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Special Concrete and Applications

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42.1 Concreting in Extreme Climatic Conditions

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Concrete is a commonly used construction material in most parts of the world. However, much of the current available knowledge on concrete technology has been mainly generated in the more developed parts of the world. These regions are mostly in the temperate zone. Hence, much of the standard specifications for concreting practice are based on experience in these cooler regions of the world. When concreting in climatic conditions that are different from the normal range, one should consider these to be extreme climatic conditions.

For the normal temperature range of 10 to 20°C current standard specifications developed for temperate regions are adequate. There are occasions when the ambient temperature falls way below this range. This is often referred to as cold weather concreting. On the other hand when the ambient temperature is much above this range, it is referred to as hot weather concreting. However, it is useful to differentiate between an unusually warm summer day in a temperature country from the constantly warm climate outside the temperature zones. STUVO (1982), the Netherlands representative of FIP, has classified climatic regions in the zone of hot countries around the equator into two separate main sub-groups. On the one hand, there is a wet and tropical (hot-humid) climate and on the other, a dry and desert-type (hot-arid) climate. These three different conditions of high temperatures at the time of placing concrete require different degrees of precautions to be taken to achieve proper performance of concrete. All these three types of climatic conditions are covered by the more general description of ACI Committee 305 as follows:

> "Any combination of the following conditions that tend to impair the quality of freshly mixed or hardened concrete by accelerating the rate of moisture loss and rate of cement hydration, or otherwise resulting in detrimental results:
>
> 1. High ambient temperature
> 2. High concrete temperature
> 3. Low relative humidity
> 4. Wind velocity
> 5. Solar radiation."

This performance approach is preferred than the alternate prescriptive approach of limiting concrete temperature at time of placing. The availability of improved chemical admixtures and the increasing acceptance of mineral admixtures in blended cements offer concrete producers new opportunities to satisfy performance requirements that are not based on previous formulation of concrete mixtures. The three different conditions of high temperatures may then be provided with more appropriate and economic solutions.

Cold Climatic Condition

One of the important effects of low temperature on concrete is the reduction in the rate of hydration. It has been found that down to about 10°C below freezing, cement hydration may be extremely low. The actual temperature at which the water within concrete begins to freeze varies with the concentration and types of chemicals present. If this liquid freezes before concrete has set, cement may never set until the climate has warmed above the freezing temperature for the liquid phase. If freezing occurs after concrete has set, then the increase in volume on solidification of the liquid phase may lead to disruption of the concrete structure if the expansion exceeds the strength of concrete at this early age. The resistance to alternate freezing and thawing cycles is provided by suitable amount of air-entrainment, in the order of 5 to 7% by volume of concrete. Air-entraining agents (complex hydrocarbons) are used to entrain the air bubbles of a suitable size and spacing for this purpose. As explained by Dodson (1990), these agents do not generate air in the concrete but only stabilize the air infolded and mechanically enveloped during mixing, already dissolved in the mixing water, originally present in the intergranular spaces in the dry cement and aggregates or in the pores of the aggregates. Entrained air bubbles are typically between 10 μm and 1 mm in diameter and essentially spherical in shape. They are different from entrapped air voids, which are 1 mm or more in diameter and irregular in shape. The spacing factor, defined as the maximum distance of any point in the paste or in the cement paste fraction of mortar or concrete from the periphery of an air bubble, is usually in the range of 0.1 to 0.2 mm. However, the presence of such high percentage of air reduces the compressive strength of the mixture. A reduction in water/cement ratio to compensate for this loss of strength is taken into consideration in the design of the concrete mixture.

ACI Committee 306 defines cold weather as a period when, for more than 3 consecutive days, the following conditions exist: (a)The average daily air temperature (average of highest and lowest from midnight to midnight) is less than 5°C, and (b)The air temperature is not greater than 10°C for more than one half of any 24-hour period.

Cold weather concreting practice should aim to prevent damage to concrete due to freezing at early ages by ensuring a compressive strength of at least 3.5 MPa before the first occasion of freezing. The concrete should develop strength levels appropriate to construction stages such as removal of forms and shores as well as for taking loads during and after construction. Required concrete temperature at the time of placing may be achieved by heating the mixing water and/or aggregates. Concrete after completion of placement should be protected against freezing by insulation and heating, if required, to promote an acceptable rate of strength development needed for the construction. Strength development may be monitored by testing specimens cured with the same temperature history as the structure or based on the concept of maturity factor (ASTM C 1074). Chemical admixtures (non-chloride based) may be added to accelerate setting and hardening. More details are available from list of publications for further information.

Hot Climatic Conditions

Three types of hot climatic conditions have been identified as follows: a hot summer day in a temperate region, hot and humid tropical conditions, hot and dry (arid) conditions.

The special issues involved due to the high ambient and high concrete temperatures are common. However, there are differences not only in the ambient relative humidity but also in the period of high ambient temperature after the placement of concrete. On the other hand, the effects of high temperature on the properties of fresh and hardened concrete are common.

Issues Relating to Properties of Fresh Concrete

The effects of both high ambient temperature and high concrete temperature on concrete and possible mitigating measures are:

1. increased water demand for a given degree of workability — 5 to 10 kg/m^3 of water for each 10^{o}C rise in temperature or a higher dosage of water-reducing admixtures to restore workability;
2. increased rate of workability loss due to more rapid rate of hydration — retarding admixtures to extend dormant period of cement hydration;
3. increased rate of setting due to more rapid rate of hydration — set retarding admixtures to prolong time before potential formation of cold joint;
4. increased tendency for plastic shrinkage cracking — reduce rate of evaporation by shielding from high wind and solar radiation and initiate curing as early as practicable after finishing;
5. increased difficulty to entrain air — higher dosage of air-entraining admixture to promote air entrainment.

Additional precautions should be taken in planning sequence of work and method of placing concrete into the form so as to minimize the need for long set retardation. Concrete has tendency to settle before setting and over retardation tends to increase potential for plastic settlement cracking.

Issues Relating to Properties of Hardened Concrete

The effects of both high ambient temperature and high concrete temperature on hardened concrete and possible mitigating measures are:

1. increased setting temperature leads to lower long-term strength — use low-heat cement and low cement content to minimise temperature rise during setting stage;
2. increased tendency for potential early age thermal cracking — use low-heat cement and low cement content to minimize temperature rise in hardened concrete and/or insulating exterior surfaces to reduce differential temperature;
3. increased early drying shrinkage due to faster rate of moisture loss from the warm concrete — longer curing period and/or applying curing compound to reduce rate of evaporation;
4. decreased durability if microcracking or surface cracks developed by one or more of the above factors — all visible cracks should be grouted to improve durability;
5. tendency to use higher water and/or cement content to provide workability aggravate the above factors — use water-reducing admixtures to compensate for loss of workability instead of increasing cement and water contents.

Additional measures include the use of blended cement when low-heat Portland cement is not readily available or not economically viable, and methods for temperature control in fresh and hardened concrete. The design of concrete mixtures with minimum water content adequate for the process of mixing results also in minimum cement content, irrespective of water-to-cement ratio required for specified strength.

Temperature Control

The need to control temperature of concrete may be divided into two stages: placing temperature of fresh concrete, and peak (maximum) temperature of hardened concrete and resultant temperature differential (potential early thermal cracking).

When a hot summer day occurs in a temperate country, the normal everyday practice of handling and placing concrete is no longer adequate. Hence, a limiting temperature for concrete at the time of placement is sometimes specified (e.g., 30°C in BS 8110). In addition, for the case of thick sections (e.g., raft foundations), when the weather has returned to cooler normal summer temperatures, the subsequent rise in temperature of the hardened concrete results in a greater thermal differential. Under such situations, specifying fresh concrete temperature at the time of placement lower than the ambient temperature of the day is beneficial. On the other hand, for the hot and humid or hot and dry climate, the daily mean temperature remains high. For such cases, cost/benefit considerations may not justify the high cost of reducing temperature of fresh concrete to below ambient temperatures. The need for this is mainly to limit the peak temperature in thick sections and also the consequential temperature differential. Other alternate approaches may provide more economic solutions. However, it is to be noted that the benefits of a lower concrete temperature include minimizing the effects listed above.

In using an initial concrete temperature below that of the ground for a thick raft foundation also reduces the temperature of the soil layer immediately below the raft. This results in a greater temperature differential with respect to the warmer interior and may be higher than that with respect to the top of the raft (exposed to ambient temperature).

Methods for minimizing the temperature rise due to heat of hydration of the cement in thick sections include:

1. select a low heat cement;
2. adopt the lowest water content for method of mixing and hence the lowest cement content for a given w/c ratio for strength;
3. use of high range water-reducing admixtures to provide workability at lowest practical water content (method (2) above);
4. partial replacement of cement with mineral admixtures (e.g., fly ash or ground granulated blast-furnace slag together with method (3) above);
5. accepting conformity of strength at a later age, instead of 28 days, to enable adopting a lower cement content for the same water content;
6. partial replacement of cement with silica fume (or micro-silica) which provides a higher strength-to-mass ratio but not significantly changing the heat of hydration per unit mass;
7. use of ice or chilled water to lower the initial concrete temperature;
8. combination of one or more of the above methods.

Tam (2000) reported on an adoption of a combination of more than one of the above methods for a 2.8m raft foundation having a specified concrete cube strength of 40 MPa. The monitored temperatures met the specified temperature control of initial concrete temperature not exceeding 30°C, peak temperature not exceeding 70°C and temperature different not more than 20°C. No insulation was needed for the top of the raft.

The potential for early thermal cracking in a structural element depends on the following factors:

1. temperature differential between the warmer interior and the cooler exterior of a thick section;
2. degree of constraint by the external boundaries
3. ultimate tensile strain capacity of the concrete mixture (depending on the thermal properties of its mix constituents) over the period where the temperature differential is significant;
4. rate of strength development over the period where the temperature differential is significant; creep relief of concrete at early ages over the period where the temperature differential is significant; drying shrinkage superimposed on thermal strain over the period where the temperature differential is significant;
5. both the space rate of change and the time rate of change of thermal strain.

Not all the above factors are independent of one another and their significance varies within the spatial location of the structural element. The whole process is complicated by the long period of time needed to place a large volume of concrete in a single continuous operation. In order to minimize the number of construction joints, each single operation often calls for a minimum volume of 2000 cubic meters. The earlier placed concrete has hardened before the final portion is placed. Even in the case of placement in a single continuous operation, the parts of the structure placed at the initial stage of placement will start their temperature history at a different time from those placed at the later stage of the placement.

In the case of hot and dry climatic conditions, the situation is more demanding than when it is only hot and humid. Some of the guidelines intended for a hot summer day in a temperate country are often not directly applicable, and appropriate adjustments should be made to take into consideration the difference in the climatic conditions. These include higher rate of evaporation reducing the stiffening time and greater potential for plastic shrinkage.

Potential Cracking in Fresh Concrete

The two main types of cracking in fresh concrete are plastic settlement cracking, and plastic shrinkage cracking.

Plastic settlement is due to the downward movement of heavier particles (particularly larger size aggregates) and the upward movement of the lighter particles (cement grout). If such movements are free to take place, the resultant segregation does not give rise to cracking. However, if such movements are hindered by top reinforcement bars, surface cracks may develop at the top surface reflecting the pattern of these bars. At local changes in cross-section, arching action may result. An internal void develops when the concrete below the arch falls away, particular for elements of a great depth, e.g., columns, walls, or deep beams. Similarly, a sudden change in depth between a slab and the ribs of a ribbed slab may give rise to a surface crack directly over the sides of a rib. A shifting of the reinforcement cage in a column may reduce the cover locally, giving rise to arching and, in general, an approximately horizontal tear develops at the side face of the column. Such cracks are formed during the first few hours before the concrete has set.

Plastic shrinkage cracks develop due to drying out of the fresh concrete. The mechanism is similar to the more familiar drying shrinkage of hardened concrete. The rate of evaporation may be estimated by the equation proposed by Uno (1998):

$$\text{Evaporation rate} = 5\left\{(\text{Tc}+18)^{2.5} - \text{R}(\text{Ta}+18)^{2.5}\right\}\{\text{V}+4\}\times 10^{-6}\ \text{kg/m}^2\text{h}$$

where Tc = concrete temperature, °C
Ta = air temperature, °C
R = relative humidity,%
V = wind velocity, km/h

The above expression points to the much more serious situation in hot and dry compared to hot and humid climatic conditions or when high wind velocity is present. Strong winds are often found along coastal regions and during placement on the high floors of a tall building. The critical rate of evaporation giving rise to cracking may be expected to vary with the type of cementitious materials used (ultimate tensile strain capacity of fresh concrete), but is often considered as 1 kg/m^{2}h in the case of commonly used Portland cement. Retardation and cohesiveness of the mixture introduce additional factors to be evaluated for their effects on potential plastic shrinkage cracking. Higher evaporation rate may be critical for mixtures with low bleeding rate or slow rate of stiffening. Evaporation of bleed water does not lead to shrinkage until the surface moisture is lost. If shrinkage occurs without any restraint, cracking may not develop. However, in practice, restraint may be due to external boundary conditions of the structural element, e.g., a slab or internal difference in moisture content within the thickness of a deep section.

Even though both types of plastic cracks may initially be very fine, subsequent drying out of the hardened concrete widens them to widths that are visible. In general, most cracks of this nature do not significantly impair the structural behaviour, nevertheless they present a reduction in durability performance unless they are grouted or sealed.

Additional details on plastic shrinkage cracking potential may be obtained from the relevant references and publications for further information listed.

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42.2 Polymer Modified Concrete

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Portland cement concrete is one of the most versatile and cost-effective construction materials. Polymer-modified concrete were developed from the 1960s to overcome some of the limitations of concrete such as low tensile and flexural strength, high porosity and low resistance to certain chemicals. The relative high cost of monomers had limited the commercial viability of certain polymer-modified concrete. The advancement in chemical admixtures and mineral additives has offered alternative solutions to overcome a range of those limitations. Certain polymer cement concrete and polymer concrete remain relevant. They offer unique solutions to a range of applications.

