Fred Andrews-Phaedonos – Principal Engineer, Concrete Technology, VicRoads
Dr Ahmad Shayan – Chief Research Scientist, ARRB Group
Dr Aimin Xu – Senior Engineer, ARRB Group

Abstract: As part of an overall strategy to better manage bridge assets and thus extend their service life, a number of investigations were undertaken in 1999, 2006, 2009 and 2010 to monitor the insitu condition of two bridge columns and the pile-cap of Pier 2 of Lynch’s Bridge in Flemington, Victoria. The investigations were aimed at comparing the present condition of the concrete with the condition assessments made previously, in order to evaluate the progress of concrete deterioration. The field investigations included visual inspection and selection of measurement sites, determination of depth of cover the steel, half cell potential, electrical resistivity, corrosion rate measurements. The laboratory work included determination of chloride content and estimation of the chloride diffusion coefficient, carbonation depth, compressive strength, volume of permeable voids (VPV) and petrographic features of the concrete.

The chloride profiles, half-cell potentials, resistivity and corrosion rate determined in the 2009 investigation are similar to those obtained in the 1999 and 2006 investigations, although some values have slightly increased in terms of likelihood of steel corrosion. Nevertheless, the corrosion rate measured some 19 years after construction of the cement only concrete (ie without any SCMs used) is still low to moderate, although it is anticipated that under the present conditions, chloride-induced cracking may occur in the base of columns and top of pile cap, earlier than seven years. Monitoring of potential crack development is required in order to be able to estimate the time for intervention in the form of protective and preventative measures. It is considered that in addition to the reasonable quality of concrete used, the specific type of acrylic coating applied to the bridge piers at the time of construction has also contributed to the improved performance of this structure in a saline environment. As such renewal of the surface coating in susceptible areas with a dedicated high quality protective coating will further slow down the rate of corrosion and further delay the time to crack development.

1.0 INTRODUCTION

Lynch’s Bridge on the inbound carriageway of Smithfield Road, over Maribyrnong River at Flemington was built in 1991/92 comprising of a steel I-Beam superstructure with a concrete deck and supported on reinforced concrete piers. It has five spans, each 21 m, and six piers. Pier 1 and 6 function as abutments standing on dry land, and piers 2 to 5 are in the saline tidal river with a tidal range of about 0.4 to 0.7 m. The bridge was constructed shortly before the introduction of major durability provisions into the VicRoads structural concrete specification which also allowed the inclusion of supplementary cementitious materials (SCMs), ie fly ash, slag, silica fume. As such the piers were constructed using a 50 MPa strength grade concrete containing cement only with a tricalcium aluminate (C3A) content of 2-5%. The concrete piers were also coated with an acrylic decorative/ anticarbonation coating (with DFT in the order of 100-140 μm) in order to improve the aesthetic appeal of the bridge in such a focal location near the Flemington racecourse. Although the expected service life of the coating is in the order of 10 to 15 years, this coating has been in place for the past 19 years without renewal.

In 1999, a diagnostic investigation was undertaken on the corrosion status of the new Lynch’s Bridge over Maribyrnong River in Flemington, as part of a strategy to better manage bridge assets and thus extend their service life. Subsequent investigations were undertaken in 2006, 2009 and 2010 in which chloride ingress tests and electrochemical measurements were conducted on the same columns and pile-cap. The initial investigation work indicated that corrosion activity is very likely to start at the base of the columns and top of pile cap. The current work was undertaken to further monitor the corrosion activity so that timely preventative measures could be implemented. Concrete powder samples were drilled from the atmospheric and tidal zones of a column and the top surface of the pile cap, and were analysed to establish their chloride content profiles. Half-cell potential mapping and corrosion rate of the reinforcement and concrete resistivity were determined on site. Depth of carbonation in the column was also measured. The apparent chloride diffusion coefficient was 1.27x10^-12 m²/s for the column and 2.85x10^-12 m²/s for the pile cap. The surface chloride content of the column has increased from 0.65% in 2006 to 0.78% in 2009. For the pile cap the surface chloride content is almost unchanged in this period (1.35% and 1.39%). For the pile cap the chloride content at the vicinity of
reinforcement has reached the threshold for corrosion initiation. However, the measured corrosion rate in this investigation is still low to moderate.

Based on the 70 mm cover thickness of columns, as stated in the drawings, it is predicted that the threshold chloride would reach the reinforcing steel in three to six years from now. However, it would take several years after that for the thickness of corrosion layer to build up to a level that would cause concrete cracking. Nevertheless, it would be important to intervene (eg use of a dedicated higher quality protective coating in the tidal splash zone area or cathodic prevention) before this stage is reached.

Changes in the deterioration factors should be monitored in about five years from now, when the threshold chloride content would probably reach the steel bars in the tidal and splash zones of columns. It may be necessary to establish the actual cover depth of the columns for a better estimation of the intervention time, or of when visible cracking may occur in concrete. The carbonation depth is very low and would not influence the durability of the structure in the foreseeable future.

2.0 SCOPE OF INVESTIGATION

The investigation work was carried out on Pier 2 of the inbound road bridge of Lynch’s bridge in Flemington. The following onsite sampling and testing were carried out:

- Determination of chloride profiles in one location in atmospheric zone and one location in tidal zone of Column 4 (the upstream second column), and one location on pile-cap. Concrete powder samples were taken at four depth increments at each location and analysed for chloride content. Note that in the 2006 investigation, the chloride samples were taken from Column 5 of this pier. Column 4, which is identical in construction and material to Column 5, is chosen in this investigation to give more information on the chloride penetration into the pier.

- Half-cell potential measurements on the left face and abutment face of Column 4 (as viewed from Melbourne abutment), on grids of 300 mm horizontally and 250 mm vertically.

- Half-cell potential measurements on Column 5 in different exposure zones; and on the pile-cap top face.

- Measurement of concrete resistivity in different exposure zones on the columns.

- Measurement of corrosion current density (corrosion rate) of reinforcement in the same areas as for the resistivity tests, as well as in two locations on the pile cap.

- Concrete cores to determine the strength using a conventional compression testing machine on cores of 100 mm diameter and a eight/diameter ratio of 2 (1999 investigation).

- Concrete cores to determine water absorption, density and volume of permeable voids (VPV) according to the Australian Standards AS 1012.21 (1999 investigation).

The major site work was carried out at low tide in late 2009 when the pile cap surface was above the water level, with a minor follow up undertaken in late 2010.

3.0 INVESTIGATION METHODS

3.1 Chloride analysis

Concrete powder samples were analysed for chloride content by treating the powder with nitric acid solution according to Australian Standard AS 1012.20, and by potentiometric titration against a standard AgNO₃ solution. The diffusion of chlorides in concrete largely obeys Fick’s Second Law of Diffusion:

\[ C(x,t) = C_i + (C_s - C_i) e^{-\frac{x^2}{4Dt}} \]

where \( C(x,t) \) is the chloride content at depth \( x \) and at time \( t \); \( C_i \) is the initial chloride in concrete, \( C_s \) is the content at surface \( (x = 0) \); and \( D \) is the coefficient of diffusion.

The chloride profile data are fitted to this equation, from which the diffusion parameters \( C_s \) and \( D \) can then be estimated.

3.2 Half-cell potential mapping

The half-cell potential survey was made in accordance with ASTM C876, and the final output presented in the form of a potential map of the concrete elements concerned. ASTM C876 notes the probability of reinforcing steel corrosion based on the measured copper-copper sulfate half-cell potential values (CSE) as shown in Table 1.

