NISTIR 6295

Curing of High-Performance Concrete: Report of the State-of-the-Art

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March 1999

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U.S. Department of Commerce

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ABSTRACT

This report describes the latest information, technology, and research on the curing of high-performance concrete (HPC). The scope of the report is expanded somewhat to examine the current body of knowledge on the effects of various curing conditions on the development of the properties of concrete in general. The significance and importance of curing and various proposed definitions of high-performance concrete are discussed. Specific types of the most commonly used high-performance concrete are described, and their properties and characteristics are highlighted. The report summarizes some of the currently accepted concepts and theories of how curing alters the physico-chemical characteristics and structure of a cement paste, since many of these are applicable to the study of high-performance concrete. The landmark studies by Powers and Brownyard in the mid 1940s on the physical and chemical properties of hydrating cement paste are summarized. The history of the American Concrete Institute (ACI) building code requirements for curing are traced from the beginning of this century to the present time. Current curing requirements in the standards of various countries are reviewed and discussed. Some of the recent research in the United States and other countries, related either directly or indirectly to the curing of high-performance concrete, is summarized, including the important work of Hilsdorf in Germany. The report concludes with a discussion of the major areas of research needed to develop optimum curing criteria for this new class of concrete.

Keywords: Building technology; curing; durability; high-performance concrete; maturity; porosity; self-desiccation; silica fume; strength.

PREFACE

This state-of-the-art report is based on a recently completed literature search on the curing of high-performance concrete (HPC). A separate annotated bibliography (NIST GCR 97-715) is available from the National Institute of Standards and Technology (NIST). The scope of the report is expanded somewhat beyond considering just high-performance concrete to summarize the effects of different curing methods on concrete in general. With regard to high-performance concrete, the amount of information available on the effects of various curing conditions on the development of properties is limited. The curing of high-performance concrete has been identified as an area in which research is needed so that the full potential of this relatively new class of concrete can be realized. In many respects the complexities of curing are not completely understood, and hopefully this report will provide guidance on the key areas of research to establish the curing methods most suited to high-performance concrete

The first author was a graduate student in Civil Engineering at The George Washington University (GWU) pursuing a Ph.D. in Structural Engineering. His dissertation research focused on the curing of high-performance concrete and was conducted in cooperation with the National Institute of Standards and Technology. This report was prepared as partial fulfillment of his doctoral requirements. Dr. Shahram Sarkani, Associate Professor and Chairman of Civil and Environmental Engineering Curriculum at GWU, was Dr. Meeks' academic advisor, and his advice and assistance are greatly appreciated. Dr. Meeks is currently an Associate Professor of Civil and Environmental Engineering at Tri-State University, Angola, Indiana. The second author is Leader of the Structural Evaluation and Standards Group, Structures Division, Building and Fire Research Laboratory (BFRL) at NIST. The authors appreciate the assistance of Dale Bentz and Ken Snyder of the Building Materials Division at NIST for their suggestions and technical input into this report. Dr. H.S. Lew and the late Dr. James R. Clifton provided critical reviews of the report.

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1. INTRODUCTION

1.1 Background

It has long been recognized that adequate curing is essential to obtain the desired structural and durability properties of concrete. Proper curing of concrete is one of the most important requirements for optimum performance in any environment or application. Historically, curing has not received the attention it deserves. It was not until the early 1950s that any significant articles devoted to concrete curing practices were published by the American Concrete Institute (ACI) (Timms 1952, Robinson 1952, Gilkey 1952). A review of the history of the ACI building code (Chapter 4) reveals little change in curing requirements in the past 50 years.

With respect to high-performance concrete (HPC), the amount of information available on the effects of various curing conditions on its properties is limited, and the current curing requirements for ordinary concrete may not be optimal for high-performance concrete. Since the strength development and durability characteristics of high-performance concrete may be different from ordinary concrete, it follows that new curing practices may be needed. Since high-performance concrete is a relatively new class of concrete, additional research is needed to understand more fully the factors affecting the development of its physical and chemical characteristics. Thus one of the critical aspects for the successful usage of high-performance concrete will be knowing how to cure it to achieve its full potential. Eventually, design codes will need to be revised to incorporate the necessary requirements for the safe and efficient use of high-performance concrete.

1.2 Significance and Importance of Curing

Poor curing practices adversely affect the desirable properties of high-performance concrete, just as they do any concrete. Proper curing of concrete is essential to obtain maximum durability, especially if the concrete is exposed to severe conditions where the surface will be subjected to excessive wear, aggressive solutions, or severe environmental conditions (such as cyclic freezing and thawing). Likewise, proper curing is necessary to assure that design strengths are attained.

Even when good quality concrete is placed on the job site, curing is necessary to ensure the concrete provides good service over the life of the structure. Good concrete can be ruined by the lack of proper curing practices. Curing is even more important today than ever before for at least three reasons (Neville 1996):

- Today’s cements gain strength earlier and allow contractors to remove formwork soon after concrete placement. This encourages discontinuing curing operations prematurely.

- The lower water-cement ratios being used with modern concretes (like HPC) tend to cause self-desiccation (Chapter 3). Ingress of water from proper curing is necessary to control this phenomenon.
- Many modern concrete mixtures contain mineral admixtures, such as fly ash and ground granulated blast furnace slag, that have slower reaction rates. Curing over longer periods of time is needed for proper development of the properties of these mixtures.

Curing has a major impact on the permeability of a given concrete. The surface zone will be seriously weakened by increased permeability due to poor curing. The importance of adequate curing is very evident in its effect on the permeability of the "skin" (surface) of the concrete.

In the United States and other countries, contractors tend to either short cut curing requirements in the field or ignore them almost completely. One survey conducted in the United States in 1979 estimated that 24 % of concrete used in nonresidential construction was not cured at all, and only 26 % was cured in accordance with project specifications (Senbetta and Malchow 1987). It is doubtful that the situation has improved very much since then. The concrete industry must do a better job of educating contractors, engineers, superintendents, and quality control personnel of the importance of good curing practices in the field. This is especially true for high-performance concrete since it has been found to be even more sensitive to curing conditions than ordinary concrete, particularly at early ages. It has been suggested that one way to highlight the importance of curing is to make it a separately billed item in the schedule of prices for the project (Cather 1994).

Contract specifications usually contain curing requirements; however, they are rarely adhered to in the field (Neville 1996). Similar to the batching and mixing operation for concrete, curing needs to be closely supervised and controlled. As a construction project progresses, it is extremely difficult to prove whether proper curing has been applied. Although specifications may be adequate and complete, one of the biggest obstacles to ensuring proper curing in the field is the lack of standard methods to verify curing adequacy. Various penetrability methods have been proposed (Kropp and Hilsdorf 1995), but none has yet to be standardized for use. Without approved testing methods, it will continue to be difficult to verify desired levels of curing in the field.

The curing of high-performance concrete has been identified as one of the critical areas in which more information and research are needed in order to realize the full potential of this class of concrete (Carino and Clifton 1990). Current national curing specifications in the United States do not include specific requirements for high-performance concrete even though its use is becoming more widespread. Existing curing criteria are based on information from ordinary concrete, and may not be appropriate for the high-performance concrete mixtures being used today. Current standards are also deficient in that they do not address proper curing for durability. Historically, curing requirements have been based primarily on obtaining adequate strength. Some of the most recent research on high-performance concrete has focused on how curing affects the surface layer and thus, the durability of the concrete. Finally, current curing requirements in the United States do not take into account the actual rate of hydration or strength development, both of which may be affected by in-place temperature and whether chemical and mineral admixtures are used.

1.3 Objectives of Report

Curing in the field is perhaps the most critical factor in the concrete construction process. Sufficient curing is essential if the concrete is to perform its intended function over the life of the structure. The key word here is "sufficient," because contractors are sensitive to the time value of money borrowed to finance construction. Excessive curing time can add to the construction cost of a project and cause unnecessary delay. If high-performance concrete is to compete with other construction materials, standards must be defined for "sufficient curing" to achieve strength and durability criteria. It is likely that this will require new or revised curing practices to optimize the improved performance characteristics of this class of concrete.

This report describes the results of a literature search on the curing of high-performance concrete. It will also examine, in general terms, the current body of knowledge on the effects of various curing conditions on concrete. Some of the basic physical and chemical effects of curing on concrete mixtures are reviewed and summarized as the basis for further technological advancements that may be needed when considering high-performance concrete. Many of the currently accepted concepts and theories of how curing alters the physical characteristics of cement paste are applicable to high-performance concrete. Since curing is closely related to moisture migration through concrete, this important subject will be covered. The issue of different curing standards for different performance requirements is also addressed; for example, curing specifications should probably be different when considering durability criteria as opposed to simply considering adequate strength for a specific application of high-performance concrete.