Classifications

There are three types of polymer-modified concrete:

- *Polymer-impregnated concrete* (PIC) is a hardened cement concrete impregnated with a monomer system that is subsequently polymerised *in situ*.
- *Polymer cement concrete* (PCC) is a concrete with polymeric admixtures or a monomer system added to the fresh concrete. The monomer system is subsequently polymerised after the concrete has hardened, whereas the polymeric admixtures cure with the hardening concrete.
- *Polymer concrete* (PC) consists of an aggregate mixed with a monomer or resin that is subsequently polymerised *in situ*.

Polymer-Impregnated Concrete

The quality of porous materials such as concrete and stone can be modified vastly by the filling of the pore system. Polymer-impregnated concrete (PIC) is produced by impregnating hardened concrete with a monomer system, either by surface application or full immersion of concrete in the monomer. The amount of monomer absorbed will depend on the porosity of the concrete, the conditioning of the concrete (drying or vacuum), and the viscosity of the monomer system. The monomer is subsequently polymerized by thermal catalysis or irradiation. The polymer impregnation process enables significant improvement in both mechanical and durability properties of the concrete.

The monomer system usually involves a monomer or copolymer, a catalyst and an additive such as a surfactant. Acrylic monomer systems such as methyl methacrylate or its mixtures with acrylonitrile are preferred because they have low viscosity, high reactivity, relatively low cost and result in products with superior properties. Thermosetting monomers and prepolymers are also used to produce PIC with greatly increased thermal stability. These include epoxy prepolymers and unsaturated polyester-styrene. The concrete may be impregnated to varying depths or in the surface layer only, depending on whether increased strength and/or durability is required.

The most important feature of PIC is that a large proportion of the void volume is filled with the polymer. This results in a remarkable improvement in tensile, compressive and impact strength [Mason, 1981, Dikeou 1978], enhanced durability and reduced permeability to water and aqueous salt solutions such as sulfates and chlorides [Steinberg et al. 1968]. The compressive strength can be increased from 35 MPa to 140 MPa, the water sorption can be reduced significantly and the freeze–thaw resistance is considerably improved. The main disadvantages of PIC products are their relatively high cost, as the monomers used are expensive and the production is more complicated.

Applications of PIC include structural floors, food processing buildings, sewer pipes, storage tanks for sea water, desalination plants and distilled water plants. Partially impregnated concrete is used for the protection of bridges and concrete structures against deterioration and repair of deteriorated building structures, such as ceiling slabs, underground garage decks and bridge decks.

Kukacka [1976] reported early applications of PIC and PC as a result of their high strengths and durability. Strength increased by a factor of 4 and water absorption was reduced by more than 99%. There were also improvements in hardness and resistance to abrasion and cavitation. Two bridge decks in the USA were partially impregnated to a depth of 25.4 mm (1 in.) as a means of preventing chloride intrusion. PIC curbstones have been installed on a bridge deck as an alternative to granite, and PC patching materials are being utilized in areas where heavy traffic conditions severely limit the time during which repair work can be performed.

Polymer Cement Concrete

There are two types of polymer cement concrete (PCC). The first involves the addition of a monomer system as part of the concreting materials, and is commonly referred to as premix polymer-cement concrete (PPC). The monomer system remains in the hardening concrete and is subsequently polymerised after the concrete has hardened. The second type involves the addition of a dispersed polymer into the mortar or concrete mix, and is usually referred to as polymer-modified cement concrete. For both types, the compatibility between the monomer or polymer and the hydrating cement system is critical to the outcome.

The monomer system and subsequent polymerisation process used in PPC are similar to those used in PIC. With limited improvements in the quality of PPC compared to conventional concrete of similar mixture proportions, and the high cost of monomer and polymerisation process, no viable commercial applications have been found for PPC.

According to Blaga and Beaudoin [1985], a range of dispersed polymer (latex: colloidal dispersion of polymer particles in water) results in greatly improved properties, at a reasonable cost. Therefore, a great variety of latexes are now available for use in PCC products and mortars. The most common latexes are based on poly (methyl methacrylate; also called acrylic latex), poly (vinyl acetate), vinyl chloride copolymers, poly (vinylidene chloride), (styrene–butadiene) copolymer, nitrile rubber and natural rubber. Each polymer produces characteristic physical properties. The acrylic latex provides a very good water-resistant bond between the modifying polymer and the concrete components, whereas use of latexes of styrene-based polymers results in a high compressive strength. Generally, PCC made with polymer latex exhibits excellent bonding to steel reinforcement and to old concrete, good ductility and resistance to penetration of water and aqueous salt solutions, and resistance to freeze–thaw damage. Its flexural strength and toughness are usually higher than those of unmodified concrete.

The drying shrinkage of PCC is generally lower than that of conventional concrete; the amount of shrinkage depends on the water-to-cement ratio, cement content, polymer content and curing conditions. It is more susceptible to higher temperatures than ordinary cement concrete. For example, creep increases with temperature to a greater extent than in ordinary cement concrete, whereas flexural strength, flexural modulus and modulus of elasticity decrease. These effects are greater in materials made with elastomeric latex (e.g., styrene–butadiene rubber) than in those made with thermoplastic polymers (e.g., acrylic). Typically, at about 45°C, PCC made with a thermoplastic latex retains only approximately 50% of its flexural strength and modulus of elasticity.

The main application of latex-containing PCC is in floor surfacing, as it is non-dusting and relatively cheap. Because of lower shrinkage, good resistance to permeation by various liquids such as water and salt solutions, and good bonding properties to old concrete, it is particularly suitable for thin (25 mm) floor toppings, concrete bridge deck overlays, anti-corrosive overlays, concrete repairs and patching.

Polymer Concrete

Polymer concrete (PC) or resin concrete is a composite containing polymer as a binder, instead of Portland cement, and inert aggregate as filler in the concrete. An epoxy or polyester is the most common polymer used. PC has higher strength, greater resistance to chemicals and corrosive agents, lower water absorption and higher freeze–thaw stability than conventional concrete. It can be produced in a similar manner to conventional concrete.

Bloomfield [1995] reported the development of PC pipes with good resistance to chemical attack from both acidic and caustic effluents inside the pipe, and from chemical attack on the outside of the pipe. Approximately 50,000 tonnes of PC pipe were manufactured by the 1960s. PC pipes made with polyester resin are reported to be corrosion resistant in continuous service with effluents ranging from a pH of 0.5 to 9.0. These pipes meet the corrosion requirements of DIN 4030, 'Assessment of Water, Soil and Gases for Their Aggressiveness to Concrete'. PC pipes can also be made with epoxy resin for a range of pH values from 0.5 to 13. In the United States, basic standards for PC pipe are being prepared by a committee on Standard Specifications for Public Works Construction, also known as the 'Green Book,' for the Los Angeles County Sanitation District.

Sewer pipes, jacking pipes, manholes, drainage system, access covers to underground services, and electrical cable jointing pits, ducting and accessories, produced with polymer concrete, are available commercially.

Schoenberner et al. [1991] compared nine general chemical family groups of polymer binders for chemically resistant concrete floor overlays. They include MMA acrylics, common epoxies, novolac epoxies, furans, polyesters, vinyl esters, potassium silicates, sulfur and urethane. It was noted that the properties were listed for average materials in the group and that the property data was obtained from manufacturers' literature on a cross-section of materials in the generic family. Vinyl ester, including vinyl ester novalacs, was reported to have better chemical resistance and to be tougher and more resilient than most polyesters. They have a lower compressive strength range and a lower coefficient of thermal expansion range, but a higher flexural and tensile strength range than polyesters. A higher full cure time of 7 days is typical for vinyl ester compared with 4 to 7 days for polyesters.

PC materials to be used in aggressive environments should be composite materials consisting of about 85% inorganic material — dry aggregate and silica (sand), which are chemically very inert and long lasting. These inorganic materials are bound together by approximately 15% of an organic matrix resin which is potentially less resistant to long-term aging in aggressive chemical environments. The vinyl ester-based matrix resin systems are known to be more chemically resistant than the polyester materials.

Limited long-term performance data on these polymer concretes are available. For unsaturated polyester concrete, an initial drop of 10% of compressive strength within the first year, followed by a period of relatively constant strength retention up to 8 years under outdoor exposure in Japan, was reported by Chandra and Ohama [1994]. It was not clear whether the concrete samples were subjected to any load during the exposure. A polyester styrene PC overlay used in the U.S. was reported by Sprinkel [1991] to provide skid resistance and protection against intrusion by chloride ions for bridge decks for 10–15 years. However, significant loss of tensile strength and bond strength, as well as elongation, were found in this polymer concrete overlay over a period of 5 to 9 years. Under accelerated deterioration tests in a weather-ometer and exposure to heat cycles, Imamura et al. [1978] found no loss in strength on a precast PC made from an unsaturated polyester resin. The concrete also exhibited good fatigue properties under bending.

For vinyl ester concrete, good resistance to water erosion, H_2SO_4, and freezing and thawing was reported. No long-term aging data were found.

Unsaturated polyester concrete is not suitable for use under severe outdoor exposure in thin section. Deterioration can occur due to photochemical reaction and hydrolysis of the ester groups in the presence of chemicals such as fuel oil (at elevated temperature) and rubber chemicals. In thicker sections, photochemical degradation could be contained in the skin, resulting in some initial drop of strength. While different degrees of degradation of polyester concrete in water were found by Imamura et al. [1978] and Mebarkia and Vipulanandan [1995], a complete recovery of the compressive strength upon drying was found by the latter.

DePuy and Selander [1978] investigated performance and durability of vinyl ester PC with initial properties of:

Compressive strength	114 MPa
Modulus of elasticity	33 GPa
Modulus of rupture	17 MPa
Flexural modulus of elasticity	35 GPa
Specific gravity	2.40

The durability of the vinyl ester PC exposed to freezing and thawing and to 5% H_2SO_4 was investigated and found to perform well. No indications of deterioration were detected after 3010 cycles of freezing and thawing, and a 0.19% weight loss was shown after 948 days of exposure to 5% H_2SO_4. The vinyl ester PC specimens were tested for creep deformation. Specimens sized 115 × 300 mm and loaded at 69.0 MPa failed within 10 to 48 days under load. This loading was about 71% of the ultimate strength for these specimens. Specimens loaded at 27.6 MPa (29% ultimate strength) and 48.1 MPa (50% ultimate strength) had creep deformation of 62.9 and 70.5 millionths/MPa respectively after 2 years. DePuy and Selander [1978] also noted that, after 2 years exposure, the experimental vinyl ester PC overlay at Shadow Mountain Dam in Colorado was in good condition, with only some minor cracking observed.

A new type of high molecular material — 3200 vinyl ester resin mortar — has been described by Lin et al. [1986], and comparison has been made with other types of polyester mortars. Vinyl ester resin, also known as epoxy acrylic resin, is formed by a ring opening addition reaction between a low-molecular weight epoxy resin and an unsaturated carboxylic acid. The 3200 vinyl ester resin mortar is vinyl ester resin modified with a small amount of fumarate, which allows free polymerization in the presence of a peroxide initiator and can be cured at room temperature.

In 1983, 3200 vinyl ester resin mortar was used during the overhaul of the sluiceway to repair the high-velocity section of a hydropower station. At the same time, unsaturated polyester resin mortar was applied as a comparison mortar. Experience with the application of 3200 vinyl ester resin mortar showed that this type of mortar had excellent resistance to cavitation erosion, abrasion erosion, chemical corrosion and freeze–thaw cycling, with long-term durability to weathering and soaking in water. It successfully withstood three years of operation for passing water, silt and debris, and remained unaltered, while the polyester mortar exhibited signs of distress.

Okada et al. [1975] investigated the thermo-dependent properties of polyester concretes made from two types of unsaturated polyester resin. The mechanical properties of the polymer concretes were affected by atmospheric temperature. The factors influencing the thermo-dependent properties of the concrete are the type of resin and resin content, as well as the maximum size of aggregate. The strength and modulus of elasticity decrease almost linearly when temperature rises from 5°C to 60°C. Creep deformation increases remarkably above a certain temperature (about 40(c). Below about 20°C, creep deformation is almost proportional to the stress induced, and the apparent viscous flow observed is very low.

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42.3 High Performance Concrete

V. Sirivivatnanon

During the 1970s, concrete having a higher strength (40 to 50 MPa) began to be specified for columns in high-rise buildings because slender columns offered more architectural possibilities and more renting space [Albinger and Moreno 1991]. With the years, the name of these initial high-strength concretes has been changed to high-performance concrete [Aïtcin 2000] because it was realized that these concretes have more than simply a high strength. These concrete started to be used outdoors and faced more severe environments such as offshore platform, bridges, roads, etc. Little by little, it was realized that the market for this concrete was not only the high-strength market, but also more generally the market for durable concrete that represented more or less one third of the present market for concrete.

Definitions

High performance concrete (HPC) was defined by the American Concrete Institute [ACI 1994] as concrete which meets special performance and uniformity requirements that cannot be achieved by using only the conventional materials and normal mixing, placing, and curing practices. The performance requirements may involve enhancements of: placing and compaction without segregation, long-term mechanical properties, early-age strength, toughness, volume stability and service life in severe environments. Most engineers have adopted the term HPC to literally describe concrete which has specifically been formulated or chosen to give a "high performance" in specific applications with no restriction on the type of concreting materials nor production methods. The choice of concrete requires, on the one hand, a good understanding of how concrete properties are derived and varied by its compositions and production processes. On the other hand, it requires the identification of vital properties resisting against the deterioration mechanism(s) associated with the target performance. For example if high "strength" is the required "performance" to carry greater load in high-rise building, a silica fume or silica fume/fly ash low water-to-binder ratio concrete is used with good strong aggregates to produce the required concrete. In most cases, silica fume and superplasticizer are used to boost the strength of the paste. Fly ash is chosen to reduce heat of hydration for large structural members such as columns and transfer girder as well as improve pumpability of the concrete to a greater height or distance or both on a construction site. Selected coarse aggregates may be required to match the paste strength and to control long-term volume stability. If durability such as long-term wear resistance is required for concrete road surface for example, the selection of high wear resistance coarse aggregate becomes the primary issue in HPC used. Other processing requirements are the provision for induced crack at appropriate timing and intervals.

