Sustainable Concrete Seminar

PROCEEDINGS

30 June 2000

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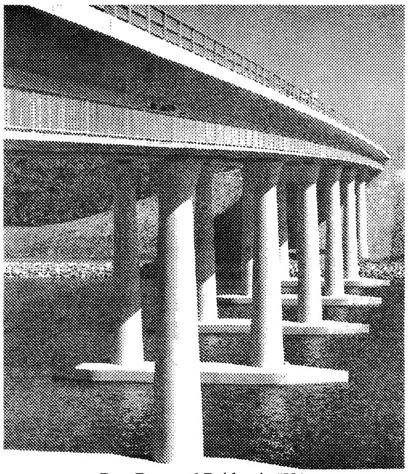

Port Bouvard Bridge in WA
Concrete In Australia (Vol21, No3, Oct 95)

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Rockhampton

Organiser

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PREFACE

Welcome to Central Queensland University for this one-day seminar on *Sustainable Concrete.* I hope you will find the day enjoyable as well as educational.

Concrete is evolving as the most popular construction material around the globe. Very high-strength, high-performance concretes and various polymer and fibre reinforced concretes are some of the state-of-the-art technologies. Pavements, buildings and hydraulic and marine structures that are built in concrete are often cited for their elegance, ease of construction and economy. However, with changes to the global environment, durability problems in concrete are attracting wider attention from most practising engineers and builders.

As organiser of the seminar, I would like to take this opportunity to thank the presenters who have given of their time to share their knowledge and expertise through the printed papers in these proceedings, and through their addresses to you today.

Finally, I would like to thank Kerrilyn Tomkins for her support as Seminar Secretary. Kerrilyn has made my job easier by taking care of all administrative tasks, including the compilation of these proceedings.

Rema Dhanasekar
Seminar Organiser

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AN OVERVIEW OF DURABILITY ISSUES AND REPAIR TECHNIQUES FOR CONCRETE STRUCTURES

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SUMMARY

The Queensland Department of Main Roads has a significant investment in concrete infrastructure in relation to Queensland's transport needs. The financial value of this investment is approximately $3 billion with the majority invested in concrete bridges followed by drainage structures and concrete roads. While our construction of concrete roads started to increase in magnitude in 1998 with the Pacific Motorway Project 104 years ago in 1896 the Lamington River Bridge was being constructed just south of Maryborough. This bridge is still functioning as access into Maryborough from the main highway and hence has achieved a functional life over 100 years. The purpose of this paper is to summarize the size of our concrete structure assets and discuss some of the important durability issues that have occurred over the last 100 years in relation to Main Road's concrete structures and our response to those events. The only way we can confidently project into the future is to have a sound understanding of the past. The information in this paper is targeted at enhancing this understanding.

INTRODUCTION

Historically asset management of our concrete structure assets has been demand based. As issues have arisen they have been researched and the fundamental cause of the problem established. Queensland Main Roads has made a significant contribution towards research and development which has had a major impact on the durability of our concrete structures. This research has also benefited the rest of Australia through technology transfer at National conferences and workshops. Once the primary cause of distress was established on a particular structure a detailed intervention strategy was designed and implemented including feedback if relevant into the design and specification stage for new structures. One unique advantage Main Roads has as an organization is being able to look across the complete spectrum of life cycle of a structure as detailed in Table 1.

Table 1 Summary of Structure Life Cycle

Stage	Description
1 – Conception	Design Concept – Specification
2 – Birth	Construction – Limitations
3 – Life	Maintenance – Access
4 – Death	Demolition – Safety

Historically it has been an important advantage to be able to collect and cycle information from one stage to another and hence optimize total life cycle management. Section 2 discusses the size of the concrete structure asset and Section 3 some of the issues that have arisen in the past.

SIZE OF QUEENSLAND MAIN ROAD'S CONCRETE ASSETS

Table 2 summarizes the approximate size of each asset in 2000. The estimate of the length of the concrete culverts is for comparison only and is based on the length along the flow of water in the drainage structures not chainage length as for bridges and roads.

Table 2 Summary of Asset Size

Asset Type	No	Length (km)
Timber Bridges	600	19
Concrete Bridges	2000	115
Concrete Culverts	7200	100
Concrete Roads	4	*120

* dual lane km

Concrete Bridges

The Main Roads bridge stock consists of approximately 2600 bridges constructed from timber, reinforced concrete, steel and prestressed concrete materials. The replacement cost of the bridge stock is approximately $2 billion with annual maintenance costs of $10 million for the timber bridges and $3 million for the concrete bridges. The timber bridges represent only 25% of the bridge stock but claim 80% of the maintenance expenditure. As a result approximately 20 timber bridges are being removed from the road bridge network each year.

Historically the first reinforced concrete bridge built was the Lamington River bridge into Maryborough in 1896. The first composite steel girder/reinforced concrete deck bridge was the Thompson River bridge on the Landsborough Highway near Longreach built in 1935. The earliest prestressed concrete deck unit bridge was the bridge over Tenthill Creek constructed in 1954. All these bridges are still in use except Tenthill Creek bridge which has been upgraded with a new structure. A new scheme to replace the Thompson River Bridge is under construction at the time of this paper presentation.

Concrete Roads

Table 3 summarizes the date of construction of the four plain concrete pavements (PCP) listed in Table 2.

Table 3 Summary of Plain Concrete Pavements (PCP)

Road Name	Year Constructed
Reedy Creek - Original Duplication	1982 1997
Caloundra – Sippy Creek	1986/87
Yandina	1997
Pacific Motorway	2000

All the above roads were built using slip form paving machines. Earlier concrete roads were built using hand placement techniques on Ipswich Road, Samford Road and the Bald Hills Flats. All of these earlier roads gave excellent service and have since been upgraded with new road designs. Hence the oldest road in Table 3 is currently 18 years of age making it a relatively new technology in Queensland in relation to the experience with the bridge stock.

MAINTENANCE ISSUES

Concrete Bridges

A number of maintenance issues have arisen within the bridge stock which have led to significant changes in design, material selection and specification. Table 4 lists the major issues that have occurred and the implication of them in other areas as detailed in Table 1.

In the area of alkali-silica reaction distress Main Road's has developed a Patented accelerated test procedure under its research and development portfolio. This test procedure is unique in its application to concrete mix designs and allows a comparison of control mixes containing GP cement and other optimized mixes containing fly ash and/or slag and silica fume.

Table 4 Summary of Significant Issues in Bridge Maintenance (Ref. 1)

Issue	Description
Chloride Diffusion in RC Piles	In marine environments chloride induced corrosion is prevalent in older structures of Class 20 to 30 Mpa due to an inadequate diffusion resistance intrinsic in that quality of concrete. New structures are designed with Class 50 Mpa and w/c < 0.4
Alkali – Silica Reaction (ASR) in prestressed piles and deck units (Ref. 2)	Longitudinal cracking 0.1 to 3.0 mm in width in prestressed concrete elements. The main reasons for the significant occurrence in prestressed concrete compared to reinforced concrete are the higher cement contents and the use of steam curing. The Main Road's Concrete Specification MRS11.70 (Ref. 3) requires the use of a minimum mass of fly ash (20%) to minimize this issue.
Carbonation induced corrosion of reinforced concrete elements eg RC piles and columns	Carbonation of the cover concrete by carbon dioxide in the atmosphere results in a reduction in pH of the pore solution surrounding the included reinforcement. With access to sufficient moisture corrosion will proceed but at a slower rate than occurs in the presence of chloride ions. The current approach is to provide concrete of sufficient quality that carbonation occurs at a very slow rate and resultant shallow depth.
Plastic cracking in slabs	Thin deck slabs of large area are at high risk of early plastic cracking. The use of aliphatic alcohol as an early protection agent is now mandatory for bridge decks in accordance with MRS11.70.
Delayed Ettringite Formation (DEF) in precast products (no proven case in Queensland at this stage)	DEF may occur when concrete is kept at a high temperature (> 75 deg Celsius) early in its hydration process and the SO3/Al203 ratio of the cement is above 0.7. This mechanism results in expansion and cracking of the concrete due to the amount of crystalline water contained in the ettringite compound. ASR and EF may be found in association however either may be the primary cause of the cracking with the other occurring as a secondary deposit of no real consequence. The MRS11.70 limits the maximum temperature of steam curing to 75 deg. C

Concrete Culverts

Main Road's has a large number of culverts as recorded in Table 2. These culverts exist in a range of environments from aggressive marine locations to very mild inland areas. Concrete culverts may take the form of RCP's, RCBC's or cast insitu.

(i) RCP's – Traditionally reinforced concrete pipes have been manufacture by the spinning process which has resulted in a very durable product in all environments. The issues relating to the poor performance of pipes all seem to be explained by cracking induced at the time of installation rather than problems with the manufacture of the pipes.

(ii) RCBC's – The major problem with RCBC's which occurred in the early 1970's was due to the use of calcium chloride as an accelerator in the precast industry. This chemical containing chloride ions has caused a significant rate of corrosion of a large number of box culverts both small and large in size. Main Road's currently has a significant liability regarding the detection and replacement of these defective culverts. One culvert in the Cairns District alone will cost over $500000 to replace. The existing MRS11.70 does not allow the use of chloride containing admixtures and the overall chloride ion content of the concrete mix is controlled by AS 1379 to 0.8 kg/m3.

(iii) Cast Insitu Culverts – These culverts have performed well in aggressive environments basically because the supporting columns have a very low amount of reinforcement. Other examples of carbonation induced corrosion of the suspended slab reinforcement may be found in inland environments. Overall these culverts have performed satisfactorily. They have not suffered from the calcium chloride issue identified with precast RCBC's

Concrete Roads

The first PCP built at Reedy Creek to Palm Beach yielded a rougher surface over some sections than the current generation of new pavements. Some longitudinal, acute angle and transverse cracking also occurred in this pavement due to a range of causes eg unplanned shrinkage and foundation settlement. The road at Sippy Creek has a crack in nearly every slab of the slow lane. The fast lane remains relatively crack free in comparison. No definitive single cause has been isolated for this cracking. However since every slab is cracked the external load must play a significant part in the observed cracking. Since the cracks occurred early in the life of the pavement fatigue of the pavement is unlikely as the cause. Hence possible causes could be overloading, excessive curling of the slab edges and foundation settlement. All of the observed cracks in the Reedy Creek and Sippy Creek pavements have been repaired by cross-stitching of the cracks. The cross-stitching provides a shear transfer mechanism across the cracks and prevents differential displacement and deterioration of the material adjacent to the cracks.

INVESTIGATION TECHNIQUES FOR CONCRETE STRUCTURES

A comprehensive range of analytical techniques is available today which enable a detailed analysis of the current condition of a concrete structure. Once the present condition has been established an estimation of the residual life of the structure can be made and assessed in relation to the forward planning for a new structure and possibly upgraded service. An accurate assessment of the economics regarding repair or replace is essential in today's tight funding situation. A forward estimate of liability regarding structure replacement is fundamental to sound investment planning regarding concrete structure management. Table 5 lists techniques which are commonly used in condition analysis.

Table 5 Investigation Techniques for Concrete Structures

No.	Technique	Purpose
1	Petrographic analysis	Assessment of source material origin. Detection of impurities and identification of any deleterious reactions
2	Cement content and w/c ratio	Audit of original mix design
3	Density and compressive strength	Audit of compaction achieved
4	Permeability of cover concrete by water or electrical methods	Assessment of durability of the structure in a given environment
5	Carbonation depth	Assessment of permeability of cover concrete to carbon dioxide gas
6	Chloride ion profile of cover concrete	Diffusion of chloride ions into concrete yields information on the current likelihood of corrosion and the residual life of the structure
7	Concrete surface potential and resistivity of the cover concrete	Non destructive tests yielding information on the probability of corrosion in a structure
8	Ultrasonic pulse velocity	Used for the detection of voids in concrete elements and hence the degree of compaction
9	Cover meter survey of an element	Assessment of depth of cover and conformance with design parameters
10	Sounding survey	Assessment of compaction uniformity
11	Crack mapping survey	Assessment of crack locations, type orientation and width.
12	Radar testing of concrete	Ground penetrating radar has been used on buildings and roads to assess the level of compaction and voids
13	Thermographic imaging	Detection of cracks and porous zones by heat diffusion. Hot and cold areas are thermographically imaged yielding a view of the heat flow patterns.
14	Scanning electron microscope (SEM)	Quantitative assessment of reactions in concrete eg ASR or DEF

REPAIR TECHNIQUES FOR CONCRETE STRUCTURES

A range of traditional and innovative repair techniques have been used by Main Roads for the repair of our concrete infrastructure. The most common form of distress in our reinforced concrete bridge substructures is corrosion of reinforcement due to the diffusion of chloride ions through the cover concrete. For prestressed concrete elements the most common form of distress is alkali-silica reaction resulting in significant longitudinal cracking which may then also allow chloride ions to penetrate through the cracks rather than the slower mechanism of diffusion. Table 6 lists the range of repair techniques in use and their area of typical application.

Table 6 Range of Repair Techniques Currently in Use

Distress	Concrete Element	Repair Technique	Description
Chloride ion diffusion from external source	RC piles	Concrete encasement	Encasement of piles with a reinforced concrete jacket to protect reinforcement and exclude oxygen.
	RC columns	Cathodic Protection (CP)	For columns which cannot be encased or have the cover concrete removed eg in tidal zone
	RC columns	Chloride extraction (CE)	For columns which are on land eg overpass bridges. This procedure is noiseless and dustless
Chloride ions from internal source	Precast bridge rails and RCBC's	Removal of affected elements	No effective repair technique can be employed for concrete elements affected by calcium chloride included in the original concrete mix
Alkali – Silica Reaction (ASR)	Prestressed Piles	Concrete encasement and/or fibre wrapping	Piles in the tidal zone or in water may suffer a rapid deterioration due to secondary corrosion of the included highly stressed strands. All piles need to be adequately encased to prevent significant corrosion occurring.
	Prestressed deck units	Monitoring	No effective repair technique is available. The superstructure drainage is critical and hence these bridges need to be effectively drained and kept as dry as possible.
Carbonation	RC Piles	Concrete encasement	RC piles typically will carbonate and suffer maximum corrosion at ground level where adequate moisture is available to drive both mechanisms.
	RC Piers	Anti carbonation coatings	A range of coatings exist which will assist in sealing the structure against carbon dioxide penetration
Plastic cracking	Bridge decks	Epoxy filling of cracks or removal of the element	In some cases isolated cracks may be filled with low viscosity epoxy. In more serious cases

			removal of the affected element is required. This type of cracking is high risk in windy hot dry weather.
Cracking	PCP's plain concrete pavements	Cross stitching	Longitudinal or oblique angle cracks may be cross-stitched to form a hinge and hence ensure shear transfer across the joint.

Traditionally Main Roads has not allowed electrical techniques for repair of our concrete bridges due to concerns with the possible aggravation of ASR distress. Two repair systems are mentioned in Table 6 using electrical techniques viz cathodic protection (CP) and chloride extraction (CE). One CE project has been completed recently which is the first Project in Australia of its type where it was deemed the risk of ASR distress was minimal. Chloride extraction uses higher current densities (1-2 A/m2) than CP (2-20 mA/m2) but is applied for a shorter period of time. Another project is about to start where CP is being used to control the rate of corrosion of RC columns in the tidal zone. The advantage of CP is that extensive removal of cover concrete is not required and hence the technique is less invasive.

Additional research work is underway at the University of Queensland to examine the use of electrical repair techniques with very reactive aggregates. This work will be reported over the next couple of years. One pile encasement project made use of both glass fibre and carbon fibre wrapping for the durability protection of 500 piles in a marine environment. In addition 6 probes have been inserted in 3 piles for long term monitoring of the actual corrosion currents within the repaired sections of piles. Main Roads has shown that it is prepared to look at innovative repair solutions and play a significant part in advancing our collective knowledge in the area of optimized repair of concrete structures.

CONCLUSIONS

In relation to the information presented in this paper the following conclusions are made:

(i) Main Roads has invested a significant amount of time and funds in analyzing the performance of its concrete structure networks ie bridges, culverts and roads. We are in a unique position to understand the issues that have arisen over the last 100 years and optimize our way forward into the 3rd millenium.

(ii) The standard of design, specification and construction required for road infrastructure to survive 100 years is higher than for typical building infrastructure with a shorter economic life of 25 to 40 years.

(iii) Main Road's investment in research and development has allowed it to minimize the impact of some potentially serious durability issues eg alkali-silica reaction in all of its concrete structures.

(iv) Main Roads has shown itself to be innovative in its attitude to new types of repair systems and has participated in carefully controlled application of these techniques eg CP , CE and carbon fibre wrapping.

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DEALING WITH DEFECTIVE AND DETERIORATING CONCRETE STRUCTURES

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SUMMARY

Australia presently faces serious technical and economic problems as a result of its ageing, deteriorating, and inadequately maintained physical infrastructure. Dealing with defective and deteriorating structures has thus become an extremely important task for structural engineers. This lecture focuses specifically on defective and deteriorating concrete structures. Before effective corrective work can be planned and carried out, a thorough investigation needs to be undertaken for the purposes of identifying all defects, diagnosing correctly their extent and causes, and then assessing the present and likely future condition of the structure. Without such an investigation, the corrective work is likely to be unsuccessful. In view of the limited time available, attention is concentrated here on the methods of investigation. Methods of treatment are mentioned only very briefly. Detailed references are given in an appendix on both investigation methods and repair and corrective work.

THE DETERIORATING PHYSICAL INFRASTRUCTURE

In the last 50 years there has been sustained economic development in most western countries, including Australia. This has created an unprecedented demand for new physical infrastructure, for structural engineers, and also for research and development work in structural engineering and in building technology. However, after many years of intense building activity, an extensive physical infrastructure has now been created, and we are experiencing a reducing demand for new construction. Of course, some building activity is always going to be needed for new and developing industries (such as, at present, information technology and communications), and for special events such as the Olympic Games. Nevertheless Australia, like many other countries with a mature economy and an extensive physical infrastructure, must expect to see a decrease in new building activity in the future, and a decrease in the demand for the traditional design skills of structural engineers.

On the other hand, an intense wave of new building activity can now be seen in some other parts of the world, such as China and Eastern Europe, where the economies are developing strongly.

Australia's extensive stock of physical infrastructure is now beginning to age, and to deteriorate. All construction is prone to deterioration over time, and in many western countries severe problems of deterioration are now starting to appear. The range of related engineering problems is vast, and well documented. More than a decade ago, in discussing the scale of the worldwide problem of ageing infrastructure, Blakey (1989) quoted the following figures:

- One in five bridges in the USA had been in need of major repair in 1981 in order to prevent closure.
- In the USA the Federal Department of Transportation estimated that required road and bridge repairs would cost 33 billion (thousand million) dollars, whereas 1.3 billion dollars had been allocated to the problem.
- Sweden would need to spend 40 billion dollars in the decade of the nineties on bridge repairs.
- Of an estimated 43,000 dams in the USA, the US Army Corps of Engineers inspected 9000 and found the majority to be faulty. There was no funding to continue the inspections, let alone to undertake any repairs.

The financial implications of the deteriorating infrastructure are severe. An example can be taken from the state of South Australia. In 1989, a four-fold increase was required in the allocated annual real expenditure on the physical infrastructure in that state in order simply to maintain the existing facilities without any further new development. Needless to say such spending did not occur, and the situation has steadily deteriorated.

While some of the above data are clearly dated, and possibly inaccurate in the fine detail, they suggest a problem of crisis proportions. The crisis has arisen because the public and private sectors in the past have generally been unable or unwilling to provide the resources needed to maintain the stock of infrastructure. For some reason, it is always far easier to find funding for new construction than for the repair and maintenance of existing construction. The result nationally is an inevitable and accelerating deterioration in the stock of bridges, roads, wharves, buildings, sewerage and water-supply systems. In the future, a greater proportion of the national annual budget will have to be devoted to repair and maintenance if the existing infrastructure is to be kept in working condition.

There is consequently a growing demand for engineering expertise to maintain and prolong the life of existing construction. The change in emphasis from new construction to maintaining and preserving the existing infrastructure is now affecting not only the work opportunities of structural engineers but also the nature of their work. Furthermore, it is affecting the type of structural research that needs to be undertaken, and indeed the content of undergraduate course programs in structural and civil engineering.

DEFECTS IN CONCRETE STRUCTURES

In general terms, a defect in a building structure is an inadequacy that has the potential to adversely affect the ability of the structure to satisfy its intended function. As already suggested, a prime cause of defects in building construction is progressive deterioration. However, defects also occur for other reasons. Minor defects are usually discovered during inspection of a new structure in the final phase of construction, and are corrected by the builder prior to commissioning. Nevertheless, other defects, resulting from design and construction errors, often go unnoticed and only become evident, if at all, when the in-service structure is subjected to abnormal conditions of load or environment. Yet other defects are caused by unusual man-made events such as accidental overload and blast, and by extreme natural events such as earthquake and high water or high wind.

It is useful to distinguish among three types of defects according to how and when they occur. **Inherent defects** exist in the structure from the time of construction. They typically occur as the result of design error or construction error, or from the use of inappropriate materials. **Induced defects** occur during the life of a structure and are usually the result of an adverse event. **Developed defects** occur gradually, and are most typically due to progressive deterioration, although other causes include progressive damage under repeated heavy loading, and material.

While defects may take many forms, those of prime concern to the structural engineer are **structural defects** and **functional defects**. Structural defects have the potential to adversely affect structural performance, through poor service load behaviour or reduced strength. Functional defects do not affect strength or serviceability, or even durability, but they affect the useability of the structure. For example, a common functional defect in office buildings is lack of watertightness in the roof, in the basement or in the cladding.

INVESTIGATION OF DEFECTS: DIAGNOSIS AND ASSESSMENT

Before any corrective work is undertaken on a potentially defective structure, it is vitally important to undertake a thorough investigation to identify all the defects, to determine their nature and causes, and then to evaluate the condition of the structure, and its ability to fulfil its functions. Without this prior investigation any corrective work is likely to be inappropriate and may not even address the real inadequacies in the structure.