3.3 Resistivity of concrete

A four-pin resistivity meter (MEGGER DET 5/4R), was used to determine the concrete resistance; the four pins having an equal spacing (d) of 5 cm. The equipment displays the measured resistance of concrete, \( R(\mu \Omega) \) from which the concrete resistivity \( r (\text{k} \Omega \cdot \text{cm}) \), is calculated using \( r = 2d\pi R \). The results are empirically interpreted as:

- 100 kΩ-cm – The resistivity will effectively stop corrosion.
- 50-100 kΩ-cm – Low corrosion rate.
- 10-50 kΩ-cm – Moderate to high corrosion rate when steel is active.
- < 10 kΩ-cm – Resistivity is not the controlling factor.

3.4 Corrosion rate of reinforcing steel

An electrochemical assessment of corrosion was made by measuring the corrosion current density of the reinforcement at selected areas, representative of the different exposure zones.

Table 1. Concrete surface potential criteria.

<table>
<thead>
<tr>
<th>Potential (vs. Copper Sulphate Electrode (CSE))</th>
<th>Indication for reinforcing steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>More positive than -200 to -350 mV CSE</td>
<td>A greater than 90% probability that no corrosion is occurring</td>
</tr>
<tr>
<td>Between -200 to -350 mV CSE</td>
<td>Corrosion activity uncertain</td>
</tr>
<tr>
<td>More negative than -350mV CSE</td>
<td>A greater than 90% probability that corrosion is occurring</td>
</tr>
</tbody>
</table>
In this investigation, the corrosion current density (rate of corrosion) was determined directly by the Gecor equipment, which employs a Cu-CuSO₄ electrode equipped with a sensor controlled guard ring to confine the area of steel bar under testing. The measured values of the corrosion current density can be interpreted in terms of corrosion activity of the steel and categorised as shown in Table 2.

### 3.5 Carbonation depth of concrete

The carbonation depth of concrete is usually determined by spraying the surface of a newly fractured specimen with a colourless solution of phenolphthalein pH indicator. In the case of this work, where holes were drilled into the concrete, the spraying was done on the inner surface of the drilled holes. The phenolphthalein indicator displays a pink colour at pH values of 10 to 13, for non-carbonated concrete. However, it remains colourless when the concrete is carbonated and the pH is lower than 9.5. Therefore, the depth of the colourless zone indicated the carbonation depth of concrete. This method is not effective if the carbonation depth is very small and there is not enough time to allow the excess alkali provided by the newly exposed cement grains (due to drilling) to be neutralised.

**An alternative way to estimate the carbonation depth:**

- The carbonation depth can be indicated by the CO₂ contained in the concrete powder which is released due to the acid treatment. According to the structure drawings for this bridge, 50 MPa concrete was specified for pier columns and pile cap of Lynch’s bridge. Considering that 50 MPa concrete has a cement content of 450 kg/m³ and the dry density of the concrete determined in the 1999 investigation was 2250 kg/m³, the fully carbonated concrete would contain 9% CO₂ by mass of carbonated concrete. The carbonated depth of the concrete sample will be proportional to the ratio of the actual CO₂ content to 9% (the fully carbonated). For a given sampling program a supplier or specifier may be able to verify conformance using a similar methodology to that applied to compressive strength in AS 1379. 1 except that in this case the “characteristic” value of W/B ratio becomes an upper bound rather than lower bound. Standard deviations on W/B ratio can be estimated from production and test data or from analysis of variation for individual parts of the process as above.

### 4.0 RESULTS OF INVESTIGATION AND TESTING

The tidal range at the pier is 0.4 m to 0.7 m and the high water mark on pier columns is at 0.40 m above the pile cap. The actual water level can change to a larger extent due to rainfall and dry weather conditions. Usually, the pile-cap and lower part of the columns are immersed in water at high tide, and exposed at low tide, which allows about 3 h to work on the pile cap. The columns and pile cap have not exhibited any major sign of deterioration. Some corrosion stains can be seen at the construction joint (same height as the water mark) of the columns (Figure 1), which are apparently due to corrosion of the tie wires. This was also observed in the 1999 investigation. As part of the 1999 investigation cores were drilled from columns 4 and 5 and the pile cap of Pier 2 as shown in Figure 1 which shows a general view of Lynch’s Bridge (Melbourne bound), and the view of Pier 2 (from Melbourne side) at low tide, where core drilling was done, and at high tide.

#### 4.1 Chloride profiles and analysis

Concrete powder samples were drilled from Column 4 at 1.32 m and 0.10 m above the pile-cap to represent the atmospheric zone and

---

**Table 2. Corrosion current density and estimated corrosion rate for steel in concrete.**

<table>
<thead>
<tr>
<th>Corrosion current density, Icorr (µA/cm²)</th>
<th>Corrosion rate category</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 0.1</td>
<td>No corrosion expected</td>
</tr>
<tr>
<td>0.1 to 0.5</td>
<td>Low to moderate rate</td>
</tr>
<tr>
<td>0.5 to 1.0</td>
<td>Moderate to high rate</td>
</tr>
<tr>
<td>&gt; 0.1</td>
<td>High rate</td>
</tr>
</tbody>
</table>

---

Note 1: Theoretically, for steel, a corrosion current density of 1.0 µA/cm² is equivalent to a corrosion rate of 11.6 µm/a any particular target slump (e.g. ± 15 mm at a target slump of 80 mm). This amount will vary a little from mix design to mix design but for mixes containing a maximum sized aggregate of between 10 mm and 20 mm will be reasonable estimate. This appears to be a greater level of accuracy than many of the aggregate moisture measurement methods discussed in section 2.2 above.
tidal zone, respectively (Figure 2). Four incremental samples were taken to a depth of about 100 mm from the top face of the pile cap, between Column 4 and Column 5. Each concrete powder was collected and sealed in a plastic bag immediately after drilling, and the hole was cleaned by using an air puff. The chloride content results are shown in Table 3 and Figure 3. According to construction documents, the clear cover to reinforcing steel is 70 mm, which is indicated in Figure 3, in which the chloride content threshold for steel corrosion is indicated, i.e. 0.4% by cement mass. This value has been acknowledged as the threshold value, above which steel corrosion is likely to occur in reinforced concrete.

The chloride profiles obtained in this investigation are almost the same as those obtained in 2006 as shown in Figure 4, except that the first increment of pile cap showed a higher chloride content in 2009. This very high value may be due to contamination from the mud left on the surface of the pile cap, although the surface was scraped of deposits before sampling.

Diffusion parameters were calculated, with values listed in Table 3. The diffusion coefficient in the tidal zone of the column was calculated to be $D = 2.85 \times 10^{-12} \text{ m}^2/\text{s}$ for both investigations and $C_r = 0.78\%$ compared to 0.65\% in 2006. Similar results were obtained for the pile cap, with $D = 2.85 \times 10^{-13} \text{ m}^2/\text{s}$ for both investigations and $C_r = 1.39\%$ compared to 1.35\% in 2006. The chloride ingress and diffusion parameters represent the combined effect of the in-situ concrete and the acrylic coating applied immediately after construction.

These values indicate that the chloride profiles are reasonably reproducible, although they may vary somewhat due to sample location. For example, the sampling location for the splash zone of columns in 2006 and 2009 were 0.27 m and 0.10 m, above the top face of the pile-cap, respectively. The location of samples from the atmospheric zone was 1.12 m above pile cap in 2006 and 1.32 m in 2009, for which the chloride profile is somewhat lower. Nevertheless, it appears that the surface coating of the columns effectively slowed the chloride ingress in the tidal zone.

The chloride content at 70 mm depth into the column is still below the threshold of active corrosion (0.4% cement mass), whereas the reinforcement in the exposed parts of the pile cap is likely to undergo corrosion. Given the cover depth of 70 mm, analysis of the data (Figure 5) shows that the reinforcing bars in the tidal zone of columns is in a situation where slow corrosion may occur, and more active corrosion is likely in three to six years. The curves in Figure 5 represent the progression of the threshold chloride content into concrete with time.