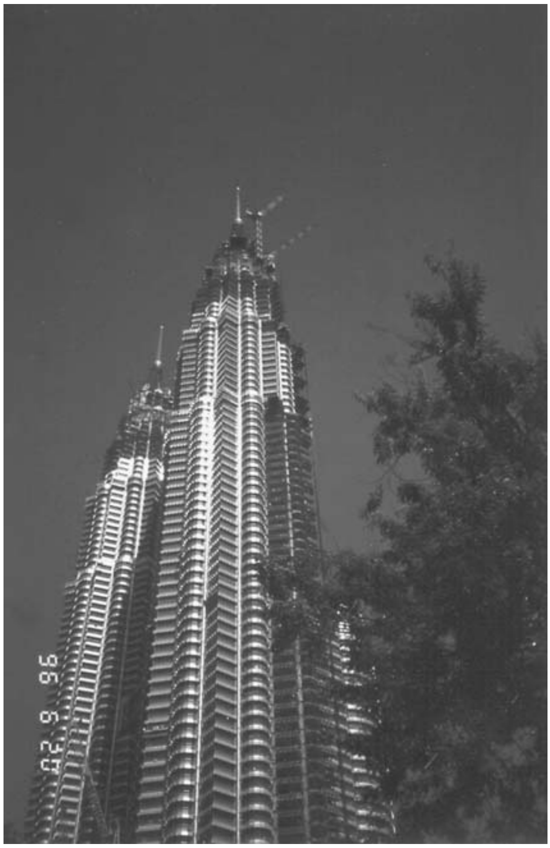

FIGURE 42.1 Petronas Towers, Malaysia.

High Performance Criteria

Modern construction and environmental obligations lead to a demand on a range of performance of modern concrete. They can be classified into those related to production, in-service performance and sustainability:

1. Production: fresh concrete properties, setting time, heat of hydration, early strength and curing requirement.
2. In-service Performance: mechanical, volume stability, and durability properties.
3. Sustainability: embodied energy, ecolabelling and lifecycle cost.

In most cases, there will be one primary and a range of secondary performance requirements necessary for the concrete to fully satisfy its intended function. In order to satisfy these ranges of performance, it is important that performance-based criteria are specified. The key performance must be vigorously acquired without losing sight on a range of secondary but complementary attributes necessary for them to be achieved in practice. (See Figs. 42.1 and 42.2.)

Performance standard and compliance criteria must be selected and tailored to rate the risks associated each performance standard according to its importance. For example, in specifying reinforced concrete subjected to chloride-induced corrosion, both the *quantity* and *quality* of concrete cover to reinforcement are necessary. However, it has been found that the quantity of cover thickness is far more important than the quality of cover influencing the service life. It is therefore necessary not only to specify both cover thickness and concrete quality as performance standard, but also devise a more strict compliance criteria

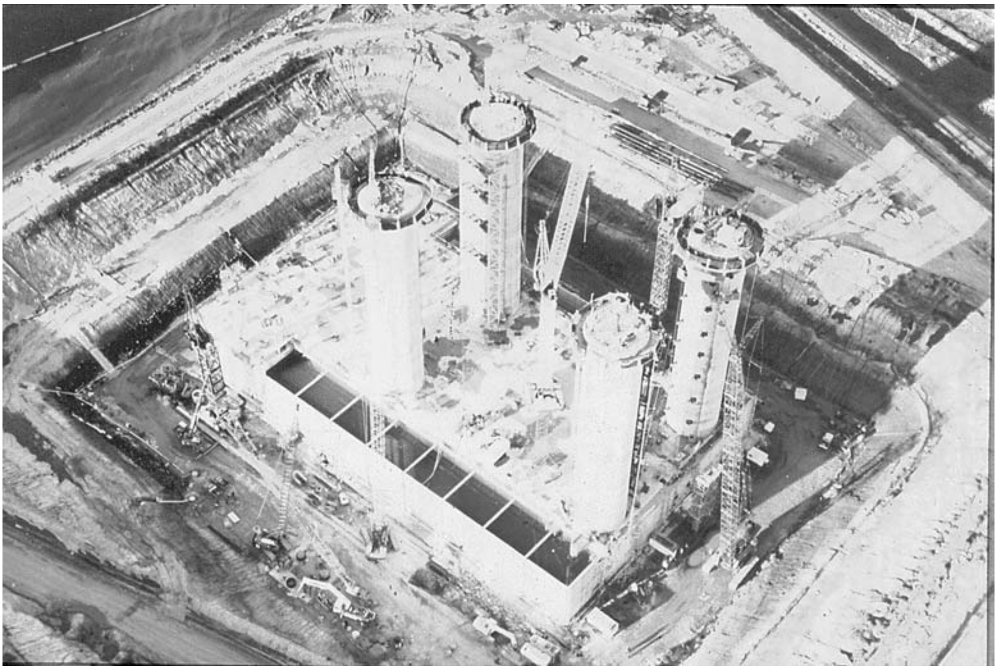

FIGURE 42.2 Wandoo platform under construction in western Australia.

for cover thickness than concrete quality. One method of balancing some of the risks associated with compliance criteria has been discussed [Sirivivatnanon and Baweja 2002].

Formulation of High Performance Criteria

Basic Constituents

Good concrete can usually be proportioned with the very basic constituents of Portland cement, water, a chemical admixture, fine and coarse aggregates. Since aggregates constitute approximately 75% of the volume of concrete, the properties of concrete are highly dependent on them. A careful choice and combinations of aggregates will enable highly dense and workable to be produced.

When greater demand is placed on specific properties of concrete, they can be met by a careful selection of the basic as well as a range of new constituents. They include specific Portland and blended cements, chemical admixtures, mineral additives and non-traditional reinforcement such as galvanized or stainless steel, fine galvanized wire mesh (ferrocement), and various fibers.

Cements, Chemical Admixtures, Mineral Additives, Fibers and Special Reinforcement

There are various Portland cements and blended cements available in many parts of the world. They enable the modification of the hydration products and the pore structures of the concrete. Blended cements incorporating mineral additive such as fly ash, blast furnace slag, silica fume and other natural pozzolans are widely used to modify the durability performance of concrete. When blended-cement concrete is proportioned to give similar mechanical properties to Portland-cement concrete, some slightly modified volume stability properties and enhanced durability properties are usually achieved.

Chemical admixtures such as water reducers are commonly used to reduce the water-to-cement ratio or maintaining the W/C with improved workability. A high-range water reducer or superplasticizer is both a powerful constituent to control the water content and to improve workability or both. It enables concrete of very low water-to-binder ratio (0.3–0.4) to be produced and placed. Either a very high strength or a highly durable concrete can be produced. A combined use of silica fume and superplasticizer has led to the early production of high strength concrete without excessive quantity of cement. This is important as a large quantity of cement may result in greater heat of hydration and the possible cracking problems.

Ground limestone has been used as an additive to modern Portland cement. It has also been used in the production of self-compacted concrete. Organic and chemical additives are used as corrosion inhibitor [Sorensen et al. 1999, Schießl and Dauberschmidt 2000] in renovation and new concrete. They should be considered when conventional solutions could not be used.

The potential of fiber reinforcement in concrete has been widely researched and published [ACI 1996]. Synthetic fibers such as polypropylene fiber are popularly used to control plastic shrinkage while steel fiber is more commonly used to control cracking and improve the impact resistance of floor slab [Knapton 1999]. Plastic fibers have also been found [Rostam 2001] to improve the fire resistance of high strength concrete. They could become the essential ingredients for structural members susceptible to hydrocarbon fire such as in concrete tunnel lining.

Ferrocement incorporating fine galvanized wire mesh in cement mortar was first developed as an appropriate technology for construction in rural areas. In the 1980s, this thin-wall yet durable material has been found to be the key performance requirement for a range of structural elements in urban construction. They include sunscreens, secondary roofing slabs and water tanks for high-rise buildings [Paramasivam 1994]. In special circumstances, galvanized reinforcement has been successfully used to combat carbonation-induced corrosion whereas stainless steel would be required for chloride-induced corrosion [Rostam 2001].

Special Processes

Apart from varying the constituents, a number of processes have been successfully used in order to enhance specific performance of concrete. They include vacuum suction, controlled permeable formwork and induced crack.

Both vacuum suction and controlled permeable formwork (CPF) are based on the concept of improving the surface quality of concrete by lowering the water-to-cement ratio of the concrete after placing. The former involves a removal of water and air void from the surface by applying a vacuum to formed or unformed surfaces of concrete immediately or very soon after the concrete is placed [U.S. Bureau of Reclamation 1981]. CPF is a special material adhered to the surface of formwork, which allows for the controlled removal of surface water by gravity [Wilson 1994]. The value of such beneficiation processes needs to be evaluated on a case by case basis.

Applications of High Performance Concrete

HPC has been used in prestigious structures such as the Petronas Towers and the Troll Platform. Petronas Towers was the tallest concrete building in the world built in Malaysia in the mid-1990s. In 1998, the deepest offshore platform, the Troll platform, was built in Norway — a structure taller than the Eiffel Tower.

In most applications, one key performance criterion may be critical but so are a number of associated criteria. It is interesting to examine a range of HPC according to their key performance criterion.

Production as Key Criterion

Self-Leveling Concrete for Foundation of Raffles City — Singapore

In order to satisfy the requirement for a large volume of concrete to be placed rapidly in huge foundation elements, a non-segregated flowing and self-leveling concrete was developed [Collepardi 1976] by the use of a superplasticizer combined with a relatively high content of powder materials in terms of Portland cement, mineral additive, ground filler, and/or very fine sand. In late 1970, such a concrete was placed

FIGURE 42.3 Concrete placing at Raffles City, Singapore.

with tremie for the construction of a dry dock [Collepardi et al. 1989]. It was also used in a system of inclined-chutes and distribution boxes for the slab foundation of the Trump Tower in New York in the late 1970s and the Raffles City in Singapore in the early 1980s. Cracking due to differential temperature movement was controlled by limiting the fresh concrete temperature at placement. This type of concrete was subsequently placed in large structural members with highly congested steel and can possibly be considered as the earlier version of self-compacted concrete (SCC) (Fig. 42.3).

Bottom Up Placement at Market City — Australia

A key feature of the Market City project (a 36-story residential tower over a 10-story podium incorporating the new Paddy's Market as well as a range of retail tenancies and cinemas) is the use of high-strength concrete-filled steel tube columns. The system combined the tube column and parallel beam concepts. The combination works well with one of the advantages being the minimization of moment transfer from the floors into the columns, thus allowing minimum column sizes to be used. The concrete was specially formulated to minimize bleeding and was pumped up from the base of the tube without vibration. A full-scale 13-m high prototype column was built prior to construction and used to investigate a number of factors including the construction procedures and the performance of the proposed concrete mix.

The mix finally adopted was an 80 MPa concrete at 28 day. Silica fume and fly ash were used and the maximum aggregate size was 14 mm. A superplasticizer was used to increase the slump from the initial 30 to 35 mm to a value of 200 mm. Actual strengths of up to 100 MPa were achieved. Core samples taken from the prototype column at various levels confirmed that a very dense uniform mix was achieved with no separation from the tube wall. No settlement voids were found in core samples taken around reinforcing bars (Fig. 42.4).

In-service Performance as Key Criterion

High Strength Concrete Projects

Silica fume and fly ash has been widely used since 1980 to produce high strength concrete for high rise buildings. Some silica fume concrete data used in a number of structures in Australia are summarized in Table 42.1. Some of the advantages of silica fume concrete include the ability to obtain high early

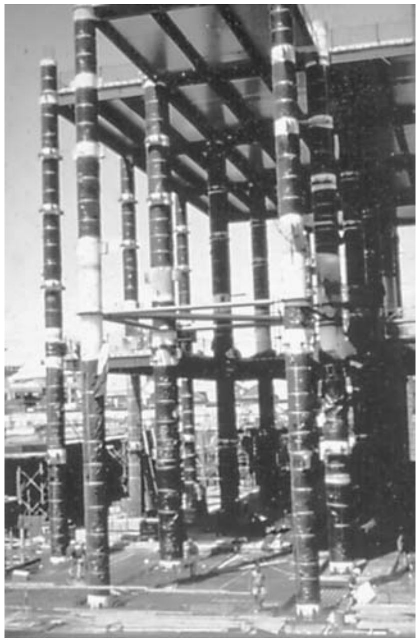

FIGURE 42.4 Steel tube columns at Market City, Australia.

TABLE 42.1 Details of Silica Fume Concretes Used in Selected Melbourne Projects

Structure	Final Slump (mm)	W:B Ratio	28-day Comp. Strength	Drying Shrink. (μstrain)	Mod of Elast. (GPa)	Creep (μstrain/MPa)	Flexural Strength (MPa)
Melbourne Central	160	0.30	89.5	590	38.3	31.5	—
Caulfield Grandstand	130	0.28	94.0	470	42.3	26.0	9.4
Southbank Boulevard	158	0.42	63.7	600	—	—	—

Notes: Final slump was measured after superplasticiser inclusion, modulus of elasticity and flexural strength measured at 28 days, drying shrinkage values are those measured at 56 days.
(After Burnett 1990)

strengths and reduced creep characteristics. The general drawback is the increase in water demand of the concrete due to the fineness of the material.

A mix design for 70 MPa high strength fly ash concrete used for the core of the main tower at the Melbourne Central project is given in Table 42.2. The mix pumped well, having an initial slump of 50 mm and rising to 170 mm in practice.