To undertake a successful investigation, a rigorous approach is needed. The investigation will usually consist of some or all of the following steps:

- An initial on-site inspection of the structure, and a study of relevant documentation.
- A diagnostic study to identify the defects, and to determine their causes. This may well require laboratory tests on material samples and, possibly, field tests on the structure.
- Theoretical analyses of the real structure, as distinct from the as-designed structure, to determine its in-service behaviour and load capacity.
- A condition assessment of the structure to decide whether it will be capable of satisfying its intended use and function, either with or without corrective work.

Initial inspection

In the initial inspection, a search is undertaken for symptoms of defects. This evidence may be in the form of symptoms and signs, and anomalous (unexpected) behaviour. Sampling and testing of materials may also may also be undertaken in this initial phase of the investigation. A study of documentation regarding the design and construction and operation of the structure can show up design and construction errors, the use of inappropriate materials, and accidents or incidents that may have led to induced defects and damage.

Signs and symptoms of defects

The most common symptoms of deterioration and defects in concrete structures include the following:

- large deflections under normal service conditions,
- excessively wide local cracks, and/or extensive cracking patterns,
- cracking in unusual and unexpected locations,
- spalling at the surface of the concrete,
- honeycombing at the surface of the concrete,
- discolouration of the surface of the concrete,
- other unexpected patterns of behaviour.

Diagnosis

The purpose of the diagnosis is first to identify positively any significant deterioration and defects and second to determine the causes. The starting point for the diagnosis is a full list of symptoms and signs and abnormal behaviour. Some of the symptoms may well have been observed prior to the inspection, ie during the in-service behaviour of the structure. Unfortunately, in most diagnostic investigations there are usually relatively few symptoms but many possible defects, and even more possible causes of defects. A careful and methodical approach to diagnosis is therefore essential. All potential defects and possible causes must be identified and then eliminated progressively by careful investigation and collection of information. Only in this way can a sound diagnosis be achieved.

For the diagnosis to be acceptable, it must explain convincingly ALL of the observed symptoms and anomalous behaviour. In order to undertake a successful diagnosis, the investigator should have a good understanding of the way defects occur in structures. In particular, the deterioration processes that lead to defects can be quite complex. Processes such as carbonation of concrete, drying shrinkage of restrained concrete elements, progressive cracking of concrete, progressive bond breakdown, corrosion of steel reinforcement embedded in concrete, and fatigue of concrete and steel, need to be understood.

Assessment

A sound diagnosis provides the starting point for an assessment of the ability of the structure to fulfil its intended function, either without or with corrective work. In carrying out the assessment, it is vitally important to consider the real structure, in its current condition, and not the ideal, as-designed structure. Theoretical analyses of the real structure, including the effects of defects and design and construction errors, will often provide the basis of the assessment. Such analyses must consider the load paths in the real structure with real loads acting, rather than the idealised models used in conventional design.

Protective actions for potentially dangerous situations

If the structure is found to be in a potentially dangerous condition, prompt protective action needs to be taken before the investigation is completed. Decisions must be made promptly concerning propping or even evacuation. An integrated methodology for investigating potentially dangerous structures, based on progressive decision-making, and a step-by-step process of diagnosis and assessment, is described in a paper by Warner (1999). A version of the paper is attached.

CORRECTIVE WORK

Sufficient information must be obtained from the investigation to allow appropriate and effective corrective work to be chosen, and then planned and designed. The purpose of the corrective work may simply be to eliminate the defects or their effects. This can be often be achieved by strengthening and stiffening key components in the structure.

Another useful option is simply to downgrade the existing structure by reducing the allowable working loads and load capacity. The prescribed design loads are often not achieved in a specific structure. In such cases a careful analysis may show that the existing structure, although defective, is still capable of safely carrying the loads which actually occur. Reduced load limits may then be a simple and inexpensive solution to an otherwise very expensive problem.

Not infrequently, the corrective work can have the further objective of upgrading the structure or modifying it so that it can be used for purposes not originally considered in its design. Upgrading frequently requires increasing its capacity to resist load, and may even require a reconfiguration of the structural members such as walls and columns to improve functionality and allow new use of the enclosed space.

In addition to any corrective work and refurbishment, protective measures may also be undertaken to prevent, or at least delay, further deterioration and damage. Improved durability of the repaired structure may be achieved by protection of surfaces and regions exposed to wear or aggressive environments.
The actions undertaken in the treatment of a defective structure thus include:

- correcting and removing specific defects;
- imposing load limits;
- introducing protective measures to halt or delay future deterioration;
- refurbishing the structure to bring it back to a condition comparable with its original state;
- modifying the structure so that it can be used for new functions.

After repairs and corrective work have been completed, a carefully determined management plan is needed for inspection and maintenance of the structure in the future.

SAFETY AND RISK CONSIDERATIONS

It is important for structural engineers to understand that there are important differences between investigating an existing, possibly defective, structure and designing a new structure. In particular, care must be taken when the design requirements of a design code or standard are used to evaluate an existing building structure.

Design codes and standards rightly treat key design quantities such as loads, material properties and structural dimensions as random variables. Provision is made, by means of safety coefficients, for considerable variability in these quantities, and hence for the possibilities of overload and under-strength. However, it is not possible to say, in advance, which structures will in fact prove to be understrength or overloaded. Because of the margins of safety embedded in the design process, most real structures are very safe, with significant excess capacity for overload, while those few structures with significant understrength, still perform at least marginally under overload.

In the case of an existing structure the situation is different. The dimensions of the components and the location of all reinforcement are fixed, and can be measured precisely. Likewise, the concrete has been cast and its strength and stiffness can be determined accurately in the relevant regions of the structure. Even the loads and the environment of an existing structure can be determined much more accurately than for a structure still to be constructed. The parameters that govern the performance of the structure can thus be evaluated with good precision. It follows that the safety concepts and safety margins prescribed in a design code may NOT be applicable or appropriate to the assessment of an existing structure.

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TERMINOLOGY

While various terms are frequently used in publications on building pathology, their meanings tend to vary. A common terminology does not yet exist. Definitions are given below for terms which are frequently used in diagnostic work and structural assessment.

Defects: are inadequacies in a building structure which have the potential to adversely affect structural performance, use, and functionality.

Defects may be **inherent** (eg due to design error or construction error), **developed** (eg as the result of attack from an aggressive environment, or deterioration over time), or **induced** (eg by accidental blast, overload, or earthquake).

Structural defects: are inadequacies which result in the non-fulfilment of one or more of the structural design requirements. They include strength defects, serviceability defects, and durability defects.

Functional defects: are inadequacies which adversely affect the function of the structure, ie its ability to serve its intended purpose.

Errors: are incorrect human acts which have the potential to produce a defect, and consequently a fault or failure in a structure.

Signs, symptoms and anomalies: are specific indications of abnormal behaviour or deterioration in behaviour which suggest the presence of defects.

Explanations: are reasons for why and how the defects are present in a specific structure.

Causes: are similar to explanations, but with legal implications of fault.

Diagnosis: is the investigation of a structure which is displaying abnormal behaviour in the form of signs, symptoms, or anomalies, with the purpose of finding explanations for the abnormal behaviour, and of detecting any defects.

Assessment: is the investigation of a building structure with the purpose of evaluating its present condition and structural adequacy, and its likely condition and structural adequacy in the future.

Repair: is the restoration of a defective or deteriorated structure to an acceptable level of functional and structural adequacy.

Deterioration: is a progressive change in the structure and its component materials with time and a corresponding progressive loss of ability to satisfy its the structural and functional requirements stipulated in the design and construction briefs.

Durability: is the ability of the structure to resist the processes of deterioration and continue to satisfy its structural and functional requirements over a reasonable period of its intended life.
Maintenance: is the regular inspection and treatment of minor progressive flaws and defects which develop with time, in order to halt or retard significant deterioration and the development of serious defects.
Repair: is the structural reinstatement of a structure or some of its components in order to bring the structural performance up to a required level.
Protection: is the taking of appropriate measures to shield the structure from the processes of deterioration, possibly in the presence of aggressive environment.

SUSTAINABLE CONCRETE AND SUPPLEMENTARY CEMENTITIOUS MATERIALS

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SYNOPSIS

This paper presents three major areas of research on supplementary cementitious materials: Specification and testing for Durability, Innovative curing techniques and Development of Ultra High Strength Cementitious Materials.

Chloride ion transport and reinforcement corrosion in high performance concrete is affected by many factors. Currently, methods available to specify concrete include compressive strength and cover, water absorption, sorptivity, chloride diffusion coefficient, rapid ion penetration test and porosity. Some of the methods are not directly related to design life or corrosion of reinforcement. Recently, a critical evaluation and modifications have been made on the rapid ion migration test and this method has been incorporated in specifications in many countries. In this paper some modifications carried out and a possible specification method for durable concrete are presented and compared with the existing standards in USA, Norway and Sweden.

Self-curing concrete is a new concept on curing which has emerged within the last 5 years where an internal, polymer based curing admixture can be used eliminating the external curing procedure. In order to investigate the feasibility of this method and to develop sufficient technical background in this area, a research project is being carried out at the University of New South Wales. Results for different curing methods including a solvent based curing membrane and 3 internal curing admixtures are presented here. An internal curing admixture with a high solids content (64%) has performed similar to a good quality curing membrane.

A new high performance cementitious material called Reactive Powder Concrete (RPC) is being developed and practical applications such as extruded pipes and bridge components are emerging in France, USA and Canada. RPC mixes have high cement content, silica fume, steel fibres and a special type of new generation superplasticizer. The water/binder ratios are in the range of 0.12-0.18 and this range will not be possible without the new type of superplasticizers. The mixes produced can have flowing consistency. With high temperature curing such as steam curing and pressure techniques, compressive strengths of 200 to 800MPa and flexural strengths up to 35MPa have been achieved. In this paper the properties of the modified mixes developed in Australia are presented and practical applications in the precast industries are illustrated.

INTRODUCTION

In Australia, for marine structures, a slag cement is often used. More recently, concrete mixes containing silica fume have been used and in this paper, an investigation carried out on HPC containing silica fume at replacement levels of 5-20% by weight of cement are reported. The concrete mixes investigated are designed for applications in a marine environment and have a 28-day compressive strength ranging from 50MPa to 100MPa.

Many researchers have indicated that ordinary Portland cement blended with a supplementary cementitious material such as silica fume significantly alters the pore structure of concrete and protects the steel reinforcement from chloride induced corrosion by decreasing the penetration of chloride ions (Virtanen, 1983; Preece et al, 1982; Marusin, 1986). More recently extensive research has been carried out on the permeability and diffusivity of chloride ions into high strength lightweight concrete with compressive strengths between 50 and 100MPa (Zhang and Gjorv, 1991; Gjorv et al, 1994). The mixes investigated contained a blended cement with 9% silica fume and the lightweight coarse aggregates used were expanded clay and sintered fly ash. Among the factors investigated, the presence of silica fume had the most dominating effect on chloride diffusivity. This paper discusses chloride induced corrosion and specifications for durable concrete.

Another main area of current research is 'self-curing concrete'. The object of curing is to keep concrete saturated or nearly saturated as possible, until the water filled spaces in the fresh cement paste have been substantially reduced by the products of hydration of the cement (Neville, 1996; Powers, 1958). For sufficient hydration to take place, the humidity in the pores need to be maintained above 80% and below this level, the degree of hydration can be affected significantly (Powers, 1958). The purpose of curing is mainly to ensure low permeability and as a result better durability of a structure. Curing also improves compressive strength, flexural strength, abrasion resistance, reduces porosity, long-term shrinkage, plastic cracking and enhances resistance to reinforcement corrosion by improving the quality of the concrete in the cover zone.

Concrete curing practices have changed through the years and in many cases, the shift has been from "external water-adding" to "external water-retaining" techniques. Water adding techniques include immersion or ponding, water mist or spray, and wet hessian or burlap. Some water retaining techniques are plastic sheeting, delayed removal of form work, and curing membranes. More recently, advancement in self-curing concrete has emerged as a potential alternative to traditional 'external curing methods' (Dhir et al, 1996, Cabrera et al, 1989).

Self-curing is an 'internal curing system' where a water soluble polymer is added to the concrete mix. This method overcomes the difficulty in ensuring that effective curing procedures are employed by the construction personnel as the internal curing admixture is a component of the mix. The mechanism of self-curing can be explained as follows: Continuous evaporation of moisture takes place from an exposed surface due to the difference in chemical potentials (free energy) between the vapour and liquid phases. The polymers added in the mix mainly form hydrogen bonds with water molecules and reduce the chemical potential of the molecules which in turn reduces the vapour pressure. This reduces the rate of evaporation from the surface.

The main objectives of this part of the study are:

(i) to assess the effect of internal curing admixtures on the properties of concrete made of ordinary Portland cement (opc), fly ash and slag binders,

(ii) to evaluate membrane method of curing and internal curing in terms of their ability to reduce evaporation of water from concrete surfaces under simulated environmental conditions,

(iii) to relate the properties of self-cured concrete to those cured by traditional methods and to assess the efficiency of these internal curing materials and

(iv) to transfer the internal curing techniques to field applications particularly for bridges and other highway structures.

The third and most recent area of investigation is the development of an ultra-high strength concrete. Reactive Powder Concrete (RPC) is an ultra-high strength, low porosity material produced from powder composition cementitious materials. The material contains no coarse aggregate and the fine aggregates are replaced by very fine sand with particle size in the range 150-600μm. The entire material is therefore composed of very fine particles and for this reason it is called "Powder Concrete". The material also contains very large quantities of reactive cementitious material and the term "Reactive" has been added. RPC has demonstrated very high compressive strengths in the range of 200-800MPa, flexural strengths of 25-50MPa and excellent durability. RPC usually incorporates large quantities of steel or synthetic fibres and has enhanced ductility and high temperature performance, enabling structural members to be built entirely from this material to resist all but direct primary tensile stresses (Richard and Cheyrezy, 1994; Gowripalan and Te Strake, 1998).

A thorough understanding of High Performance Concrete and its component materials led to the development of RPC. Traditional concrete is a heterogeneous material and the large particle size difference leads to a weak transition zone between the aggregate and the cement paste (HDR, 1994). In RPC the homogeneity of the mix is improved by the size distribution of the fine particles. The microstructure in RPC is improved by the reduction in $Ca(OH)_2$ primarily due to the use of a supplementary cementitious material in the binder. By applying a heat treatment after setting, the pozzolanic reaction is activated and RPC is often cured at a high temperature such as 90°C (Richard and Cheyrezy, 1994). When ground quartz is added to the mix even high curing temperatures, from 250°C to 400°C may be used. Samples cured at 350-400°C, without pressure have given compressive strengths of 488-524 MPa. With the application of a pressure technique, even higher strengths of up to 673MPa have been reported. By increasing the compacted density of the dry solids the amount of water required for fluidising the mix can be reduced. Optimisation of grain size and application of pressure can increase the compacted density of the dry solids. Usually ductility of RPC is achieved by the addition of large quantities of fibres. At the dosages used, usually, the material flexural strengths are also increased.

While the main objectives in developing RPC were to increase strength and ductility, it was also necessary to maintain mixing and casting procedures as close as possible to existing practice. RPC has a very low water/binder ratio (typically around 0.15) and hence high dosages of superplasticizers are required for it to be readily mixed and placed (HDR, 1994). The use of a modified carboxylic ether polymer superplasticizer, makes the task considerably easier.

Currently, RPC is substantially more expensive than conventional concrete. With such high ultimate strength and improved characteristics, however, it is often compared with steel sections and prestressed concrete beams. In some cases, due to the reduced thickness of member sizes and speed of manufacture, RPC has provided more economic solutions when compared with steel and prestressed concrete. Some actual practical applications include, cycle-pedestrian bridge in Canada, hazardous waste containers, precast floor beams, pipes, frames, cladding panels and tilt-up panels. With the modification of the mixes with the view to reducing the binder content and faster production techniques, currently investigated, RPC may have more practical applications.

The main advantages of RPC include, significant weight reduction, superior ductility and hence greater structural reliability, enhanced abrasion resistance, impermeable nature of pores and hence better durability, possible new structural shapes due to the absence of conventional reinforcement and the self-healing potential under cracking conditions due to a significant amount of unhydrated cementitious component in the finished product.

RESEARCH ON MATERIALS AND TECHNIQUES

Chloride ion penetration studies

Ten high performance concrete mixes with varying percentages of silica fume up to 20% of replacement of ordinary Portland cement have been investigated for chloride ion penetration tests. Series I was based on accelerated chloride ion penetration test in accordance with ASTM: C1202-91. Three identical concrete specimens of 100mm dia. and 50mm thick were cut from a 100x200 mm cylinder cured for 28, 56 and 1 year in a fog room at 23°C and 95% relative humidity (R.H.). The specimens were tested for 6 hours at 60V or for 24 hours at 30V.

In Series II, 200x200x100 mm slabs were cast from the above mixes and cured for 28 days in the fog room. At the age of 28 days, the slabs were coated with a silicon rubber compound on five sides and immersed in a 5% NaCl solution with the exposed surface of the slab maintained at 40mm below the salt water level. The slabs were stored in a laboratory atmosphere at 23°C and about 50% R.H. After 28, 56 days and 1 year of immersion time, the slabs were removed from the salt water and were split in the middle using a hydraulic testing machine. Silver nitrate solution of 0.1M concentration was sprayed on to the freshly broken surface. The depth of penetration of chloride ions as indicated by the colour change due to the formation of AgCl was measured.

Series III tests were carried out on steel bar reinforcement embedded in a cube or cylinder with various cover thickness ranging from 10mm to 40mm. The specimens were subjected to 24 hour wetting (by spraying a 4% salt water) and 24 hour drying cycles(under high intensity lights). The maximum (during drying) and minimum (during wetting) temperatures were 35°C and 20°C, respectively. Half cell potential measurements were taken on these specimens over a period of 1 year using a $Cu/CuSO_4$ standard electrode.

Studies on Internal Curing Admixtures

The effect of curing, particularly new techniques such as "self-curing", on the properties of high performance concrete is of primary importance to the modern concrete industry. As an initial step, the effect of self-curing admixtures on moisture retention, strength development, porosity, permeability and shrinkage is being investigated. Long-term strength development is also included in the test programme. A research project is being carried out with one type of curing membrane and three internal curing admixtures and these are being compared with the traditional methods of water curing in a laboratory experimental programme in the Civil and Env. Engineering, at the University of New South Wales.

Materials

The properties of the curing membrane and the internal curing admixtures used are given in Table 1. Internal curing admixture 3 is generically similar to the material used in a previous investigation (Dhir et al, 1996). The other two are locally available products. The curing compound used on the surface of the specimens is a solvent borne resin with an efficiency of 94% as determined in accordance with AS 3799 (1998). The mix proportions, used in the initial investigation, with Type GP cement are shown in Table 2.

Preparation of Specimens

Concrete slabs of 200x200x100mm were cast in steel moulds. Soon after the surface water had disappeared (2-3 hours after casting), the specimens were transferred to the controlled environment. The specimens were demoulded after 1 day and kept in the controlled environment of 23±2°C and 50±5%R.H. Cylinders and prisms were also cast and kept under the same environment until testing for compressive strength or shrinkage. To avoid any possible interaction between the mould releasing agent and the polymers used, the steel moulds were lined with 'Rencourse' (aluminium core damp course) lining which served as a moisture barrier on five sides of the slabs exposing only the top surface. The following curing methods were used:

(i) no curing
(ii) 3 day water curing by ponding and these specimens were stored in the fog room at 23°C. The specimens were removed to the controlled environment after 3 days.
(iii) 7 day water curing by ponding and these specimens were stored in the fog room at 23°C. The specimens were removed to the controlled environment after 7 days.
(iv) curing by the application of a curing membrane (see Table 2) applied evenly at a rate of 0.2 litres/m^2 with a spraying equipment.
(v) curing using the internal curing admixture at a dosage of typically 5 litres per cubic m.

Test Methods

Moisture loss due to evaporation from the surface of the concrete slabs was measured periodically up to an age of 28 days using an electronic balance with a resolution of 0.1g. The exposed surface of the slab is as cast and the other five surfaces were sealed with a water proofing sheet which consists of an aluminium foil core (Rencourse). The top edges of the specimens were also sealed. Three identical slabs were tested for each curing method.

Compressive strength was measured at ages of 3, 7 and 28 days using 100 mm diameter concrete cylinders, cured and stored in the same environment as the slabs. In addition a number of cores cut from the slabs will also be tested for comparison with the cylinders. Six specimens were tested for each curing method with sulphur caps and all specimens were tested in the same laboratory by the same operator.

Porosity of concrete was determined using a vacuum saturation method (Cabrera and Lynsdale, 1988). The oven-dried specimens were evacuated dry for 1 hour and a further evacuation was carried out for 1 hour after introducing water in order to saturate the specimens. The porosity was then calculated from the oven-dry weight, saturated weight and submerged weight of the specimens.

From the vacuum saturation method for porosity measurement, it was also possible to obtain values for the total water absorption. These values based on an oven-dry basis of the specimens are reported here.

Reactive Powder Concrete (RPC) Investigation

Materials for RPC

In the preliminary investigation on RPC, locally available materials have been used. The main objectives of this investigation were to minimise porosity, reduce binder content and achieve higher strengths with adequate ductility using readily available ingredients. Different types of cement, and three different types of fibres were investigated. The cements used included Type GP, and two micro-cements with fineness of $650 m^2/kg$ and $900 m^2/kg$. Fibres used were straight steel, crimped polypropylene and PAN based carbon. The influence of grading of fine aggregate was investigated by assessing the suitability of four different types of sand. The curing of RPC was identified as critical and therefore several different curing regimes were investigated. The age at which high temperature curing commenced was considered as a critical factor influencing strength development. Details of the curing regimes adopted are given in Table 3.

Some high temperature tests were carried out in a furnace, heating the specimens up to 1000°C, to assess the fire resistance and residual strength characteristics of the material.