Recently completed research is reviewed as well as current efforts underway to address the curing aspects of HPC. The contents of the remaining chapters of the report are summarized below:

Chapter 2 describes the major types of high-performance concrete in use today and highlights some of their important properties.

Chapter 3 summarizes many of the properties of cement paste related to curing that were presented in the series of nine articles by Powers and Brownyard written in the mid 1940s. It also summarizes some of the curing-related work of Neville from the 1970s, covers recent percolation theory, describes the impacts of self-desiccation and low penetrability on curing, and discusses carbonation.

Chapter 4 summarizes the history of the ACI building code requirements for curing from the beginning of this century to the present.

Chapter 5 surveys existing standards and criteria for the curing of concrete in the United States and in other countries.

Chapter 6 discusses recent research in the United States and in other countries that is related either directly or indirectly to the curing of concrete.

Chapter 7 includes concluding remarks concerning the curing of high-performance concrete and discusses critical research needs to develop curing standards for this new class of concrete.

It is hoped that this state-of-the-art report will provide the framework for additional studies dealing with the complexities of the curing processes applicable to high-performance concrete. Further experimental and analytical research is needed to determine optimum curing practices for all types of high-performance concrete.

1.4 Definitions of High-Performance Concrete

Several different definitions of high-performance concrete have been proposed. Currently there is no one definition that is universally accepted either within the United States or in other countries. Some of these definitions are summarized below:

1. *Strategic Highway Research Program (SHRP) definition* (Zia et al. 1991):
   a. High-performance concrete shall have one of the following strength characteristics:
      1. 28-day compressive strength greater than or equal to 70 MPa (10 000 psi), or
      2. 4-hour compressive strength greater than or equal to 20 MPa (3 000 psi), or
      3. 24-hour compressive strength greater than or equal to 35 MPa (5 000 psi)
   b. High-performance concrete shall have a durability factor greater than 80 % after 300 cycles of freezing and thawing.
   c. High-performance concrete shall have a water-cementitious materials ratio[1] less than or equal to 0.35.

The SHRP definition encompasses specific strength, durability, and mixture proportioning characteristics. It should be noted that this definition was developed primarily to address requirements for highway construction.

[1] Unless specified otherwise, water-cement ratio or water-cementitious materials ratio refers to the ratio of the mass of water to the mass of cement or cementitious materials.

2. *NIST/ACI Workshop definition* (Carino and Clifton 1990):
   High-performance concrete is concrete having desired properties and uniformity that cannot be obtained routinely using only traditional constituents and normal mixing, placing, and curing practices. As examples these properties may include:
   1. Ease of placement and compaction without segregation
   2. Enhanced long-term mechanical properties
   3. High early-age strength
   4. High toughness
   5. Volume stability
   6. Long life in severe environments

This is a more general definition that attempts to include a variety of concretes having special properties not attainable by ordinary concrete.

3. *University of Tokyo definition* (Carino and Clifton 1990):
   In this definition, high-performance concrete is characterized as a “forgiving concrete” that compensates for poor construction practices and structural detailing, and has the following features:
   1. Ability to fill forms with little or no external compactive effort
   2. Cohesive mixture with low segregation
   3. Minimum cracking at early ages due to shrinkage and thermal strains
   4. Sufficient long-term strength and low permeability

This definition is a reflection of the Japanese emphasis on constructability as well as strength and durability of concrete.

4. *Prestressed Concrete Institute definition* (PCI Committee on Durability 1994):
   High-performance concrete is concrete with or without silica fume having a water-cement ratio of 0.38 or less, compressive strength at or above 55.2 MPa (8 000 psi) and permeability 50 % lower (by AASHTO T-259 or T-277 methods) than that of conventional mixtures.

5. *Civil Engineering Research Foundation definition* (CERF Technical Report 1994):
   Unlike conventional concrete, high-performance concrete meets one or more of these requirements:
   1. Places and compacts easier
   2. Achieves high strengths at early ages
   3. Exhibits superior long-term mechanical properties such as strength, resistance to abrasion or impact loading, and low permeability
   4. Exhibits volume stability and thus deforms less or cracks less
   5. Lasts longer when subjected to chemical attack, freezing and thawing, or high temperatures
   6. Demonstrates enhanced durability

This definition is an outgrowth of the earlier NIST / ACI workshop definition.

1.5 Increasing Use of High-Performance concrete

The use of high-performance concrete in construction is expected to increase as we move into the next century in both the United States and other areas of the world. Estimated usage of HPC in the United States is currently no more than about 10 % of the total annual concrete production (CERF Technical Report 1994). Throughout the United States, it is expected to play a significant role in the massive effort that will be required to rebuild the nation’s aging infrastructure (Carino and Clifton 1990). Working with the American Concrete Institute, NIST has taken a leadership role in proposing a national program on HPC to develop the necessary design standards and criteria to ensure its safe use by the construction industry (Carino and Clifton 1990). The curing of HPC is one of the areas that must be further researched to ensure complete confidence in its safe, efficient, and economical usage in construction.

High-performance concrete is expected to result in more economical construction of major concrete projects, such as high-rise buildings. For example, high-strength concrete (HSC) can reduce significantly the size of primary structural members such as columns. With greatly increased strength and load carrying capacity, some structural members may also be safely designed with less reinforcing steel, saving on both labor and material costs. Greatly enhanced durability properties of high-performance concrete should make its use very attractive in environments where ordinary concrete would not suffice.

Life cycle cost analysis is being used more frequently than ever before by the Department of Defense and other government agencies as well as the private sector. Life-cycle costs will likely become equally important, if not more so, than first-time costs for major structures. Life cycle cost estimating will provide the opportunity to highlight the benefits of HPC compared with other structural materials (Carino and Clifton 1990).

1.6 Applications and Design Considerations

Designers and constructors of concrete structures have been somewhat hesitant in specifying and recommending high-performance concrete because of the absence of standards, codes, and other reliable information concerning its use. A good example is high-strength concrete. Since most of the properties of high-strength concrete have been determined from ideal curing conditions in the laboratory, more studies are needed on how actual curing practices in the field affect its strength development. More information is also necessary on environmental effects, on how various admixtures affect performance, and long-term durability. Silica fume is being widely used as a mineral admixture for high-strength concrete, and despite a considerable amount of research on its positive benefits, more specific knowledge will be required to ensure complete confidence in its use (ACI 363 1987).

Design codes presently in use do not provide criteria for the high strengths and durability commonly available with high-performance concrete. As an example, in Australia the structural design codes currently in use are only applicable for concrete strengths up to 50 MPa (7 300 psi) (Papworth and Ratcliffe 1994). High-strength concrete in the range of 70 MPa to 100 MPa (10 000 psi to 15 000 psi) is commercially available in many areas of the world today (CERF Technical Report 1994). The latest ACI building code for structural concrete (ACI 318-95) essentially limits the usable strength of high strength concrete to 70 MPa (10 000 psi) in computing shear strength and development length. This limit can only be exceeded if additional shear reinforcement is provided beyond that normally used in beams and joists.

There are many examples of successful projects around the world where high-stregnth concrete has been used (ACI 363 1992). It is finding frequent applications in a variety of construction projects—columns and core walls in high rise buildings, marine structures, offshore oil platforms, and long-span bridges (Mak and Torii 1995). As the body of knowledge of high-performance concrete increases and construction standards are modified to take full advantage of its potential, the level of confidence in its use will grow.

2. CHARACTERISTICS OF HIGH-PERFORMANCE CONCRETE

Just as there are numerous definitions of high-performance concrete, there are different types of high-performance concrete. Generally, the term high-performance concrete refers to concrete with a variety of enhanced properties and characteristics. In order to understand the significance that good curing plays in the use of this type of concrete, the various kinds of high-performance concrete and their properties should be understood. Therefore, this chapter summarizes the major categories of high-performance concrete and some of their important properties. As will be seen, the majority of the descriptions of high-performance concrete are based on compressive strength, and they, therefore, use the term "high-strength concrete." However, a unifying feature is that most types of high-performance concretes have low water-cementitious materials ratios.

2.1 Strategic Highway Research Program Types of High-Performance Concrete

In a report prepared for the Strategic Highway Research Program, the following types of high-performance concrete are defined for highway applications (Zia et al. 1991):

Very early strength (VES)—Concrete with a compressive strength of at least 21 MPa (3 000 psi) within 4 hours after placement. Curing beyond the 4-hour period is not expected; however, additional curing is recognized as being beneficial. This concrete is intended mainly for making repairs that require a minimum time of traffic shut down.