Cao et al. [1989] conducted studies into the properties of concretes made using silica fume, slag and fly ash. They concluded that the inclusion of silica fume was very effective in achieving strength in excess of 70 MPa at 28 days. Binders having silica fume coupled with either slag or fly ash resulted in concretes having a similar 28 day strength as the above mentioned mix with silica fume alone. For the triple blend mixes, an added 25 kg/m^3 of binder was needed to achieve the strength performance. The triple blend mixes were noted to have a significantly lower superplasticizing admixture demands for given weights of binder when compared to the silica fume concretes alone. In addition, the later age strength gains were greater for the triple blend concretes over the silica fume concretes alone.

TABLE 42.2 F′c (90 days) = 70 MPa Mix Proportions

Cement (kg/m$^{3)}$)	Geelong Type A (OPC)	470
Fly Ash (kg/m$^{3)}$)		150
Aggregates (kg/m$^{3)}$)	Deer Park (14 mm)	950
	Bacchus Marsh (10/7 mm)	310
Sand (kg/m^3)	Bacchus Marsh	430
Admixtures	Water reducing agent	470–600
(ml/100kg of binder)	Superplasticiser	400–800
Water (l/m^3)	Maximum	180

FIGURE 42.5 Towers with batch plant, Malaysia.

In 1998, the tallest concrete building in the world, the Petronas Twin Towers, was built in Malaysia. A silica fume/fly ash blended cement concrete was successfully used to produce high strength, high pumpability with low heat of hydration (Fig. 42.5).

Hibernia Offshore Platform — Grand Banks, offshore Newfoundland, Canada

The Hibernia Offshore Platform was reported by Hoff [1998] to be a gravity base structure (GBS) built for the recovery and processing of hydrocarbons on the Grand Banks, 315-km offshore Newfoundland in 80 m of water (Table 42.3). The platform is essentially a cylindrical concrete caisson that extends from the seabed to 5 m above the waterline. Four shafts extend above the caisson another 26 m to support all the equipment (Topsides) of the platform. It was designed to resist the impact of icebergs in a severe marine environment. It is expected to have a service life for as long as 70 years. The concrete used will undergo continual wetting and drying by seawater, seasonal freezing and thawing, abrasion from floating debris (principally ice), and be subjected to both operational and accidental loads. The concrete had a design strength of 69 MPa but produced concrete typically averaged 80 MPa. The majority of the concrete was pumped and placed by slipforming. The structure was constructed in the period of December 1991 and November 1996.

TABLE 42.3 Hibernia Offshore Platform Concrete Specifications

Performance Requirements		Specifications	
Type	Attributes	Test	Criteria
Physical	Density	Use of normal and light weight aggregate	Unclear
	Watertight	Water permeability under 2760 kPa	10^{-14} m/s
Mechanical	Strength	Compressive strength, MPa	69 at 1 year
	Volume stability	Drying shrinkage and Creep in 23 ± 1°C, 75 ± 4%RH	Tested
Durability	Abrasion	Compressive strength, MPa	49 at 1 year
	Alkali-silica reactivity	Na_2O equivalent [Fournier et al. 1994]	0.72%
	Freezing and thawing in the splash zone	Air entrainment, and ASTM C666 Proc. A	5 ± 1%
	Resistance to chloride	ASTM C1202, coulomb	<1000
Production	Crack controlled	Maximum temperature	70°C
		Max temperature gradient within concrete section	20°C in 300 mm

TABLE 42.4 SHT Immersed Tube Unit : Concrete Properties

Performance Requirements		Specifications	
Type	Attributes	Test	Criteria
Physical	Consistent Density	Density, kg/m^3	2260 ± 40
Mechanical	28-day Strength	Compressive strength, MPa	40
	Volume stability	56-day Drying shrinkage, μstrain	500
Durability	Low permeability	Water to binder ratio	0.38
	Crack control	Maximum crack width	0.1 mm
Production	Workability	Initial and superplasticized slump	65 and 150
	Crack controlled	Peak adiabatic temperature rise	40°C
	Strengths	3-day tensile and	2.5 MPa
		5-day compression	10 MPa

Harbour Tunnel — Australia

The Sydney Harbour Tunnel is a 2.3-km long land and underwater tunnel linking the northern and southern suburbs of Sydney. The underwater section is approximately 1 km in length and was constructed from eight precast concrete 'immersed tube' units, floated into position and then sunk into a prepared trench in the harbour bed. The multi-cell immersed tube units were cast at Port Kembla, 60 kms south of Sydney. Concrete thicknesses are typically in the order of 1 m. A HPC was specified for the units as they were required to meet a number of severe constraints, the most important of which are as follows:

- design life of 100 years
- low permeability with high resistance to chloride ingress
- limited cracking from thermal and drying shrinkage effects
- predictable and consistent density (to suit strict tolerances on buoyancy of the floated units)
- composition from readily available materials at an economical price, with consistently high quality control over a two year period.

The specification for the concrete was developed in the mid-1980s and is summarized in Table 42.4.

A blended cement comprising a 40/60 blend of portland cement and slag was selected. The total binder content was 380 kg/m.^{3} The laboratory testing was followed by the construction of a full scale prototype section of wall and floor by the contractor. This trial proved to be extremely valuable for testing construction

FIGURE 42.6 Picture of SHT unit, Australia.

procedures, fine tuning reinforcement retailing, and the like. The trial wall was also instrumented to record temperature and strain profiles against time. Crack widths were also measured against time and generally stabilized after a few days at 0.05 to 0.08 mm, within the 0.05- to 1.0-mm range predicted from earlier finite element analyses (Fig. 42.6).

Parallel Runway Sea Wall — Australia

HPC was specified for major elements in the new Parallel Runway project at Sydney's Kingsford Smith Airport, particularly in the runway and main taxi ways, the sea wall, sewer outfalls and the taxiway bridges.

In the case of the Sea Wall (Fig. 42.7), durability was the key issue for the concrete used. *A 100-year design life was required*. The construction comprised a "reinforced earth" wall faced with reinforced concrete panels and wave deflectors of approximately 180 to 200 mm thickness. The concrete was required to comply with the following specifications shown in Table 42.5 [Laurie and Gross 1993].

The concrete selected was a 90/10 blend of OPC and silica fume mix with a binder content of 380 kg/m^{3}and a well-graded aggregate. A combined water reducer/retarder and a superplasticizer were used to produce a working nominal slump of 100 mm. The reinforced concrete panels and wave deflectors were manufactured at a precast plant, which is approximately 40 km from the site.

Sustainability as Key Criterion

Consideration for sustainability in concrete is a relatively new concept. One key performance criterion adopted is in limiting the greenhouse gas associated with the production of binder such as Portland cement. However, the scope is considerably wider as discussed in the section on Concrete for Sustainable Development.

FIGURE 42.7 Aerial picture of seawall around Sydney Parallel Runway, Australia.

TABLE 42.5 Sea Wall Concrete Specifications

Performance Requirements		Specifications	
Type	Attributes	Test	Criteria
Physical	Consistent density	Density, kg/m^3	2400 ± 20
Mechanical	28-day Strength	Compressive strength, MPa	40
	Volume stability	56-day Drying shrinkage, μstrain	600
Durability to AS3600	Low permeability	Water to cement ratio	0.38
		Minimum binder content, kg/m^3	380
		Chloride permeability, coulomb	1000
Production	Workability	Slump, mm	80
	Strengths	20 hours Compressive strength	7 MPa

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42.4 Self-Compacting Concrete

D.W.S. Ho

In terms of construction, self-compacting concrete (SCC) is a relatively new technology. Since its introduction over 10 years ago in Japan, the concept of SCC has captured the imagination of researchers and practitioners around the world. This material can be considered as a high performance composite, which flows under its own weight over a long distance without segregation and without the use of vibrators. For the past decade, the focus on SCC has been on its fresh properties. Research and practical experience were well documented in the first symposium of self-compacting concrete held in Stockholm [RILEM

1999], and later in the state-of-the-art RILEM report [2000]. More information, particularly hardened properties, can be found in the second symposium held in Tokyo [SCC 2001].

The complete elimination of the consolidation process in SCC can lead to many benefits. Besides the obvious benefit of improved concrete quality in difficult sites relating to access and congested reinforcements, the use of SCC increases productivity, reduces the number of workers on site, and improves working environment. The reduction in overall construction cost could be around 2 to 5%. Depending on competition, the supply cost of SCC could be from 10 to about 50% higher than that of conventional concrete of similar grade. This leads to the low consumption of SCC in practice amounting to less than 5% of total concrete production. With improved quality control by suppliers and increased competitiveness in the market, the use of SCC is accelerating in many developed countries.

Fresh SCC must possess high fluidity and high segregation resistance. Fluidity or deformability means the ability of the flowing concrete to fill every corner of the mould as well as the ability to pass through small openings or gaps between reinforcing bars, often referred to as filling ability and passability of SCC respectively. To satisfy this high fluidity requirement, the maximum size of aggregate is generally limited to 25 mm. To improve flow properties, the amount of coarse aggregates is reduced and balanced by the increase in paste volume. Superplasticizer is needed to lower the water demand while achieving high fluidity. The common superplasticizer used is a new generation type based on polycarboxylated polyether, which is considerably more expensive than the traditional type used in conventional concrete. For SCC to have high segregation resistance, high powder content ranging from 450 to 600 kg per cubic meter of concrete should be specified. Powder generally refers to particles of sizes less than 0.125mm. Since cement content of 300 to 400kg/m^3 is often available, SCC usually incorporates 150 to 250 kg/m^3 of inert or cementitious fillers. Limestone powder is the common filler used, with fly ash and blast furnace slag enjoying increased popularity. Viscosity agent is sometimes incorporated to minimize the addition of fillers. This admixture is similar to that used in under-water concreting. It increases the viscosity of water, thereby increasing segregation resistance.

The rheology of fresh concrete is most often described by the Bingham model. According to this model, fresh concrete must overcome a limiting stress (yield stress, τ_o) before it can flow. Once the concrete starts to flow, shear stress increases with increase in strain rate as defined by plastic viscosity, μ. The target rheology of SCC is to reduce the yield stress to as low as possible so that it behaves closely to a Newtonian fluid. The other target property is "adequate" viscosity. The addition of water reduces both the yield stress and viscosity. Too much water can reduce the viscosity to such an extent that segregation occurs. The incorporation of superplasticizer reduces the yield stress but causes limited reduction in viscosity. The use of Bingham parameters is useful in describing the behavior of fresh concrete, but there is no consensus, at least at this stage, on their limiting values appropriate for SCC.

For site quality control, tests requiring simple equipment are often performed to indicate qualitatively or quantitatively the three basic properties of SCC: filling ability, passability, and segregation resistance. Slump-flow test is the most popular test method used because of it simplicity. A representative sample of concrete is placed continuously into an ordinary slump cone with a jug without tampering. The cone is lift and the diameter of the concrete (i.e., slump flow value) after the concrete has stopped is measured. The time to reach a flow diameter of 500 mm and final flow diameter are also noted. The degree of segregation can be judged to a certain extent by visual observation. This test reflects the filling ability, but the passability is not indicated. L-box, U-box, and V-funnel are other common tests available to assess one or more of the basic properties of SCC. Details of these tests can be found in the state-of-the-art RILEM document [2000].

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42.5 High Volume Fly Ash Concrete

V. Sirivivatnanon

Fly Ash and High Volume Fly Ash Concrete

The use of fly ash (FA) in structural concrete dates back to 1937 [Davis et al. 1937] with the construction of the Hungry Horse Dam in the U.S. in 1948 and Keepit Dam in Australia in 1957. Its early use in mass concrete structures was in order to reduce the heat of hydration. With the introduction of concrete pump in the 1970s, fly ash concrete was popularly used as pumpable concrete mixture especially in areas where there was a shortage of well-graded sand. In this case, the amount of fly ash (ASTM C618 Type F or its equivalence) in typical concrete mixtures varies from 60–100 kg/m.[3] This represents about 20 percent or less by weight of the total binder used. This typical dosage also probably reflects the optimum fly ash content in terms of cost related to compressive strength [Butler 1988]. However, when durability is of prime concern, the optimum dosage would need to be re-examined [Sirivivatnanon and Khatri 1998].

High volume fly ash (HVFA) concrete usually refers to structural concrete with fly ash content substantially higher than that used in conventional fly ash concretes. The concept of high volume replacement of cement with fly ash was recognized more than 35 years ago [Mather 1965]. Structural grade high fly ash content concrete has been tried at Didcot Power Station in 1981 [Proctor and Lacey 1984]. The Canadian Centre for Mineral and Energy Technology (CANMET) has carried out a major research project developing high volume fly ash concrete since 1985 [Malhotra 1985]. CANMET has adopted the approach of producing concrete with high volume (>50%) of low-calcium fly ash with water to cementitious materials ratio of 0.32 and a relatively high dosage of superplasticizer to achieve the required consistency. In the U.K., the focus has been on slightly higher water to cementitious materials ratio of 0.40 or more [Swamy and Hung 1986], and the addition of a small amount of highly reactive pozzolan such as silica fume to accelerate early hydration reactivity [Swamy and Hung 1998]. In Australia, a range of HVFA concrete was developed by the CSIRO in the late 1980s [Sirivivatnanon et al. 1995] and concrete with fly ash making up to 40–50 percent by weight (wt.%) of binder was first tried in a large scale in 1991 [Sirivivatnanon et al. 1993]. With the emphasis on concrete with similar fresh concrete characteristics as conventional concrete, the Australian industry has preferred HVFA concrete with no more than 40 wt.% of binder. HVFA concrete is now commonly specified for concrete exposed to aggressive chloride or sulphate environment in the eastern states in Australia. In Japan, HVFA porous concrete is being developed for concrete structures in river or seashore from the viewpoint of providing habitat for living organisms [Torii et al. 2001]. In this chapter, concrete with 30 wt.% and above of binder is classified as HVFA concrete.