Most of the investigation was carried out using Type GP cement. A limited number of specimens made with two micro-cements with specific surface areas $650 m^2/kg$ and $900 m^2/kg$ were also investigated. The silica fume used was an Australian silica fume with the characteristics given in Table 4. In some mixes precipitated silica was used instead of undensified silica fume. For most mixes Sydney sand with particle sizes between 100 to 400µm was used. This is similar to the sand recommended by Richard and Cheyrezy (1994) for RPC production. They used a sand with particles sizes between 150-400µm. In the present investigation a sand with a similar particle size distribution was used to produce the specimens. In some mixes a ground silica flour with particle sizes less than 4um was used to replace part of cement.

In the initial investigations, a Naphthalene Formaldehyde Condensate based superplasticizer was used. Water/binder ratios of 0.14 could be achieved with this superplasticizer but with a considerably long mixing time. **With the introduction of a new generation modified polycarboxylic ether polymer superplasticizer, however, production of RPC became easier and w/b ratios of 0.11-0.14 could be consistently achieved.** This type of superplasticizer is now available in Australia.

An 18mm long straight steel fibre with ends modified, having a cross-section of 0.6x0.4mm, was used for the steel fibre reinforced mixes. A crimped, new type of polypropylene fibre with an effective length of 15mm was used. A PAN(Polyacrylonitrile) based fibre with a modulus of 238GPa was used. The length of the fibres was either 6mm or 12mm having a diameter of 7μm.

Mix Proportions and Preparation of Specimens

Several mixes with different compositions, as shown in Table 5, were investigated in order to obtain an optimum mix. The mixing sequence of Mix 1 is described below: A Hobart or similar mortar mixer was used for the bulk of the investigation. For large scale casting a pan mixer was used. In order to standardise the mixing procedure with the Hobart mixer, among the three laboratories participating in this investigation in Sydney, the following mixing sequence was adopted. First, water and superplasticizer were mixed together and introduced into the mixing bowl. Saturated surface dry (SSD) sand was then introduced followed by the silica fume. The mixing of the above materials was carried out within 2 minutes. The cement was then added and the mixing continued for a further 5 minutes. During this 5 minute period, the steel fibres were added.

The flow test was carried out immediately after mixing, and thereafter at 30 minute intervals. After the initial flow test, the batch was left in the bowl of the Hobart mixer and covered to minimise evaporation. Just prior to the 30 min. test period, the bowl was uncovered and the material given a 15 seconds re-mixing before checking the flow. For flow tests at different temperatures, a control plain mix (identical to Mix 1 but without fibres), a mix with steel fibres (Mix 1) and a mix with polypropylene fibres (Mix 2) were used.

Cubes (50mm size) were cast for the compression test, from the different mixes. It was decided to use cubes due to the difficulties in capping cylinders of such a high strength material. The smaller size for cubes was chosen for development purposes of the mixes and due to the smaller size particles in RPC. Load capacities of testing machines was also a consideration. For flexural strength determination, 50x50x600mm long prisms were cast. However, limited number of large cubes (100mm size) and cylinders (75mm diameter) were also cast and the results compared. Special beam moulds were used to cast the 600mm long prototype beams.

RESULTS AND DISCUSSION

Chloride ion penetration and Design Life

The 28-day compressive strengths obtained from 100mm diameter cylinders (Table 6) indicate that at 10% replacement level of silica fume and at 0.35 w/(c+sf) ratio, there was an increase of about 19% in the compressive strength when compared with the respective control mix without any silica fume. At lower w/(c+sf) ratios or at 20% replacement level of silica fume, the increase in compressive strength was moderate, possibly due to compaction difficulties.

Results of a typical mix obtained from the accelerated chloride ion penetration test at 60V and 30V at the age of 1 year are compared in Tables 7(a) and 7(b), respectively. At 60V, the temperature rise, over the 6 hour period, of both NaCl and NaOH solutions was more than 10°C. For more porous, normal strength concrete mixes a temperature rise as much as 20°C was recorded. However, tests carried out at 30V and below showed a temperature rise not more than about 3°C for the high strength concrete mixes investigated. At 60V the ions were driven through the concrete specimen at varying temperatures. The current flow and the charges passing through the specimen were dependent on the temperature during testing and hence the comparison of different mixes is difficult. At 30V or below the temperature rises in all the specimens were very small, making the comparison of charges passed for different mixes valid.

Supplementary cementitious materials are effective in reducing chloride ion penetration. For example, as the percentage of flyash, slag or silica fume was increased the total charge passed decreased. At 20% replacement level, the charged passed was a minimum. The presence of silica fume showed a significant effect on the chloride ion diffusion.

The 56-day effective coefficient of diffusion (Dc) calculated based on Fick's Law of diffusion showed a remarkable reduction with increasing percentage of silica fume as shown in Table 8. The estimated time for corrosion to initiate based on the above diffusion coefficients and 0.4% chloride ion concentration by weight of cement are given in Table 9.

For a given w/(c+sf) ratio, as the percentage of silica fume increased, Dc decreased significantly. With sufficient cover and proper mix design, expected life of reinforced concrete structures can be extended. Similar results have been compared for lightweight high strength concrete with 25, 50 and 75mm cover by Gjorv et al (1994). Control mixes without any silica fume showed considerably lower estimated life time.

Careful observation of the silver nitrate sprayed split surface showed that chloride ions moved through the pores in the paste and not around the aggregates, indicating a dense interface between aggregates and cement paste. Chloride ions are 3.6°A (0.36 nm) in diameter (Smith, 1990) and if the pores are very small such as 100°A (10nm) and below, then chloride ions can become adsorbed on to the gel surface. In addition to the physical adsorption, there is also chemical reactions, particularly with unhydrated C_3A (Tricalcium Aluminate) to form calcium chloroaluminates (Midgley and Illston, 1984). According to Tuutti (1980), chlorides are present in concrete in three forms; chemically bound, physically adsorbed and free chlorides. It is the free chlorides which diffuse further into the concrete, mainly through pores of 100°A and above, and attack the passive layer of the steel reinforcement. Presence of silica fume together with low w/(c+sf), impede the movement of chloride ions due to an improved pore structure of the concrete (higher proportion of smaller size gel pores rather than large capillary pores).

An ion migration test, similar to ASTM C1202-91 has been developed in Norway and it has been investigated in Australia. The fundamental difference between this method and ASTM C1202-91 method is that in this modified ion migration method a chloride permeability is obtained based on chloride ion concentration measurements under steady state flow under a low potential difference.

Increase in chloride ion concentration (from zero) passing through a saturated concrete sample under a low potential difference is monitored in both initial non-steady state and subsequent steady state, over a period of 2 to 4 weeks. Based on the ion concentration values in the steady state flow a chloride permeability is calculated. Limits on chloride permeability is specified for a durable concrete. For marine structures, typically, a permeability of less than 10×10^{-13} m^2/s is specified. The classification of concrete quality is somewhat arbitrary. However, the chloride permeability is directly linked to chloride induced corrosion.

A chloride diffusivity is calculated based on the following equation:

$$\mathbf{D = \beta_0\,(300kT/ze_0\,\Delta\psi).\,(LV/C_0A_0).(dc/dt)}\ldots\ldots(1)$$

Where

D-Diffusion Coefficient (cm^2/s)
β_0 - Correction factor for ionic interaction
k - Boltzman constant (1.38×10^{-16} ergs/K)
T - Temperature (K)
z - Valence of chloride ion
C_0 - charge of proton (4.8×10^{-10} e.s.u.)
$\Delta\psi$ - applied electrical potential (V)
L - Specimen thickness (cm)
V - volume of chloride collecting cell (cm^3)
e_0 - initial chloride concentration in chloride source solution ($mmol/cm^3$)
A- cross-sectional area of specimen (cm^2)
dc/dt - steady-state migration rate of chloride ions ($mmol/cm^3.s$)

With increasing concentration of chloride ion concentration of the chloride source solution from 0.1 to 0.5M NaCl, the correction factor β_0 varies from 1.22 to 1.70. Since this method is based on the steady-state migration rate of chloride ions through the concrete specimen, the test duration is 2 to 4 weeks and usually carried out on 28-day (age) specimens. The diffusion co-effiecient obtained from this test can be related to design life of concrete structures based on chloride induced corrosion of reinforcement.

Although many accurate methods are available for the measurement of rate of corrosion, in the present investigation, a simple half cell potential measurement was used as a comparative tool and this method can give reasonable results. It is clearly evident that with 40mm cover, in the mix containing silica fume, the half cell potential was maintained above -350mV for a longer period of time. This investigation is continuing at present.

Results on Internal Curing Admixtures

Compressive Strength Development

Compressive strength developments of concrete curing under different curing methods are compared in Figures 1(a) and 1(b). At 7 days of age, water cured specimens for 7 days showed the highest compressive strength of 40MPa and specimens water cured for 3 days showed a strength of 38MPa. Membrane method of curing showed a comparable strength of 38MPa. Internal curing admixtures 1 and 2 showed a strength above 35 MPa at dosages of 2 and 5 l/m^3. Internal curing admixture 3 showed a strength slightly below 35 MPa at the above dosages. At a dosage of 10 l/m^3, internal curing admixtures 1 and 2 showed a slight reduction in strength. However, admixture 3, at a dosage of 10 l/m^3, showed a considerable reduction to give a strength of only 29MPa. When compared with 3-day water cured and membrane cured specimens, this is a reduction of about 24%.

At 28 days of age, specimens with 7-day and 3-day water curing showed strengths of 58MPa and 55MPa, respectively, and the membrane method of curing showed a strength of 49MPa. At a dosage of 5 l/m^3, internal curing admixture 2 gave a comparable result (to the membrane method of curing) of 51MPa but internal curing admixture 3 gave a lower result of 47MPa. This is a reduction of 15% when compared with 3-day water curing.

At dosages of 2 and 5 l/m^3, internal curing admixtures 1 and 2 give compressive strengths comparable to those specimens cured using a high quality membrane. Higher dosages of these admixtures are not recommended. Internal curing admixture 3 appears to give significantly lower compressive strengths.

Porosity

Porosities of concrete, as determined by a vacuum saturation method, at 7 and 28 days of age are compared in Figure 2. Internal curing admixture 2 gave porosity values similar to the membrane cured specimens but internal curing admixture 3 showed no reduction in porosities when compared with a non-cured specimen, particularly at an early age of 7 days.

Rate of Evaporation

The rate of evaporation at 23°C and 50% R.H. of 50mm thick concrete slabs are compared in Figure 3. Internal curing admixture 2 is performing similar to the membrane after 28 days and is better than 3-day water curing. A specimen cured in water for 3 days (and then exposed to the same environment) lost more moisture after 28 days when compared with both membrane method and internal curing admixture 2 method. In both these cases, the rates of evaporation of moisture from the slabs were considerably lower than that of a non-cured slab. The surface quality of slabs containing internal curing admixtures were similar to water-cured slabs and any subsequent finishes can easily adhere to these surfaces.

Water Absorption

The water absorption, determined from the saturated, submerged and oven-dry weights of specimens cured under different conditions are shown in Table 10. Water absorption clearly distinguishes the different curing methods. Water curing for 3 days, membrane method and internal curing admixture 2 clearly show a substantial reduction in the absorption values when compared with the other methods.

Performance of RPC

Flow Characteristics and Plastic State Properties

The flow of the Control Mix (without fibres) immediately after mixing and flow retention characteristics are presented in Fig. 4. Initially, the mix had a flow of 70%. A few minutes after mixing, the control RPC mix appeared to stiffen due to the high thixotropy induced by the low water/binder ratio and high silica fume content. The material, however, showed a significant flow once agitated or vibrated as seen by the extended flow retention times. Within the temperature range of 13-33°C, good flow characteristics can be maintained for more than 1 hour after mixing.

The flow, plastic density, air content, initial set and final set of three mixes (control, Mix 1 and Mix 2) at 23°C, are compared in Table 11. Even at high fibre contents with steel and polypropylene fibres, similar flow characteristics were observed at room temperature.

The air contents at plastic state can be further reduced by the addition of a chemical admixture. A further reduction in the air content is expected to significantly improve the strength characteristics. Methods to reduce air contents are currently being investigated.

Compressive Strength Development

Selection of the appropriate type of cement for RPC can influence the strength development of RPC. Since RPC is (generally) subjected to heat treatment, the type and morphology of hydration products and the stability of the paste system with time will be governed by the type of cement. With the appropriate heat treatment applied at a chosen age, Type GP cement can be used for RPC. Micro-cement 1, with a reduced C_3A content (0.5-2.5%) and 650m^2/kg fineness, also appears to be a viable alternative for the production of RPC.

Four different sand types (Kurnell sand, Anna Bay sand, Ottawa sand and Normalised sand) were investigated (Paraschiv and Gowripalan, 1999). A Sydney sand with particle sizes varying from 50-600μm was also investigated in an earlier investigation (Te Strake, 1997). It appears that particle sizes up to 600μm can be utilised for RPC and a cube strength of 189 MPa was obtained (with a naphthalene formaldehyde superplasticizer) at the age of 7 days for specimens cured at 90°C hot water. The sand used had particle sizes varying from 100-400μm.

Compressive strength development of cubes cured in hot water at 90°C (Condition 2 in Table 3) is shown in Figure 5. It is possible to achieve 200MPa cube compressive strength with Mix 1, at 28 days of age, and these values are slightly higher than those reported by Collepardi et al (1997), for 90°C steam-cured cube specimens. The high temperature curing commenced at 24 hours of age. Specimens where the hot temperature curing commenced at the age of 3 hours after casting (Condition 4 in Table 3) showed somewhat lower results. Early age hot temperature curing, heating the specimens gradually up to 80°C, is currently being investigated to simulate steam-curing conditions. Prolonged curing in hot water for 28, 56 and 128 days did not show any benefits and in some cases prolonged heat curing may cause harmful effects such as the development of micro-cracks. Specimens cured for 3 or 7 days at

90°C maintained their strength values up to 128 days. Further long-term testing is currently under way.

On selected specimens, high temperature autoclaving was carried out at 160°C and 225°C (Conditions 6 and 7 in Table 3). It is generally not advisable to apply such high temperature curing at 3-4 hours after casting due to the development of cracks, expansion and failure of specimens. It is, however, possible to apply this technique at 24-72 hours after casting. At 3 days of age, heat treatment at 160° C improved the compressive strength by 20-60% of the strength of specimens cured at room temperature. Hence for precast products with RPC, autoclaving at a high temperature can offer advantages. Heat treatment at 225°C is currently being investigated. A comparison of cube and cylinder strengths of RPC specimens cured in hot water at 90°C is shown in Figure 6. The ratio of cylinder strength/cube strength for these RPC mixes is in the range of 0.843 to 0.886. These values are slightly higher than the values reported for normal strength concrete.

Modulus of Elasticity

Static modulus of elasticity on 75mm diameter cylinders and dynamic modulus of elasticity on 50x50x600mm prisms (using fundamental resonant frequency method) were determined and these results are shown in Table 12. These are non-standard size specimens. However, for development work and high temperature testing, it was not possible to test larger size specimens with the existing capacity of test machines and muffle furnaces.

The variation of results among specimens for the static modulus test was high, possibly due to preparing the 75mm cylinders for testing (e.g. capping), and as a result the dynamic modulus of elasticity was determined. The dynamic modulus results are very consistent (The values for the 4 specimens are 54.66, 51.30, 52.59 and 50.41 GPa) and relatively easy to obtain. For normal concrete mixes, the dynamic modulus is higher than the static chord modulus. With RPC also slightly higher values for the dynamic modulus was observed. The modulus values are comparable to very high strength concrete mixes but RPC mixes do not have coarse aggregates which contribute considerably to the modulus of elasticity of concrete.

Flexural Strength

Flexural strength and load-deflection curves were obtained on 50x50x600mm prisms. The water/binder ratio in this mix was 0.13 and the water/cement ratio was 0.16. These results are summarised in Table 13 while a typical load-deflection curve of Mix 1 is given in Figure 7.

The flexural strengths obtained from the prisms compare well with the 28-day results reported on 40x40x160mm prisms by Collepardi et al (1997), for 90°C steam-curing. These high flexural strengths together with high toughness indices make RPC an ideal material for pipes, panels and floor beams. Structures subjected to impact loading such as explosion chambers are also possible applications. In a typical pipe design, for example, it is expected to allow a hoop tensile stress of up to 10MPa, resulting in a significant reduction in the production cost (Dowd and Dauriac, 1998). Other thin wall products can also result in economic designs. To verify this, two prototype beam sections were wet-cast with external vibration and tested in flexure, under two point loading, with a span of 450mm. In these sections flexural strengths in excess of 25 MPa have been achieved. These values are slightly higher than the values reported by Collepardi et al (1997) on 150x150x600mm prisms (about 20MPa) cured under similar conditions.

The high compressive and flexural strengths of RPC make it an ideal material for certain structural applications. Currently, code provisions, world over, do not cover such high strength materials and hence structural components have to be carefully designed with special design manuals.

High Temperature Performance of RPC

Since the matrix of RPC is very dense, it is important to assess the fire rating and high temperature performance of RPC mixes. Mixes with steel and carbon fibres have been tested up to 500°C at this stage. Testing is continuing up to 1000°C. A panel of about 930x775x50mm in size will also be tested for fire resistance. Some preliminary results at high temperatures are shown in Fig.8 Carbon fibre reinforced mixes with 12kg/m^3 and 20kg/m^3 performed better than metallic fibres in terms of retaining higher residual strengths and ductility after subjecting them to 500°C (Nguyen, 1998). This investigation is continuing.

CONCLUDING REMARKS

Chloride ion diffusion coefficients and penetration depths of high performance concrete mixes with silica fume are significantly lower when compared with the corresponding mixes having only ordinary Portland cement. The presence of silica fume and the lower w/(c+sf) ratio of the high performance mixes yielded a dense pore structure with considerably lower porosity.

The accelerated chloride ion penetration test according to ASTM: C1202-91 requires modifications to obtain more meaningful results. With normal strength concrete mixes the temperature rise during the test can be as much as 20°C. Even for HPC mixes the increase in temperature can be as much as 10°C. Factors such as lower voltage, temperature sensitivity, isothermal conditions and measurement of ion concentrations as suggested by other researchers can be further investigated. A longer-term test for diffusion coefficient appears to be a promising development.

The HPC mixes with silica fume showed a subtantially slower rate of steel corrosion in a salt water environment. Together with diffusion and permeability measurements, actual corrosion rate measurements of steel reinforcement are important in assessing the suitability of concrete mixes for marine applications.

Internal curing admixture 2 retained moisture similar to the solvent borne resin membrane and performs better than 3-day water curing. At dosages from 2 to 5 l/m^3, the strength development of the three internal curing admixtures were compared. Internal curing admixtures 1 and 2 give compressive strengths similar to a high quality solvent borne resin membrane. However, internal curing admixture 3 appears to show a significantly lower strength, particularly, at the highest dosages.Porosity and absorption values obtained with internal curing admixture 2 are comparable to that cured with the solvent borne resin membrane. These values are also comparable to 3-day water curing. Internal curing admixtures provide a reliable means of ensuring that sufficient curing is carried out and may eliminate the need for external curing procedures.

With the materials available currently in Australia, with Type GP cement, it is possible to produce RPC mixes with cylinder compressive strengths of 175MPa (cube strengths of above 200MPa). A finer cement with low C_3A content and low alkali content is being investigated. Sands having particles sizes up to 600μm can be used for the production of RPC. Only a high temperature curing, such as steam curing, is adequate to produce the above compressive strength and no pressure technique or vacuum technique has been investigated yet. Further optimisation of the mix is possible.

Application of high temperature water curing and autoclaving have a beneficial effect on strength development. The heat treatment to be used, however, depends on the age of RPC, and type of cement. Static modulus of elasticity above 50GPa can be achieved. This is slightly higher than the values recorded for high strength concrete mixes. Flexural strengths of 30-35MPa can be achieved for RPC with steel fibres, and these values are more than twice the flexural strengths of high strength concrete mixes. Toughness of RPC is also considerably high enabling this material to be used without conventional reinforcement in certain applications. In prototype beam sections flexural strengths of 25MPa have been achieved.

High temperature explosive spalling can be controlled by appropriate type and quantities of fibres. The behaviour of RPC appears to be similar to very high strength concrete mixes. Further investigation is under way. Possible applications of RPC can include precast pipes, panels, floor beams, tilt-up construction, culverts and sheet piles. Continuous research investigation with local materials to further improve the mix design of RPC, curing and assessment of high temperature performance is essential.