Even though active corrosion may start in 3-6 years, it would take several years after that for the thickness of corrosion layer

Table 3. Chloride content of Pier 2 concrete.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Location</th>
<th>Depth from concrete to surface</th>
<th>Result</th>
<th>Diffusion parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Location</td>
<td>From (mm) To (mm) Mean (mm)</td>
<td>Cr (%)</td>
<td>C (%) D (m²/s)</td>
</tr>
<tr>
<td>H1</td>
<td>Col. 4</td>
<td>0 10 5</td>
<td>0.150</td>
<td>0.19 2.9E-12</td>
</tr>
<tr>
<td>H2</td>
<td>1.32 m</td>
<td>10 27 19</td>
<td>0.025</td>
<td></td>
</tr>
<tr>
<td>H3</td>
<td>above pile cap</td>
<td>27 35 31</td>
<td>0.028</td>
<td></td>
</tr>
<tr>
<td>H4</td>
<td></td>
<td>35 69 52</td>
<td>0.013</td>
<td></td>
</tr>
<tr>
<td>L1</td>
<td>Col. 4</td>
<td>0 13 7</td>
<td>0.667</td>
<td>0.78 1.3E-12</td>
</tr>
<tr>
<td>L2</td>
<td></td>
<td>13 27 20</td>
<td>0.410</td>
<td></td>
</tr>
<tr>
<td>L3</td>
<td>0.10m</td>
<td>28 53 41</td>
<td>0.212</td>
<td></td>
</tr>
<tr>
<td>L4</td>
<td>above pile cap</td>
<td>54 80 67</td>
<td>0.152</td>
<td></td>
</tr>
<tr>
<td>C1</td>
<td></td>
<td>0 7 4</td>
<td>2.495</td>
<td>1.39 4.1E-12</td>
</tr>
<tr>
<td>C2</td>
<td></td>
<td>7 32 20</td>
<td>0.954</td>
<td></td>
</tr>
<tr>
<td>C3</td>
<td></td>
<td>32 51 42</td>
<td>0.556</td>
<td></td>
</tr>
<tr>
<td>C4</td>
<td></td>
<td>53 97 75</td>
<td>0.509</td>
<td></td>
</tr>
</tbody>
</table>

Note: Chloride ingress and diffusion parameters represent the combined effect of the in-situ concrete and the acrylic coating applied immediately after construction.

Figure 2. Column 4 – Concrete powder sampling at the low level of the column (l) and Column 4 – Measuring spots and corrosion stains at the construction joint (r).
to build up to a level that would cause concrete cracking. Nevertheless, it would be important to intervene (eg use of a higher quality dedicated protective coating in the tidal splash zone area, cathodic prevention etc) before this stage is reached.

4.2 Half-cell potential

Half-cell potential was measured on two adjacent faces (left face and Melbourne abutment face) of Column 4. The 2006 investigation showed that, in terms of half-cell potentials, Column 5 was similar or less negative than Column 4; and the half-cell potentials of the pile cap was almost the same on the whole area between Columns 4 and 5. Therefore, the mapping was made only on Column 4, whereas for Column 5 and the pile cap, the half-cell potential measurement was performed on a number of representative spots.

The results for Column 4 are presented as a contour map (Figure 6). Compared with the results of 2006 investigation, the half-cell potential values are more negative (the average potential at the column base was -800 mV CSE, in 2009, compared to -700 mV of the 2006). Figure 6 shows that from the surface of the pile-cap to a height of about 1 m, the half-cell potential is more negative than -400 mV CSE indicating that corrosion is likely to occur. The half-cell potentials of Column 5 were: atmospheric zone (1.09 m) -312 mV CSE; 0.58 m height above pile cap, -467 mV CSE; and splash zone (0.30 m) -653 mV CSE.

The half-cell potential of the pile-cap on the left side of Column 4 was -950 mV CSE, and on the right side -945 mV CSE. This could partly be due to corrosion activity and partly due to lack of oxygen in the water-saturated concrete in this location.

4.3 Resistivity of concrete

Resistivity was measured on Column 4 and the pile cap. The locations and results are given in Table 4. The resistivity of concrete below the water line is below 10 kΩ-cm, where corrosion would be easy to proceed once it started. The resistivity results are similar to that measured in 2006.
Corrosion rate was measured on the left face of Column 4 and Column 5, on the same spots which were measured in the 1999 and 2006 investigations, as well as on the pile cap. The vertical reinforcing bar in the middle of the columns face was used for the measurements (diameter 32 mm according to drawings). For the pile cap, the measurement was performed over a transverse bar (20 mm) near Column 4. The results are presented in Table 5, and indicate that the corrosion activity is negligible in the atmospheric zone and low to moderate in the tidal and splash zones.

Figure 7 shows a comparison between the 2009 results and those of the 1999 and 2006 investigations. Figure 7 indicates that the corrosion current density (Icorr), and hence the corrosion rate of both columns has been increasing over the past 10 to 11 years.

It should be noted that corrosion rate monitoring undertaken in 2010 has confirmed this small increase in both columns, although in Column 4 an unusually "moderate-to-high" Icorr of 0.76 μA/cm² was recorded at the low location of 0.2 m from the pile cap.

At this stage this is considered to be a once off reading influenced by localised environmental conditions at the time of testing and as such it would have to be reconfirmed as part of more comprehensive future monitoring including chloride and other electrochemical testing.

<table>
<thead>
<tr>
<th>Component and face</th>
<th>Height above pile cap (m)</th>
<th>Resistivity (kΩ.cm)</th>
<th>Interpretation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column 4, left face</td>
<td>1.40</td>
<td>15.3</td>
<td>• 100 kΩ.cm - The resistivity will effectively stop corrosion</td>
</tr>
<tr>
<td></td>
<td>0.20</td>
<td>6.0</td>
<td>• 50 - 100 kΩ.cm - Low corrosion rate</td>
</tr>
<tr>
<td>Column 4, abutment face</td>
<td>1.40</td>
<td>73.0</td>
<td>• 10 - 50 kΩ.cm - Moderate to high corrosion rate when steel is active</td>
</tr>
<tr>
<td></td>
<td>0.20</td>
<td>2.2</td>
<td>• &lt; 10 kΩ.cm - Resistivity is not the controlling factor</td>
</tr>
<tr>
<td>Pile cap, top face</td>
<td>0</td>
<td>1.8</td>
<td></td>
</tr>
</tbody>
</table>

Table 4. Resistivity of concrete.

<table>
<thead>
<tr>
<th>Element</th>
<th>Location, height above pile cap (m)</th>
<th>Corrosion current density (Icorr) (µA/cm²)</th>
<th>Corrosion rate (µm/year)</th>
<th>Interpretation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column 4</td>
<td>1.22</td>
<td>0.027</td>
<td>0.3</td>
<td>No corrosion expected</td>
</tr>
<tr>
<td></td>
<td>0.72</td>
<td>0.117</td>
<td>1.4</td>
<td>Low to moderate rate</td>
</tr>
<tr>
<td></td>
<td>0.20</td>
<td>0.158</td>
<td>1.8</td>
<td>Low to moderate rate</td>
</tr>
<tr>
<td>Column 5</td>
<td>1.09</td>
<td>0.023</td>
<td>0.3</td>
<td>No corrosion expected</td>
</tr>
<tr>
<td></td>
<td>0.58</td>
<td>0.061</td>
<td>0.7</td>
<td>No corrosion expected</td>
</tr>
<tr>
<td></td>
<td>0.30</td>
<td>0.090</td>
<td>1.0</td>
<td>No corrosion expected</td>
</tr>
<tr>
<td>Pile cap</td>
<td>Transverse bar</td>
<td>0.229</td>
<td>2.7</td>
<td>Low to moderate rate</td>
</tr>
</tbody>
</table>

Table 5. Corrosion rate of reinforcing steel.