Mixture Proportion and Properties

There have been different philosophies to mixture proportioning of HVFA concrete around the world. Details can be sought from literature listed in the "Further Information" section. The Australian approach and some performance data will be discussed in this section.

HVFA concretes can be proportioned using conventional mix design philosophy such as the ACI method [ACI 1989] or those originated from Road Note No. 4 [Teychenne et al. 1975]. Examples of mixture proportions given are those based on the *fixed dosage* of chemical admixture. They enable the design of concrete mixes with a reasonable amount of free water and hence a workability which does not differ significantly from conventional concrete. The *increased dosage* technique is useful in designing concrete mixes with low heat of hydration by limiting the amount of binder used.

Compressive Strength and Water-to-Binder Ratio

The compressive strength of HVFA concrete can be related to water-to-binder or water-to-cement ratio in a similar manner to Portland cement. In Fig. 42.8, the relationships between 28-day compressive strength and the water-to-binder ratio of HVFA concretes manufactured from a fly ash from New South

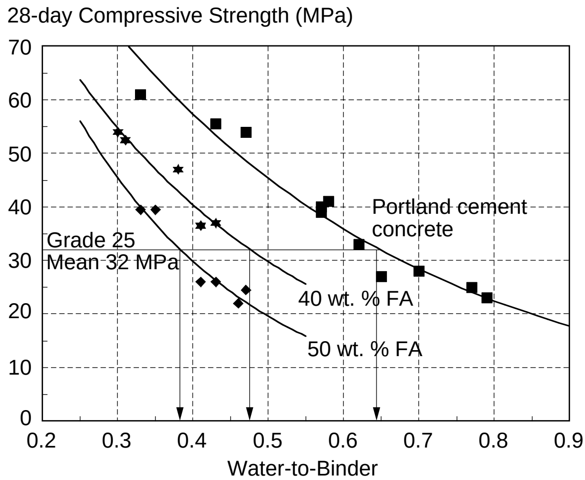

FIGURE 42.8 28-day compressive strength to water-to-binder ratio relationship.

Wales Australia, an ordinary Portland cement, and a 20 mm maximum size crushed basalt aggregate are given. While these relationships are distinguishable for different percentages of fly ash in the binder, they appear to be independent of the type of chemical admixture used. At a fixed water-to-binder ratio, lowering the percentage of fly ash results in an increase of corresponding strength of the concrete.

Fresh Concrete Properties

Water Demand

The water demand is found to depend on the type of chemical admixture used and the total binder content. Typical free water demands are given in Table 42.6. It has been found that the water demand depended more on the binder content than on the percentage of the fly ash in the binder.

Consistency, Setting Times and Early Strength

HVFA concretes generally contain higher binder contents than equivalent grade Portland cement concrete. This usually results in fresh HVFA concretes, which are more cohesive and sometimes very sticky. There is usually some delay in the setting times of HVFA concrete compared to Portland cement concrete. The extent of the delay depends on the particular cement and fly ash combination. In Fig. 42.9, the setting times of Waurn Pond (WP) cement and its combinations with 40 wt.% fly ashes from Eraring (E) and Vales Point (VP) are given. The delays in initial and final set are of the order of 1 and 1.5 hours, respectively. While these lengths of delay are quite acceptable in most applications, the use of a certain type of chemical admixture with a particular cement/fly ash combination could cause an unacceptable length of delay. Precaution should therefore be taken in checking the compatibility between all concreting materials prior to the production of HVFA concretes.

The early strength development of HVFA concrete is found to be slightly lower than Portland cement concrete as shown in Fig. 42.10. The 7-day to 28-day compressive strength ratio was 0.61 and 0.53 for Portland cement and HVFA concretes, respectively.

Mechanical Properties

Three specific types of hardened concrete properties are of interest to engineers. They are mechanical, volume stability and durability properties. In evaluating these properties of HVFA concrete, comparisons are usually made to Portland cement concrete of equivalent 28-day compressive strength.

For specific structural grades, the nominal water-to-binders of Portland cement and HVFA concrete are given in Table 42.7. It should be noted that HVFA concretes have W/B ratio ranging from 0.12 to 0.16 below Portland cement of equivalent grade. These differences will prove to be of significance in durability performance as discussed in a subsequent section.

TABLE 42.6 Approximate Free-Water Content (kg/m^3) Required to Produce HVFA Concrete with 20 mm Maximum Size Crushed Aggregate and Natural Sand

Chemical Admixture	Water Reducer			Superplasticizer		
Slump, mm	Binder, kg/m^3			Binder, kg/m^3		
	450	550	650	450	550	650
50–100	160	180	205	140	145	160
100–150	180	200	225	160	165	180

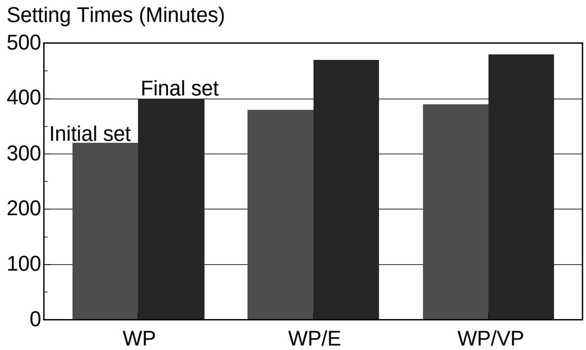

FIGURE 42.9 Initial and final setting times of concrete made from Waurn Pond cement and its combinations with Eraring or Vales Point fly ash.

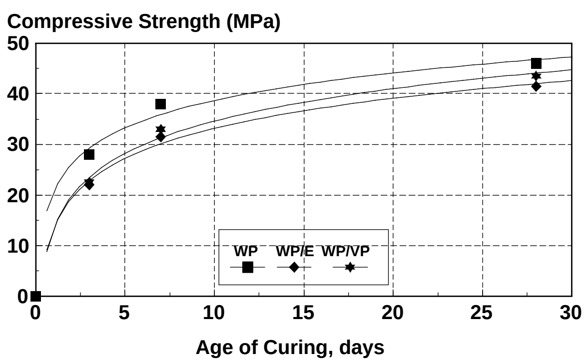

FIGURE 42.10 Early compressive strength development.

TABLE 42.7 Water-to-Binder Ratio of Portland Cement and HVFA Concrete at Corresponding Grade

Structural Grade MPa	Portland Cement Concrete	40% High Volume Fly Ash Concrete
25	0.64	0.48
32	0.56	0.41
40	0.49	0.35
50	0.40	0.28

TABLE 42.8 Mix Design of Concretes Given in Kilogram per Cubic Meter

Mix Designation	Cement	Fly Ash	Free Water	20 mm Agg.	10 mm Agg.	Sand	Admixture	W/B	W/C
200AL	245	0	170	605	605	820	WRA	0.7	0.7
205AL	185	185	160	605	605	680	WRA	0.43	0.87
320AL	315	0	175	600	595	765	WRA	0.57	0.57
324AL	235	160	160	615	610	645	WRA	0.41	0.68
450BL	355	0	155	620	605	770	SP	0.43	0.43
454BL	265	175	135	730	480	660	SP	0.31	0.51
200AH	270	0	175	590	580	810	WRA	0.65	0.65
205AH	200	200	165	580	575	680	WRA	0.41	0.82
320AH	335	0	195	570	570	770	WRA	0.58	0.58
324AH	255	170	180	565	560	655	WRA	0.43	0.72
450BH	340	0	160	620	615	760	SP	0.47	0.47
454BH	340	225	170	535	535	585	SP	0.3	0.5

Aggregates at s.s.d. All mixes had either a water reducing agent (WRA) or a superplasticizer (SP) at a dosage of 0.4 and 1.0 liter per 100 kg of binder, respectively.

TABLE 42.9 Mechanical and Drying Shrinkage Properties of the Low Slump HVFA and Portland Cement Concrete of Equivalent 28-day Compressive Strength

Mix Designation	Fly Ash %	Grade MPa	Slump mm	Flow mm	Compressive Strength, (MPa)		28-day Elastic Modulus GPa	28-day Flexural Strength MPa	Drying Shrinkage at 56 days $\times 10^{-6}$
					7-day	28-day			
200AL	0	20	50	370	22.0	27.0	35.5	3.7	575
205AL	50	20	55	315	18.5	26.0	35.5	3.5	455
320AL	0	32	60	350	30.5	41.0	44.5	4.3	605
324AL	40	32	50	365	28.0	37.0	44.5	4.6	525
450BL	0	45	60	370	49.5	55.5	49.0	5.3	615
454BL	40	45	85	290	39.5	52.5	49.0	4.7	525

TABLE 42.10 Mechanical, Drying Shrinkage and Properties of the High Slump HVFA and Portland Cement Concrete of Equivalent 28-day Compressive Strength

Mix Designation	Fly Ash %	Grade MPa	Slump mm	Flow mm	Compressive Strength, (MPa)		28-day Elastic Modulus GPa	Creep Rate 10^{-6*1}	Drying Shrinkage at 56 days $\times 10^{-6}$
					7-day	28-day			
200AH	0	20	115	465	23.0	28.0	36	38.3	600
205AH	50	20	125	425	18.5	26.0	36	16.7	465
320AH	0	32	110	435	34.5	40.0	47	23.9	650
324AH	40	32	100	420	26.5	36.5	42	14.0	605
450BH	0	45	80	345	44.5	54.0	50	—	540
454BH	40	45	130	390	40.5	54.0	53	—	595

Note: 1. Creep rate is given in 10^{-6} MPa/ln(t + 1).

Typical mixture proportions of two series of concretes are given in Table 42.8. The first L series is a low 50 ± 15 mm slump HVFA and portland cement concretes designed for pavements (and other slab applications). The other H series are pump mixes with 100 ± 25 mm slump HVFA and portland cement concretes developed for other structural works.

The mechanical properties of the mixtures given in Table 42.8 are summarized in Tables 42.9 and 42.10. The results indicated that the flexural strength and elastic modulus of HVFA concretes are similar to

Portland cement concretes of equivalent 28-day compressive strength. HVFA concretes can thus be used for concrete structures in the same manner as Portland cement concretes. Their elastic properties can also be predicted from the compressive strength and density (ρ) in the same manner as portland cement concretes as given in Concrete Structures standard such as the Australian Standard AS 3600 [SAA 1988].

Drying Shrinkage and Creep Characteristics

HVFA concretes can have a similar or up to 20% lower drying shrinkage than portland cement concretes. The reductions in shrinkage are more significant in concretes of lower grades. Typical drying shrinkages at 56 days for both the low and high slump concretes are given in Tables 42.9 and 42.10. The shrinkage values are well below 700 × 10^{-6} recommended in AS 3600. The increase in drying shrinkage with time up to 91 days for Portland cement and HVFA concrete is shown in Fig. 42.11 for the pump mix H-series. The reduction in shrinkage of HVFA concrete can be observed as early as 28 days for the lower grade 20 concrete. This trend remains up to 91 days and beyond.

The creep characteristics of concrete are significantly improved with the use of high volume of fly ash as shown in Fig. 42.12. A creep rate, F (K), is determined from the slope of the line relating creep strain per unit stress to the natural logarithm of time $\log_e(t+1)$ where t is the time of loading in days. The creep rates were reduced by 40 and 55 percent in grade 32 and 20 HVFA concretes respectively compared to portland cement concretes of equivalent 28-day compressive strength as shown in Table 42.10. There can therefore be clear advantages in the use of HVFA concretes for structural members that are sensitive to high creep strain such as long span bridge girders and columns in high rise buildings.

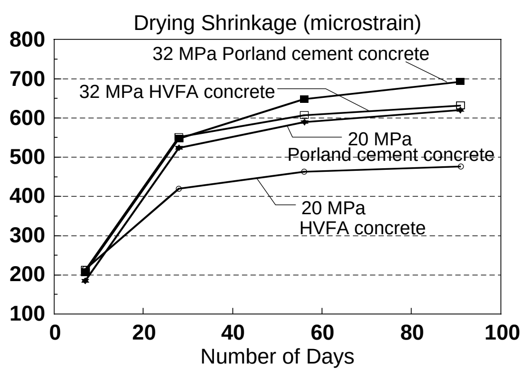

FIGURE 42.11 Drying shrinkage of portland cement and high volume fly ash concrete of various grades with high slump of 100 ± 25 mm. Creep Strain (microstrain per MPa)

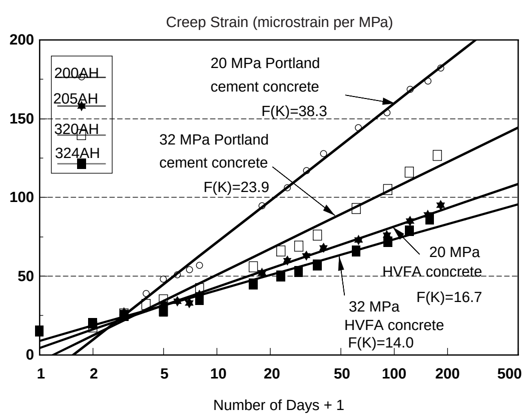

FIGURE 42.12 Strain due to creep of portland cement and high volume fly ash concrete of various grades.

Durability Properties

The durability of HVFA concretes with respect to the protection of steel reinforcement against corrosion and the resistance to deterioration in sulphate environments has been studied. Three fly ashes, FA1, FA2 and FA3 from three different States in Australia were examined. Most durability studies were carried out using mortars.

It is well known that when the pH of the concrete surrounding steel reinforcement is sufficiently lowered by *carbonation* or when there is a sufficient level of *chloride ions* at the steel surface, steel corrosion occurs. This could eventually result in cracking of concrete and loss of structural integrity. The *service life* of reinforced concrete structure is closely related to properties of the concrete such as its resistance to carbonation, carbonation-induced steel corrosion, resistance to chloride penetration and chloride-induced steel corrosion. These properties of HVFA concretes are discussed in this section.