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Table 1 Characteristics of Internal Curing Admixtures and Membrane-Forming Curing Compound Used (Supplied by the Manufacturer)

Curing Material	**Curing Membrane**	**Internal Curing Admixture 1**	**Internal Curing Admixture 2**	**Internal Curing Admixture 3**
Base material	Solvent borne resin with dye	Water based Modified Wax blend	Water based Modified Wax blend	Water based Polyethers
Solids content (%)	48	25	64	71
Specific Gravity	0.89	0.978	0.934	1.110
Curing Efficiency(%)	94*	-	-	-
Appearance	Clear Red liquid	Milky Emulsion	Milky Emulsion	Dark liquid
Viscosity	Low	Low	Low	Low
Solubility in Water	Not Soluble	Soluble	Soluble	Soluble

* Tested according to AS 3799 - 1998

Table 2. Details of the mix with Type GP cement per m^3

	Mix 1	Mix 2	Mix 3	Mix 4
Binder type	Type GP cement	Type GP cement	Type GP cement	Type GP cement
Cement (kg)	470	470	470	470
Sydney sand (kg)	565	565	565	565
Nepean gravel -20mm crushed(kg)	940	940	940	940
Water (kg)	188	188	188	188
Internal curing agent 1 (litres)		2,5,10		
Internal curing agent 2 (litres)			2,5,10	
Internal curing agent 3 (litres)				2,5,10

Table 3. Curing Regimes Used in the Experimental Investigation

Curing Regime	Description
Condition 1	Demoulded after 24 hours and cured at 23°C in water for up to 3 or 7 days
Condition 2	Demoulded after 24 hours, cured at 90/95°C hot water/steam up to 2 or 7 days and then stored in water at 23°C until testing
Condition 3	Demoulded after 3 hours and cured at 23°C in water for 1, 3 or 7 days
Condition 4	Demoulded after 3 hours, cured at 90/95°C hot water/steam for 1or 7 days and then stored in water at 23°C until testing
Condition 5	Demoulded after 21/24 hours and cured at 80°C in hot water up to 3 or 7 days
Condition 6	Demoulded after 21/24 hours, cured at 160°C autoclaving for 6 hours and then stored in water at 23°C until testing
Condition 7	Demoulded after 21/24 hours, cured at 225°C autoclaving for 4 hours and then stored in water at 23°C until testing

Table 4. Physical Properties of Cement and Silica Fume Used

Physical Properties	Type GP (Type A) Cement	Micro Cement 1	Micro Cement 2	Undensified Silica fume
Specific Gravity	3.15			2.20
Fineness (Surface Area) m^2 /kg	360	625-675	875-950	19,000
Compressive strength at 28 days (MPa)	41	45-52		-

Table 5. Details of Mix Proportions of RPC

Materials	Mix Proportions (kg/m^3)					
	1	2	3	4	5	6
Cement Type GP	955	955	955			
Micro-cement 1 (650m^2/kg)				875	660	
Micro-cement 2 (900m^2/kg)						660
Silica Fume	240	240	240	220	220	220
Water	170	170	170	370	290	290
water/binder	0.14	0.14	0.14	0.33	0.33	0.33
Fine Aggregate	1100	1100	1100	1010	1305	1305
Superplasticizer (litres)	30	30	30	60	60	60
Steel Fibres	190			175	175	175
Polypropylene fibres (kg)		26				
Carbon (kg)			12			

Table 6 Compressive strength of HSC mixes (age:28 days)

W/C+SF	Mix Type	SF %	Mix No.	Slump (mm)	Comp. Strength* (MPa)
0.25	Control	0	1	105	99.3
	SF	5	2	85	97.7
	SF	10	3	85	99.7
0.3	Control	0	4	110	84.8
	SF	5	5	100	82.2
	SF	10	6	90	85.6
	SF	20	7	85	93.4
0.35	Control	0	8	100	65.7
	SF	5	9	95	66.8
	SF	10	10	100	78.2

*** Average of 3 specimens**

Table 7(a) Acclerated chloride ion penetration test at 60 V at the age of 1 year

Mix 10 **W/C+SF = 0.35** **Silica Fume = 10%**

Time (minutes)	Temperature of NaOH (° C)	Temperature of NaCl (° C)	Current (mA)	Charge Passed (Coulombs)
0	16.8	16.8	48.0	0
5	17.5	17.5	51.6	15
15	18.4	18.4	53.6	47
30	19.4	19.4	55.2	96
60	20.6	20.6	57.5	197
120	23.1	23.1	60.9	410
180	24.5	24.5	64.4	636
240	25.8	25.8	67.2	873
300	26.9	26.9	69.4	1119
360	27.8	27.8	70.5	1371

Table 7(b) Acclerated chloride ion penetration test at 30 V at the age of 1 year

Mix 10 **W/C+SF = 0.35** **Silica Fume = 10%**

Time (minutes)	Temperature of NaOH (° C)	Temperature of NaCl (° C)	Current (mA)	Charge Passed (Coulombs)
0	17.2	17.2	20.4	0
30	18.0	18.0	21.1	37.4
60	18.3	18.3	21.5	75.7
120	18.7	18.7	22.0	154.0
240	18.9	18.9	22.4	313.8
360	19.2	19.2	22.6	475.8
480	19.4	19.4	22.7	638.9
600	19.7	19.7	22.4	801.3
720	19.5	19.5	22.1	961.5
1440	19.0	19.0	21.5	1903.3

Table 8 Chloride ion diffusion Coefficients (D_c)

W/(C+SF)	Mix Type	SF (%)	Mix No.	Diffusion Coefficient D_c (x 10^{-8}cm2/s)
0.25	Control	0	1	0.92
	SF	5	2	0.72
	SF	10	3	0.61
0.3	Control	0	4	1.16
	SF	5	5	0.76
	SF	10	6	0.73
	SF	20	7	0.48
0.35	Control	0	8	1.63
	SF	5	9	1.38
	SF	10	10	1.06

Table 9 Estimated design life (in years) for different concrete cover (Assumed threshold level: 0.4% by weight of cement)

Mix No.	Cover: 25mm	Cover: 40mm	Cover: 50mm
1	6	15.5	25
2	8	20	31
3	10	23.5	37
4	5	12.5	20
5	8	20	30
6	8	20	31
7	12	30	50
8	3.5	9	14
9	4	10.5	16
10	5.5	14	22

Table 10 Water Absorption Under Different Curing Conditions (Based on Oven-Dried Basis)

Curing Condition	7 Day Absorption (%)	28 Day Absorption (%)
No Curing	5.10	5.04
3 Day Water Curing	3.24	3.15
Curing Membrane	3.88	3.28
Internal Curing Admixture 1	4.90	4.78
Internal Curing Admixture 2	3.99	3.31
Internal Curing Admixture 3	4.71	4.45

Table 11. Plastic State Properties of RPC at 23°C

	Control (Plain)	Mix 1 (with steel fibres)	Mix 2 (with polypropylene fibres)
Flow (%)	65	64	60.5
Air Content (%)	8.4	9.2	8.6
Plastic Density (kg/m^3)	2260	2370	2210
Initial Set (min)	330	-	-
Final Set (min)	430	-	-

Table 12. Modulus of elasticity of RPC (cured at 90°C hot water) at different ages

Age of Concrete (Days)	Static Chord Modulus (GPa)*	Dynamic Modulus (GPa)+
3	45.68	50.67
7	48.81	-
28	49.63	52.24
56	51.76	54.90

* *Average of 3 specimens*
\+ *Average of 4 specimens*

Table 13 Flexural Strength Development of RPC (cured at 90°C water) (Mix 1)

Age of Concrete (Days)	Flexural Strength (MPa)*
3	30.08
7	34.29
28	35.2

- *Average of 3-4 specimens*

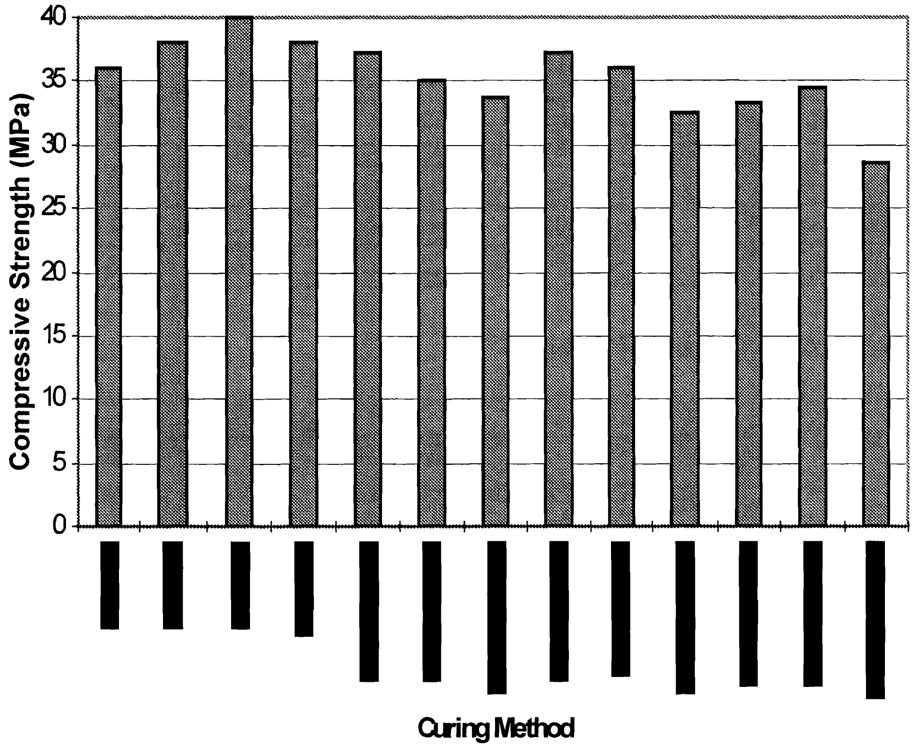

Figure 1(a). Compressive Strength at 7 Days of Age of Specimens Cured Under Different Methods

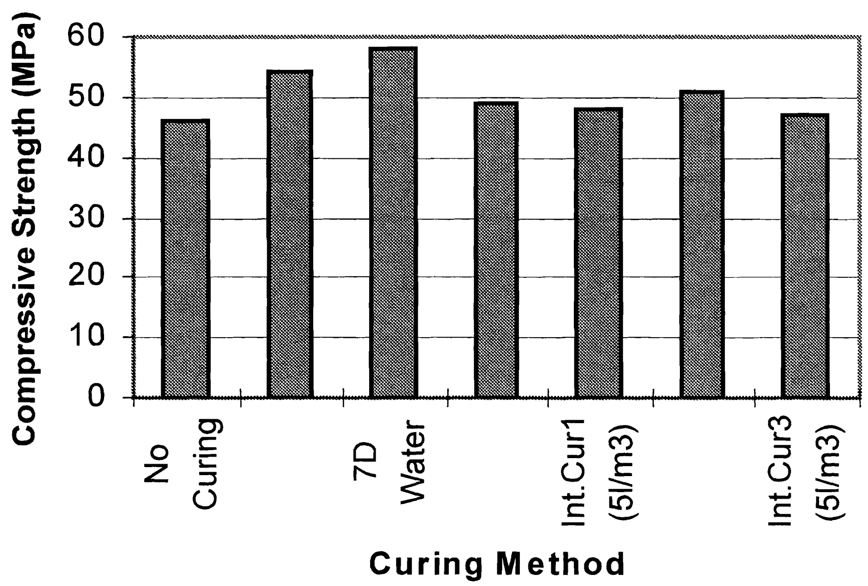

Figure 1(b). Compressive Strength at 28 Days of Age of Specimens Cured Under Different Methods

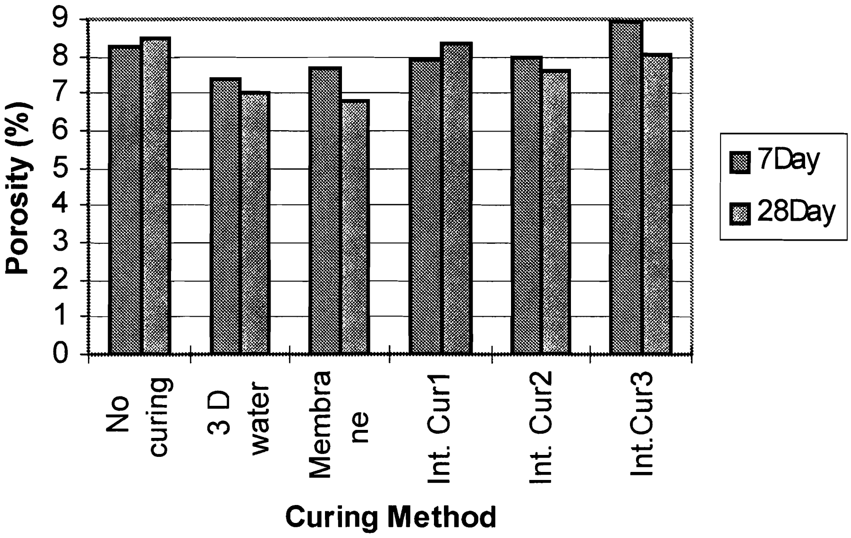

Figure 2 Porosity of Concrete Cured Under Different Methods

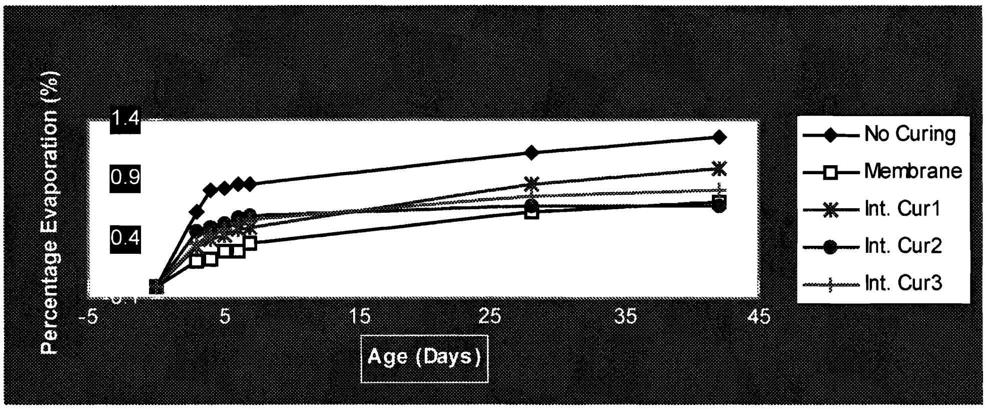

Figure 3. Rate of Evaporation versus Age, from 200x200x50mm Concrete Slabs Cured Using Internal Curing Admixtures (Dosage 5 l/m^3)

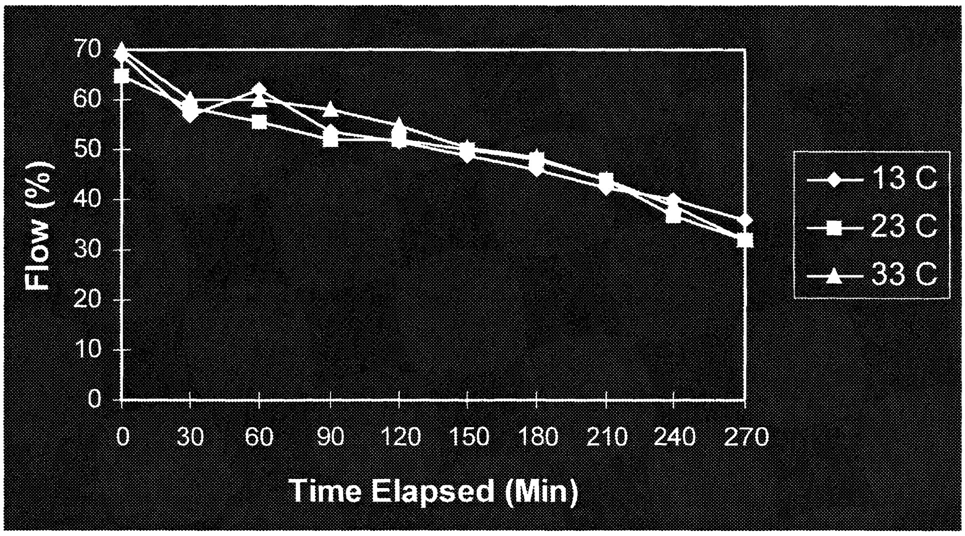

Figure 4. Flow Characteristics of RPC at Different Mixing Temperatures

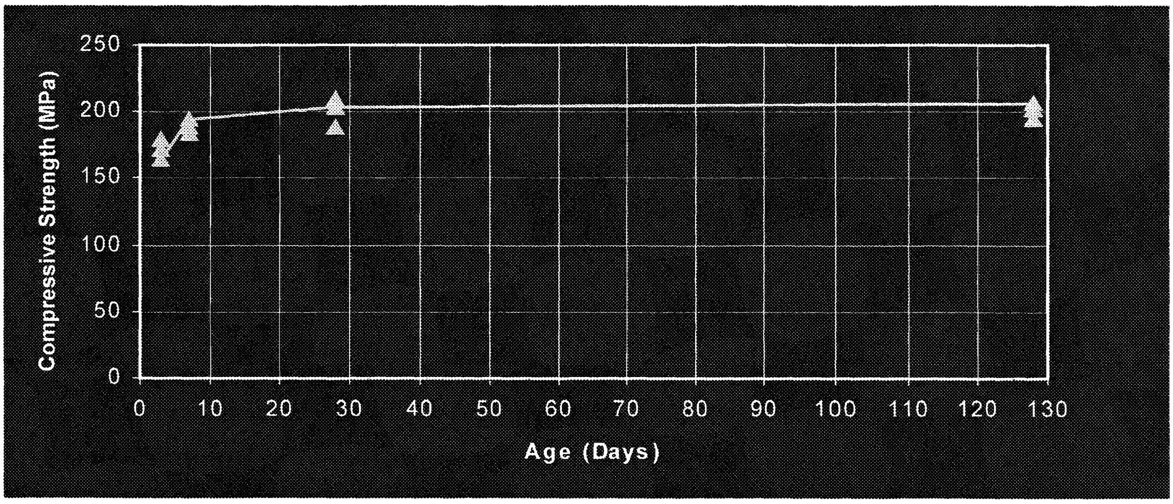

Figure 5. Cube Strength Development of RPC (Mix 1)

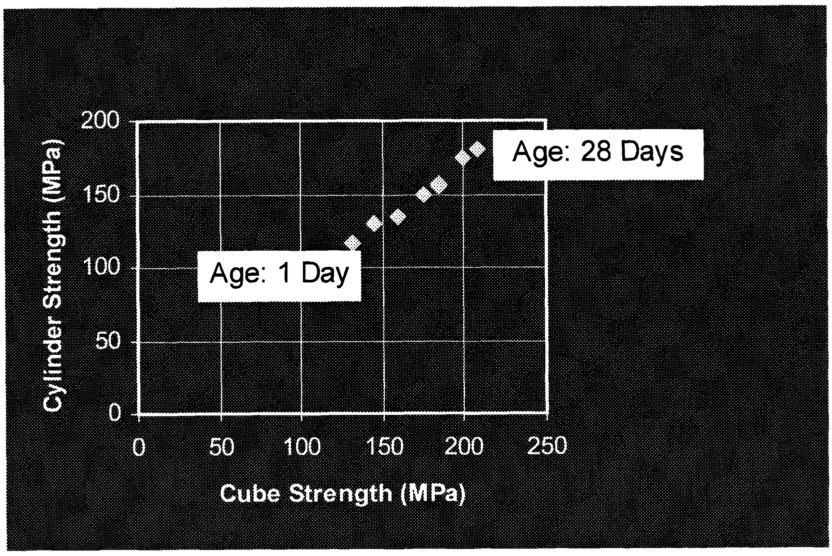

Figure 6. Comparison of Cube and Cylinder Strength (75mm size cubes and 75mm diameter cylinders)

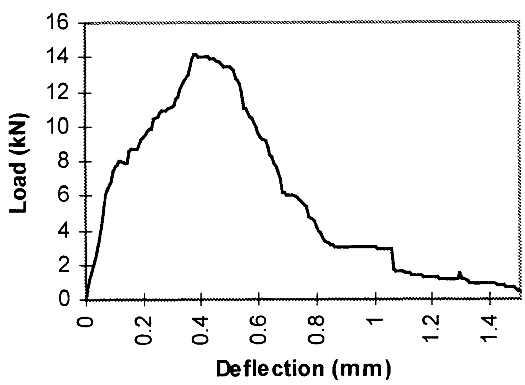

Figure 7 Load-Deflection Characteristics of Prisms (Cross-section : 50x50mm; Span: 300mm)

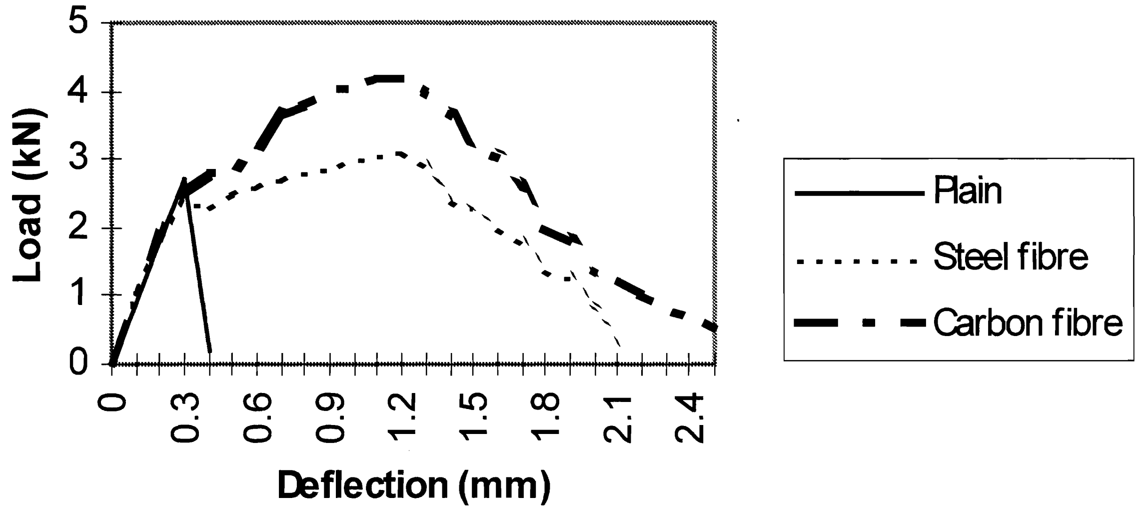

Figure 8 Residual Load-Deflection Curves of Prisms (50x50x600mm) Subjected to 500°C

STUDIES ON THE CENTRAL QUEENSLAND CONCRETES: USE OF REGIONAL RAW MATERIALS AND INDUSTRIAL BY-PRODUCTS

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ABSTRACT

It is well known that the properties of concrete improves with the addition of industry by-products such as fly ash, silica fume and slag when carried out in moderation under supervision Prior to using the by-products, however, significant research and investigation must be carried out. Central Queensland region has several industries whose by-products are useable in concrete and other building materials. Research in the use of CQ industrial wastes is widespread and offers the potential to further investigate the technology of concrete making. This report includes the results of the usage of the industrial by-products from the CQ region in plain and fibre reinforced concretes.

LITERATURE REVIEW:

Fibre reinforced concrete is a relatively new innovation in concrete technology and involves the inclusion of fibres into the concrete matrix. In steel fibre reinforced concrete(SFRC) steel fibre is distributed more uniformly throughout the concrete mix thus minimising the crack width due to either loading or shrinkage. The crack arresting properties of SFRC has encouraged the researchers to use it as a grout material in masonry construction (Dhanasekar & Steedman 1998). Romualdi & Batson 1963, Shah & Rangan 1971, Edginton & Hannant 1972 have reported the strength and other properties of SFRC. The benefits of the addition of steel fibre as coarse aggregate replacement include the increase in compressive and tensile strengths. Volume fraction of steel fibre has substantial influence on the properties of SFRC (Shah 1991). However because the cover to steel fibres in SFRC is not well defined, there is no clear understanding of its durability. Durability of SFRC is only sparsely discussed in the literature. The available literature ranges from environmental conditions such as industrial atmosphere, sea water, and sewage water contact to pure acidic attack such as sulphur and chlorine. This project focuses on the sea water attack on SFRC. Similar work is also reported by other researchers (Mangat & Gurusamy 1987a, 1987b, 1987c, 1987d, Mangat & Gurusamy 1988) using laboratory and beach situations.