4.4 Corrosion rate of reinforcing steel
Corrosion rate was measured on the left face of Column 4 and Column 5, on the same spots which were measured in the 1999 and 2006 investigations, as well as on the pile cap. The vertical reinforcing bar in the middle of the columns face was used for the measurements (diameter 32 mm according to drawings). For the pile cap, the measurement was performed over a transverse bar (20 mm) near Column 4. The results are presented in Table 5, and indicate that the corrosion activity is negligible in the atmospheric zone and low to moderate in the tidal and splash zones.

Figure 7 shows a comparison between the 2009 results and those of the 1999 and 2006 investigations. Figure 7 indicates that the corrosion current density (Icorr), and hence the corrosion rate of both columns has been increasing over the past 10 to 11 years.

It should be noted that corrosion rate monitoring undertaken in 2010 has confirmed this small increase in both columns, although in Column 4 an unusually "moderate-to-high" Icorr of 0.76 μA/cm² was recorded at the low location of 0.2 m from the pile cap.

At this stage this is considered to be a once off reading influenced by localised environmental conditions at the time of testing and as such it would have to be reconfirmed as part of more comprehensive future monitoring including chloride and other electrochemical testing.
4.5 Carbonation of concrete

The inner surface of each drilled hole was cleaned by water, dried by tissue paper, and then sprayed with the phenolphthalein solution. The indicator turned the exposed surface into pink colour, i.e., no carbonation, and the stone closest to the column surface remained this colour during the rest of the in-situ investigation.

As described in the methodology section, the carbonation depth was estimated according to the mass of CO₂ released from the concrete powder samples. The carbonation depth of column concrete in the atmospheric zone is 6 mm, and that in the tidal zone is 1 mm. These carbonation depths which have also been inhibited by the acrylic anticarbonation are very low and indicate that carbonation will not be a concern with respect to reinforcement corrosion in the tested areas.

4.6 Other test results including VPV of the insitu concrete

Table 6 summarises the results obtained on the concrete samples for carbonation, water absorption, VPV, density and compressive strength. It can be noted that the VPV test results (~ 15.5%) represent the quality of the concrete after eight years and therefore it is estimated that the initial 28 day VPV (ie initial permeability performance) of the concrete would have been in the order of 17.5%. This would represent an insitu quality performance equivalent to a concrete grade of VR330/32 to VR400/40 as specified in the Section 610 of the required quality performance of a concrete grade VR450/50 which is essentially what was specified for use in the construction of the concrete piers for the bridge. This outcome supports the fact that the acrylic surface coating applied to the columns during construction effectively slowed the chloride ingress in the tidal zone. This is supported by the performance of this coating as discussed in Section 5 and shown in Figures 8 and 9.

4.7 Adhesion and thickness testing of acrylic coating applied onto Lynch’s Bridge

The original decorative/anticarbonation acrylic coating was applied onto the Lynch’s Bridge substructure prior to the standardisation of the VicRoads concrete coating specification. The original coating was applied in two coats with a variable dry film thickness (DFT) of 100 to 140 micron.

The coating adhesion strength testing was undertaken using a pull-off tester with 50 mm diameter aluminium dollies and was determined to be in the order of 3.7 MPa which is unusually high for this type of coating. It is possible that the effects of the epoxy adhesive which was left in place with the acrylic coating may have been more effective than originally anticipated.

The thickness of the coating was examined by placing the pulled-off pieces under microscope and compared with a 0.08 mm (1 pt) line, although the accuracy of such determination may be in question if the coating material remnants attached to the aluminium dollies do not remain intact. The insitu coating thickness using this method of determination ranged between 30-100 μm (compared to original DFT in the order of 100-140 μm). It is considered that the insitu use of a paint inspection gauge consisting of a

![Figure 8. Coated slabs (including acrylic coating) in exposure environment.](image)

Table 6. Results of various laboratory tests.

<table>
<thead>
<tr>
<th>Core Sample (Fig. 1)</th>
<th>Carbonation depth (mm)</th>
<th>Water absorption (%)</th>
<th>Volume of Permeable Voids (VPV) (%)</th>
<th>Density (kg/m³)</th>
<th>Compressive Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lyn 1</td>
<td>&lt; 1</td>
<td>6.7</td>
<td>15.7</td>
<td>2248</td>
<td>-</td>
</tr>
<tr>
<td>Lyn 2</td>
<td>&lt; 1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Lyn 3</td>
<td>&lt; 1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>54</td>
</tr>
<tr>
<td>Lyn 4</td>
<td>6.0</td>
<td>6.7</td>
<td>15.6</td>
<td>2263</td>
<td>63.5</td>
</tr>
<tr>
<td>Lyn 5</td>
<td>2.0</td>
<td>6.6</td>
<td>15.3</td>
<td>2235</td>
<td>71.5</td>
</tr>
</tbody>
</table>

The corresponding 28 day estimate of VPV is in the order of 17.5% which represents the initial permeability condition and therefore microstructure performance of the original concrete.
Laboratory 6. Each of four slabs was coated with one of the coatings and a set of five concrete slabs was prepared in a

5.1 Sample, preparation, testing and exposure environment

To examine the performance of a number of protective coatings, a set of five concrete slabs was prepared in a Laboratory. Each of four slabs was coated with one of the trial coatings on all faces in accordance with the material data sheet requirements. One of the four slabs was coated with the same acrylic coating previously applied to the inbound Lynch’s Bridge over the Maribyrnong River. The fifth slab was left uncoated as a Control sample. The concrete was of 40 MPa Grade, produced using a mix design which typically meets the VicRoads V400/40 specification (typical 400 kg/m³ Type GP cement @ w/c <0.45) and cured to 14 days age in wet hessian. This reflects typical current practice for construction of new bridges in saline exposure conditions in Victoria.

The slabs were positioned vertically, secured to two pier columns of the Bunyip River Bridge on the South Gippsland Highway (Figure 8). The environment is a saline tidal estuary, and varies from tidal to splash zone exposure of up to 2.5 m with seasonal variation of the river level.

After curing of the coatings, the adhesion strength of the film forming materials was determined. Chloride profiles were obtained by grinding of core samples retrieved from the slabs at 6 and 16 months exposure, at the same depth increments as used in the laboratory testing. Only the latter results are discussed here for space reasons.

5.2 Results

Adhesion strength results are recorded in Table 7. In all cases, the values obtained and modes of failure were comparable to those obtained on the laboratory plaques.

Figure 9 shows the chloride profiles obtained from core samples taken at 16 months exposure. All of the coatings showed measurable penetration of chloride at that stage, however the film forming coatings are acting as effective barriers with Efficiency Indices above 96%. The acrylic coating (same as that applied at Lynch’s Bridge) appeared to be performing better insitu than in the laboratory tests. This was attributed to lower temperatures on-site and the exposure being wet/dry cycles rather than continuous wetting, the former being a more favourable type of exposure for this generic type.

The relatively high penetration in the silane treated slab is attributed to the effect of shallow immersion head pressures during tidal periods. The observed penetration had occurred prior to the six month sampling, little change in the profile being seen between the two samplings. It is considered that the concrete became essentially saturated relatively quickly, the incoming water carrying chloride salts, but it was subsequently in an equilibrium state where little mass transfer takes place across the silane treated zone during each successive wet cycle.

Using Ficks Second Law of Diffusion, the chloride penetration coefficient of the Control slab was estimated to be $3.5 \times 10^{-12}$ m²/sec at six months exposure, $1.4 \times 10^{-12}$ m²/sec at 16 months. This decrease was expected, and it is considered that the 16 month value is indicative of long term performance.