The benefit of the use of fly ash concretes in moderate sulphate environment has been recognized in current British and Australian Standards [BSI 1985, SAA 1978]. In this work, the sulphate resistance of fly ash blended cement concretes in a 5% sodium sulphate solution as well as solutions at pHs of 7 and 3 are given.

Corrosion of Steel Reinforcement

Corrosion of steel reinforcement is one of the most common durability problems in reinforced concrete structures. This problem is caused by carbonation or chloride penetration or both. Generally the deterioration of concrete due to corrosion of steel reinforcement is characterized into three stages, i.e., *initiation*, *propagation* and *accelerated corrosion*. In the initiation stage, steel is protected by its passivation in high pH condition and the absence of chloride ions. The corrosion rate (steel loss) during this stage is very small and is considered negligible for engineering purposes. When the alkalinity of the concrete surrounding the steel is lowered sufficiently by carbonation and/or when there are sufficient chloride ions at the steel/concrete interface, steel passivation is destroyed. Corrosion rate of steel becomes significant. This is the propagation stage. The accelerated corrosion stage occurs when there is severe cracking and damage to the concrete cover caused by the cumulative effect of the steel corrosion.

Carbonation-induced corrosion — The rate of carbonation or the advance of the carbonation front depends on many factors. Some of the important factors are time of exposure, the nature of cementitious matrix, its "permeability" to carbon dioxide and the condition surrounding the concrete (moisture, temperature). The carbonation rate can vary significantly with the climate. Furthermore, carbonation is not a problem in itself but carbonation-induced corrosion of steel reinforcement is. Hence it is necessary to consider the corrosion rate in conjunction with the carbonation rate.

With the length of long-term exposure limited to two years in a standard laboratory condition of 23°C 50% RH, a condition that results in a significantly higher carbonation rate than that expected in an exposed outdoor condition, the depth of carbonation of a range of Portland cement and HVFA concretes is compared on the basis of equivalent 28-day compressive strength (F_c). Figures 42.13 and 42.14 show

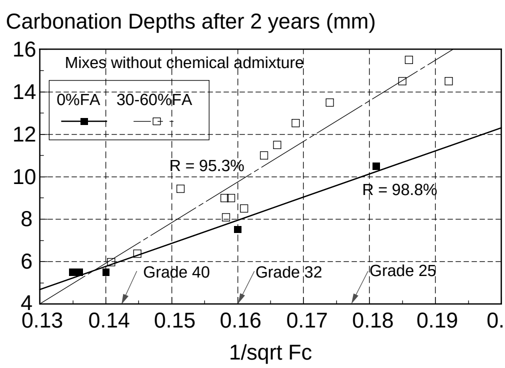

FIGURE 42.13 Carbonation depth of concretes without chemical admixture after 2 years exposure in 23°C 50% RH.

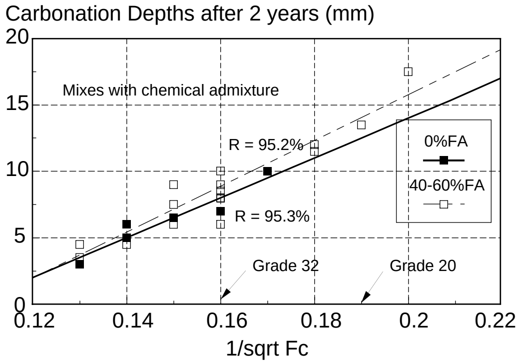

FIGURE 42.14 Carbonation depth of concretes with chemical admixture after 2 years exposure in 23°C 50% RH.

the relationship between carbonation depth and the reciprocal of the square root of compressive strength 1/sqrt F_c of concretes proportioned without and with chemical admixture respectively.

For concretes without chemical admixture, the carbonation depth of HVFA concretes can be higher than that of corresponding portland cement concrete as shown in Fig. 42.13. The differences in the carbonation depth of the two concrete increases with the reduction in the strength level.

For concretes designed with a standard chemical admixture dosage, that is 400 ml of water reducer or 1000 ml of superplasticizer per 100 kilogram of binder, the differences in the carbonation depth of HVFA and portland cement concretes, as shown in Fig. 42.14, are not significant especially for concretes in the grade range of 25–32 MPa. According to AS 3600, grades 25 and 32 are recommended for exposure classification A2 and B1. These classifications cover the conditions where carbonation could pose a threat to the durability of concrete structures.

It is noted that HVFA concretes designed with chemical admixtures are the types of structural concrete that should be chosen for most building and civil engineering works. Based on the short-term data, these HVFA concretes would be expected to perform as well as Portland cement concretes.

Chloride-induced corrosion — Chloride ions can penetrate into concrete through the effect of concentration gradient and/or through the effect of capillary action. The mechanism of chloride penetration is often described as diffusion. This may be an over simplification for the process. The transportation of ionic species into a concrete medium is complicated. This is because of the possible reactions between the chloride ions and the hydrates that constantly alter the pore system. Regardless of the mechanism of transportation of chloride ions, it is known that when the amount of chloride ions at the steel/concrete interface is higher than a critical concentration, steel corrosion will occur. This critical chloride concentration is called the *chloride threshold* level. It is also known that chloride threshold level depends on binder content and the chemistry of the pore solution.

It has been suggested that the hydroxyl concentration is the controlling factor with regard to the chloride threshold level. A relationship, such as $Cl^-/OH^- = 0.6$, has been suggested for the estimation of chloride threshold level. Recent work by *Cao* et al. [1992] indicated that this is not the case since blended cements can have similar chloride threshold level to Portland cements despite having lower OH^- concentrations in their pore solution.

When passivity of steel cannot be maintained, corrosion of steel occurs. The service life of a reinforced concrete structure is directly related to the development of the corrosion and its rate. However, it must be stressed that the mode of corrosion should also be considered. For example, metal loss due to pitting corrosion may be much smaller than that of general corrosion. However, the effect of pitting corrosion (concentrated metal loss in a small area) can be very dangerous to the integrity of the structure in terms of loss in load carrying capacity.

The detection of corrosion and the measurement of corrosion rate of steel can be used to compare the behavior of different binders in the *initiation* and *propagation* stages of the deterioration. One of the

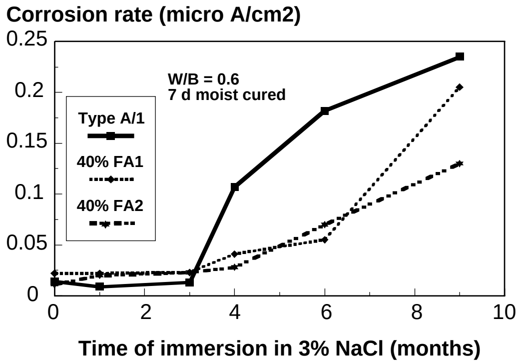

FIGURE 42.15 Effect of binder on the corrosion rate of steel embedded in 7-mm thick mortars.

possible methods of comparatively assessing the initiation stage is by monitoring the corrosion rate of steel embedded in concrete or mortar. The change in corrosion rate from "negligible" to "significant" can be used to determine the effect of binder type on initiation period. These data are presented in Fig. 42.15. The corrosion rate was determined by using polarization resistance technique. It must be stressed that the initiation stage is controlled by the rate of chloride penetration, the chloride threshold level, and the concrete cover thickness.

The effect of 40 wt.% fly ash binder systems on the development of corrosion of steel is shown in Fig. 42.15. From this figure, it can be seen that the use HVFA binders leads to a similar or longer initiation period as compared to portland cement mortar of the same W/B and with limited initial curing period of 7 days. The cover of the mortars over steel sample in this case is about 7 mm. It can be seen that the initiation period where the corrosion rate of steel is negligible for this configuration is about 3 months for portland cement and about 3 to 4 months for both HVFA binders. It is expected that for a realistic concrete cover in a marine environment, say about 50 mm, the effect of HVFA binder in terms of the increase of the initiation period will be further magnified. This conclusion is based on better chloride penetration resistance performance data at larger cover depths [Thomas 1991, Sirivivtnanon and Khatri 1995] and the lower W/B ratio used to produce HVFA concrete of the same strength grade as portland cement concrete (Table 42.2). The propagation period, characterized by significant corrosion rate, is also a very important to the maintenance-free service life. The main reason is that, for a binder system in which a low corrosion rate can be maintained, the service life will be prolonged by an extended propagation period.

The effect of fly ash on the corrosion rate of steel embedded in mortars W/B ration of 0.4 and cured for 7 days, is shown in Fig. 42.16. It is clear that the use of high volume fly ash binder systems can result

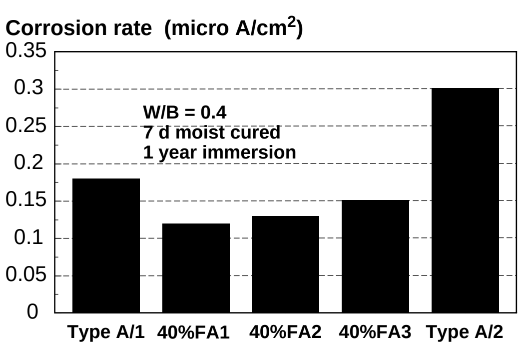

FIGURE 42.16 Effect of binder on the corrosion rate of steel embedded in 7-mm thick mortars after 1 year of immersion in 3% NaCl.

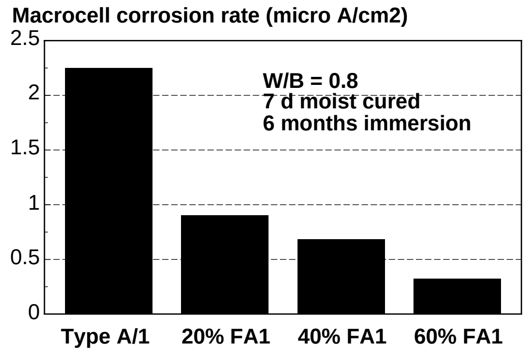

FIGURE 42.17 Effect of varying dosage of fly ash on the *Macrocell* corrosion rate of steel after 6 months of immersion in 3% NaCl.

in reduced corrosion rate of steel in comparison to the Portland cements. The extent of the reduction depends on the source of the fly ash.

It should be noted that the corrosion rate mentioned above is that of *microcell* corrosion rate where the anode and cathode of the corrosion cell are in close proximity and sometimes are not physically distinguishable. For most reinforced concrete applications, there are situations where the anode and cathode of the corrosion cell can be physically separated. In such a situation, termed *macrocell* corrosion, the characteristics of the concrete, such as the resistance to ionic transportation and its resistivity, will have important influence on the corrosion rate of steel. By using fly ash blended cement, both of these characteristics of the concrete will be improved and hence the corrosion rate will be reduced. This has been confirmed experimentally as shown in Fig. 42.17 in which the macrocell corrosion rates were determined using a model of equal areas of anode (chloride contaminated area) and cathode (chloride free area) and the mortar medium was 15 mm. It can be seen that the beneficial effect of HVFA binder systems in reducing the macrocell corrosion rate compared to Portland cement mortar of the same W/B is very significant. The effect of increasing the fly ash proportion on the reduction of macrocell corrosion rate is also clearly evident.

The overall conclusion is that the use of HVFA concrete can result in extended maintenance-free service life of reinforced concrete structure in marine environments. This is based on its potential in increasing the initiation period and reducing the corrosion rate in the propagation period of the deterioration process due to chloride-induced corrosion of steel reinforcement.

Sulphate Resistance

Sulphate resistance of a cementitious material can be broadly defined as a combination of its *physical* resistance to the penetration of sulphate ions from external sources and the resistance of the *chemical* reactivity of its components in the matrix to the sulphate ions. Both factors are important to the overall resistance to sulphate attack of a concrete structure. However, the chemical resistance to sulphate attack is considered to be more critical for long-term performance. The physical resistance can be improved by good concreting practice such as the use of concrete with low W/B, adequate compaction and extended curing. The extent of chemical reactivity of hardened cement paste with sulphate ions, on the other hand, depends very much on the characteristic of the binder system. It may appear obvious that for a long service life, a good concrete is required. However, this will be better assured if the concrete is made from a binder that has low "reactivity" with sulphate ions.

Sulphate resistance of cementitious materials can be assessed by a variety of methods. The performance of two fly ashes and three Portland cements was examined in terms of *expansion characteristic* and *strength development* of mortars immersed in 5% Na_2SO_4 solution. In addition, the performance of mortars in 5% Na_2SO_4 solution at lower pHs of 3 and 7 were determined.

All the expansion mortars were made with a fixed sand-to-binder ratio of 2.75 and variable amount of water to give a similar flow of 110 ± 5%. In most cases, the fly ash mortars had lower W/B than

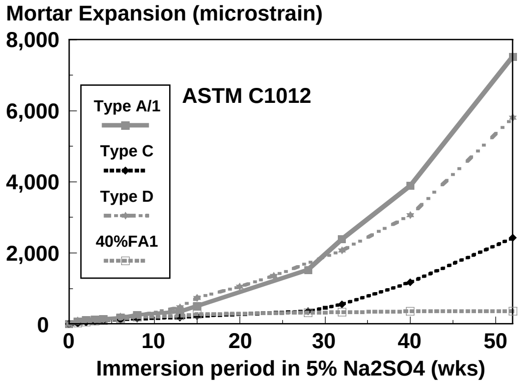

FIGURE 42.18 Expansion of mortar bars in 5% Na_2SO_4 solution to ASTM C1012.

Portland cement mortars. When a test was performed to ASTM C1012 [1989], the samples were cured for 1 day at 35°C and subsequently at 23°C until they reached a compressive strength of 20 MPa before immersion in the Na_2SO_4 solutions. Mortars used in the evaluation of compressive strength retention had the same sand-to-binder ratio of 2.75 and a W/B ratio of 0.6.

Apart from two fly ashes, FA1 and FA2, the three portland cements used were Type A, C and D cement (normal, low heat and sulphate-resisting cement respectively) according the superseded AS 1315–1982.