High early strength (HES)—Concrete with a compressive strength of at least 34 MPa (5 000 psi) within 24 hours of placement. When used in highway construction, HES concrete would be placed by machine and receive little or no curing beyond the 24 hours.

High strength (HS)—Concrete with a compressive strength of at least 42 MPa (6 000 psi) at 28 d.

Very high strength (VHS)—Concrete with a compressive strength of at least 69 MPa (10 000 psi) at 28 d. VHS concrete would find applications where structural capacity is a primary consideration. Extended curing would be used with this material to ensure optimum results.

Fiber-reinforced concrete (FRC)—Concrete with sufficient fiber reinforcement to provide a ductility or toughness equal to at least five times the area under the stress-strain curve for the same concrete mixture without fiber reinforcement. Fiber-reinforced concrete is normally associated with toughness, i.e., the ability to absorb energy. This energy absorption occurs primarily after the ultimate strength of the concrete has been attained.

High-durability concrete—Concrete with a minimum durability factor (freezing and thawing) of 80 %, as measured by AASHTO T 161 (Method A) (or ASTM C 666), and a water-cement (w / c) ratio of 0.35 or less. A maximum w / c of 0.35 will provide a paste with a discontinuous capillary system after a relatively short curing period (normally about a day). This provides improved resistance to moisture penetration and chemical attack from the environment.

High-strength lightweight concrete—Concrete produced by using lightweight aggregates, such as expanded clay, shale, and slate aggregates, so as to reduce the mass 20 % to 25 % below that of conventional concrete. Some lightweight concrete can attain compressive strengths greater than 69 MPa (10 000 psi). This type of high-performance concrete is desirable in applications where reduction of dead load is a significant consideration.

2.2 French Definitions of High-Strength Concrete

Researchers in France (de Larrard and Bostvironnois 1991) distinguish high-strength concrete from very high-strength concrete as follows:

High-strength concrete—Concrete containing chemical admixtures, primarily for water reduction, and having a compressive strength between 50 MPa and 80 MPa (7 250 psi and 11 600 psi).

Very high-strength concrete—Concrete containing, in addition to chemical admixtures as found in HSC, a finely graded pozzolanic material, such as silica fume, and having a compressive strength in excess of 80 MPa (11 600 psi).

2.3 ACI Definition of High-Strength Concrete

Committee 363, on high-strength concrete, of the American Concrete Institute defines high-strength concrete as follows (ACI 363 1992):

> *"Concretes that have specified compressive strengths for design of 41 MPa (6 000 psi) or greater, and excluding concrete made using exotic materials or techniques. The word 'exotic' refers to materials such as polymer-impregnated concrete, epoxy concrete, and concrete with artificial normal and heavyweight aggregates."*

2.4 FIP-CEB Definition of High-Strength Concrete

The Federation Internationale de la Précontrainte-Comité Euro-International du Béton (FIP-CEB) defined high-strength concrete as follows (FIP-CEB Working Group 1990):

Concretes with a cylinder compressive strength above the present existing limits in national codes, i.e., about 60 MPa (8 700 psi), and up to 130 MPa (18 800 psi), the practical upper limit for concretes with ordinary aggregates.

2.5 Mineral Admixtures

2.5.1 *Silica fume*—Silica fume (SF) is probably the most common addition to concrete mixtures to produce high-performance concrete. This material, also called *condensed silica fume* or *microsilica*, is a finely-powdered amorphous silica that is highly pozzolanic (develops cementing properties in the presence of water and calcium hydroxide). Its use is becoming so common around the world that many consider high-performance concrete to be synonymous with silica fume-concrete.

Silica fume is a by-product from electric arc furnaces used in the manufacture of elemental silicon or ferro-silicon alloys. Silica fume contains large amounts of silicon dioxide (between 85 % and 98 %) and consists of extremely fine particles. It is collected by filtering the escaping furnace gases[2]. The average size of these spherical particles is less than 0.1 µm which is approximately one hundred times finer than cement. The small particle sizes lead to important benefits when SF is used in concrete. For example, the extremely fine particles can fill spaces between cement particles, which results in a more refined microstructure and a more dense cement paste. As the pores within the paste become finer and more dispersed, the permeability is reduced considerably.

The beneficial effects of silica fume result from its highly reactive pozzolanic characteristics. The high reactivity can be attributed to its very high silicon dioxide content, small particle size, and large specific surface area. Because of the very high specific surface area, the use of silica fume leads to increased water demand. For this reason, it is necessary to use high-range water-reducing admixtures (superplasticizers) with silica fume to produce workable concrete.

Since the early 1980s, silica fume has been used to improve concrete properties such as compressive strength, abrasion resistance, and durability. It is frequently used as a replacement for a portion of the portland cement in a mixture. Silica fume-concrete has become quite common throughout the United States when high compressive strength is required. Other performance improvements from silica fume replacement (about 10 % by mass appears to be optimum) include higher resistances to sulfate attack, alkali-aggregate reaction, and freezing and thawing (Hooton 1993).

Studies have shown (Gjorv 1991) that silica fume can be very effective in producing highly impermeable concretes for use in harsh environments. In fact, the benefits of silica

[2] Aitcin, P. C., 1983, "Condensed Silica Fume," Brochure, 52 pp.

fume on reducing permeability of concrete may be more important than its benefits in improving strength.

One of the concerns that must be addressed with the use of silica fume is the high potential for plastic shrinkage cracking. This is the most common complaint from users of SF (Holland 1989). Silica fume-concretes normally have low water-cement ratios and experience little bleeding. The surface of silica fume-concrete tends to dry quickly, subsequently causing shrinkage and cracking prior to final setting. This is one reason why early-age moist curing of silica fume-concrete is important. Immediately after placement, steps must be taken to prevent drying of the surface (Ozyildirim 1991).

2.5.2 *Fly ash*—Fly ash is the finely divided residue resulting from the combustion of ground or powdered coal and is transported by flue gases (ASTM C 125). This waste by-product is used extensively in the production of high-performance concrete. Due to its outstanding pozzolanic and cementitious properties, fly ash is used to improve durability and enhance strength gain. As concrete containing fly ash is cured, the products of the pozzolanic reaction fill in the spaces around cement particles. This results in a paste of lower permeability and greater resistance to chemical attack. Because the pozzolanic reaction is slower than the hydration of portland cement, fly ash is often used to control the amount of early heat generation and the detrimental effects of early temperature rise commonly experienced in massive concrete structures.

2.5.3 *Ground granulated blast-furnace slag*—This is a glassy material that is formed when blast-furnace slag is rapidly cooled, such as by immersion in water (ACI C 125). It is composed essentially of silicates and aluminosilicates of calcium. When it is ground to cement fineness, it is referred to as ground granulated blast-furnace slag (GGBFS), and it is commonly used in HPC mixtures. The use of GGBFS reduces the permeability of the mature concrete. It is believed that this improvement is a result of the reaction of the GGBFS with the calcium hydroxide and alkalis released during hydration of the portland cement (ACI 233.1R). The reaction products fill pore spaces in the paste and result in a denser microstructure. In addition to reducing the permeability of concrete, GGBFS also improves resistance to sulfate attack because of the low calcium hydroxide content (Neville 1996). Like fly ash, GGBFS is also used to reduce temperature rise in mass concrete. Ground granulated blast-furnace slag also improves the workability of fresh concrete. It is believed that the smooth, dense surfaces of the slag particles result in very little water absorption during the mixing process. Also, cement pastes containing GGBFS have exhibited better particle dispersion and higher fluidity, leading to improved workability (ACI 233.1R).

2.6 Chemical Admixtures

Silica fume is not the only additive commonly used in HPC; high-range water-reducers (HRWRs), or superplasticizers, are also commonly used. The emergence of HRWRs has facilitated the production of very low water-cement ratio concretes, and good workability can be maintained even at water-cement ratios below 0.40 (Aitcin 1994).

The term “high range” means that the admixture permits at least a 12 % reduction in the water content of a concrete mixture while maintaining the same slump. Two ASTM specifications describe the requirements for high-range water-reducing admixtures. ASTM C 494 covers two types of HRWR admixtures to produce conventional slump concretes. Type F is used for concretes when normal setting times are desired, and Type G is used for concretes when retarded setting times are desired. ASTM C 1017 covers two types of high range water-reducers for producing “flowing concrete,” i.e., concretes with slump greater than 190 mm (7 ½ in). Type I is for concretes with normal setting times and Type II is for concretes with retarded setting times.