Studies on the effect of adding of fly ash to concrete have also been well documented. The results generally show that the addition of the fly ash increases the durability of the concrete (Wesche 1991; Cao et al 1997, Ramezaniapour 1995; Chen 1997, Wong et al 1997; Khatri and Sirivivatnanon 1993; Abdul Awal 1997; Butler 1997; Saricimen 1995). Fly ash also provides better protection against the effects of infusion (Alhozaimy et al 1996 and Wong 1997).

This report describes two ongoing pilot studies on the use of the by-products from several CQ industries in concrete. Fly ash (FA) from Stanwell Power Corporation and Washed Lime Stone (WLS) from Pacific lime are particularly referred. While the FA was used as a cement replacement agent, the WLS is used as the aggregate replacement agent. Both plain and steel fibre reinforced concretes have been investigated and the strength and durability properties were investigated.

PROJECT #1 **STANWELL POWER STATION –FLY ASH & CSR HUMES HIGH STRENGTH CONCRETE.**

The objective of this project is to investigate the effect of adding steel fibre in high and normal strength concretes with fly ash towards the strength and durability of concrete. High and normal strength concretes referred in this paper are relative terms – with the "high" standing for 50 MPa and "normal" standing for 32 MPa grade.

The steel fibre used in this experimental study was FS186EE in which 'FS' and 'EE' stands for fibre steel and enlarged end respectively. The nominal dimensions of the fibres are 18 mm length, 0.6 mm width and 0.3 mm thickness with an average tensile strength of 850MPa (BHP).

Experimental program:

Cylindrical specimens of diameter 150mm with height 300mm were prepared and tested in the concrete laboratory, CQU. All the specimens were compacted by table vibration and cured for a normal curing period of 28days and after that the cylinders were fully immersed in either sea or pure water. Immersion of the test cylinders extended over a period of four months.

Standard slump tests were carried out on fresh concrete while compression testing and scanning electron microscope(SEM) analysis were conducted on hardened concrete cylinders every two weeks for four months after the normal curing period. To observe the behaviour of the concrete cylinders at the microscopic level, SEM technology was utilised. In this investigation information on the chemical profile of the nominated test cylinders were gained. By observing the cylinders from a chemical point of view, any unusual chemical reactions occurring that may or may not be the result of infusion could be identified. The presence of extreme levels of certain chemicals, inadequate bonding between the concrete matrix and aggregate (including fibre), disruption of the concrete matrix and a time-dependant presence of some chemicals could all be indicators of infusion and all can be identified in the SEM technology.

High strength concrete:

High strength concrete mix design obtained from CSR Humes, one of the leading concrete manufactures in Rockhampton, was slightly modified to account for no addition of admixture and method of curing. The W/C and A/C ratios, however, remained constant in the modified mix deign throughout the program. The compression test results of high strength plain and SFRC concretes are shown in Figure 1.

The strength generally has increased with the increase in sea water immersion period; an increase of 26% in compressive strength over a 3 month period was observed. However, after 3 months of seawater exposure the compressive strength of the plain concrete specimens has exhibited a tendency for rapid reduction. Similar finding was also reported by Mangat 1988 and is in good agreement with the current result.

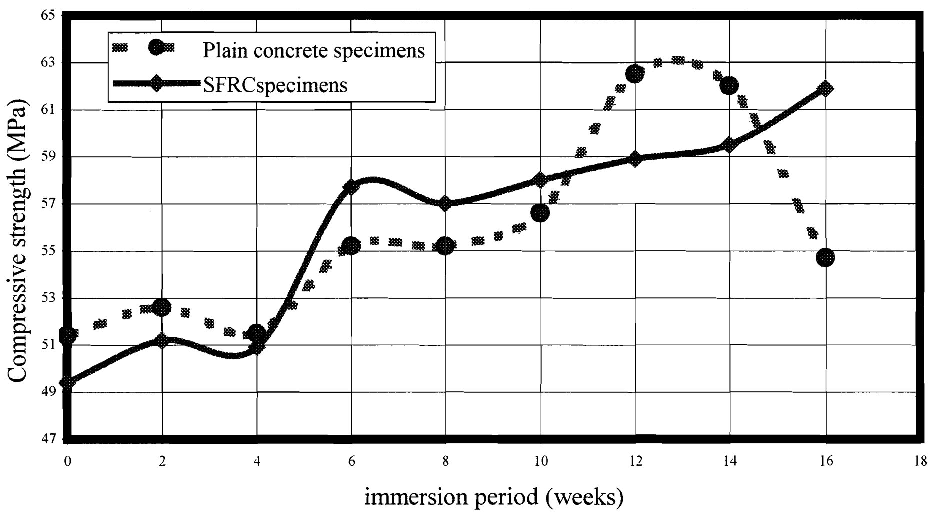

Figure 1. Compressive strength of high strength concretes

After 8 weeks of seawater exposure most of the plain concrete specimens failed in an explosive manner. SFRC specimens on the other hand remained intact after failure. This might have been the effect of the enlarged head of the steel fibres that resists lateral tension of the specimen to a greater extent.

SEM analysis provided the chemical composition of the specimen. Several chemicals including sodium, potassium and calcium were detected(Hudson 1998). Only the distribution of Cl ion concentration is shown in Figure 2. It could be seen from the graph that up to 2 weeks of immersion into sea water, there was no chloride ion infiltration in both plain and SFRC concretes. However after 16 weeks of sea water immersion, both concretes have suffered chloride ion infiltration – SFRC exhibiting more or less uniform levels of Cl concentration throughout the body and the plain concrete exhibiting higher levels of the chlorine concentration closer to the surface. This indicates that the porosity of SFRC is larger than that of the plain concrete.

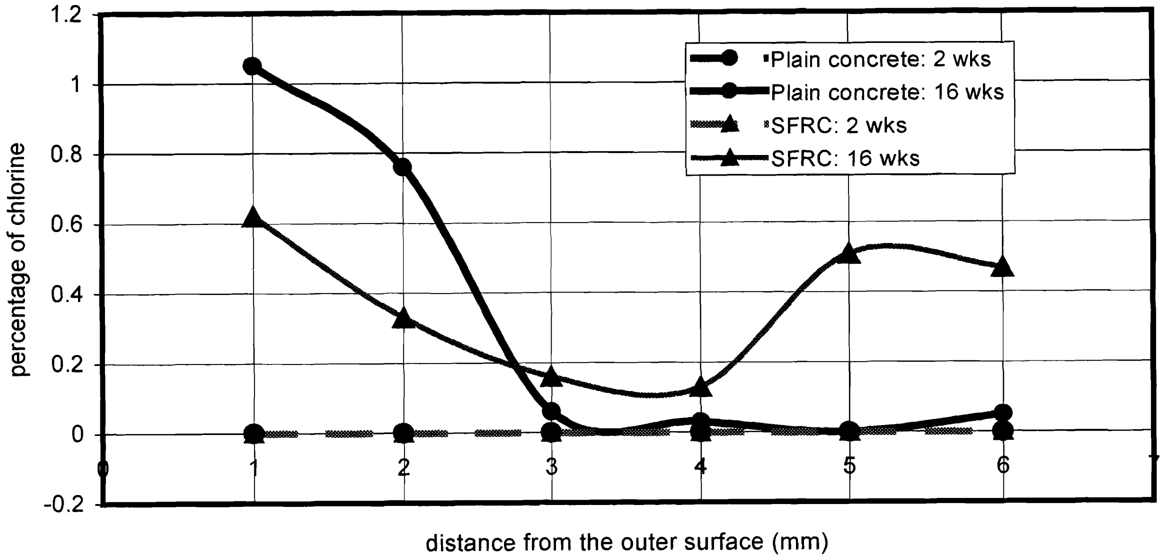

Figure 2 Chloride profile of high strength concretes

Normal strength concrete.

Normal strength SFRC mix was designed with target strength 32MPa (Bako 1997). The amount of steel fibre used in this investigation was established by an earlier thesis. This figure, three percent by volume of concrete, was proven to provide optimum strength to the concrete as opposed to the economic optimum fibre content. The decision to utilise 22.5% fly ash by weight of cement in the test mix was made due to the information received from industry source. The above mentioned source went on to recommend that no more than 25% cement be replaced by fly ash, as an industry rule-of-thumb. However, literature on this topic suggests that the optimum content of fly ash for performance and workability lies between twenty and forty percent by weight of concrete (Wesche 1991; Cao 1997, Chen 1997, Wong et al 1997; Abdul Awal and Warid Hussin 1997; Butler 1997; Hannant 1978; Swamy 1984). The W/C(0.55) and A/C (4.4) remained constant through out the study.

The compression test results of concrete is shown in Figure 3. The compressive strength of specimens increased over the sixteen weeks with an impressive 30% increase in strength.

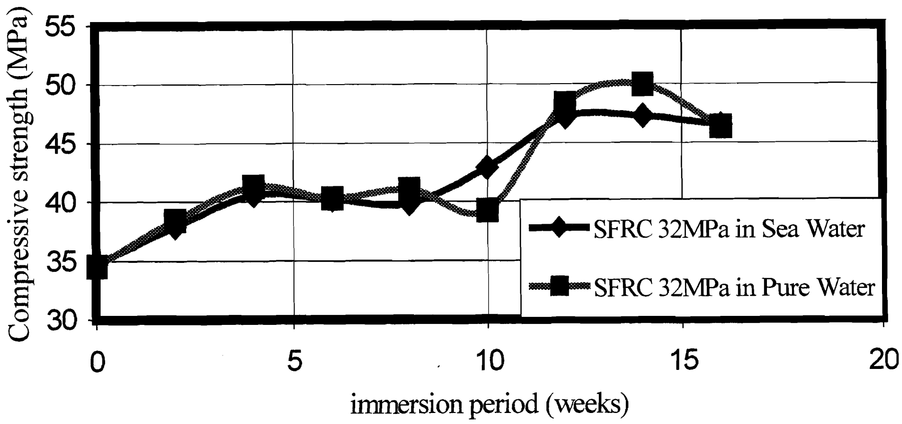

Figure 3 Compressive strength of normal strength concrete

Upon compressive failure, pure water specimens generally displayed hairline fracturing while sea water specimens displayed a widespread network of cracking as shown in Figure 4.

(a) Specimen in pure water 16 weeks (b) Specimen in sea water 16 weeks

Figure 4 Failure mode of normal strength SFRC specimens.

Literature research into this topic also revealed that infusion is usually limited to 12-15mm from the surface (Sirivivatnanon et al 1997). Hence scanning electron microscope study was carried out on specimens over a length of 12mm in six, 2mm-steps. Detailed information regarding the percentage concentration profiles of elements (aluminium, silicone, calcium, sulphur, potassium, iron, titanium, magnesium, sodium and chlorine) were studied (Murphy 1999). Figure 5 shows the percentage profile of chloride ion in normal strength SFRC for a distance of 12mm from the surface over a period of 11 weeks exposed to sea water.

Figure 5 Chloride ion profile in normal strength SFRC concrete

For a period of two weeks exposure the SFRC specimens did not show the trace of chloride ion penetration irrespective of the type of water. SFRC immersed in sea water, as anticipated, has shown a remarkably increase in the level of chloride penetration after the period of 16 weeks. The magnitude of the chlorine detected in the pure water sample remained lower than that detected in sea water samples. A summary of the chloride ion penetration is shown in Table 1.

TABLE 1 Effect of Steel Fibre on Chloride Ion Penetration

Concrete type		f'c MPa	Chloride ion penetration after 16 weeks of immersion in sea water		
			2mm	6mm	12mm
HSC	Plain concrete	51.4	0.8	0.1	0.0
HSC	SFRC	49.4	0.3	0.5	
Normal	SFRC	34.7	0.9	0.2	0.1

From the above table it could be concluded that addition of steel fibre reduces chloride ion concentration, the lower the strength of SFRC, higher the chloride ion concentration.

PROJECT #2 **PACIFIC LIME - WASHED LIMESTONE**

The objective of this study is to investigate the effect of replacing sand aggregate by washed limestone on the fresh and hardened properties of concrete. At this stage washed limestone has been used as a filler material. The particle size distribution of the material and the chemical composition were analysed. Sieve analysis results of the washed lime stone (WLS) was compared with the particle size distribution graphs of fine and coarse sand particles as shown in Figure 6.

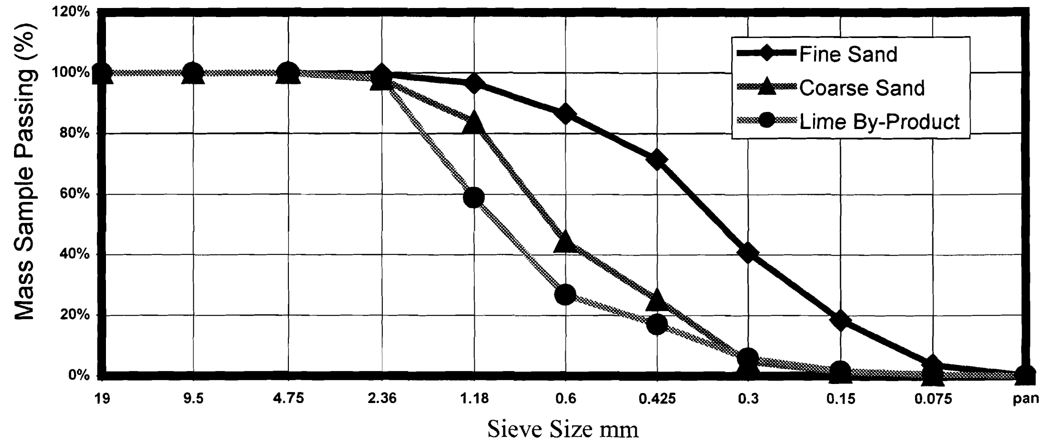

Figure 6 Particle size distribution

Figure 6 shows that the coarse sand particle size distribution curve is closer to the particle size distribution curve of the WLS. Hence it was decided to replace the coarse sand by washed limestone and to study the fresh and hardened properties of concrete.

Chemical composition:
The chemical composition of washed limestone and coarse sand were analysed using scanning electronic microscope (SEM) and the data are tabulated in Table 2.

When writing results from the chemical analysis, the SEM system groups all elements with atomic numbers 10 and below together and labels the group as "Matrix". This occurs due to the low energy x-rays emitted by these elements; such low-energy x-rays can not pass through the beryllium window within the SEM detector. The picture of the sample showing a representation of grains is shown in Figure 7.

TABLE 2 Chemical Composition of WLS

Sample Identification	Ca	Si	Al	K	Fe	Na	Other Matrix
Washed limestone	45.8	4.0	4.2	0.4	1.4	0.0	44.0
Coarse sand	0.0	31.2	4.1	3.1	1.2	0.9	

Figure 7 View of washed limestone under SEM

Properties of concrete with WLS aggregate:

The concrete mix was designed with reference to grading curve 3, for a target strength of 32 MPa (Bako 1997). The coarse sand particles are replaced by washed lime stone in steps of 10% and the fresh and hardened properties were studied. The results were compared with concrete without washed lime stone.

A sample graph of grading curve 3 and the simulated mix design curve with 60% addition of washed lime stone is given in Figure 8.

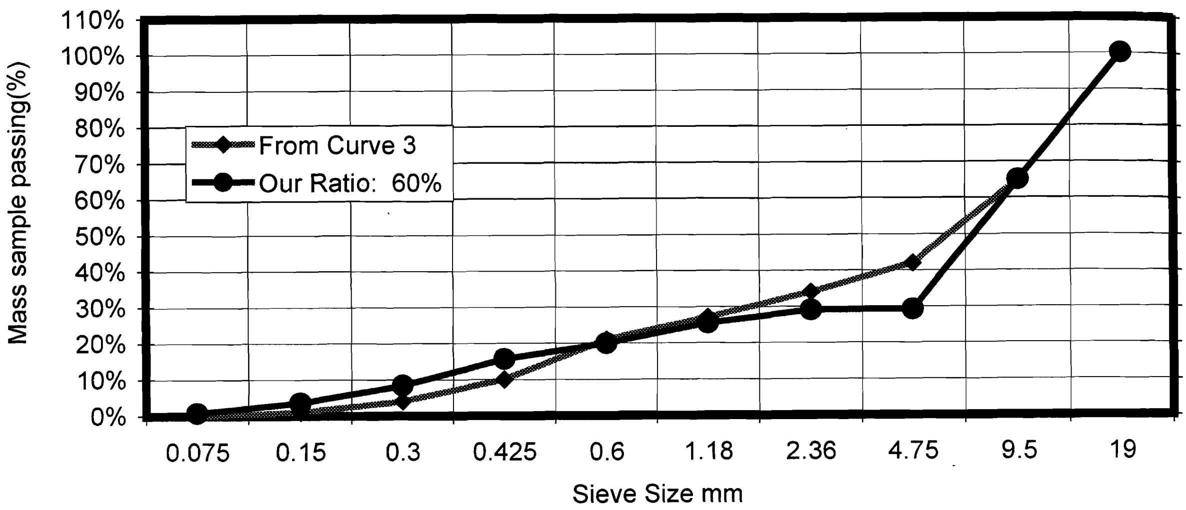

Figure 8 Grading curve3 and simulated mix design curve

The W/C ratio of 0.57 and A/C ratio of 5.1 remained constant throughout the testing program. The specimens were allowed to cure for 28 days under moist condition. The workability of the mix was calculated by slump and compaction factor methods. Slump values are given in a graphical form as in Figure 9.

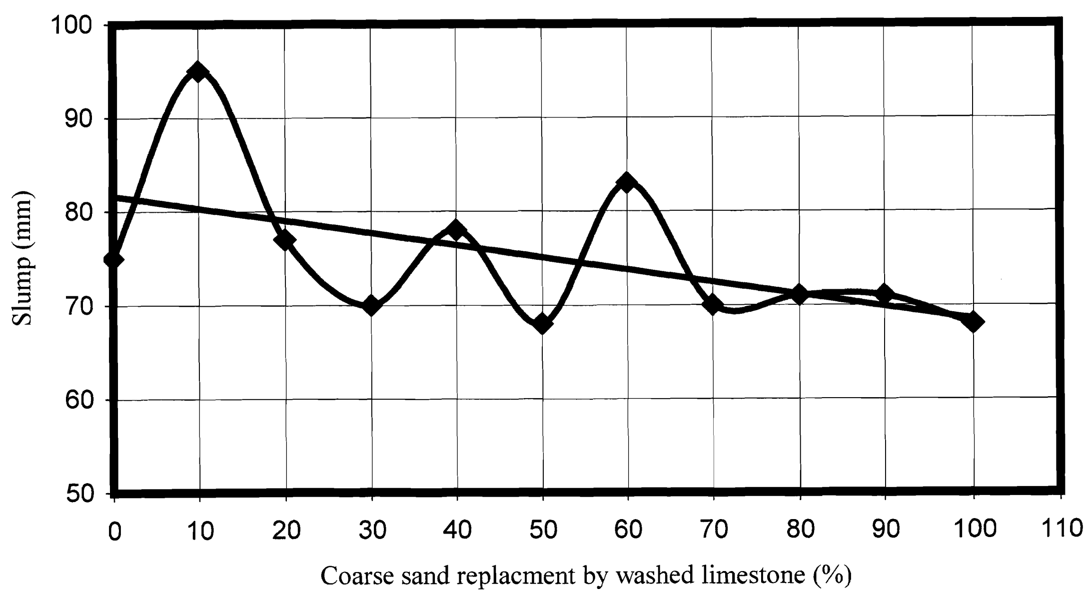

Figure 9 Workability of concrete mix with washed limestone as an aggregate.

In the above figure the percentage addition of washed limestone is given on the x-axis and the slump value is marked on the y-axis. From Figure 9 it was clear that as the percentage of washed limestone increases, there is a decrease in the amount of workability. The trend line also explained the same.

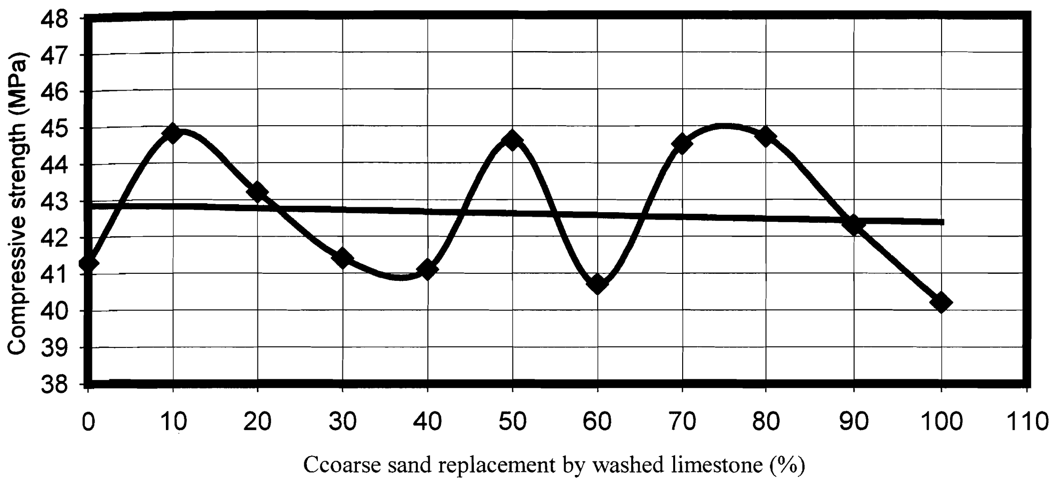

Figure 10 Compressive strength of concrete mix with washed limestone

Figure 10 shows that the compressive strength is reducing gradually as the amount of washed lime stone is increased. However the compressive strength remained above the target strength.