6.0 EFFECTIVENESS OF ACRYLIC COATING USED AT LYNCH’S BRIDGE

A comparison of chloride diffusion coefficients between the concrete used in the control slab of previous coating research work and the combined effect of the insitu concrete and

Figure 9. Chloride profiles from site test slabs after 16 months exposure.

Table 7. Adhesion strength results, site exposure slabs (6).

<table>
<thead>
<tr>
<th>Coating</th>
<th>Cementitious</th>
<th>Epoxy</th>
<th>Acrylic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trowelled Face</td>
<td>0.9 MPa, Adhesive</td>
<td>1.4 MPa, Substrate</td>
<td>0.5 MPa, Adhesive</td>
</tr>
<tr>
<td>Plywood Formed Face</td>
<td>1.0 MPa, Adhesive</td>
<td>2.3 MPa, Substrate</td>
<td></td>
</tr>
</tbody>
</table>
7.0 DISCUSSION ON FUTURE PERFORMANCE

The corrosion rate measured in the 1999 investigation is still low to moderate. The chloride diffusion coefficients determined from the chloride profiles of this investigation are identical to those obtained in 2006, while the maximum chloride content at the concrete surface is higher. The chloride content at the vicinity of the main reinforcing steel is below the “threshold level” of 0.4% chloride by cement mass. The estimation of the chloride ingress indicates that the threshold level will reach the reinforcing bar in three to six years. As stated before, it would take several years after that for the thickness of corrosion layer to build up to a level that would cause the concrete to develop cracking. Nevertheless, it would be important to intervene (eg higher quality protective coating in the tidal splash zone area, cathodic prevention etc) before this stage is reached.

Half-cell potential, resistivity and corrosion current density results all indicate that the reinforcing steel in the lower part of the columns (about 0.5 m above pile cap) is subject to very slow corrosion at present, despite the negative half-cell potential values. The half-cell potential values for the pile cap are very negative, although this is probably due to the lack of oxygen in the saturated concrete rather than aggressive corrosion. The corrosion activity could be low, as there is no visual evidence of corrosion. Thus it is still too early to confidently estimate the time of major deterioration for this structure.

Changes in the deterioration factors should be monitored in about five years when the threshold chloride content would be likely to reach the steel bars in the tidal and splash zones of columns. It may be necessary to establish the actual cover depth of the columns for a better estimation of the time when visible cracking may occur in concrete. The carbonation depth is very low and would not influence the durability of the structure in the foreseeable future.

It is considered that the acrylic coating which was applied to the concrete surface in the early stages of exposure (DFT in the order of 100-140 μm) contributed significantly to slowing down the ingress of chlorides into the concrete and lowering the potential for corrosion activity. However, it is likely that the effectiveness of the acrylic coating has been reduced and while the bond appears to be mostly good, the chemical structure of the material is unlikely to be performing fully after 19 years in the exposure environment as it is approaching the end of its service life. As such a higher quality protective coating (ie epoxy coating suitable for tidal exposure conditions in accordance with the requirements of Section 610 5) should be applied in the tidal zone in a timely manner both on the pile caps and the lower portions of the columns.

The current testing and monitoring of the 19 year old Lynch’s Bridge suggests that the cement only concrete (ie without any SCMs used) continues to provide reasonable protection for the structure in such an aggressive saline environment. However, it must be emphasised that the acrylic surface coating (which is not a dedicated protective coating against chloride ingress) applied to the columns effectively helped to slow the chloride ingress in the tidal zone. This result further supports the multi-level approach to durability adopted by the VicRoads specification Section 610 5 which includes the use of additional high quality protective barriers against the ingress of chlorides, and supported by strict specification and quality control measures as an effective combination suitable for structures situated in aggressive exposure conditions.

8.0 CONCLUSION

Two columns and the pile cap of Pier 2 of Lynch’s Bridge (inbound carriageway for Smithfield Road) were assessed for the corrosion state of the reinforcing steel. The chloride profiles, half-cell potentials, resistivity and corrosion rate determined in the 2009 investigation are similar to those obtained in previous investigations over the past 11 years, although some values have slightly increased in terms of likelihood of steel corrosion. Corrosion rates measured in the
1999 investigation are still low to moderate and furthermore the chloride diffusion coefficients in 2009 are identical to those from 2006.

The main outcomes of the Lynch’s Bridge investigation/monitoring can be summarised as follows:
(a) Early application of a coating provided enhanced performance, even though it was not a dedicated protective coating.

• The acrylic coating contributed significantly to slow down the ingress of chlorides into the pier concrete and lower the potential for earlier corrosion activity.

• However, the effectiveness of the acrylic coating has been reduced.
  – Whilst the bond is mostly good, the coating is unlikely to be performing fully after 19 years in the exposure environment as it approaches the end of its service life.

• Higher quality protective coating should be applied in the tidal zone in a timely manner.
  – Epoxy coating for tidal exposure as per VicRoads Section 610.5.
  – Both on pile caps and lower portions of columns – up to 1 m high.

(b) The bridge monitoring has reinforced the durability strategy adopted by VicRoads since 1993 (ie introduced after the construction of Lynch’s Bridge) and the enhanced durability provisions incorporated in its design drawings and specifications (ie Section 610.5 etc). This durability strategy which has been used successfully over the years in the construction of bridges such as the Patterson River Bridge (Nepean Highway over Patterson River at Carrum, 1994/95) and the North Arm Bridge at Lakes Entrance (1995) include the following which is in contrast to the Lynch’s bridge construction.

• Strict Specification Requirements
• High Performance SCMs Concrete
  – Low VPV (Volume of Permeable Voids) – impermeable
  – Chemical and Dimensional Stability
• Multi-level Protection Approach & Other Durability Provisions
  – Protective coating systems
• Improved Construction Practices
• Attention to Technical/Practical Process Interaction – Ensure Compliance
• Ongoing Testing and Monitoring – Excellent Performance

9.0 ACKNOWLEDGEMENT
The authors wish to thank VicRoads for permission to publish this paper. The views expressed in this paper are those of the authors and do not necessarily reflect the views of VicRoads.

REFERENCES
Self compacting concrete for superior marine durability and sustainability

Len McSaveney – Golden Bay Cement, New Zealand
Frank Papworth – BCRC, Western Australia
Michael Khrapko – CBE Consultancy, New Zealand

Abstract: Concrete made with a blend of Portland Cement, microsilica and fly ash, has for some time been recognised as delivering superior long-term durability in marine structures. These triple-blend binders are increasingly being used for major structures internationally, but have only recently been tested with New Zealand-sourced materials. The ability to use local materials is of course one of the key sustainability attributes in favour of the use of high-performance concrete for major structures. The impact of blended cements on sustainability is multi-faceted: not only is the durability enhanced for a long, low-maintenance life, but the structure’s carbon footprint and embodied energy are also substantially reduced.

In addition to the normal concrete attributes that can so significantly influence durability in a marine environment, the fine-end particle size-grading achieved by the use of selected cement and pozzolanic fillers in such high-performance concrete mixes, means that the binder combination makes an ideal medium for self-compacting concrete. This enables the contractor to take advantage of all the quality improvements; the environmental benefits of a clean, quiet site; and the cost-saving advantages that self-compacting concrete can bring to a well-managed construction project. The boost in productivity can be quite outstanding and as vibration during casting is not required, durability risks such as displacement of reinforcement, over or under compaction, and honey-combing at regions of congested reinforcement are also virtually eliminated. This allows tighter tolerances on bar locations to be used in the durability modelling for a hundred-year design life.

The application of these insights to a recent “Design and Construct” contract for the Tauranga Harbour Link project, in New Zealand, has verified the cost advantages of smarter concrete technology – when it is backed by rigorous performance-based testing and detailed durability modelling.