2.7 Self-desiccation

One of the potentially detrimental side effects from the use of the low water-cement ratio concretes is self-desiccation, Self-desiccation refers to the process by which concrete dries itself from the inside. Internal moisture is consumed from within the paste by the hydration reactions, and the internal relative humidity continues to decrease to the point at which there is not enough water to sustain the hydration process. The result is that the hydration and maturity of the concrete will terminate at an early age if additional moisture is not provided. Therefore, self-desiccation effects are important considerations in the performance of high-performance concrete, particularly in the curing practices that involve “sealing” the concrete.

Self-desiccation may be especially harmful to the durability properties of high-performance concrete since the microstructure of the paste is adversely affected. With inadequate hydration, the near-surface regions become more susceptible to the penetration of deleterious materials from the surrounding environment. For this reason, proper curing of low water-cement ratio concrete at early ages is essential if the concrete is to attain its potential properties. The detrimental effects of self-desiccation can be largely controlled by careful attention to curing, especially during the initial 7 d after placement. Acceptable curing practices will be discussed in greater detail later in this report.

2.8 Workability

Another concern with the use of high-performance concrete, particularly mixtures with silica fume, is to be able to achieve the workability necessary for ease of placement. Large cement paste volumes are required to achieve workable concretes having low water-cement ratios. In cement-rich mixtures, common to high-strength concrete and other types of high-performance concrete, reduced workability and “stickiness” (flocculation of cement particles) can lead to placement and finishing problems (ACI 363 1992). The use of high range water-reducing admixtures has been required with these types of high-performance concretes to provide the fluidity desired (Holland 1989; Domone and Soutsos 1994). Recent tests on concretes with various amounts of silica fume (Sabir 1995) have confirmed that high range water-reducing admixtures are required to achieve acceptable levels of workability.

The lack of an adequate test for the workability of low water-cement ratio concretes is a major problem. While the slump test has proven useful for normal concrete mixtures, it does not characterize the rheological behavior of concrete under high strain rates as would occur during consolidation by vibration.

2.9 Creep and Shrinkage

The amount of information available on the creep and shrinkage properties of low water-cement ratio concretes is limited. Tests on high-strength concrete have shown that modest improvements in creep and shrinkage properties can be obtained by lowering the paste content and maximizing the size of the coarse aggregate. Concretes with relatively large-sized aggregate of 38 mm (1 ½ in) and a low paste content have been shown to exhibit the least amount of creep and shrinkage (Collins 1989). It was found that shrinkage strains are inversely proportional to the moist curing time—the longer the curing time, the lower the amount of shrinkage. Creep testing did not investigate the effect of different curing conditions. There were no significant effects on shrinkage and creep from the use of high range water-reducing admixtures. Creep has the beneficial effect of reducing internal stresses caused by restrained shrinkage, and thus helps to mitigate cracking (Neville 1996). Creep can be considered a balancing influence on the harmful effects of shrinkage cracking by relieving the internal stresses that arise when shrinkage is restrained.

With respect to silica fume-concrete, replacing cement with silica fume does not increase long term shrinkage; however, adding silica fume without reducing the corresponding volume of cement tends to increase long term shrinkage (PCI Committee on Durability 1994). The reason is that the higher paste content results in more shrinkage deformation, which is consistent with the test results described above.

2.10 Modulus of Elasticity

Designers can take advantage of the higher modulus of elasticity associated with low water-cement ratio concretes to control concrete structural deflections. Current ACI code provisions to estimate the modulus of elasticity are based on the density and compressive strength of the concrete. Although more research is needed on predicting the modulus of elasticity of HPC, investigations already completed verify that the currently used empirical relationships developed for ordinary concrete are not always appropriate for HPC (Sabir 1995). There is a need to develop more reliable methods to estimate the elastic modulus based on the applicable properties and proportions of the materials used in the concrete

(Carino and Clifton 1990). As these improved methods for accurately estimating the elastic modulus are perfected and accepted, designers will be able to take advantage of the higher stiffness of high-performance concrete with more confidence.

3. PHYSICAL AND CHEMICAL PROPERTIES OF CEMENT PASTE RELATED TO CURING REQUIREMENTS

It is important to understand the basic physical and chemical effects of curing on concrete mixtures. A fundamental understanding of these effects and how the curing process influences them will be the foundation upon which further advancements can be made in the curing requirements for high-performance concrete. Since the properties of concrete develop as result of hydration, much can be learned by studying the effects of different curing conditions on the characteristics of cement paste. This chapter summarizes many of the characteristics of cement paste related to curing from the works of T. C. Powers and T.L. Brownyard from the 1940s and the more recent contributions of A.M. Neville. Other relevant subjects addressed include percolation theory, self-desiccation, and carbonation.

3.1 Contributions by Powers and Brownyard

Perhaps the most well-known series of papers in the literature on the detailed physical and chemical properties of cement paste are the nine articles by Powers and Brownyard published in the mid 1940s (Powers and Brownyard, Parts 1-9, 1946-1947). These classical papers reported on studies dealing with, among other topics, water fixation in hardened paste, evaporable water, density of the solid material, and porosity (Powers and Brownyard, Synopsis, 1946). Many of the properties of the paste that are related to curing are covered in these studies, such as absorptivity, permeability, moisture diffusion, and capillary action. This section summarizes the significance of these and other properties that are described in these nine articles, as related both directly and indirectly to curing of cement paste. It should be kept in mind that the conclusions of Powers and Brownyard are based on the experimental tools, knowledge, and materials available 50 years ago. Nevertheless, they provide important basic concepts about the nature of hardened cement paste.

3.1.1 *Properties of cement paste*—Hydration products of portland cement have been determined to be primarily colloidal[3]. The paste is composed predominantly of "submicroscopic" particles that appear crystalline in nature when viewed with an electron microscope. Colloidal particles are generally about 1 nm to 100 nm in diameter and form a rigid network with water adsorbed on the surface of the particles. Powers and Brownyard describe the gel formed by the hydration reactions as a "coherent mass of colloidal material." Frequently referred to as the *cement gel*, this colloidal material is the most important constituent of hardened cement paste.

[3] A colloid refers to a suspension of finely divided solids in a liquid. Colloids range from *sols*, which have the consistency of a fluid, to *gels*, which are semi-solids like gelatin.

Some of the important characteristics of the hardened paste are its chemical composition, the physical state of the solid phase (hydration products), and the affinity these solids have for water. Pore structure also has a very important relation to the performance of the paste. The moisture content of the paste, which depends on curing, is a primary factor in some of its most important properties, such as strength and volume changes.

Hardened cement paste can have two general types of pores, *capillary* and *gel pores*. Capillary pores are the spaces between the masses of cement gel formed during hydration of cement grains, and they make up what is called the "capillary system." Depending on the degree of hydration and the initial separation of the cement grains, capillary pores may be interconnected (percolated). The gel pores are spaces between the solid products of hydration within the cement gel. Gel pores are normally filled with water that is strongly held to the solids. Capillary and gel pores will be filled with water if the paste is saturated. When the paste is exposed to drying conditions, these pores empty, as the evaporable water is lost.

Powers and Brownyard defined *evaporable* and *non-evaporable* water in terms of an initially saturated, hardened paste according to its volatility. Evaporable water was defined as the portion of the water that escaped from a sample of saturated cement paste after it had been dried at 23 °C (73.4 °F) to constant mass in a vacuum desiccator containing a strong drying agent (magnesium perchlorate). The remaining non-evaporable water is essentially the chemically combined water plus some portion of the water in the gel pores, whereas the evaporable water comes from the capillary pores and some of the gel pores. These types of water will be discussed further in the section to follow on water fixation.

An increase in compressive strength and reduction in permeability of a given concrete is directly related to an increase in the extent of hydration of the cement paste. When cement and water are mixed, the cement particles are separated by water-filled space. As hydration progresses, the solid products of hydration are deposited within the initially water-filled space. There is thus an increase in the *gel-space* ratio. Strength gain was found to be proportional to V_m/w_o , where V_m is defined as the "quantity of adsorbate required for a complete condensed layer on the solid, the layer being 1 molecule thick," and w_o is the original water content of the mixture. The quantity V_m is an indicator of the surface area of the cement gel, and is, therefore, also an indicator of the amount of cement gel that has formed.