The effect of fly ash blended cement on expansion of mortar using the ASTM C1012 procedures is shown in Fig. 42.18. A 5% Na_2SO_4 solution, without any control on the pH of the solution, was used in this case.

It can be seen that the use of 40 wt.% fly ash blended cement greatly reduces the expansion of mortar. In fact, the expansion of fly ash blended cement is much lower than that of the three Portland cements Type A, C, and D. When all the mortars were cured for a period of 3 days, the effect of both fly ashes on the reduction of expansion was also very clear, as shown in Fig. 42.19.

The improved expansion characteristic of fly ash blended cement mortars was maintained in sulphate environments of low pHs as shown in Figs. 42.20 and 42.21. In fact, the same effect has been observed for most Australian fly ashes when used at a replacement level of about 40% [1994].

Apart from a much lower expansion characteristic in sulphate environments, the use of binders with "high" fly ash percentage leads to superior strength retention in a sulphate solution. Figure 42.22 shows that for most Portland cements, the loss of compressive strength was observed after about 6 months in sulphate solution. Whereas, the 40 wt.% fly ash blended cement mortars showed strength increases even after 1 year.

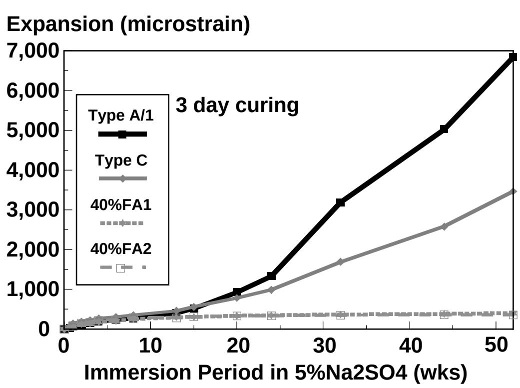

FIGURE 42.19 Expansion of mortar bars in 5% Na_2SO_4 solution to ASTM C1012 but with 3 days moist curing.

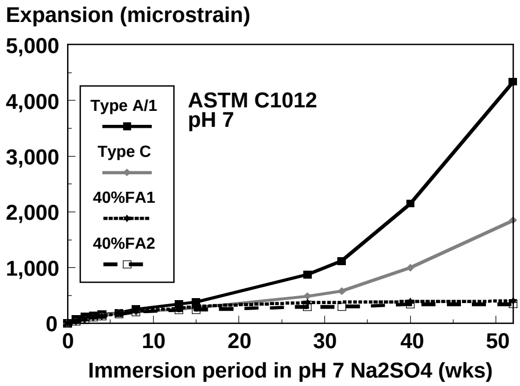

FIGURE 42.20 Expansion of mortar bars in a 5% Na_2SO_4 solution (low pH 7) to ASTM C1012.

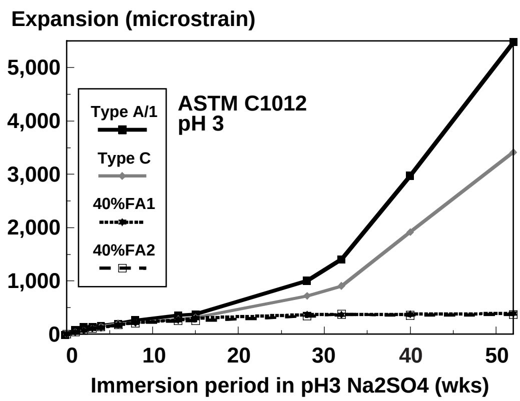

FIGURE 42.21 Expansion of mortar bars in a 5% Na_2SO_4 solution (low pH 3) to ASTM C1012.

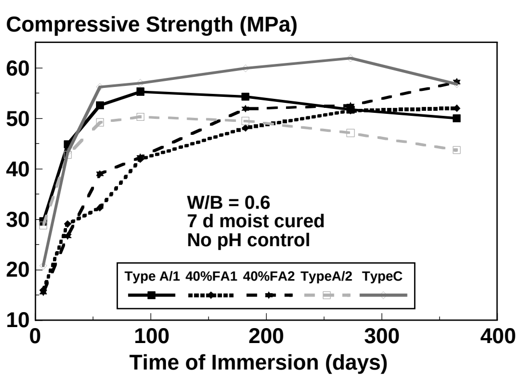

FIGURE 42.22 Compressive strength of mortars after different periods of immersion in sulphate solution.

In low pH sulphate solutions, the beneficial effect of high replacement fly ash binder was even more pronounced as shown in Figs. 42.23 and 42.24.

The results clearly indicate that for concrete application in sulphate environment, particularly in those where the pH is low, the use of HVFA concrete has the highest probability of extending the service life.

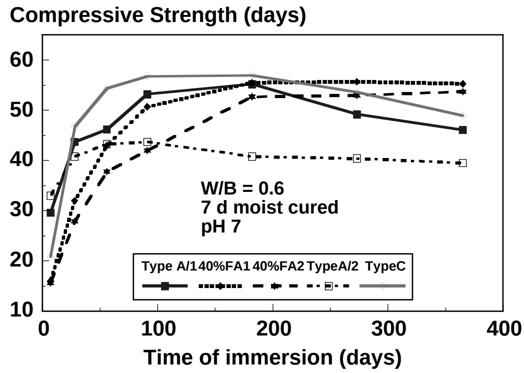

FIGURE 42.23 Compressive strength of mortars after different periods of immersion in a pH 7 sulphate solution.

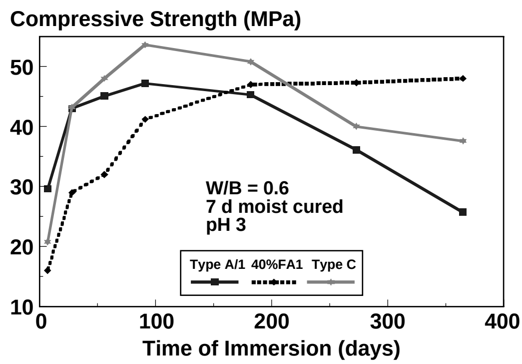

FIGURE 42.24 Compressive strength of mortars after different periods of immersion in a pH 3 sulphate solution.

Optimum Dosage for Durability

In marine and sulphate environments, the optimum dosage of fly ash has not been strictly determined but is expected to be around 40 wt.%. When the proportion of fly ash by weight exceed about 50%, the lowering in compressive strength becomes significant. With the present cost structure of concreting materials including fly ash, a HVFA concrete with 40 wt.% fly ash is marginally more expensive than portland cement concrete of equivalent strength grade. However, with the significantly increased expected service life, it is argued that the optimum dosage with respect to service life of HVFA concrete would be around 40% by weight of binder.

Basis for Applications

Sufficient knowledge is now available on the design, production and properties of HVFA concrete for its applications to be identified. The inherent properties and limitations of HVFA concrete are the keys to its selection in suitable applications. They are given in relative terms to the properties of Portland cement and conventional fly ash concrete as follows:

- good cohesiveness or sticky in mixes with very high binder content;
- some delay in setting times depending on the compatibility of cement, fly ash and chemical admixture;
- slightly lower but sufficient early strength for most applications;
- comparable flexural strength and elastic modulus;
- better drying shrinkage and significantly lower creep;

- good protection to steel reinforcement in high chloride environment;
- excellent durability in aggressive sulphate environments;
- lower heat characteristics; and
- low resistance to de-icing salt scaling [Malhotra and Ramenzanianpour 1985].

Built Structures

Examples of built structures are given in accordance with the primary basis for which HVFA concrete was selected. It is emphasised that the fresh and mechanical properties were usually comparable to conventional concrete it replaced unless highlighted. The economy of HVFA concrete depends on the transport cost. This has resulted in the tendency for its popularity in locations near the supply sources.

Pioneering Firsts

High volume fly ash concretes have already found applications in major structures in many countries. The first field application in Canada, carried out in 1987, was reported by Malhotra and Ramenzanianpour [1985]. This consisted of the casting of a concrete block, 9m × 7m × 3m, at the Communication Research Centre in Ottawa. The block, cast indoors in permanent steel forms, is being used in vibration testing of components for communication satellites and was required to have as few microcracks as possible, a compressive strength of at least 40 MPa at 91 days, and a Young's modulus of elasticity value of at least 30 GPa. The mixture proportions are: 151 kg/m^3 Portland cement ASTM type II, 193 kg/m^3 of ASTM Class F fly ash, 1267 kg/m^3 coarse aggregate, 668 kg/m^3 fine aggregate, 125 kg/m^3 water, 5.6 kg/m^3 superplasticizer, and 680 mL/m^3 AEA. The recommended placing temperature of the concrete and ambient temperature was 7° and 24°C, respectively. At the end of placing, the temperature was reported to be 12°C because of delays in placing. A peak temperature of 37.5°C was reached in the block after 7 days of casting at which time the block was performing satisfactorily for the intended purposes. In 1988, *Langley* [1988] reported its use in the Park Lane and Purdys Wharf Development in Halifax, Nova Scotia, Canada. It is also believed that a 40 to 50% wt. fly ash concrete was used in the construction of the caissons of the famous Thames River Flood Barrier in London and in bridge foundations in Florida by the Florida Department of Transportation.

In 1992, Nelson et al. [1992] reported the application of concrete with 40% wt. fly ash in the construction of sections of road pavement and an apron slab at Mount Piper Power Station in New South Wales, Australia. The casting of the apron slab is shown in Fig. 42.25. At about the same time, Naik et al. [1992] reported the successful use of three fly ash concrete mixtures, 20% and 50% ASTM C618 Class C fly ash and 40% ASTM C618 Class F fly ash to pave a 1.28 km long roadway in Wisconsin.

Service Life Designs

The largest volume of HVFA concrete used in Australia was in the construction of the basement slabs and walls of Melbourne Casino in 1995. Figure 42.26 shows concreting activities on the site. According to *Grayson* (pers. Comm.), of Connell Wagner, low drying shrinkage and durable concrete was required for the construction of the 55,000m^2 basement which was located below the water table. Saline water was found on the site situated near the Yarra River. Slabs with an average thickness of 400 mm were designed to withstand an uplift pressure of 45 kPa. The concrete was specified to contain at least 30 kg/m^3 of silica fume or 30 wt.% of fly ash or 60 wt.% of a combination of slag and fly ash. Drying shrinkage within 650 microstrains was also specified. A 40 MPa HVFA concrete containing 40 wt.% fly ash was selected for the 40,000 m^3 of concrete required for the basement. The fresh concrete was reported to behave similarly to conventional concrete and a drying shrinkage of lesser than 500 microstrains was achieved. In addition, a similar concrete was used in the construction of the pile caps and two raft slabs in the same project. In Malaysia, concrete containing 30 wt.% fly ash was used for the substructure and piers of the Malaysian-built half of the Malaysia Singapore Second Crossway in 1996 (Fig. 42.27). The HVFA concrete was chosen for its chloride and sulphate resistance [Sirivivatnanon and Kidav 1997]. Ordinary Portland cement concrete was used for the superstructure of the Crossway.

FIGURE 43.25 First pour of 32 MPa HVFA concrete in an apron slab at Mount Piper Power Station in New South Wales in 1991.

FIGURE 45.26 Crown Casino under construction in Melbourne Australia.

FIGURE 42.27 Construction activities at the Malaysia Singapore Second Crossway in 1996.

One interesting application recently reported [Mehta and Langley 2001] is the use of unreinforced HFVA concrete in the construction of the foundation of the San Marga Iraivan Temple along the Wailua River in Kaua'i, Hawaii. This is a unique temple in the Western Hemisphere as it is constructed of hand carved white granite stone from a quarry near Bangalore in India. The temple is constructed of highly durable stone, which will contribute to the design service life of 1000 years. The foundation slab was required not to settle more than approximately 3.2 mm in a distance of 3.66 meters because the free-standing components such as columns and lintels would separate beyond safe limits. The temple foundation was designed to have low shrinkage, slow strength development, low heat evolution and improved microstructure particularly in the paste aggregate transition zone. A concrete containing a high volume of Class F fly ash was used to meet the design criteria and emulate the ancient structures.

Construction Economy

In Perth, Western Australia, a 50 wt.% fly ash concrete has been used [Ryan and Potter 1994] for the construction of the secant piles at Roe Street Tunnel. The ground water in the area was tested and found to be abnormally acidic, pH = 4.0. Thus it was necessary for all piles to contain a high binder content to limit the attack of the ground water on the concrete. A minimum binder content of 350 kg/m^3 and W/B = 0.5 was specified. The requirements posed problems for the low early-age strength needed to allow the soft piles to be bored. A number of trial mixtures were cast and the preferred option for the binder was a 50:50 Portland cement fly ash blend.

In 1990, Heeley [1999] reported the development and use of HVFA shotcrete in the construction of the Penrith Whitewater Stadium, shown in Fig. 42.28, for the Sydney Olympic Co-ordination Authority. The design was based on shotcrete because conventional formwork would have been prohibitive. In this shotcrete, ultra-fine fly ash was used to replace 44 wt.% of the binder. This provides the cohesiveness normally achieved by the use of silica fume.

Choice for Sustainability

Following the success of the use of HVFA concrete at the Liu Centre on the campus of the University of British Columbia in Canada, a range of HVFA concrete, covered by an EcoSmart™ Concrete Project, with FA contents ranging from 30 to 50 wt.% of binder was successfully used in a number of structures including the Ardencraig residential development and 1540 West 2nd Avenue — an Artist Live/Work studio near Granville Island in Vancouver, 50% *in situ* fly ash concrete and 30% fly ash concrete in precast

FIGURE 42.28 Arial view of the Penrith Whitewater Stadium showing the complexity of forms, which favors the use of shotcrete (picture courtesy of the Sydney Olympic Co-ordination Authority).

elements at the Brentwood and Gilmore SkyTrain Station, and the majority of concrete building elements at Nicola Valley Institute of Technology/University of the Cariboo in the interior of British Columbia [Bilodeau and Seabrook 2001].