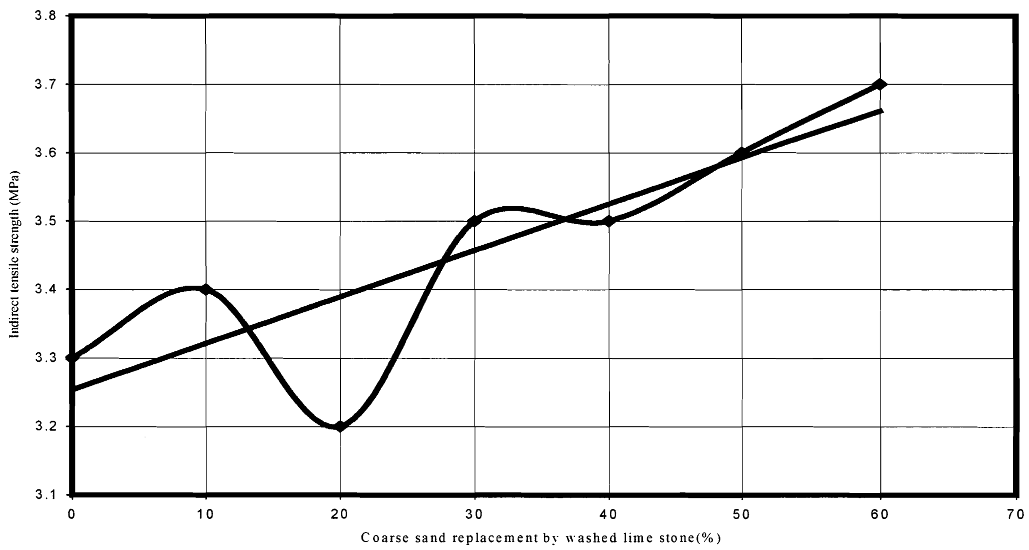

Figure 11 Indirect tensile strength with addition of washed lime stone

The tensile strength with washed lime stone shows an increase in tensile strength with the percentage increase in WLS. At 0% WLS level, the tensile strength was roughly 7.3% of the compressive strength whilst at 60% WLS level the tensile strength was 8.5% of the compressive strength.

In summary, the replacement of coarse sand by washed limestone decreases the workability, increases the tensile strength and decreases the compressive strength. The durability studies are not yet carried out.

Acknowledgement:

The author wishes to acknowledge the industries, Pacific lime, Queensland Cement limited, Stanwell power station, CSR Humes, BHP, CQU research fund and her project students for their support.

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HIGH PERFORMANCE CONCRETE – A MANUFACTURERS PERSPECTIVE

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SUMMARY

Manufacturing industry is increasingly faced with matching the need for higher performance of materials with increasing environmental demands. The drive to improve the long-term performance of concrete and concrete structures must continue, but it must enable manufacturing industry to utilise different techniques, raw materials and fuels if this improvement is to meet environmental and sustainability requirements.

The cement manufacturing industry has made significant advances in these areas over recent years. Many of these advances have addresses performance based issues and the associated regulatory, standards and specification changes that are needed in future to further enhance this process and facilitate the push for higher performance concrete and concrete structures.

WHAT GIVES CONCRETE A HIGH PERFORMANCE?

The majority of discussions on High Performance Concrete (HPC) are merely an analysis of high strength concrete under another name. Indeed it may be that as high strength was not acceptable to the industry or researchers, discussions moved toward high performance to gain the recognition and support needed for the work.

Many specify high strength in the hope they will get high performance, as they have nothing else they can set down and that can be realistically assessed.

The two terms are not the same, as both technology and performance in the field has proven. The enormous growth of the concrete repair industry is testament to this fact.

Strength alone is not the answer to most of the problems that beset the concrete industry and is not the answer to the sustainability needs of the industry in future.

High performance comes from the ability to perform the required duty over the entire life of the structure with little maintenance or upkeep.

Aspects such as

Low shrinkage
Freedom from cracks
Chemical resistance
Abrasion

All become important

Other issues, such as Alkali Aggregate Reaction can be added, particularly with the high cement contents used in HPC.

High performance is all about getting it right the first time.

Are the benefits being sought really achieved through High Performance Concrete (HPC)?

It is important to assess the real needs as this issue is explored.

What is the ultimate aim?

High Performance Cement
High Performance Concrete
or
High Performance Structures

Each has different implications, although the last, which reflects the needs of the asset owner, depends on the others.

At what level the performance is measured is a critical question to the future development of materials, performance criteria and standards.

BENEFITS OF HIGH PERFORMANCE CONCRETE (HPC)

There is no doubt that benefits in moving toward HPC accrue to the User/Owner, Contractor, Producer and Society

Quality Issues

The industry must focus more on the quality of the finished structure.

The key ingredient is the measurement of the potential performance of the concrete as delivered to site and this must then be matched by the on site practices.

While efforts to produce high performance cements are to be applauded that material is very early in the food chain and any benefits can be negated by subsequent actions.

It is important to ensure the performance measurement is moved as close to the completed structure as possible.

Environmental Issues

It is important to add environmental issues to the debate.

Blast Furnace Slag, Fly ash, Silica Fume, Limestone and other materials in future have a positive impact on this issue.

There remains some resistance to the use of these materials in certain sections of the industry. However it should be questioned as to whether use of these materials in cement and or concrete really matter provided the performance criteria are met?

Production of High Performance Concrete

To be fully effective and enable the HPC to be produced in a sustainable and environmentally friendly manner requires a change to the way in which standards and regulations for materials are approached.

Put simply it requires a greater move toward performance standards and specifications.
Currently the industry is faced with a difficult mixture of these and prescription standards.

Australian Standards

The Australian Standard for Concrete, AS1379, uses performance criteria for slump and strength but that is the limit.

There are no performance criteria for other parameters that are of any practical value.

In many situations properties other than strength are of greater importance, except they are hardly ever formally required or proved. Hence concrete may be seen to fail in these situations and the producer and contractor shoulders the blame even though they were not asked to meet any criteria other than strength.

It is important to determine the best assessment methods and performance parameters to enable these special requirements to be specified in addition to strength.

AS3972 Portland and Blended Cement has made excellent progress in terms of performance criteria for aspects such as Sulphate resistance, but meeting those does not guarantee performance of a concrete structure.

To promote this adequately, the industry needs to determine the key performance criteria for materials, to establish the practical tests and assessment methodology for these and then to let the manufacturing industry use whatever raw materials they require to meet those criteria.

Any attempt to impose prescription requirements on top of the performance criteria will lead to confusion, dispute and will not move the industry forward.

HPC is more than materials

It becomes increasingly obvious that the achievement of high performance structures, ie the ultimate aim, also involves on-site activity.

This may simply refer to aspects of construction such as
Compaction
Protection and Curing
Finishing

Historical developments in materials and techniques have not provided the solution to many of these issues. It is perhaps time to recognise they are not going to be performed to the ultimate degree on many sites and to commence developing materials that will eliminate the need for these practices.

Consider the development of HPC materials such as:

Self Compacting Concrete
Self Curing Concrete
Shrink free concrete

Design and Specification

HPC also requires additional attention be given to design, detailing and specification

- Ensure the concrete specification is suited to the project
- Ensure it is practical
- Ensure it is possible to meet with the materials available
- Watch the reinforcement detailing
- Ensure adequate crack control provisions
- Detail joints correctly

Where these and other similar areas are not covered it will be impossible to meet the requirements for high performance structures regardless of the materials used.

Whatever is specified must
- Be practical
  What may be achievable in one area may not in another
  eg Shrinkage is affected greatly by the aggregate type.
- Suit the concrete elements being produced.
  Even in a single structure one method may not suit all elements and general specifications adopted from previous structures frequently do not suit the new development.
- Ensure the specification is kept up to date with latest developments in materials and techniques.
  At the very least ensure that it covers the most recent Australian Standards.

PERFORMANCE SPECIFICATION

Prescriptive standards can work provided the criteria are adequately understood and the specifier is willing to take the responsibility for the final performance.

However such specifications tend to limit innovation and this can have a marked effect on issues such as sustainability.

- They restrict the use of new materials
- They limit the energy and environmental savings

One should always question whether the make up of the concrete or the chemistry of the cement is of any real interest to the asset owner.

Ultimately it is more important to develop the performance criteria to be met in the completed concrete structure. This is the asset owners' real intent.

In the interim, in terms of cement and ultimately concrete, it is necessary to lay down additional performance criteria for the products, but having done so to leave it to the manufacturer to use methods and materials required to provide this performance.

Performance Criteria

To establish realistic and useable performance criteria involves a number of matters.

- Adequate definition of the performance parameter
- Setting the criteria for the specification of the particular performance property.
- Development of the test/assessment method

This development must be technically defensible and internationally benchmarked and reflect products that are known to perform. It must stand on its merits and not merely pass everything that is currently in the market. The test method must be a clear, reliable method that can be used quickly to measure the performance parameter

AS3972 Portland and Blended Cement

AS3972 has undertaken some significant work on performance criteria for cement.

It includes performance criteria for soundness and cement strength. The latter is based on the European ISO test with modifications to suit Australian temperature conditions.

It assesses criteria for maximum temperature rise in cement through calorimetry.

Most significantly it developed performance criteria and tests for Sulphate Resisting and Shrinkage Limited Cement.

This necessitated:
- Assessing methodology world wide
- Developing criteria for performance
- Setting limits for criteria
- Benchmarking criteria world wide
- Developing suitable test procedures

The process has been successful and the Australian cement standard is one of the most advanced performance based cement standards in the world.

As a result it is also one of the most simple and easiest specifications to use and permits good flexibility for manufacturers to use a range of raw materials.

It permits up to 5% Mineral Fillers in cement and hence provides manufacturers with a degree of flexibility in meeting the technical product needs while lessening impacts environmentally. Limestone, Fly ash and Slag may be used in this manner.

However even this Standard is not the ultimate performance document.

It is still restrictive in what the manufacturer can use to make cement and this has great impact on the industry's ability to meet future Greenhouse targets and to utilise waste materials from other industries and hence benefit the environment.

Sustainability of concrete is adversely affected by these restrictions and they must be lifted by developing more performance based criteria and then gaining acceptance for these from the design, specification, contracting and owner sections of the industry.

This is neither an easy, nor a short-term development proposal, but it must continue if those objectives are to be met.

Concrete Specification

Strength is currently the governing criteria for concrete properties and the basis for most specifications, but it is now well recognised that other parameters need greater attention.
Durability is frequently mentioned but there is not and cannot be a performance parameter set for durability per se.

This condition is too broad. It is necessary to look at the various aspects of durability and to establish performance criteria and tests for these.

Conditions and situations that demand attention include

Sulphate resistance
Chloride resistance
Abrasion
Reinforcement Corrosion
Waterproofing
Shrinkage
Absorption
Sorptivity

Carbonation
Alkali Aggregate reaction

These are all issues that provide the need for performance criteria to be set. Whether these criteria should be assessed in cement or concrete or the completed structure or a combination of these is open to debate.

USE OF MINERAL COMPONENTS

Great use is being made of a variety of other materials to solve technical, manufacturing and environmental issues.

The principal materials being used within the cement and concrete industry are:
Limestone
Fly ash
Blast Furnace Slag
Silica Fume

These have great benefits technically and also have impact on the energy reduction and environmental issues facing manufacturing.

Almost all concrete produced in Queensland contains Fly ash and Australia has led the world in the development of the use of this material and hence gained great advantage.

Technically no advantage is greater than the capacity the Queensland Fly ash has for eliminating the potential trouble caused by Alkali Aggregate Reaction.

Use of these materials enables a greater production of concrete without the need for additional energy and has the long term potential to extend the life of existing cement plants, thereby bringing economical and environmental benefits to Australia.

SUSTAINABILITY

Every industry must look to the production of sustainable solutions and materials and consider, in all its actions, the impacts on these issues

Minimisation of energy use, the use of waste materials, elimination or minimisation of waste and reduced Greenhouse impact are just some of the issues.
Unfortunately they are frequently being assessed totally at the manufacturing industry level without adequate assessment of the impact the structures created have over their lifespan.

Energy consumption and greenhouse are important to the long term performance of all structures and not just to the materials with which they are made and these issues must be handled over a life cycle time frame rather than a short-term construction time frame.

The massive nature of concrete can provide significant long term energy and emission savings which can make it a material that places less impact on the environment than alternatives.

Sustainable development involves increased life spans and improved quality of products such that maintenance is low. The structure must fulfil the serviceability requirements and function adequately over its entire life.

The lifetime performance of concrete structures is reasonable but with improved performance parameters it can be improved.

While this performance must be realised with the lowest possible environmental impact, it must also be recognised that all activity results in some degradation. The challenge is to minimise this and set the limits at acceptable levels. They cannot be zero.

Adoption of HPC principals will lead to more efficient use of resources than that attainable with lower quality materials.

PRODUCTION ISSUES

Sustainability and Environmental issues have impacts on manufacturers.

- Production must not produce waste materials that cannot be recycled or used elsewhere.
- Any by products produced should not be harmful

The Cement industry does not impact on this in other than the Carbon Dioxide Greenhouse issue.

Dust emissions

Dust emissions in new cement plants are negligible

Carbon Dioxide Emissions

Carbon Dioxide emissions are of concern to the industry and come from three sources.

- The fuel – thermal emissions
  There are methods being used to reduce these
- The chemical process - unavoidable process emissions
  This produces around 1 tonne of CO_2 for every tonne of clinker produced.
  This is the only real ecological issue with cement production
- External power sources eg power stations (indirect emissions)

The use of Fly ash and Slag is important in this carbon debate.
Queensland concrete uses around 25% of replacement materials, in this case chiefly Fly ash, to replace Portland cement. This reduces the need to produce clinker and hence reduces the total volume of carbon dioxide produced from the conversion of limestone.

Energy

Energy conservation is vital and the cement industry in Australia has made large scale changes to its operation over recent years providing significant reduction in energy consumption.

The change to dry process plants, such used in QCL's Gladstone plant, can reduce heat needs by as much as 45% when compared to the older wet process plants

The industry also uses alternative fuels such as used tyres to provide thermal benefit and raw materials that are needed for the production of cement and as a result it overcomes one of the major environmental issues, that of disposing of old tyres. This can also extend to other materials such as oils, solvents and other solid fuels.

- Some are fuel replacements
- Some provide replacements for materials that would otherwise need to be mined
- Some reduce cost

All are beneficial in a sustainability sense.

Greenhouse Challenge

The cement industry is a signatory to an agreement with the Greenhouse Challenge Office and measures and reports on energy use and emissions as part of this agreement.

The management systems introduced clearly show a reduction in the gas emissions of recent years

While acknowledging these issues it is also important to address these in a manner that is not detrimental to the Australian manufacturing industry.

Any policy that advantages imported product over those locally produced will not provide reduction in greenhouse impact. Indeed if the imported materials are produced from less efficient plants, it will not only merely shift the location of the source, but may increase the total impact.

MANUFACTURING CHALLENGES

Modern Cement Manufacturing Technology

Modern dry process cement plants:
- Minimise the CO_2 emissions
- Use energy efficiently
- Substitute CO_2 producing ingredients with other material where possible
- Provide excellent recycling of waste materials

The cement industry is working towards sustainable solutions and this provides challenges for the future.

- Continued effort to reduce the production of waste
- Reduction in greenhouse gases
- More efficient use of resources
- Increased recycling or multiple use of materials

Recycling

Sustainability has implications for efficient use on non renewable resources and recycling of materials that are otherwise merely dumped in landfill. The cement manufacturing process is uniquely situated to provide these improvements.

High Performance Concrete offers benefit as it can be used (recycled) a number of times

- Firstly it can be recycled as concrete aggregate
- Secondly it can be used again as fill and road base aggregate

Low grade concrete can generally only be recycled once back into road base and not into concrete and therefore misses the opportunity to be recycled twice.

Hence HPC offers advantage in recycling.

While concrete can be recycled it is evident that it cannot produce a material that performs as well as the original and there are significant commercial implications that must be resolved to make it viable.

THE FUTURE

There are many developments under way that will enhance the high performance nature of concrete.

- Reactive Powder Concretes
  Imagine concrete stronger than steel and almost as ductile. Strength measured in gigapascals.
  Currently 200-400 MPa is possible and only require a breakthrough on the economical front.

- Problem solvers
  Work is progressing on products to solve particular problems such as:

  Self Compacting Concrete
  Self Curing Concrete
  Non Shrink Concrete
  Waterproof Concrete

HIGH PERFORMANCE CONCRETE – THE MATERIAL OF THE FUTURE

Cement based materials such as concrete are perhaps the most widely used building material in the world. The developed world is literally built on these materials and they will play a dominant role in the developing countries as they cope with the needs for modernisation and population growth.

High Performance Concrete

- **Conserves valuable resources**
- **Saves energy over the life time of structures**
- **Reduces waste**
- **Minimises Greenhouse impacts over the life of structures**
- **Provides durable, functional and efficient structures**

The future for High Performance Concrete is huge and the benefits to the industry and society are great. However it does require the development of suitable performance criteria and the assessment methods that match those criteria.

Equally the design, specification, construction sectors of the industry and the asset owners need to accept these performance parameters, to enable manufacturers to meet the performance needs in a sustainable manner and with due consideration of all the environmental influences so important to the future.

LIFE CYCLE COST ADVANTAGES OF USING STAINLESS STEEL IN MARINE ENVIRONMENTS

GREG YORKSTON
Managing Director
Arminox Australia P/L

INTRODUCTION

Today I will be discussing with you the life cycle cost advantages of using stainless steel as reinforcing in concrete structures in marine environments. My discussion will be trade based as I have been working with stainless steel for over 20 years. I will address this topic using the following 4 headings.

O/H 4 topic headings

- ❖ The problem. The corrosion of reinforced concrete structures
- ❖ The cause. Why does the reinforcing corrode
- ❖ The solutions. A brief examination of the main solutions
- ❖ The stainless steel solution. A study of the stainless steel family and why I believe it to be the most cost effective solution to the corrosion problem.

The Problem

Concrete structures in marine applications have always sustained major deterioration due to the corrosion of carbon steel reinforcing by the chloride ions, which travel through the concrete cover. Upon reaching the reinforcing they set up a corrosion cell and so begins the degradation of the structure. (O/H2). The speed of this degradation is reliant on several factors.

- ❖ The temperature of the surroundings.
- ❖ The relative humidity
- ❖ The salinity levels around the structure

- ❖ The proximity to the splash zone (suction action due to wetting and drying)
- ❖ The density of the concrete cover.

Such corrosion will always cost the owner money for the repairs to, or replacement of the structure. However, should these repairs not be timely then the result could be the loss of human life and this is a cost that no community can afford to pay.

The Cause

O/H Corrosion Cell

All of the above factors determine the rate of the onset of corrosion but once the corrosion cycle has started its rate of destruction can be exponential. As corrosion is an electro - chemical reaction the reaction will continue so long as the required elements of the equation remain in place. These elements are, chlorides, oxygen and a transport mechanism for the chlorides such as water. If we remove the transport mechanism there will be no reaction. This may sound very basic and it is, **in the short term**. But how is this achieved in the long term? The simple answer is that it isn't or the millions of dollars being spent worldwide on rehabilitation could be used more beneficially elsewhere.

The Solutions

There are several options that can be taken to delay the commencement of rebar corrosion, and I will examine these briefly (O/H) I must stress however, that no one of the following solutions gives total long term, maintenance free protection against rebar corrosion. I suggest for long life cycles, in excess of 50 years, one or several of these options be used in concert. I believe this to be current industry practice.

O/H How to avoid corrosion

- ❖ **SURFACE TREATMENTS/ SEALANTS :** Are effective in blocking chlorides only in the short term

- ❖ **CONCRETE COVER:** The most effective barrier to the ingress of chlorides is the use of an adequate cover of good quality concrete. Much research is continually being carried out to bring us better concrete. We now have what are termed HPC"s or high performance concretes. They do form very dense barriers to the ingress of chlorides but do so at a cost to workability and a lowering of the protective pH level of the concrete.

- ❖ **SURFACE COATED REINFORCING:** The use of epoxy coatings and or galvanizing as a single barrier to chloride ingress in a marine environment are at best short term and at worst foolhardy. Evidence is available to demonstrate that galvanized steel offers a marginal life extension and many volumes have recently been written on the dangers of using epoxy coatings in chloride effected concrete. I refer you to the US road administration reports.

- ❖ **CORROSION INHIBITORS:**. The theory behind the use of chemical inhibitors that are homogeneously in the mix, then travel to the reinforcing to protect it from the aggressive chlorides is I believe just that theory. That these highly mobile inhibitors will still be in place at the rebar in say 20 or 30 years time is yet to be proven. I admit to being a skeptic.

❖ **CATHODIC PROTECTION:** Cathodic protection is a widely used, very effective tool in the fight against corrosion. CP systems range widely in complexity and therefore cost. The one drawback that I see is that these systems have to be continually monitored and maintained adding to the life cycle cost of a structure.

❖ **NON CORRODING REINFORCING:** The use of non-corroding reinforcing is relatively new due to the advent of glass fibres and carbon fibres. Both of these forms have their qualities and their weaknesses that have meant very limited use to date. Stainless steel reinforcing however, is a very effective material when used as reinforcing in marine applications. Let us examine stainless steels more closely

Stainless Steels:

Stainless steels are a family of metallic materials with a huge range of physical and mechanical properties. They are loosely defined as steels containing a minimum of 10.5% chromium. There are 5 main members in the stainless family, they are: austenitic, ferritic, martensitic, duplex and precipitation hardening but today we will discuss only three (3) Ferritic, Austenitic and duplex.

Most of us will be familiar with the austenitic grades commonly referred to as the 300 series. They are non-magnetic and in addition to chromium (typically 18%) they contain nickel (which increases corrosion resistance). Grades such as 304 are used in our kitchen sink, bench tops etc and of course the marine grade 316 is used for applications on and around the sea. 316 gains its added corrosion resistance from the addition of molybdenum.

The ferritic grades are magnetic, have low carbon content and contain chromium as the main element, typically from 13% to 17%.