Although the degree of hydration of the cement paste is a primary factor in the strength of concrete, other characteristics will also affect the strength, such as air content and quality of aggregate. The higher the air content, the lower is the strength of the concrete, all other characteristics being equal. The type of cement affects the rate strength development. For example, cements that contain high amounts of tricalcium aluminate (C_3A) and tricalcium silicate (C_3S) will hydrate and gain early strength at a faster rate than cements with lesser amounts of these compounds.

3.1.2 *Water fixation*—Water movement into and out of the paste has significant effects. As water leaves, the paste shrinks; and as water enters, the paste swells. The forces of adsorption on the surfaces of the solid hydration products provide the driving force for water movement into paste and provide the resistance to drying out (desorption). The amount of water that is free to move in and out of the paste as the surrounding environment changes is strongly related to the capillary porosity of the hardened paste. The capillary porosity is, in turn, affected by the original water-cement ratio and the degree of hydration.

The total amount of water in a saturated, hardened cement paste can be categorized as being composed of three different kinds of water:

- Water of constitution is the water that is bound chemically in the hardened paste and is an integral part of the solid hydration products (chemically bound water).
- Water contained in the gel pores of the paste that is bound physically by the surface forces of adsorption (gel water).
- Water that is held within the capillary pores of the paste (capillary water).

As mentioned previously, the water in a saturated cement paste can be characterized as either evaporable or non-evaporable. However, these are arbitrary distinctions because whether a given kind of water will evaporate depends on the specific drying conditions. Powers and Brownyard made a distinction on the basis of drying a paste to constant mass at 23 °C (73.4 °F) in a vacuum desiccator. Alternatively, evaporable water can be defined as the water that is removed upon drying to constant mass at 105 °C (221 °F) at atmospheric pressure (Neville 1996). The key point is that the three types of water are held within the paste with forces of different magnitudes. The water of constitution is held most strongly through chemical bonds. The gel water is held less strongly by secondary van der Waals' forces. Capillary water is beyond the range of influence of the surface forces of the solid phase; therefore, in a saturated paste, it is under no stress and evaporates readily. However, if the paste is partially saturated, the capillary water will be held more strongly by tensile stresses related to the curvature of the air-water interface (related to the size of the capillary) and the surface tension of the water. When drying conditions prevail, the most loosely bound water evaporates first, followed by the more tightly bound water. Under ordinary drying conditions, some of the adsorbed gel water is lost along with water from the smaller capillaries. As the paste is cured for longer periods, the amounts of chemically-bound water and gel-pore water increase while the amount of capillary water decreases.

3.1.3 *Adsorption and evaporable water*—Some of the key aspects of adsorption and evaporable water in a cement paste have already been described above; however, they deserve additional discussion. In a cement paste, when drying occurs, the evaporable water leaves the pore spaces. This results in shrinkage of the paste, some of which may be irreversible.

In concrete with a low water-cement ratio, extended curing may eventually eliminate all the water in the capillaries. The initially water-filled capillaries may be replaced substantially with hydration products, and further hydration is not possible.

Adsorbed gel water plays a major role in the volume changes (shrinkage and swelling) of cement paste. Shrinkage is the result of water withdrawal from the gel during drying. Swelling is the result of water being drawn into the gel by the surface adsorption forces of the solid phase.

When the capillary pores in the paste become partially emptied due to the consumption of water during hydration, self-desiccation may occur (self-desiccation is an important consideration in the curing of high-performance concrete and will be discussed in more detail later in this chapter). Studies have shown that, for low water-cement ratios, even specimens cured properly in moist surroundings are not able to adsorb enough water to avoid some level of self-desiccation. This is why mature (high degree of hydration) cement pastes of high-performance concrete will not, normally, be saturated.

The amount of evaporable water (capillary plus gel water) in a saturated paste will be less for lower water-cement ratio mixtures, and also will be lower as the curing period is lengthened. However, as reported by Powers and Brownyard, tests have shown that the quantity of evaporable water in a saturated paste will not be lower than about $4V_m$, which corresponds to the condition of only gel pores and no capillary pores. (Recall that V_m is related to the surface area of the gel and represents the volume to form a mono-molecular layer of water on the surfaces of the solid particles of the cement gel.) Thus it appears that in saturated paste, the solid surfaces in the cement gel are coated with a layer of water approximately four molecules thick. This amount of adsorbed water per unit volume of cement gel is independent of the water-cement ratio or degree of hydration. If a saturated paste has an evaporable water content in excess of $4V_m$, that excess water is contained in the interstitial space comprising the capillaries in the paste.

When water reacts with cement, the hydration reactions result in new solid phases and the evolution of heat. This heat is sometimes referred to as the *heat of combination*, and the amount of evolved heat is directly proportional to the non-evaporable (chemically bound) water content. As these new solids are formed within the paste, they adsorb water with the resultant release of additional heat, called *heat of adsorption*. The sum of the *heat of combination* and the *heat of adsorption* will be the *total heat of hydration* in the paste. Normally, the *heat of combination* will be about 75 % of the total heat of hydration.

The gel resulting from the hydration of portland cement and subsequent adsorption of water belongs to the “limited-swelling” classification, which means it will only take up a limited amount of water with a corresponding limited amount of swelling. Likewise, it will shrink when evaporable water is lost during drying. Some of the shrinkage that occurs during initial drying is irreversible, that is, the paste will experience some permanent shrinkage deformation upon rewetting. This permanent shrinkage results from plastic flow of the solid phase of the paste due to the stresses induced by the effects of shrinkage. This plastic deformation causes a permanent change in the structure of the cement paste.

It is important to realize that in applications where concrete is exposed to frequently changing environmental conditions, the evaporable water will be in a state of flux. Therefore, the concrete will be subjected to continuous moisture movements, and the accompanying volume changes, throughout its life.

3.1.4 *Hydration*—The significance of the degree of hydration in the proper curing of any type of concrete, particularly high-performance concrete, cannot be over emphasized. Hydration produces the solid binding material that gives concrete its most basic engineering properties—strength and durability. The rate of hydration is greatest immediately after final setting and decreases gradually with age. Under certain conditions, hydration can continue for years. The key feature for continued hydration is the presence of water-filled capillaries. When water-filled capillaries cease to exist, hydration stops and the mechanical properties cease to develop.

Self-desiccation can influence the maximum degree of hydration that is achievable in a given mixture. If there is not a sufficient supply of water, it will not be possible to attain the potential maximum degree of hydration. Proper curing practices may help to offset the tendency toward self-desiccation, which can be a problem in low water-cement ratio concretes. However, for a low water-cement ratio concrete, the moisture transport properties of the outer layer become very low with continued hydration, effectively preventing the passage of an external supply of moisture to the self-desiccating interior region. Thus it is not clear whether self-desiccation in low water-cement ratio concrete can be prevented.

Tests by Powers and Brownyard (February 1947) showed that water-cured pastes with water-cement ratios less than about 0.4 will not hydrate completely. The reason is that, in low water-cement ratio pastes, there is less capillary space available for the deposition of hydration products. This means that for concretes with a water-cement ratio less than 0.40, the ultimate degree hydration can be expected to be limited to less than 100 %. Therefore, for high-performance concretes with low water-cement ratios, the relative strength development with age will not have the same characteristics as for ordinary concretes.

3.1.5 *Permeability*—Permeability refers to flow of a fluid through a material under the action of a pressure gradient. In hardened cement paste, permeability is related to the connectivity of capillary pores. Powers and Brownyard (March 1947) noted that for a properly cured, low water-cement ratio paste, the permeability can be expected to be as low as that of granite. Thus such pastes can be considered, for all practical purposes, to be impermeable. The aggregate in concrete can help to reduce permeability. The presence of non-porous aggregate reduces the number of possible flow channels in a given cross section and also lengthens the flow paths through which the fluid must move in a given direction. In properly designed concrete with a well graded aggregate, the permeability will be reduced as the maximum size of the aggregate increases because less paste volume is necessary. There are, however, negative effects due to the presence of the aggregate. Porous zones can be formed under aggregate particles due to the accumulation of bleed water. These zones of high porosity are obviously detrimental to the impermeability of the concrete. In addition, the interfacial region (referred to as the *transition zone*) between paste and aggregate will have a higher porosity due to less than optimum packing of hydration products at the aggregate boundaries.

Bleeding also has a detrimental effect on the permeability of the paste. As bleed water rises, cement grains are brought to the surface and channels are created in the paste. These channels provide little hindrance to the flow of fluids. In a similar manner, connected capillary pores provide a convenient flow path for the fluid to bypass the relatively impermeable gel pores.