1.0 INTRODUCTION

The Tauranga Harbour Link duplicates an existing Harbour Bridge and includes approach ramps and over-bridges (Figure 1) to carry traffic to the Port of Tauranga – connecting the coastal main highway, via Mt Maunganui, to the motorway routes through the city of Tauranga. The tender for construction was let as a design-build contract in 2007.

The winning bid was based on the use of high durability, self-compacting concrete (SCC); as permitted by NZS 3101:2006, with specific durability modelling. Recent international research had indicated that a hundred year design life could be achieved with 40 mm of concrete cover, but local New Zealand materials had not yet been tested.

A triple-blend binder, consisting of HE Cement, Class C fly ash and natural geothermal microsilica makes a very good SCC – at the binder content that was required. The contractor was therefore able to set up the site-precasting yard and formwork to take full advantage of self-compacting concrete and to evaluate its full impact on cost, and on the environmental constraints at the site.

The twin technologies of the high-durability triple-binder paste, and self-compacting concrete proved to be a winning combination. The cost advantage to the successful design-build team was 20% of the bid price: a cost reduction of some $20 million over the conventional concrete design.

2.0 EXPOSURE CONDITIONS

The bridge is located within Tauranga Harbour on the east coast of New Zealand (Figure 2). The height of most of the structure far exceeds the wave height in this protected location and design was generally based on a code exposure of no splashing, but high deposition of wind-blown salts.
The surface chloride levels were discussed between the client’s engineers and the design engineers, to set the design basis for the 100-year life. The project’s durability consultant BCRC had also by that stage identified an extreme durability condition for the new bridge, which was not included in NZS 3101; or in the client’s design brief. This was the low soffit of the precast concrete T-Roff beams that were within 300 mm of extreme tides (Figure 3b). With wave splashing, droplets of water could hang on the soffit, giving extended time for chloride ions to soak in.

The T-Roff beam (Figure 3) has become a de facto industry standard in New Zealand, as it has in Australia. The cross-section copes easily with the typical span ranges of easily transported pretensioned, precast sections and simplifies the onsite deck construction. Moulds are simple to construct and easily adjusted for variable lengths and end skews.

### 3.0 TYPICAL T-ROFF BEAM DETAILING

For this project, the ability to reduce the concrete cover minimised the self-weight of the beams, allowing the units to span further while remaining within the lifting capacity of available cranes. This resulted in a dramatic reduction in the cost of the deep pier foundations and was responsible for a significant part of the total cost saving with this alternative.

The reduction in concrete volume, and the use of pozzolanic fillers, also significantly reduced the carbon footprint and the embodied energy of this design.

### 4.0 CONCRETE PROPERTIES

#### 4.1 Mix Designs

An early decision was that self-compacting concrete, with a 60 MPa, 28-day cylinder strength, would be the basis of the design. For the pretensioned T-Roff beams, the overnight strength after accelerated curing would need to be 35 MPa; while for the incrementally launched box girder, the required strength after two days of ambient curing was 30 MPa.

The beam cross sections and reinforcement details were not particularly complex (Figure 3), so an average slump flow of 680 mm was selected, with a T500 value not exceeding 3 seconds. The selected coarse aggregate was local crushed greywacke of 19 mm maximum size. The sand component was a mixture of greywacke manufactured fines, and marine quartz sands. The Triple-blend binder consisted of High Early Strength Portland cement, a local Class C fly ash, and natural geothermal microsilica with proportions of 62/30/8.

A comprehensive development program was initiated in Golden Bay Cement’s laboratory at the Portland cement manufacturing plant. The testing procedure for each successful laboratory batch of concrete consisted of the following:

- Slump flow test.
- Time for 500 mm slump flow spread.
- L-Box, for passing ability.
- Air content and wet concrete density.
- Compressive strength in 16 hours (cured for 10 hours at 650 °C), 1-day and 28-day.
After enough test data had been collected for the required types of self-compacting concrete, the three best performing mixes were selected for each type for further durability, shrinkage and creep testing. At the end of the mix developing program, two SCC mixes, for two different exposure conditions were offered to the contractor.

These two concretes had the characteristics shown in Table 1.

To achieve the strength requirements a target w/c ratio of 0.32 was set. The accuracy of batching meant that the maximum w/c ratio could be 0.34. Durability analyses indicated that the design life reduced by 20% when the w/c ratio increased from 0.32 to 0.34 hence the durability design was based on a maximum w/c ratio of 0.34. The concrete mix finally selected for the most severe exposure, following trials of various mixes, is referred to in this paper as “Mix M”.

“Mix H” was selected for the rest of the superstructure concrete that was subjected to less severe exposure conditions.

4.2 Curing

Concrete, particularly that incorporating fly ash and microsilica, is highly sensitive to curing. Often 14 days curing is stipulated for fly ash concrete. In this case seven day curing was proposed and a small allowance was made on predicted concrete properties. This allowance was based on research into the effect of curing on concrete strength (Haque, 1990) and then related back to an effective w/c ratio.

4.3 Chloride ion permeability

Three heat-cured representative samples of “Mix M” were tested by BCRC (using the NT Build 443 Method and gave a diffusion coefficient of 1.37 x 10^{-12} m^2/sec. For chloride diffusion tests, two months was allowed for curing before commencing chloride exposure as experience suggested that poor early results would be obtained due to the slow hydration of fly ash if 28 day curing was adopted. Three months is generally allowed for exposure prior to chloride profile testing when using high performance concrete to ensure that the chloride profile is sufficiently established. The resultant five month lead time was untenable from a construction perspective. Hence modelling based on expected chloride diffusion results was undertaken and the diffusion values were subsequently confirmed by testing. The bid team and their consultants drew heavily on the experience of BCRC, and on Golden Bay Cement’s experience with similar concrete mixes.

4.4 Chloride ion permeability

Where concrete is subject to wetting and drying, water (and salts dissolved in it) is drawn into the concrete by capillary action. As the initial water ingress is relatively fast (ie a few millimetres in a few minutes with conventional concrete splashed by seawater) chloride penetration into the sorption layer is rapid. Between splashes, the water dries out but the chlorides remain. Subsequent splashes provide further doses of chlorides to that sorption layer. As this build-up of chlorides in the sorption layer occurs quickly it must be deducted from the cover when doing the diffusion analysis based on Fick’s Law. The thickness of the sorption layer is a function of the concrete sorptivity and the time of wetting. On vertical surfaces, water runs off so a wet time of 15 minutes was used in the analysis. On horizontal surfaces, water droplets hang on so for “low soffits” a wet time of 200 minutes was used. The significance of these parameters is seen in Table 2. For the low-soffit exposure condition (and in the splash zone on the beam sides) this parameter was critical. Fortunately, recent research conducted by Professor Peter Bartos (Wenzhong, 2003) had discovered that the undisturbed interfacial zone between the binder paste and the coarse aggregate results in well-designed SCC mixes which have amazingly low sorptivity.

<table>
<thead>
<tr>
<th>Paste System</th>
<th>Sorptivity (mm/min^{0.5})</th>
<th>Calculated sorption layer thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>15 min wet time</td>
</tr>
<tr>
<td>GP cement, w/c = 0.50</td>
<td>0.38</td>
<td>15</td>
</tr>
<tr>
<td>GP cement, w/c = 0.40</td>
<td>0.28</td>
<td>11</td>
</tr>
<tr>
<td>8% Microsilica w/c = 0.40 (Vibrated)</td>
<td>0.07</td>
<td>3</td>
</tr>
<tr>
<td>8% Microsilica w/c = 0.32 (Vibrated)</td>
<td>0.05</td>
<td>2</td>
</tr>
<tr>
<td>Mix M (SCC)</td>
<td>0.036</td>
<td>1</td>
</tr>
</tbody>
</table>
Table 3. Cover requirements from durability modelling.