Duplex grades have a mixed ferritic / austenitic structure. Chromium varies from 18% to 28% and nickel from 4.5% to 8%. Duplex grades have a higher strength and an increased corrosion resistance in comparison to the austenitic grades. The development duplex stainless steels are very significant, as it will be these grades that will revolutionize the use of stainless steel in the construction industry.

O/H Composition of stainless steel
O/H Stainless steel Characteristics

Stainless steels gain their corrosion resistance from a naturally occurring chromium rich oxide layer, which is present on its surface. This invisible film, microns thick, is inert and tightly adherent to the metal, which allows the film to reform instantly when the surface is damaged. This self-healing action requires the presence of oxygen, but even very low amounts will be sufficient. Special attention must be given to restoring this layer after welding and I will discuss this shortly.

O/H Time Initiation of Chloride induced Corrosion
O/H Pitting Resistance Equivalent

WHY USE STAINLESS REBARS:

O/H Rebar bundles
O/H Grades available in stock

Stainless steel rebars are produced in 4 grades Gr304, Gr316, 316Ti and Duplex 2205. Sizes from 3mm to 16mm are produced by cold rolling. It is this rolling process that increases the tensile strength as most stainless steels characteristically work harden. Hot rolling produces the sizes from 18mm to 40mm in the grades Gr304LN, 316LN and duplex 2205.The high strength in the hot rolled sizes is gained by the addition of nitrogen.

In Australia only 2 grades are stocked, Gr. 316 /316LN and Duplex 2205. This is based purely on an economic basis. The main use for stainless rebars are in marine environments and Gr316 is the lowest grade to use in these applications

I commented earlier that the duplex grades would revolutionize the construction industry, why is this? There are 3 main reasons

- High strength
- High corrosion resistance
- Competitively priced (due to low Nickel content)

The yield strength is shown on the overhead. Higher strength allows the engineer the flexibility to use smaller diameter bars to achieve the same strength. Stainless is sold by weight, hence considerable savings can be achieved. The higher corrosion resistance speaks for itself. The price advantage comes indirectly by way of the nickel content. Duplex grades have lower nickel contents and it is nickel that drives the price of stainless steel. Nickel accounts for 30% of the cost in making stainless steel.

Building codes now require workmanship and materials to deliver a lifecycle of 50 years as standard. Further, demands by owners for no maintenance periods of 100 years or more are becoming commonplace. (refer Walsh Bay redevelopment on Sydney Harbour). On such projects, designers and contractors alike are being required to give a written guarantee that these life cycles will be achieved. Are these demands reasonable? Are they achievable? I can confidently say yes to both these questions if stainless steel is used.

The following example highlights the reason for my confidence. We have living proof that stainless steel can stand the test of time.

Pier Progresso Presentation

The pier Progresso was constructed under the worst possible conditions. These were

- Using seawater as the pour water.
- Crushed coral as aggregate.
- High salinity
- High temperature variation
- High humidity
- Porous concrete cover (<30mm average)

Stainless steel has lasted 63 years and very possibly will last another 60 years. I need say no more

The ability of stainless steel to withstand extreme marine attack is further documented in a report by Professor I.D. McGregor. At the time of the report Prof. McGregor was Associate professor Dept. of Building & Construction at the City University of Hong Kong.

(O/H) No1 the Strand Wellington New Zealand

THE COST OF STAINLESS STEEL V LIFE CYCLE COST

O/H Port of Brisbane

The initial cost of stainless steel rebar is in the order of 6.5 times that of carbon steel rebar. This is a very significant difference. It is obvious that with a price tag like this stainless steel is not the answer in every case. But , as previously stated "no one solution was applicable when life cycles in excess of 50 years were expected"
We at Arminox have coined a phrase, *The Intelligent Use of Stainless Steel.* What this simply means is that stainless steel should be used in the critical areas of a structure and carbon rebar is used in the balance of the areas. Typically, this could mean using stainless steel in the order of 15% maximum (depending on the structure) thus adding an insignificant amount to the overall project cost.

O/H Intelligent use of stainless

STAINLESS STEEL IN CONTACT WITH CARBON STEEL REINFORCING IN CONCRETE
Many engineers have an unfounded fear of using stainless steel and carbon steel together in the same concrete structure. Research carried out by the Force Institute of Denmark (formerly the Danish corrosion Centre) has shown that this is in fact good safe practice.

Stainless steel freely exposed to seawater may if in galvanic contact with a less noble metal, such as carbon steel, initiate a rapid galvanic type of corrosion of the less noble metal. The otherwise slow cathodic oxygen reduction at the stainless steel surface is catalyzed by a bacterial slime, which forms after a few weeks in salt water.

When cast into concrete, however, the cathodic oxygen reaction is a very slow process and no such catalytic activity takes place on the stainless steel surface. Force Institute research indicates that the cathodic reaction is inhibited on stainless steel embedded in concrete, by comparison to the cathodic reaction on ordinary carbon steel reinforcing in galvanic contact with corroding carbon steel.

Consequently, connections between stainless and carbon steels will not promote galvanic corrosion. As far as corrosion of stainless steel is concerned a galvanic connection between stainless and carbon reinforcement would result in a partial cathodic protection of the stainless steel, as a consequence of the lower passive potential of the carbon steel. There fore stainless steel is an excellent material to use for all components, which are only partially embedded in concrete e.g.: bolts, ladder rungs inserts etc.

The fact that stainless steel is a far less effective cathode in concrete than carbon steel offers possibilities for applications in repair projects. When a part

Of the corroded reinforcement, close to the surface, needs to be replaced it is advantageous to use stainless steel instead of carbon steel. Because stainless steel is a poor cathode it will minimize eventual problems which may occur in neighboring corroding and passive areas after repair.

Copies of the latest Force Institute report are available on request. Please ask me at the end of the session

At a recent seminar held in Australia Dr. Steen Rostam and Dr. Gro Markeset, (both eminent spokespersons in the field of durable concrete structure design)
discussed the use of stainless steel reinforcing in marine environments. Both Drs. have come to the same conclusion.

I quote Dr. Rostam. *"During the past few years stainless steel reinforcing has become commercially available in dimensions strengths and alloy types which are fully compatible with normal structural requirements for reinforced concrete structures- and at competitive prices relatively speaking. Such Stainless reinforcement may be corrosion resistant even in highly chloride-contaminated environments. Used selectively, in the most exposed zones of the structure the increased cost per kilogram of stainless compared to the cost of normal steel will have a marginal or negligible effect on the overall initial construction cost. In addition the service life cost will be reduced due to savings in future repairs and maintenance. (Concrete Society (1998)) (Abbott (1999)).*

Copies of both Dr. Rostam's and Dr. Markeset 's papers are available for a nominal fee. Please see me after this session for further details.

SUMMARY

In summary we should look at the direct cost advantages of using stainless steel in an intelligent way.

O/H Advantages of stainless steel rebar

ADVANTAGES

- No maintenance
- Excellent mechanical properties (Corrosion resistance/ strength)
- Easy to work and readily weldable
- Easily handled on sight
- Traceable from cradle to grave
- Certified , documented , controlled
- Improved life cycle economy
- Decreased concrete cover (refer comments by Dr. Gro Markeset)

PRECAUTIONS:

O/H Precautions

- Stainless steel should be stored separately from carbon steel on sight
- All welds should be carried out using the correct stainless consumable
- After welding the welds should be cleaned by mechanical or chemical means to reinstate the chromium –oxide layer. Specification sheets available.

Over the past few months. Arminox Australia has conducted many seminars both in Australia and New Zealand on the use of stainless steel as one option to allow our engineers to make durable concrete structures with the confidence that these structures will stand the test of time. As a direct result of our efforts here, and similar organizations in Europe, Canada and North America I can confidently say that stainless steel reinforcing is now very widely accepted and it is being specified in many major projects.

In Australia, there are now 5 major projects in which stainless steel has been or will be specified. Unbelievable growth when you consider that 3 years ago nobody really wanted to know about stainless steel. In North America one state alone has let contracts or is calling tenders for structures which will consume 3000 tonnes of stainless rebar over the next 12 months and this is only one state. But, why the sudden interest in stainless steel?

Owners, be they government or private enterprise are now very aware of shrinking budgets and need to plan ahead for maintenance costs in 10, 15, or 20 years time. Thanks to the exposure given to the Pier Progresso report they have clear proof that extremely long **no maintenance periods** are achievable for a small, sometimes insignificant up front cost at construction. This is clearly the thinking of the Tasmanian government who has just made the decision to use stainless steel in their coastal infrastructure. They know that a higher initial cost will save future generations major funding problems in the not too distant future.

We can take the easy way out and say, "Well, I won't be around in 20 or 30 years time, so let someone else worry about it". Please don't be surprised by this statement, I hear it every day from people who I would think should know better.

The challenge has been given. We have the knowledge and materials to meet such challenges but do we have the will. I guess the future holds the answer to this question.

APPENDIX

KEYNOTE ADDRESS

CONCET '99

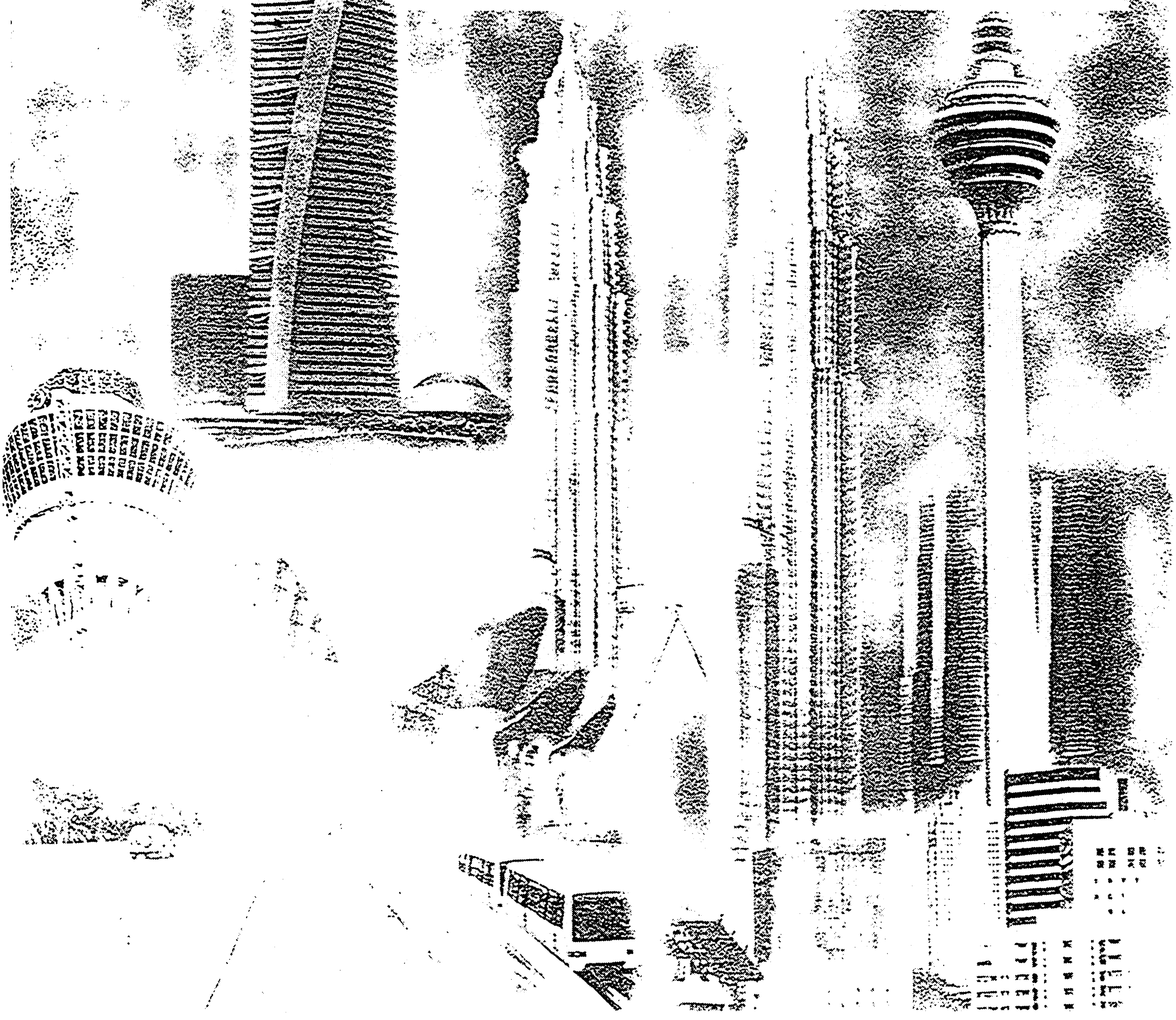
SIXTH INTERNATIONAL CONFERENCE ON CONCRETE ENGINEERING AND TECHNOLOGY

THEME

CURRENT TRENDS IN CONCRETE ENGINEERING AND TECHNOLOGY

Official Opening: **YB Datuk Law Hieng Ding**

Minister of Science, Technology and The Environment, Malaysia



29 JUNE - 1 JULY 1999

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The Institution of Engineers, Malaysia

MARA Institute of Technology

University of Malaya

Diagnosis, assessment and decision-making for defective concrete structures

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University of Adelaide

ABSTRACT

This paper describes procedures for investigating potentially defective concrete structures. The practical problem of deciding how to deal with a defective structure is also considered in some detail. Possible errors in dealing with defective structures are considered, and the importance of risk management is emphasised. An integrated procedure for investigation and decision-making is proposed. It begins with a tentative diagnosis and assessment made using the information initially available. In subsequent iterative steps new information is obtained and used to improve the diagnosis and assessment and decrease the risk of error. The process can be continued until accurate results are assured, or until the costs of further investigation outweigh those of any potential error.

1. INTRODUCTION

In the late afternoon of 16 September, 1996, the Koror-Babeldaob Bridge in Palau collapsed suddenly, apparently without any warning. Several people were killed. The loss of the bridge has severely disrupted life in this small community in the western Pacific.

The K-B Bridge was the longest prestressed-concrete, balanced-cantilever bridge in the world at the time of its construction in the early 1980s. Throughout its life, it had a continuing history of structural problems. A sag at the middle of the interior span, ie where the two balanced cantilevers were pinned together, had progressively increased with time to reach values well in excess of a metre. Mid-span sag was a not-uncommon problem in this form of construction in the 1980s. Significant deterioration of the concrete was also evident, as well as inclined cracking in the end spans of the bridge. Nevertheless, a number of studies by organisations from the United States and Japan had all concluded that the bridge was safe, and not in danger of collapse. Substantial corrective work to control the sag at mid-span had been completed several months before the collapse occurred.

The collapse of the K-B bridge is a stark reminder that building structures are prone to defect, and, in extreme cases, failure. The various technical studies of the bridge, all with positive conclusions regarding safety, are a clear warning to structural engineers that the investigation of defective structures can be a difficult and complex task which is prone to error.

Fortunately, relatively few new building structures display serious defects in their early years of use. On the other hand, all building structures are subject to deterioration processes over time, and eventually become defective. Those structures with good durability may deteriorate slowly, but those with poor durability can deteriorate at a very fast rate indeed, especially when exposed to severe environments.

In countries with rapidly expanding economies, the pressure for rapid creation of new infrastructure often results in relatively high rates of defect in new construction, and also poor durability. On the other hand, ageing and deteriorating physical infrastructure is common in countries with mature economies and can pose very severe structural and economic problems. For example, in regard to the United States of America, Silva-Araya et al [1998] state: "Most of the buildings, highways, bridges, airports and transit systems are deteriorating at a rate faster than our ability to renovate them". The situation is similar in Western Europe and in countries such as Australia [Blakey, 1989].

In the first decades of the new millenium, the problems of deteriorating and defective structures are inevitably going to intensify in most countries, irrespective of whether their economies are developing or mature. Structural engineers will have to be prepared to deal with them. This paper discusses procedures which can be used to investigate potentially defective concrete structures and to make sound structural and economic decisions concerning their treatment.

2. PATHOLOGY OF CONCRETE STRUCTURES

Building pathology has been a serious area of study for many years, but unfortunately there still does not seem to be a generally accepted terminology for even the most basic concepts, such as "defect". The following definitions are therefore introduced to provide a basis for the subsequent discussions in this paper. They refer specifically to concrete structures.

Defect: a defect in a building structure is an inadequacy which has the potential to adversely affect its structural performance and function.

Structural defect: a structural defect exists in a building structure if any of the structural design requirements are not satisfied. The structural design requirements for a structure are specified in the relevant building standards and codes. There are three categories of structural defects, according to the types of design requirements, namely *strength* defects, *serviceability* defects, and *durability* defects.

Functional defect: a functional defect adversely affects the structure in such a way that it cannot be used in the manner intended. A common example of a functional defect is lack of water-tightness.

Inherent defect: is a defect which exists in the structure from the time of its construction, and may be due to a design error or a construction error, or the use of inappropriate materials.

Induced defect: is a defect which occurs at a specific time in the life of the structure, usually as the result of a significant event such as overload, collision, fire, earthquake, explosion, etc.

Developed defect: is a defect which develops progressively during the life of the structure, and as the result of some deterioration process. Examples are steel corrosion, alkali-silica interaction in the concrete, progressively increasing deflections, and structural fatigue.

Defective structure: is a structure which contains structural or functional defects.

Diagnosis: is a process of investigation which aims to determine whether a specific structure is defective, and, if so, to identify the types of defects and their causes.

Assessment: is a process of investigation which aims to determine the condition of a structure and its ability to perform satisfactorily and safely, at present and in the future.

Symptom: is a piece of evidence which suggests that a structure may be defective.

Sign: is similar to a symptom.

Test: any procedure undertaken to obtain additional information to assist in the diagnosis and assessment of a structure.

3. RISK MANAGEMENT

Most tasks undertaken by engineers are risky, in the sense that they are prone to error, and to some extent uncertain of outcome. The nature of the risks and the risk levels which engineers have to deal with vary greatly according to the type of project, and the way it is undertaken.

One of the prime responsibilities of an engineer in regard to any project is to achieve acceptably low risk levels by careful risk management.

In the design of new building structures, legal code requirements assist structural engineers to keep risk levels sufficiently low to be acceptable to the community. For example, safety coefficients are introduced into the design procedures to allow for random adverse variations in material properties and workmanship, while factors applied to the design loads allow for overloads to occur without structural failure. Of course, codes and standards cannot prevent gross errors from being made. Design codes are developed for a population of buildings of a particular type, and statistical and probabilistic concepts are applied to evaluate safety coefficients which keep the number of potential failures to very small and economically justifiable levels.

When dealing with defective concrete structures, engineers face a completely different situation. Codification is largely non-existent, and so there is no pool of useful information, comparable to the data available to designers of new buildings, for studying structural defects.

In investigating a potentially defective structure, the most effective way to reduce the likelihood of error, and hence control and manage risk, is to acquire information specific to the structure. For example, quantities such as material properties, quality of construction and steel locations, which have to be treated by the designer as subject to random variations, are in fact fixed in the completed building, and can be determined by test and by investigation. Likewise, the degree of deterioration present in an ageing building can be determined by appropriate tests at the time the structure is investigated. Even when a stock of existing building structures is to be investigated, such as a class of bridges, statistics for this stock (as distinct from the entire population) can be determined by test and investigation.

Risk can thus be reduced progressively by obtaining additional information. However, the cost of acquiring the information can be very significant, and difficult decisions sometimes have to be made to balance the level of risk, and the associated expected costs of error, against the costs of further investigation. This is a crucial problem which affects the investigation of defective structures, and is discussed in some detail in a later section of this paper.

In summary, the risks involved in dealing with defective structures can usually be controlled, and reduced to any desired level, by paying for the additional information needed.

4. METHODS OF INVESTIGATION: DIAGNOSIS AND ASSESSMENT

4.1 Introduction

At the commencement of an investigation of a potentially defective building structure there is rarely sufficient information available to allow a reliable diagnosis or assessment to be completed. The investigation therefore needs to proceed in a progressive, step-by-step manner, beginning with an initial, but inconclusive, diagnosis and assessment. In each new step, additional information is acquired and used to improve the accuracy of the current diagnosis and assessment, and thus reduce the risk of error. At each step a check is made to determine whether sufficient information has been acquired to allow reliable conclusions to be reached, or whether the investigation should conyinue.

Time limits and resource limits mean that the investigation must be carried out efficiently by maximising the usefulness of all the information obtained. The updated diagnosis and assessment at each step of the investigation can be used to decide which information should be obtained in the following step.

It will be useful to discuss the processes of diagnosis and assessment separately, before looking at an integrated approach.

4.2 Diagnosis

The purpose of diagnosis is to explain any anomalies observed in the appearance and behaviour of a structure and hence to identify any defects and faults. To undertake this successfully an appropriate diagnostic method is needed.

Fig 1 shows an iterative procedure for diagnostic investigation which has been derived by adapting the Scientific Method and the method of engineering design [Warner, 1998]. Initially, the available information is assembled, including all reported signs and symptoms of defects and any observations made during an initial inspection, and used to create a number of alternative feasible hypotheses. To be feasible, a hypothesis must account for all the signs, symptoms and observations. The competing hypotheses, H_1, H_2, .. H_i, are then investigated iteratively.

In each new cycle of investigation additional information is obtained to support or disprove one or more of the hypotheses. In this way, the hypotheses are progressively eliminated, or modified and improved, until, ideally, one is identified as being correct. The acquisition of information may occur in many different ways, for example by restudying available design and construction data, by physical testing of materials sampled from the structure, by theoretical modelling and analysis, or even by a load test. For convenience this information-gathering step is referred to generically in Fig 1 as a "test".

The key step in this process is the initial one of setting up the alternative competing hypotheses. This requires lateral thinking, as well as experience. It is important to note that the various signs, symptoms, and anomalies in the behaviour of a structure are not necessarily all attributable to one cause. There may be several independent defects, in which case a composite hypothesis is needed to explain the observed phenomena. In the initial stages of the investigation, when the explanations arefirst being sought, it can be advantageous to introduce an extra dummy hypothesis to cover all the as-yet unthought-of composite explanations and possibilities.