3.1.6 *Absorption*—Absorption is the tendency of the cement paste to take on water in the absence of any external hydraulic pressure. This is believed to occur almost completely in the capillary pores outside the gel. The liquid water is pulled into these pores by surface tension, just as water will rise in a glass capillary. Absorption is the main mechanism whereby dry or partially dry cement paste takes on water. If the water contains deleterious chemicals, these will also penetrate into the paste. *Sorptivity* is the material property used to quantify the resistance to absorption.

3.1.7 *Diffusion*—Diffusion is another mechanism by which moisture can move through cement paste. Moisture diffusion is the result of inequalities in free energy due to moisture gradients (Powers and Bownyard, April 1947). Changes in the moisture content of a portion of the paste will change the free energy of the water in that region, resulting in diffusion from regions of high moisture to regions of low moisture. Evaporable water redistributes itself throughout the cement paste by this primary mechanism.

3.1.8 *Freezing of evaporable water*—When a saturated hardened cement paste is subjected to a temperature below freezing, not all of the evaporable water will freeze. Due to the dissolved salts in the capillary water, the evaporable water will not begin to freeze at a temperature of 0 °C (32 °F), but at a somewhat lower temperature in the range from -0.05 °C to -1.6 °C (31.9 °F to 29.1 °F). The freezing temperature is essentially a function of the alkali content of the cement, that is, the more alkali present, the lower the initial freezing point temperature. As freezing begins, the capillary solution becomes more concentrated, which lowers the temperature for additional freezing to occur. If the same paste is not saturated, the initial freezing point temperature is reduced further.

When ice forms in cement paste, it is the result of freezing of the water in the capillary pores. This occurs outside the gel so that gel water is not involved initially, although at very low temperatures gel water also contributes to ice formation (Powers and Brownyard, April 1947). At temperatures above -12 °C (10.4 °F), all the frozen water in a paste comes from the capillary water. The gel water that eventually freezes below this temperature probably flows into the capillary pores prior to turning to ice; therefore, all the frozen water in a paste is thought to reside in the capillaries. It is possible to freeze all the evaporable water, but this requires reducing the temperature to about -78 °C (-108 °F).

3.2 Contributions by Neville

In more recent times, the book by Neville, describing the properties of concrete, includes an excellent treatise on the physical and chemical characteristics of cement paste (Neville 1996). Neville makes use of the basic knowledge from the earlier research by Powers, Brownyard, and others, while expounding on its significance in light of more recent technology. This section summarizes some of Neville's discussions that relate to the significance of proper curing of concrete. The summary will by necessity repeat and reinforce ideas presented in previous sections.

3.2.1 *The effects of hydration*—Freshly mixed cement paste is considered to be in a plastic state and is comprised of a dispersion of cement grains surrounded by water. As hydration begins to occur, cement gel is one of the hydration products deposited in the water-filled space between cement grains. As hydration proceeds, two types of pores are formed: capillary pores and gel pores. As mentioned in the previous sections, the capillary pores are cavities or voids remaining between the hydrating cement grains. Within the mass of cement gel there are interstitial spaces which are the gel pores. Gel pores have nominal diameters of about 3 nm, and capillary pores are one to two orders of magnitude larger. The other major product of cement hydration is calcium hydroxide, which is deposited as relatively large crystals outside of the cement gel.

Hydration increases the amount of solid phase of the paste as water is consumed by chemical reactions. In addition, some of the water is adsorbed onto the surfaces of the solids in the cement gel. If the supply of water is insufficient to keep these surfaces saturated, the relative humidity in the paste will decrease (self-desiccation). When the relative humidity drops below 80 %, the hydration rate slows down and it becomes negligible when the internal relative humidity drops to 30 %. Self-desiccation will not occur if the initial water-cement ratio of the paste is sufficiently high and the mix water is prevented from evaporating (sealed condition). The next section discusses the minimum water-cement ratio to prevent self-desiccation under sealed conditions. If the water-cement ratio is less than the minimum value, moist curing (supply of external moisture) is required to control self-desiccation.

3.2.2 *Volumetric compositions* of *paste*—The changes that take place in the paste as hydration progresses can be understood by analyzing the volumetric make-up of a unit volume of paste. Consider Fig. 3.1, which depicts the volumetric proportions of a paste with a water-cement ratio of about 0.48 at three stages—prior to hydration, at 50 % hydration, and at 100 % hydration. Assume that the paste has initial volumes of 60 ml of water and 40 ml of cement, and it is in a sealed condition (no outside moisture is involved). Bleeding is

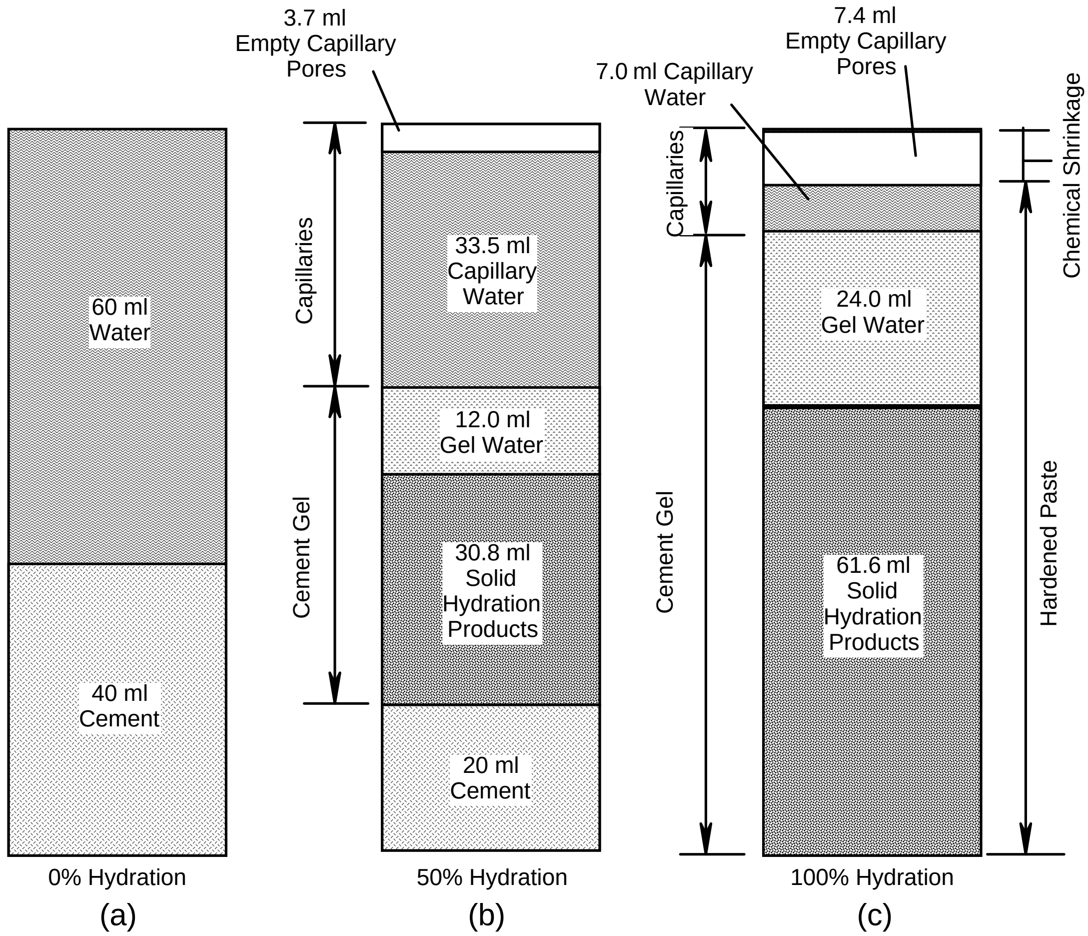

Figure 3.1 Schematic representation of the volumetric proportions of sealed cement paste (w/c = 0.475) at different stages of hydration (adapted from Neville 1996)

not considered. Figure 3.1(a) represents this initial state, and Fig. 3.1(c) represents the volumetric composition when the paste has fully hydrated. The 40 ml of cement produces 61.6 ml of solid hydration products, which include 21.6 ml of chemically combined water. These solid products represent the solid constituent of the cement gel. Within the cement gel there are 24.0 ml of pores that are filled with adsorbed gel water. Of the original 60 ml of mix water, 7 ml remains in the capillary pores. The volume of cement gel plus water in the capillaries equals 92.6 ml, which is 7.4 ml less than the original volume of 100 ml. Thus 7.4 ml of the capillary pores are empty. Figure 3.1(b) shows the composition when 50 % of the cement has hydrated; the volume of cement gel is exactly 50 % of the volume for the fully hydrated condition.