<table>
<thead>
<tr>
<th>Location</th>
<th>Exposure</th>
<th>Concrete cover</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low soffit</td>
<td>Low soffit in splash zone – rebar</td>
<td>43mm</td>
</tr>
<tr>
<td>T-Roff soffit</td>
<td>Low soffit in splash zone – prestress</td>
<td>55mm</td>
</tr>
<tr>
<td>T-Roff webs</td>
<td>Near to vertical – splash zone</td>
<td>55mm</td>
</tr>
<tr>
<td>T-Roff internal</td>
<td>Drained and vented, but no access</td>
<td>25mm</td>
</tr>
</tbody>
</table>

values. Tests at BCRC confirmed a sorptivity value of 0.036 mm/min^0.5 for Mix M, and the model was adjusted to accept this low number.

4.5 Resistivity
The modelling of reinforcement corrosion includes two phases:
• Initiation is the time for chlorides to reach the steel in sufficient quantity to initiate corrosion.
• Propagation is time from corrosion initiation to concrete spalling at the stirrups, or to potential stress-corrosion failure of prestressing tendons.

The fib Bulletin 34 deals only with time to initiation. Other models (Life 365 and Concrete Society) make a set allowance for the propagation, regardless of the mix properties.

The model used recognises that the concrete resistivity varies significantly with w/c ratio and with the cement system, and notes that it is inappropriate to treat concrete with very different resistivities in the same fashion when assessing the propagation phase. Hence, the actual concrete resistivity was used to calculate the time to spalling, and a high safety-factor was applied to allow for the current uncertainties in this calculation method.

Saturated SCC concrete samples were tested using the four-probe Wenner Method, to give a worst case resistivity. The tight internal structure of the triple-binder, low w/c, low-bleed self-compacting concrete, gave a very high resistivity of 45.6 kΩ.cm.

4.6 Bar Spacers
A durability weakness can exist where plastic spacers contract on cooling to form an ingress plane which results in localised high chloride ion penetrability. Grout and mortar spacers are often made using GP cement and with a w/c ratio lower than the surrounding concrete. In such cases the cement system and high paste volume can result in a spacer with far lower performance than the surrounding concrete. For this project, high-performance, fibre-reinforced concrete bar spacers of at least comparable durability to the adjacent concrete were used. These were positioned in compliance with BS 7973, to achieve a minimum concrete cover of spacer height minus 2 mm. Table 3 sets out the design cover requirements for the various parts of the structures, with the 2 mm placing tolerance deducted.

5.0 CONCRETE MANUFACTURE, BEAM PRODUCTION AND QUALITY CONTROL

5.1 Concrete manufacture
After completion of the mix development program at Golden Bay Cement’s laboratory, a series of full-scale trial mixes were conducted at a concrete plant located in the precasting yard. This was to evaluate the effects of variation in the quality of the mix materials, and production variations (batching accuracy, weather condition, etc) on the quality and consistency of the SCC. Only minor adjustments to the mix designs were required. For ease of batching and for rapid truck-mixing, the concrete supplier (Firth Industries) chose to use the slurry form of geothermal microsilica.

5.2 Beam production
The bridge beam production facility had been set up to maximise the productivity savings available with SCC: the delivery trucks drove along an elevated ramp, discharging their loads directly into the formwork. The time to discharge a 4.0 cubic metre load of concrete was typically 15 to 20 minutes. This made it possible to reduce the required open time of the concrete from 45 to 25 minutes, presenting an opportunity for further admixture cost savings. Both of the SCC mixes exhibited very good flowing characteristics. Moulds were quickly filled without vibration, and with minimal site labour. The lack of mechanical vibration meant the noise level limits at the site boundaries were never exceeded and there were no constraints on the hours of the day when concrete could be placed. Surface finishes were excellent, requiring very little remedial work. An anti-graffiti coating was applied to the beams prior to their installation because of their easy access by boat, but as this coating was an after-thought the beneficial effect of it was not modelled in the durability analysis.

5.3 Quality control
A dedicated mobile concrete batching plant was erected at the precast yard and started producing conventional concrete a few months prior to the commencement of concrete supply for the beam production. This allowed for a number of SCC trials to be conducted, and also allowed for Certification of the plant in accordance with NZS 3104:2003 Specification for Concrete Production. The certification warranted the plant’s capability of producing quality concrete, within standard tolerances.

Every batch of SCC was checked for slump flow at the batching plant and then at the beam mould before discharging the concrete. All tests were within the specified tolerances.

Part of the beam production quality control was to monitor concrete curing temperatures and release strengths. Temperature was monitored by a series of temperature probes and controlled by circulating hot water through the rectangular tubes attached to the outside of the forms.

5.4 Durability modelling and test results
The design life was calculated as time to corrosion initiation plus the time from initiation to spalling. Time to initiation was calculated using the fib Bulletin 34 formula:
Table 4: Calculated lives for Tauranga Bridge.

<table>
<thead>
<tr>
<th>Durability Item</th>
<th>Time to activation (Yrs)</th>
<th>Time to spalling (Yrs)</th>
<th>Design life (Yrs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low Soffit 43mm min cover</td>
<td>113</td>
<td>39</td>
<td>152</td>
</tr>
<tr>
<td>Low Soffit 38mm cover</td>
<td>68</td>
<td>35</td>
<td>103</td>
</tr>
<tr>
<td>Low Soffit prestress 55mm min cover</td>
<td>156</td>
<td>-</td>
<td>156</td>
</tr>
<tr>
<td>Web Splash 38mm min cover</td>
<td>92</td>
<td>35</td>
<td>127</td>
</tr>
</tbody>
</table>

\[ C(x=a,t)=C_o+(C_s-\Delta x) \left[ 1-\text{erf} \left( \frac{a-\Delta x}{2\sqrt{D_c}t} \right) \right] \]

Where
- \( a \) = minimum concrete cover [mm] ie nominal cover, less placing tolerance
- \( \Delta x \) = sorption layer thickness (see section 4.3)
- \( t \) = exposure time [years]
- \( C_{i(x,a,t)} \) = chloride level at the bar at time \( t \) = \( C_l \) - threshold for time to initiation
- \( C_o \) = initial chloride content [wt.-%/c], ie maximum permitted at time of delivery
- \( C_{S(x,t)} \) = chloride content at surface and through sorption layer, \( \Delta x \) [wt.-%/c]
- \( x \) = Depth with corresponding chloride content, \( C(x,t), \) [mm]
- \( D_c \) = Diffusion coefficient at each time interval
- \( D=DT \)
- \( m \) = a factor relating diffusion at any time, to initial diffusion

The chloride threshold level was taken as 0.77% by weight of cement, for passive reinforcement. Although high, compared to some standard threshold values quoted, this includes a significant safety-factor based on threshold levels found by Pettersson (1998) and Polder (1996) for low w/c ratio concrete. The activation level for prestressing steel corrosion was taken as 0.4% by weight of cement.

The surface chloride level was taken as 7% by weight of cement for the low soffit and 5.5% by weight of cement for vertical surfaces in the splash zone based on the use of fly ash which tends to have a higher surface chloride level than GP cement. The very high surface chloride level for low soffits was based on the durability consultant’s experience, while the lower level for vertical surfaces is in keeping with published data (Bamforth, 1996) for splash zones. The diffusion coefficient was adjusted to allow for the in situ concrete temperature based on the Nernst-Einstein equation. The reduction in chloride diffusion with time \( (m) \) was based on Bamforth’s 0.62 value, for fly ash. This is a value commonly used in durability modelling around the world, due to the scarcity of long-term data. However, the \( m \) value is a very significant factor in the life calculation and there was significant concern that local materials may prove to be less effective in the long term than those used in Bamforth’s research. Application of a high safety-factor led to an actual \( m \) value of 0.43 being used for this bridge.