4.3 Confidence factors and relative probabilities

To improve the efficiency of the diagnostic procedure (and also to improve the procedures for assessment and decision-making, which are to be discussed shortly) it is useful to introduce confidence factors. As the name suggests, a confidence factor expresses the investigator's confidence that a particular hypothesis will eventually prove to be correct.

A confidence factor is simply a number, chosen between 0 (zero) and 10, as shown in Fig 2. The lower extreme value 0 means that the hypothesis is impossible, while the upper extreme value 10 means that it is correct. In practice intermediate values are chosen. These are subjective and intuitive, and represent the current views of the investigator. The values are chosen on the basis of the information at hand, and also on experience and intuition, and comparisons with previous analogous situations.

Initially the values may well be based on little more than intuition, but as the investigation proceeds and more reliable information becomes available, the values become less subjective and more reliable. The diagnosis is formally complete when the confidence factor for one hypothesis approaches 10, and zero for the others. If at the start of the process it is impossible to choose in any other way, then the middle value of 5 can be applied to an hypothesis. The factors can be chosen individually for each hypothesis, or alternatively a group of hypotheses may be considered together and compared, in order to choose relative values for the confidence factors. At any stage in the investigation, the range of hypotheses H_1, H_2, .. H_i, will have confidence factors $CF[H_1]$, $CF[H_2]$, .. $CF[H_i]$, .

The confidence factors are useful at all stages of the diagnosis for choosing which tests to undertake and which hypotheses to investigate. Usually the diagnosis is most efficiently carried out, and most quickly completed, by concentrating the testing on those hypotheses which are most likely to be correct, ie the ones with the highest confidence factors.

The confidence factors can easily be transformed into probabilities. The sum of all confidence factors is first obtained, ie $\Sigma_i\ CF[H_i]$, and used to normalise each $CF[H_i]$ to produce a relative probability:

$$P[H_i] = \frac{CF[H_i]}{\sum_i CF[H_i]} \tag{1}$$

It follows that the sum of probabilities is unity:

$$\sum_i P[H_i] = 1.0 \tag{2}$$

4.4 Assessment

The aim of assessment is to establish the present and likely future condition of the structure, and hence its likely structural performance and its functional and structural adequacy. The assessment of the structure is most easily undertaken if the diagnosis has already been completed. In order to deal with the condition and performance of the structure at future stages in its design life it is necessary to predict the rate of progress of the deterioration processes in the materials and in the structure.

It is often difficult to determine accurately the condition of a structure and then to predict with precision its likely present and future performance. A realistic approach to assessment is therefore to create several scenarios to cover the range of possibilities, and include, say, an optimistic, a pessimistic and a realistic option. The various assessment scenarios are represented as S_1, S_2, .. S_j, .

A complete and exhaustive assessment of each scenario can be time-consuming and expensive. In many situations it is therefore appropriate to undertake a preliminary assessment, followed by a full investigation if the preliminary indications suggest that this is needed. Depending on the outcome of the preliminary assessment it may be possible to terminate the investigation immediately. A flow chart for a preliminary assessment is shown in Fig 3, and for a full assessment in Fig 4.

To assist with the assessment, it is useful to obtain confidence factors and relative probabilities for the scenarios at each stage of the assessment. The confidence factors can be constructed in exactly the same way as previously described for diagnostic hypotheses. For a set of assessment scenarios S_1, S_2, .. S_j, .., we thus obtain $CF[S_1]$, $CF[S_2]$, .. $CF[S_j]$, . The normalising procedure used previously to obtain $P[H_i]$ can be applied to obtain relative probabilities:

$$P[S_j] = \frac{CF[S_j]}{\sum_j CF[S_j]} \tag{3}$$

4.5 Integrated procedure for investigation and decision-making

The flow charts in Figs 1, 3 and 4 are idealised because they imply that the diagnosis and assessment can be carried out separately and in sequence. This is not so, because decisions often have to be made at the commencement of the investigation and at subsequent stages, concerning temporary protection of the structure and its contents against failure.

To provide the best available basis for decision making at the start of the process a tentative, preliminary diagnosis is needed. Clearly this cannot be fully reliable, but it should identify the likely range of possibilities regarding the presence or absence of defects and deficiencies. The possibilities are represented by an initial set of hypotheses, H_1, H_2, .. H_i,. The consequences of these hypotheses can also be identified by initial assessments, which can be made for each of the hypotheses. Fig 5 shows a tree of diagnostic hypotheses H_i, each with a separate set of assessment scenarios, S_{i1}, S_{i2}, .. S_{ij}. It should be noted that the assessment scenarios are not necessarily all different. It is useful to evaluate confidence factors and hence relative probabilities for each hypothesis and for each assessment scenario, in the manner already discussed above. The confidence factors are $CF[H_i]$ and $CF[S_{ij}]$.

Although this information has to be assembled quickly, and is incomplete and not fully reliable, it provides a good preliminary overview of the problem, and is the best available basis for making initial decisions.

In each subsequent iterative cycle a "test" is carried out to provide additional information so that hypotheses and assessments can be eliminated or improved. Following on from the tentative diagnosis and assessment, decisions are made on whether initial protective action is needed and whether the investigation needs to continue. The details of these decisions are discussed in the next section.

The integrated investigation procedure is shown in the flow chart in Fig 6. In both the initial cycle and the subsequent cycles, the procedure basically involves assembling all the available information in a systematic way, in order to allow rational decisions to be made. Apart from the test, which may be time-consuming, each cycle can be undertaken in a short period of time.

5. DECISIONS AND ACTIONS

The second decision shown in Fig 6 is whether or not to continue the investigation. This decision will take account of the available time and available funding, as well as the information contained in the hypotheses H_i, the confidence factors $CF[H_i]$, the assessments S_{ij}, and the confidence factors $CF[S_{ij}]$. Basically, the decision to continue the investigation is taken if additional information is needed to identify the best course of action to deal with the defects in the structure.

The first decision shown in Fig 6 is whether protective action is needed, for example if the structure is thought to be seriously understrength, or overloaded, and is in danger of collapse.

A range of alternative possible actions has to be considered. These will usually include the following:

- take no action and terminate the study, in the belief that the structure is sound;
- take no action, but monitor the structural performance on a continuing basis;
- take no immediate action, but continue the investigation;
- take action to strengthen the structure on a temporary basis before continuing the investigation;
- take the structure out of service temporarily, before continuing the investigation;
- take the structure out of service permanently.

The choice of any course of action is prone to error. Clearly, the risk of serious error is greatest in the first cycle of the investigation, but should reduce progressively as more and more information is acquired.

In assessing a structure it should be noted that two types of error are liable to occur. These have been referred to [Warner, 1998] as Type I and Type II errors and are defined as follows:

Type I Error: incorrectly assessing the structure to be inadequate.

Type II Error: incorrectly assessing the structure to be adequate.

Clearly, Type I errors can have serious cost implications for the owners and users of the building structure, while Type II errors can have very serious consequences such as danger to life and property, and professional reputation.

The decisions taken in each cycle of the investigation need to allow for the possible consequences of any actions. Again, it is advantageous to lay out the decision situation in a logical, ordered manner. This is best done using statistical decision theory [Dandy & Warner, 1989]. The steps required for the decision in Fig 6 are elaborated in Fig 7, and will now be discussed in detail.

We first identify a range of possible actions to deal with the structure. These are denoted as A_1, A_2, .. A_k, .., and might include those listed above. From the updated tentative diagnosis and assessment in the current cycle of the investigation it will be possible to identify a range of possible outcomes for each action. For the typical action A_k, the outcomes are denoted as O_{kl}, O_{k2}, .. O_{kl}, . For each outcome O_{kl}, a relative probability can be estimated, ie $P[O_{kl}]$ using confidence factors and the method previously applied to diagnostic hypotheses and assessment scenarios.

If the cost of each action and the costs of all possible outcomes, written as $C[A_k]$ and $C[O_{kl}]$, are estimated, the expected cost of action A_k is:

$$EC[A_k] = C[A_k] + \sum_l C[O_{kl}]P[O_{kl}] \tag{4}$$

According to the expected value criterion of statistical decision theory, the best action is the one that minimises the expected cost:

$$Min_k\,[EC[A_k]]$$

This criterion is appropriate in situations where the possible losses are not excessive. If, however, the possible losses are excessive and could result in financial difficulties or even bankruptcy, the expected value criterion is not appropriate. There is a valid and rational approach for such circumstances, which is based on the von Neumann Morgenstern utility function [Dandy & Warner, 1989]. This allows any desired level of risk aversion to be built into the decision. The utility function transforms the dollar costs into non-dimensional utility units, which, although subjective, reflect the decision maker's attitude to risk.

Apart from the numerical values provided, an advantage of this approach is that it encourages the decision maker to use an ordered and logical arrangement of the available information and to try to identify all the possible options and outcomes. An example of the use of this investigation procedure, and in particular the confidence factors and expected cost criterion for decision making, has been given recently [Warner, 1998].

5. CONCLUDING REMARKS

Decision making is an essential part of the process of investigating and assessing a potentially defective building structure. The method of investigation described here includes decision making as well as diagnosis and assessment in an integrated process.

At first sight, this process might appear to be complex, and therefore difficult to use. This is not so. In fact, it is derived from simple common sense. The underlying idea is to consider all options and explanations at each step, and progressively gather new information until accurate, reliable decisions can be made at low risk.

It might also appear that the use of confidence factors and relative probabilities will be time consuming and of marginal value. Again, this is not so. With a little practice, they become easy to use and can assist greatly in organising all the information, both subjective and objective, available to the investigator.

6. REFERENCES

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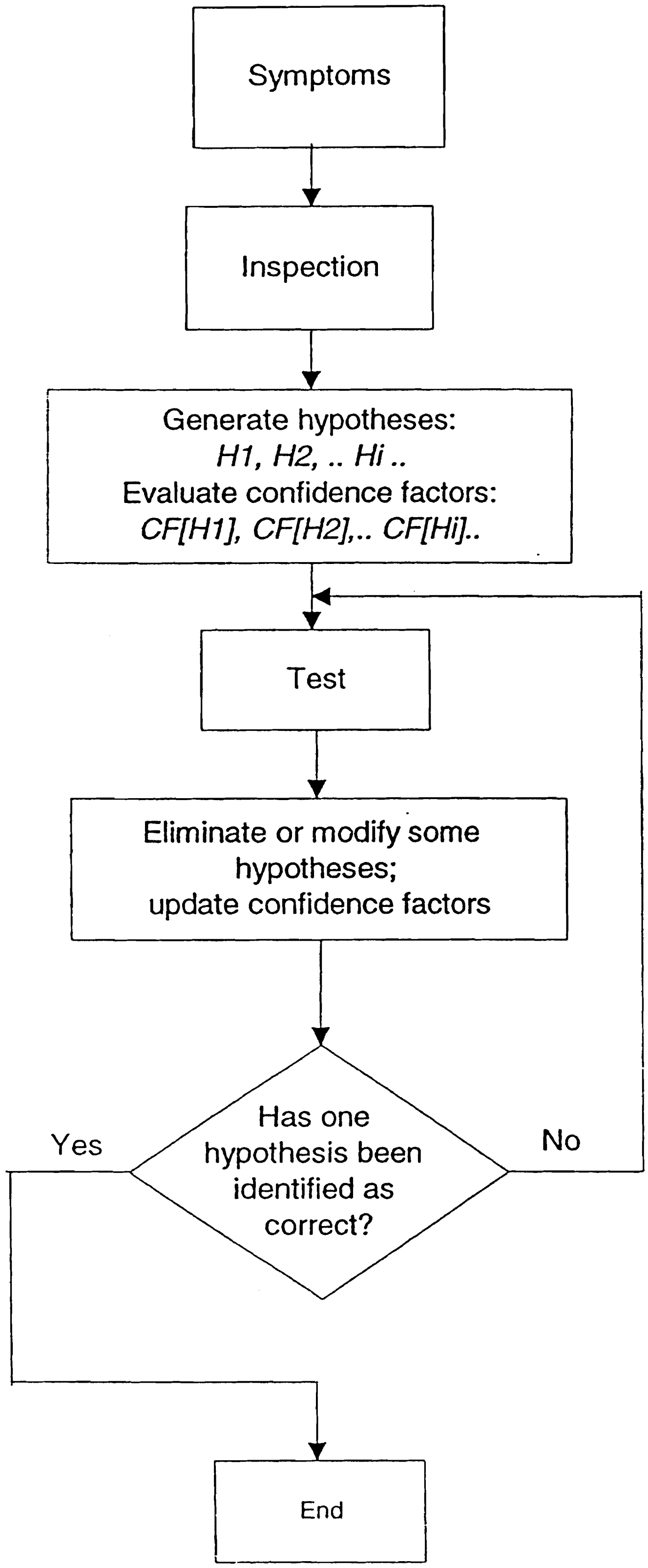

Figure 1: Diagnostic procedure

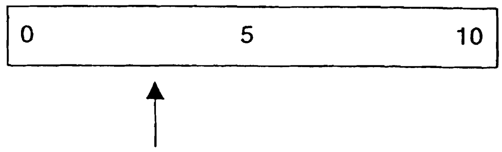

Figure 2: Confidence factor

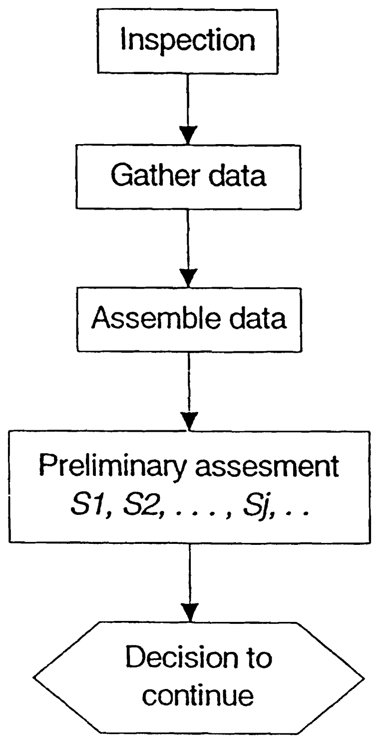

Figure 3: Preliminary assessment

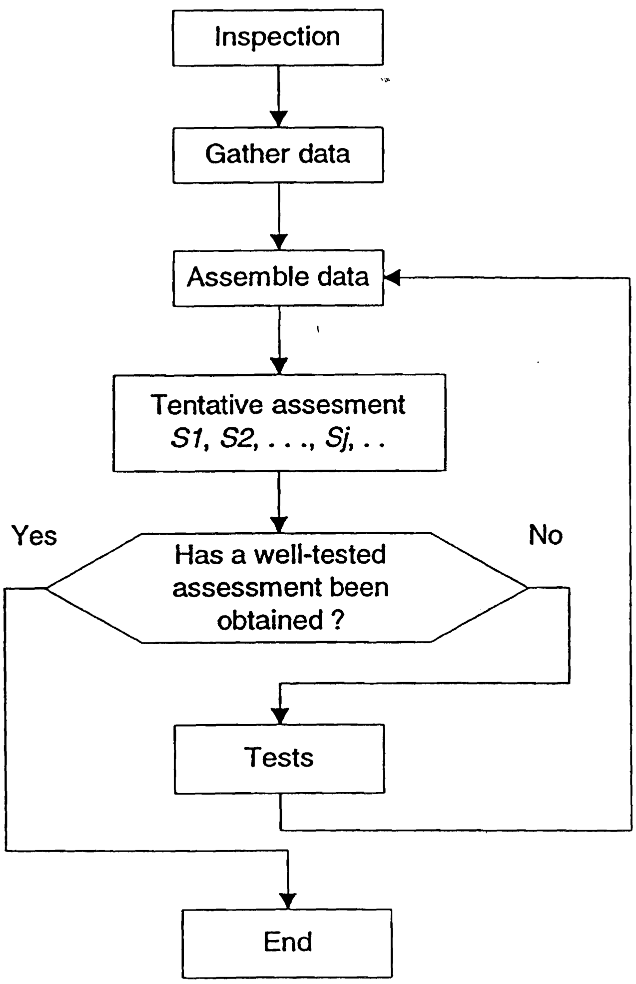

Figure 4: Assessment procedure

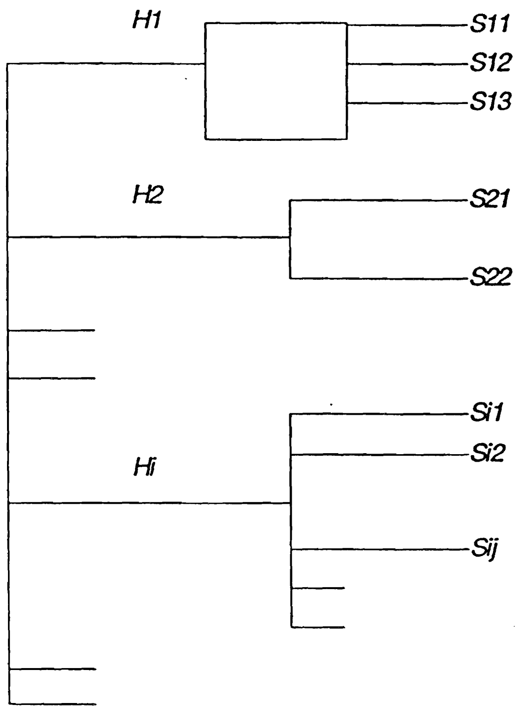

Figure 5: Hypotheses and assessment scenarios

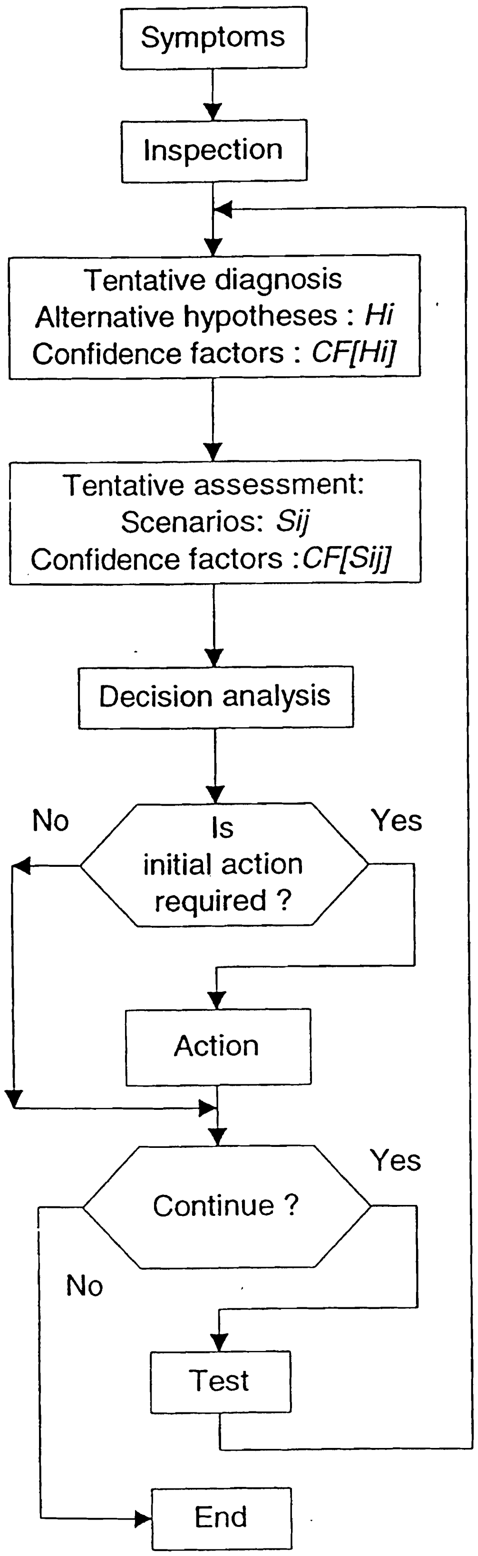

Figure 6: Integrated procedure

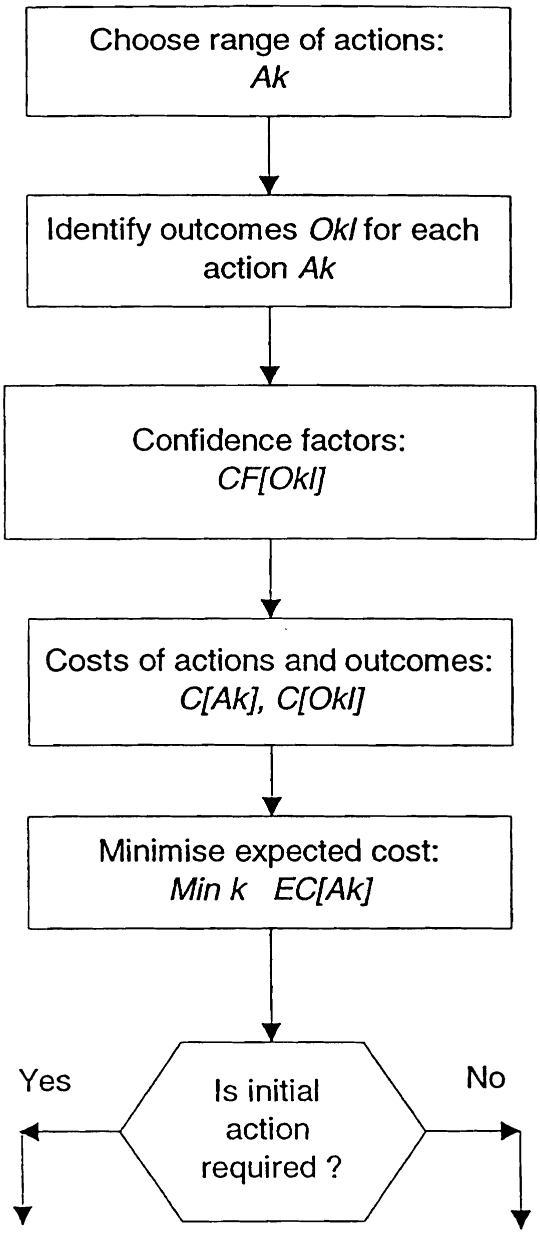

Figure 7: Decision analysis