With the emphasis on sustainability in the 2000 Sydney Olympic, a HVFA concrete containing 46 wt.% of binder was the chosen for slab-on-grade of a number of houses in the Athletics Village. A 56-day 20 MPa specification was used and achieved.

Current Developments

While the potential applications of HVFA concrete are numerous, three recent developments are worth noting. The first is in the use of fibre-reinforced HVFA shotcrete to cap degraded rock outcrops and to cover mine waste dumps, the second is in High Performance Concrete for massive marine structure, and the third is in the upgrading of dam structures.

Morgan et al. [1990] found polypropylene fiber-reinforced HVFA shotcrete to be applied satisfactorily using conventional wet-mix shotcrete equipment and that it required a minimum amount of cementitious material and water content of around 420 and 150 kg/m^3, respectively. The polypropylene fiber content required to provide a satisfactory flexural toughness index appeared to be between 4 and 6 kg/m^3. They suggested its use in capping rock outcrops which are susceptible to degradation and for covering mine waste dumps. Seabrook [1992] identified the same technology to produce a lower cost shotcrete while maintaining reasonable quality and durability for application on waste piles to prevent leaching of acids and heavy metals. Heeley [ASTM 1989] reported the successful application of HVFA shotcrete in the construction of the Penrith Whitewater Stadium for the Sydney Olympic Co-ordination Authority. The design was based on shotcrete because conventional formwork would have been prohibitive. In marine and offshore structures such as bridges, wharfs, sea walls and offshore concrete gravity structures, concretes with excellent sulphate and chloride durability are required. Where large or long-span structural members are used, low heat development and low creep characteristics are of vital importance. These are applications where HVFA concretes would be considered an ideal solution. Attention would need to be given to the use of compatible concreting materials if high early strength is required in precasting or slip forming construction.

In rehabilitation of massive concrete structures such as the raising of dam height for improved safety and flood mitigation, new high strength and high elastic modulus concrete matching existing concrete is often required. The new concrete must have low shrinkage to minimize the effects of the new concrete on existing concrete and low heat development characteristics to avoid potential cracking problems during construction. HVFA concretes have been found to have all the required attributes for such use.

Summary

High Volume Fly Ash (HVFA) concrete is relatively new concrete for the concrete industry. The range of engineering properties including: consistency and setting times of fresh concrete, mechanical, volume stability and durability properties of hardened concrete are found to be suitable for a wide range of applications [Malhotra and Ramenzanianpour 1985, Swamy and Hung 1998, Sirivivatnanon et al. 1995]. These are vital to satisfy the structural and serviceability requirements of concrete structures. Experiences gained from field trials and large-scale implementations around the world confirmed the *practicality* of this new concrete. Significant improved volume stability, in terms of reduced drying shrinkage and better creep characteristics, can result in new solutions to many engineering problems. Most important of all, the improved *durability* performance of HVFA concrete in marine and high sulphate environments signaled the tremendous economic gains that could be derived from the expected increased service life. In most cases, such technical benefits can be gained with significant contribution to sustainable development as discussed in the following chapter. Exciting new developments in the applications of HVFA concrete have been highlighted. It remains for the construction industry to adopt and advance this new technology to its full potential.

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42.6 Concrete for Sustainable Development

V. Sirivivatnanon

Sustainable Development

Sustainable development means different things to different people. In one context, it deals with how the world's diminishing resources are managed to sustain the rapid increase in the population in terms of provision of infrastructure to the *built environment* to provide physical comfort such as shelters, public utilities and transportation with minimum adverse effect to the *natural environment*. A Brundtland report of the World Commission on Environment and Development [1987], reiterated and broadened at the 1992 Rio Environmental Summit, defines sustainable development as "development that meets the needs of the present without compromising the ability of future generations to meet their own needs."

Sustainable construction is defined by the UK Government Construction Client's Panel [2000] as "the set of processes by which a profitable and competitive industry delivers built assets (buildings, structures, supporting infrastructure and their immediate surroundings) which:

Enhance the quality of life and offer customer satisfaction;
Offer flexibility and the potential to cater for user changes in the future;
Provide and support desirable natural and social environments;
Maximise the efficient use of resources."

In his wisdom in sustainable development, Pierre–Claude Aïtcin [2000] had predicted binders and concrete of tomorrow in a review published in *Cement and Concrete Research* at the beginning of this new millennium. He passionately elaborated that:

> The binders of tomorrow will contain less and less ground clinker; they will not have necessarily such a high C_3S content; they will be made with more and more alternative fuels. The will have to fulfil tighter standard requirements and they will need to be more and more consistent in their properties, because the clinker content will be lower in the blended cements. The binders of tomorrow will be more and more compatible with complex admixtures and their use will result in making more durable concrete rather than simply stronger concrete. The concrete of tomorrow will be GREEN, GREEN AND GREEN. Concrete will have a lower water/binder ratio, it will be more durable and it will have various characteristics that will be quite different from one another for use in different applications. The time is over when concrete could be considered a low-priced commodity product; now is the time for concrete "à la carte.

Contractors and owners have to realize that what is important is not the cost of 1 m^3 of concrete but rather the cost of 1 MPa or 1 year of life cycle of a structure."

Moving Forward

In the U.K., BRE has published the *Green Guide to Specification* [2000], which provides guidance for designers on the relative impact of different construction assemblies against a range of environmental criteria, including resources use, toxicity, embodied energy and durability.

In Denmark, a Green Concrete program [Glavind et al. 1999] was launched in 1998 with the goal to develop the technology necessary to produce resource-saving concrete structures, by means of new binding materials in new concrete combined with the possible reuse of materials.

There is a large European project focused on cleaner technologies in the life cycle of concrete products (TESCOP), which aims to develop and implement cost-effective cleaner technologies to reduced the environmental taxes, fulfill environmental requirements in the concrete industry, and reduce the environmental impact of concrete products [Haugaard and Glavind 1998].

The Australian Government has a clear and definite commitment to ensure that the construction industry moves toward ecological sustainability through voluntary means. There is an increasing awareness amongst practitioners (designer and builders) that there are many environmental issues that need to be considered in the design, construction and operation of buildings to ensure the built environment is sustainable. For example, the Greenhouse Challenge Program was launched by the Commonwealth of Australia in 1995. The Australian cement industry has been a keen supporter of this program, and each cement company has introduced into its operations a Greenhouse Energy Management System modeled on the principles of ISO 14001. Results to date [Cusack 1999] show that the industry's Cooperative Agreement with the Government has been very successful and mirrors the success of the program at the industry level.

In the U.S., Vision 2030 [ACI 2001] establishes goals and describes the future for the U.S. concrete industry, concrete products, suppliers, and customers. It communicates the fact that the U.S. concrete industry is committed to being a model of sound energy use and environment protection; making concrete the preferred construction material based on life-cycle cost and performance; and to improving efficiency and productivity in all concrete manufacturing processes while maintaining high safety and health standards.

The world is moving forward towards sustainable development. Civil engineers have a social responsibility and a leading professional role in implementing sustainable development. Our profession is responsibility for the efficient creation of built assets which require the use of increasingly scarce resources.

Cement and Concrete in Sustainable Development

The technical and economical characteristics of any built asset are comparatively easy to quantify. However, an assessment from the ecological point of view is more difficult to carry out. To address the latter, the concept of life-cycle assessment (LCA) has been evolving and it is considered to be an appropriate tool in sustainable development evaluation. LCA is a method that systematically assesses the environmental effects of a product, process or activity holistically by analyzing its entire life cycle. This includes identifying and quantifying energy and materials used and waste released to the environment, assessing their environmental impact and evaluating opportunities for improvement. The benefits of LCA are summarized as followed [AS/NZS ISO 14040, 1998]:

- Identifying opportunities to improve the environmental aspects of products at various points in their cycles.
- Decision making in industry, governmental or non-governmental organizations (e.g., strategy planning, priority setting, product or process design or redesign).
- Selection of relevant indicators of environmental performance, including measurement techniques.
- Marketing (e.g., an environmental claim, ecolabelling scheme or environmental product declaration).

A number of references are listed for further information on the subject. Aspects relevant to civil engineers follow.

There are three approaches that are considered appropriate for civil engineers:

1. Optimum use of natural, industrial by-products and recycled materials.
2. Choice of cleaner production technologies.
3. The application of design principles with respect to life-cycle cost.

Optimum Use of Natural, Industrial By-Products and Recycled Materials

The balance in the economic and environmental cost of the production of cement and concreting materials from naturally won material sources is rapidly changing. The production of Portland cement has been identified as one of the processes with the highest greenhouse gas emission. While the cement industry has greatly improved its environmental performance [Cusack 1999], well-informed users can further reinforce these measures by correctly specify and using appropriate binders. There is also greater knowledge in the use of industrial by-products, such as fly ash, blast furnace slag and silica fume, and other mineral additives as part of a binder. The result is the possible use of a variety of blended cements in concrete with improved workability and enhanced durability [Sirivivatnanon et al. 2000, Cao et al. 1997]. The economy of the use of industrial by-products is highly dependent on transport cost and the correct usage to improve the service life of concrete structures in different aggressive environment [Khatri and Sirivivatnanon 2001]. The economic balance is likely to change with improved quantification of the environmental cost associated with their disposal and reduction of greenhouse gas emission of the Portland cement they replace. Industrial by-products are used in larger proportions in a range of special concrete such as roller-compacted concrete (RCC), self-compacting concrete (SCC) and high performance concrete such as HVFA and high slag blended cement concrete.

Concrete waste can be processed to produce roadbase/fill material, recycled concrete aggregate and recycled concrete fines. Recycled concrete aggregate (RCA) may result in higher absorption, water demand, shrinkage and creep, and lower density, durability, permeability and strength [Sagoe–Crentsil 1999]. Its use in structural elements is therefore limited. However, RCA concrete is readily suitable for footpaths, bike paths and low strength concrete (e.g., 15 MPa concrete for footing, blinding). The primary use of recycled concrete in Australia, for example, is for use as roadbase material, which not only reduces the need for natural fill, but is also commercially viable [Sautner 1999].

There are also economical and technical benefits in the use of industrial by-products in roads and embankment stabilisation [ADAA 1997, ASA 1993], as well as in asphalt and thin bituminous surfacing. Electric arc furnace slag [CSIRO 2001] can also be used as synthetic aggregates.

Cleaner and Greener Production Technologies

There is a range of cleaner and greener technologies that are readily applicable in the production of construction materials. However, because of the high capital investment associated with the current production industry, their introductions are more likely in new plants and in countries with an environmental protection policy in place.

Examples include the temperature reduction in cement kilns by having a better control of the use of some mineralizers [Taylor 1997] and the elimination of numerous pollutants or industrial waste [Uchikawa 1996]. Low-Energy Accelerated Processing (LEAP) technologies are being developed by CSIRO for rapid curing of precast concrete products. Compared to conventional heating practice, energy consumption per tonne of product processed can be significantly lower using industrial microwave heating technology. With no significant liquid or gas emissions at the site of use, properly designed industrial microwave heating technology can potentially provide a clean manufacturing solution for the precast concrete industry [Mak et al. 2001]. Low-energy vertical concrete pipe-making technology is also replacing traditional horizontal concrete pipe manufacturing technology. In many applications, the use of flowing concrete [Collepardi 2001] since the 1970s or SCC may prove to be economical in energy saving and noise-sensitive environment. A new internal curing admixture has also been developed for self-curing concrete [Marks et al. 2001].

Application of Design Principles With Respect to Life-Cycle Cost

With increasing emphasis on service life design and a greater knowledge in the use of mineral additives, there is a huge potential for greater use of industrial wastes as mineral additives to improve the durability of concrete structures. Thus, two positive aspects are simultaneously realised in sustainable development. There are many examples given of the applications of HPC. The choice will be quite clear if the design for durability principle and life-cycle analysis is applied in the preliminary stage of project design. The correct use of fly ash, slag or silica fume has generally been found to improve durability of concrete with respect to chloride-induced corrosion, sulphate attack and alkali–aggregate reactivity. Their use in high proportions may, however, result in an increased risk of carbonation-induce corrosion. It is important to note that the performance of these mineral additives tends to vary from source to source. Performance data of local materials should be examined. A great deal of research findings have been published in a number of international conferences such as the CANMET/ACI international conference series on durability of concrete and a series on fly ash, silica fume, slag and natural pozzolans in concrete; University of Dundee series of international congresses; and a series of three international seminars on blended cements — Singapore in 1992, Kuala Lumpur in 1994 and Singapore in 1998. These are given in the "Further Information" section.

It has been realized very recently that HPC is more ecologically friendly, in the present state of technology, than usual concrete because it is possible to support a given structural load with less cement and, in some cases, one-third of the amount of aggregates necessary to make a normal strength concrete [Aïtcin 2000]. Moreover, the life cycle of high-performance concrete can be estimated to be two or three times that of usual concrete. In addition, high-performance concrete can be recycled two or three times before being transformed into a roadbase aggregate when structures have reached the end of their life.

Applications

LCA has been applied widely to buildings rather than civil engineering structures. The Quebec Ministry of Transportation has calculated that the initial cost of a 50 to 60 MPa concrete bridge is 8% less than that of a 35 MPa concrete without taking into consideration the increase in the life of the bridge [Coulombe and Quellet 1994]. The Internationale Nederlanden (ING) Bank headquarters in Amsterdam, completed in 1987, uses only 10% of the energy of the bank's old building and has cut worker absenteeism by 15%. The combined savings are estimated at U.S.$2.6 million per year [Romm and Browning 1998]. The outcome of an LCA study of different building types with various forms of construction in Australia [Slattery and Guirguis 2001] has shown that for each building type, there was no significant difference between the different forms of construction studied in terms of energy and greenhouse gas emissions, but significant differences in ozone depletion and heavy metal over three life cycles of 50, 75 and 100 years. The operation was the most important phase of the life cycle for energy usage. It was thus recommended that the environmental assessment of a building should not be based on just one or two indicators (e.g., energy and greenhouse gas).

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