For passive reinforcement, the propagation phase was calculated from the corrosion current – being the potential difference between anode and cathode, divided by the resistance between the anode and cathode. Macro-cell corrosion between the top and bottom mats of reinforcement was assumed. The results gave corrosion rates consistent with Polder (1996) however recognising the uncertainty in many aspects of the calculations, a factor of 3 was applied to the calculated propagation phase. The propagation time for prestressing steel was taken as zero. The calculated lives are given in years, in Table 4. The models are of course not so precise, but considering the conservatism built into the various factors of safety, it was agreed that these may be taken as the minimum lives to be expected.

A risk assessment was undertaken using the AS 4360 risk matrix. The consequence of corrosion of reinforcement was found to be extreme and the likelihood “unlikely”. The associated resultant risk was “high” and unacceptable. Installation of corrosion monitoring probes and designing the bridge for future cathodic protection meant the consequence of failure of the durability design became “moderate” and the risk became “low”.

6.0 CONCLUSIONS

The successful completion of this project, on time and within budget, has demonstrated that self-compacting concrete (SCC) technology can deliver significant cost savings to a high-performance concrete project of this type, as well as significant environmental benefits over the lives of the bridges. The technology is ideally suited to a design and construct type of project, were the bid team can evaluate each of the cost-benefit attributes of the technology and make design choices to optimise the total cost savings.

The project was the first detailed appraisal of the use of SCC for marine durability to be undertaken in New Zealand. The cost advantages, while no surprise to the concrete technologists, amazed the bridge designers and the contractor. Conventionally vibrated concrete was very quickly ruled out as a viable option for the critical spans in this project.

Proving the durability performance of this high performance “Mix M” has enabled it to be used as a “Standard” high-durability mix, in other applications with similar exposure.

REFERENCES


Re-inventing construction
RUBY, I., RUBY, A. (Eds.)
Ruby Press, Berlin, 2010

How must architecture, engineering and construction evolve so that sustainability is automatically embedded in the way the built environment is designed, constructed, used and recycled? This challenging question was the nucleus of the 3rd International Holcim Forum held in Mexico City and inspiration for Re-inventing Construction. The 440-page book features articles and case studies by 38 internationally renowned architects, engineers and scholars including Keller Easterling, Bjørke Ingels, Anne Lacaton and Jean-Philippe Vassal, Amory Lovins, Elinor Ostrom, Jeremy Rifkin, Michel Rojkind, Werner Sobek, Michael Sorkin, and many others.

fib Bulletin 54: Structural Concrete Textbook on behaviour, design and performance,
Second edition, Volume 4: Design of concrete buildings for fire resistance, design of members, practical aspects
International Federation for Structural Concrete (fib)
Lausanne, Switzerland, 2010

The second edition of the Structural Concrete Textbook is an extensive revision that reflects advances in knowledge and technology over the past decade. It was prepared in the intermediate period from the CEP-FIP Model Code 1990 (MC90) to fib Model Code 2010 (MC2010), and as such incorporates a significant amount of information that has been already finalised for MC2010, while keeping some material from MC90 that was not yet modified considerably.

The updated textbook provides the basics of material and structural behaviour and the fundamental knowledge needed for the design, assessment or retrofitting of concrete structures. It will be essential reading material for graduate students in the field of structural concrete, and also assist designers and consultants in understanding the background to the rules they apply in their practice. Furthermore, it should prove particularly valuable to users of the new editions of Eurocode 2 for concrete buildings, bridges and container structures, which are based only partly on MC90 and partly on more recent knowledge which was not included in the 1999 edition of the textbook.

Design and Control of Concrete Mixtures,
15th edition
KOSMATKA, S. H. AND WILSON, M. L.
Portland Cement Association, Skokie, ILL, 2011

This reference on concrete technology covers the fundamentals of freshly mixed and hardened concrete. This edition discusses sustainability; durability; materials for making concrete, such as portland cements, supplementary cementing materials, aggregates, water, admixtures, fibres, and reinforcement; procedures for mix proportioning, batching, mixing, transporting, handling, placing, consolidating, finishing, and curing concrete; precautions necessary during hot- and cold-weather concreting; causes and methods of controlling volume changes; commonly used control tests for quality concrete; special types of concrete, such as high-performance, lightweight, heavyweight, no-slump, roller-compacted, shotcrete, mass concrete, and many more.

Concrete floors and moisture, 2nd ed.
KANARE, H. W.
Portland Cement Association, Skokie, ILL, 2008

Unwanted moisture in concrete floors annually causes millions of dollars in damage to buildings in the US. Problems from excessive moisture include deterioration and de-bonding of floor coverings, trip-and-fall hazards, microbial growth leading to reduced indoor air quality, staining, and deterioration of building finishes.

This publication discusses sources of moisture, drying of concrete, methods of measuring moisture, construction practices, specifications, and responsibilities for successful floor projects. The second edition incorporates a brief discussion of terrazzo issues, an extended discussion of issues with ASTM F1869 moisture emission testing, additional references on drying times of lightweight concrete, and updates to the sources of supplies and standards.

The sustainable concrete guide: applications
SCHOKKER, A. J.
U.S. Green Concrete Council, 2010

A companion resource to the “The Sustainable Concrete Guide – Strategies and Examples”, ”The Sustainable Concrete Guide – Applications” provides readers with specific sustainable benefits of concrete’s various applications to assist in selecting/specifying concrete materials and products. Also included are tips and case studies on specifying concrete materials, constructing for sustainability, integrating into sustainable structures, and navigating “green” codes and standards.

SP-274: Fiber reinforced self-consolidating concrete: research and applications [CD-ROM]
ALDEA, C.M., FERRARA, L.
American Concrete Institute, Farmington Hills, MI, 2010

This CD-ROM contains eight papers that were presented at technical sessions sponsored by ACI Committees 544 and 237 at the 2009 ACI Fall Convention in New Orleans, US. The topics of the papers range from mixture composition and influence of fibres on the fresh state performance, to the connection between fresh state behaviour, fibre dispersion and orientation and mechanical properties of the fibre-reinforced composite, to full-scale testing and development of prototype applications for structures and infrastructures.
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These companies and people recently became members of the Concrete Institute.

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- Janet Murphy
- Michael Netherton
- Robin Netterfield
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- Cihan Onay
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- Rick Phillips
- Bryan Pisani
- Sudharshan Naidu Raman
- Andrei Rotaru
- Francisco Santana
- Nur Shams
- Lee Siong
- Robert Sirasch
- Russell Smith
- Scott Smith
- Khin Soe
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- Andreas Wang
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**DTMT Construction Co**
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**Etec Consultants**
**Georgiou Group**

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**Hallett Concrete**
**ITLS-TWA Australia**
**KBR**
**Mahaffey Associates**
**Main Roads WA**
**Nuplex Construction Products**
**Peerless Industrial Systems**
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**The Reinforced Earth Company**
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**Wood & Grieve Engineers**
### Academic Institutions

- Curtin University of Technology
- Deacon University
- James Cook University
- Monash University
- Northern Melbourne Institute of TAFE
- Queensland University of Technology
- RMIT University
- Swinburne University of Technology
- University of NSW
- University of Sydney
- University of Southern Queensland
- University of Technology, Sydney
- University of Queensland
- University of South Australia
- University of Western Australia

### Bronze Members

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12 – 14 October 2011. Perth, WA.

CONCRETE 2011
BUILDING A SUSTAINABLE FUTURE

The Concrete 2011 Technical Program will be highlighted by International Invited Speakers:

Professor Ravindra Gettu: Indian Institute of Technology, Madras, India.
Linda Figg: CEO Figg Engineering Group, Tallahassee, Florida, U.S.A.
Martin Clarke: CEO British Precast, Leicester, U.K.