The above volumetric relationships were derived using the following assumptions based on previous experimental findings (Neville 1996):

- The specific gravity of the unhydrated cement equals 3.15.

- The chemically combined water equals 23 % of the mass of hydrated cement.
- The porosity of the cement gel is equal to 0.28 (porosity equals volume of gel water divided by the volume of cement gel).
- The volume of the solid reaction products equals the volume of hydrated cement plus 74.6 % of the volume of chemically combined water.
- The gel water cannot migrate into capillary pores.

The following observations can be made about the hydration of this particular paste under sealed conditions:

- At complete hydration, the hardened cement paste is made up of the solid particles, gel water, capillary water, and capillary voids (pores).
- The capillary water is consumed in the formation of the solid hydration products and filling the gel pores.
- There is a reduction in the volume occupied by hardened paste compared with the original volume of the freshly mixed paste. This volume reduction is termed *chemical shrinkage* and results in partially empty capillary pores under sealed conditions.

Using the above assumptions, as presented in Neville (1996), it is possible to predict the volumetric proportions of fully hydrated cement pastes with different water-cement ratios and for two curing conditions: sealed and submerged under water. First, consider the sealed condition. If the water-cement ratio is less than a critical value, there will be insufficient water (self-desiccation) to hydrate all the cement, and the fully hardened paste will consist of cement gel, empty capillaries, and unhydrated cement. If the water-cement ratio is greater than this critical value, all of the cement can hydrate, and the capillary voids will be partially filled with water. This was the case shown in Fig. 3.1. By using the above assumptions and the condition that there is just enough water to chemically react with all the cement and fill the gel pores, it can be shown that this critical water-cement ratio is 0.42.

Figure 3.2 shows the predicted volumetric proportions for fully hydrated[4] pastes of different water-cement ratios cured under sealed conditions. The curve with the solid circles represents the original volume of unhydrated cement. The distance between the curve with the diamonds and the abscissa represents the volume of cement gel after hydration. For low water-cement ratios, the volume of cement gel is the distance to the curve with the open circles, which represents volume of unhydrated cement remaining after all the water is consumed by hydration. In reality, hydration ceases when the relative vapor pressure (relative humidity) within the capillaries drops below a critical value. Hence the actual critical value of the water-cement ratio above which complete hydration can occur is greater than 0.42, and Neville suggests that a more realistic value is 0.50.

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[4] In this case "fully hydrated" refers to the condition where all the cement that is capable of hydrating has done so. For low water-cement ratios this means that there may be unhydrated cement remaining that cannot hydrate because there is insufficient volume available for additional hydration products.

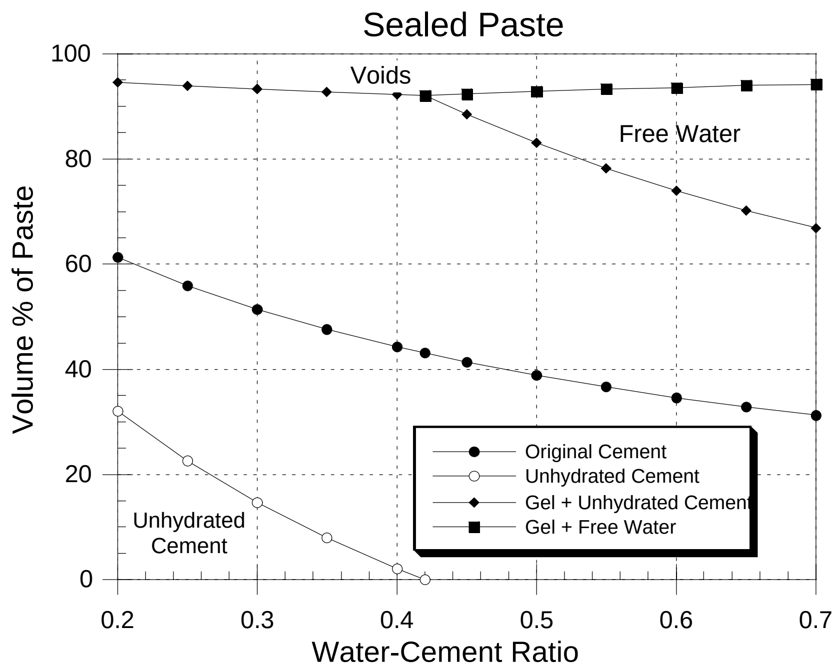


Figure 3.2 Theoretical volumetric composition of fully hydrated cement pastes cured under sealed conditions.

The other curing condition assumes that an external supply of water is available to maintain the capillary pores saturated as the cement hydrates. The additional water that enters the paste to maintain saturation can be used to hydrate additional cement until one of two conditions is attained: (1) all the cement has hydrated, or (2) all the available space is filled by cement gel. By using the condition that the volume of cement gel equals the original volume of cement plus water, it can be shown that the dividing line between these two conditions occurs at a water-cement ratio of about 0.36.

Figure 3.3 shows the volumetric proportions in fully hydrated cement pastes cured under water. For a water-cement ratio less than 0.36, there is insufficient space to accommodate additional cement gel, and a portion of the cement remains unhydrated. Comparing Fig. 3.3 with Fig. 3.2 shows that more of the cement hydrates under submerged curing conditions compared with sealed conditions. An important assumption used to derive Fig. 3.3 is that the capillaries are always saturated. This requires pathways from the exterior to the interior of the paste. As will be discussed, at low water-cement ratios and after a certain degree of hydration, the capillaries become disconnected, and it may not be possible for them to remain continuously saturated. Thus, in reality, there may be empty capillary voids present, even when paste is cured under water.

Even though Fig. 3.2 and Fig. 3.3 are based on simplifying assumptions, they are helpful in understanding how water-cement ratio and curing method affect the final volumetric proportions of the fully hydrated cement paste. For example, it can be seen that

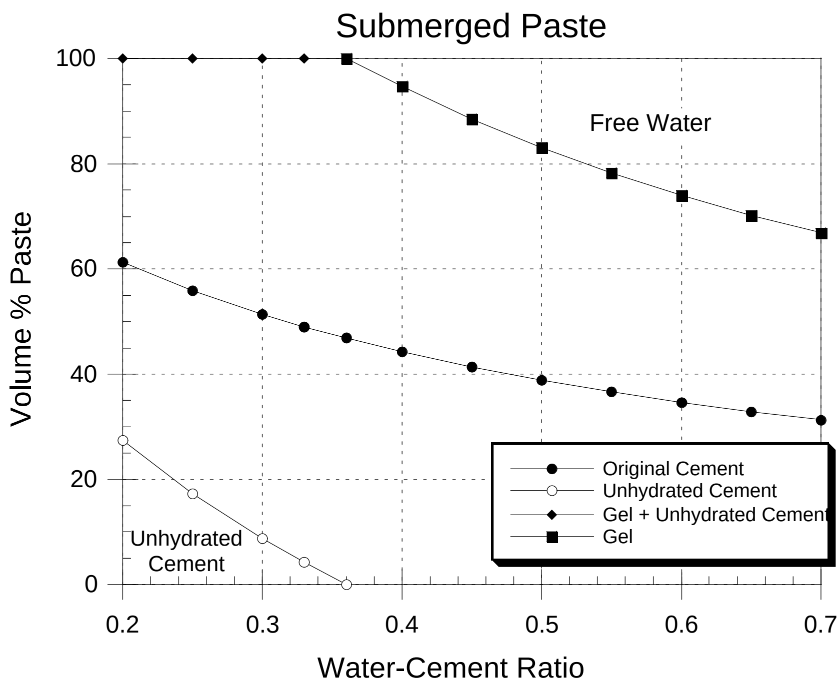


Figure 3.3 Theoretical volumetric composition of fully hardened cement paste cured under water provided capillaries are maintained saturated.

under submerged condition, it is theoretically possible to have a fully hydrated paste without capillary pores, while sealed conditions will result in capillary pores in all cases.

3.2.3 *Pore structure characteristics*—Capillary pores are the spaces in the paste that have not been filled by hydration products. Upon initial mixing, all the water occupies capillary space. As hydration proceeds, the volume of the solid phase (cement + cement gel) increases, and the capillary pore volume decreases. Depending on the degree of hydration and the water-cement ratio, the capillaries may comprise an interconnected system of randomly distributed pores throughout the paste. These interconnected capillary pores affect strongly the permeability properties of concrete and its resistance to the penetration of aggressive ions.

As the paste approaches complete hydration, there may be sufficient cement gel to isolate the capillary pores. This will eliminate continuous capillary channels since the pores will be intercepted by highly impermeable cement gel. This condition may be achieved by a sufficiently long period of moist curing. The required curing duration depends primarily on the water-cement ratio. Powers et al. (1959) developed the relationship shown in Fig. 3.4, which indicates the degree of hydration to achieve discontinuous (segmented) capillary pores. This figure shows clearly that pastes with higher water-cement ratios have to attain a higher degree of hydration before the capillaries become segmented. The curing times required for the capillary pores to become discontinuous depend on the hydration rate characteristics of the cement and on the temperature of the paste. Approximate curing times

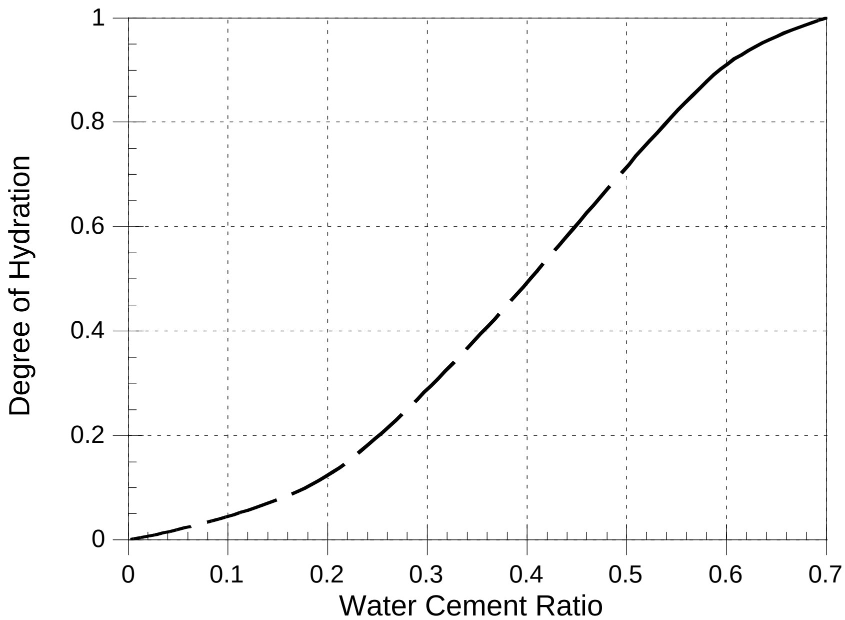


Figure 3.4 Degree of hydration at which capillary pores become discontinuous (adapted from Powers et al. 1959).

for ordinary portland cement pastes at standard temperature are shown in Fig. 3.5. For example, for an ordinary portland cement with a water-cement ratio of 0.40, 3 d of moist curing would be required to segment the capillaries. If the water-cement ratio is greater than about 0.70, even fully hydrated pastes will be unable to produce sufficient amounts of gel to attain this state. Obviously, it is very desirable to attain segmentation of the capillary pores, because this results in impermeable concrete with improved durability.

The gel pores are quite different from the capillary pores (see Fig. 3.6(a)). They are best described as small spaces intermingled between the solid particles of the cement gel. Figure 3.6(b) shows one of the popular models of the structure of the cement gel. Gel pores have small dimensions that are only an order of magnitude larger than the size of a water molecule (Neville 1996). In contrast to capillary pores, as hydration progresses, the total volume of gel pores increases along with an increase in the volume of cement gel. As was stated, the porosity of cement gel is about 28 %, independent of water-cement ratio or degree of hydration. Thus gel pores constitute about one-third of the total volume of cement gel.

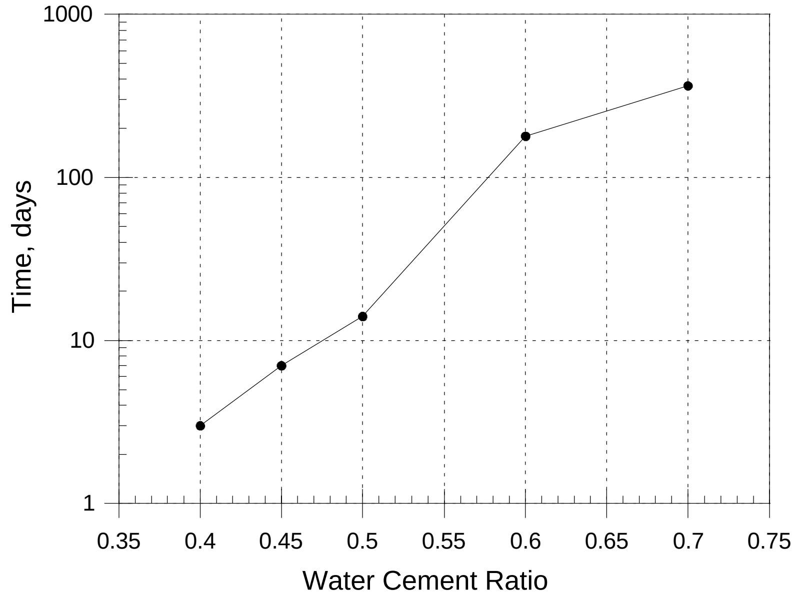


Figure 3.5 **Approximate time for capillary pores to become discontinuous for ordinary portland cement paste (adapted from Powers et al. 1959).**

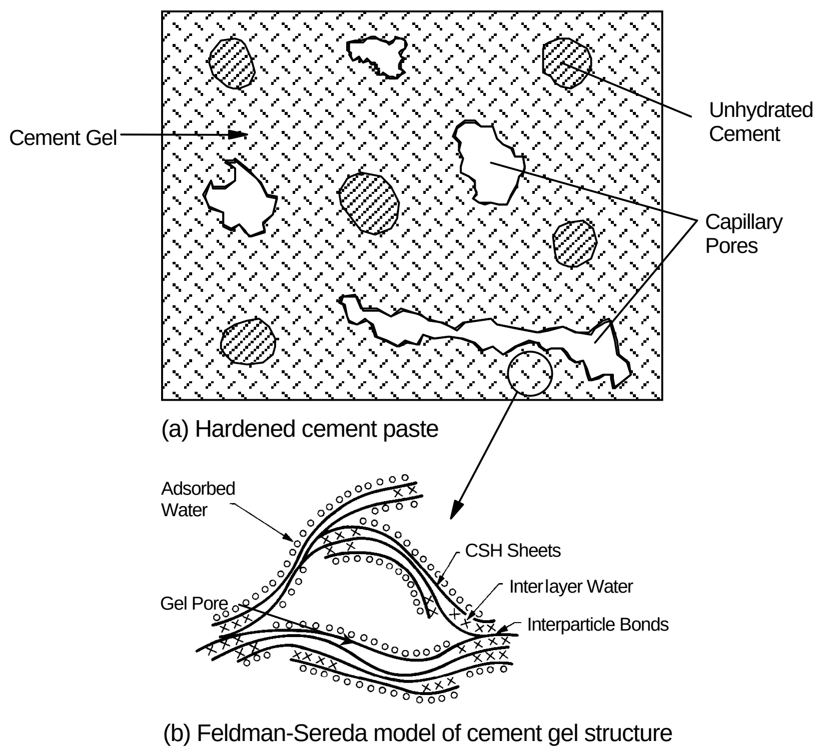


Figure 3.6 **(a) Hardened cement paste showing cement gel, capillary pores, and unhydrated cement: (b) one of several proposed models of the cement gel.**

3.2.4 *Summary*—This section has examined the basic volumetric relationships associated with the transformation of portland cement to cement gel. Using these relationships and other simplifying assumptions, predictions have been made of the volumetric proportions in fully hardened cement paste as a function of water-cement ratio and curing condition (sealed and submerged). While the calculations neglect such factors as bleeding and the effects of the internal relative humidity, they provide insight into the differences between moist curing (excess water is available) and sealed curing (no moisture exchange with surroundings). They also highlight the differences between high water-cement ratio pastes and low water-cement ratio pastes that are characteristic of most types of high-performance concrete. As noted, water-filled capillary pores are essential for continued hydration. When the capillaries dry out or become filled with hydration products, hydration ceases. It has also been shown that below a certain water-cement ratio, complete hydration is not possible and the fully hardened paste will include unhydrated cement. Finally, it was mentioned that capillary pores become discontinuous at critical degrees of hydration. These critical values increase with increasing water-cement ratio. When capillary pores become discontinuous, the moisture penetrability of the paste is reduced drastically. The consequence of this reduction in penetrability on the practicality of moist curing high-performance concrete beyond a certain level of maturity appears to be an important subject for additional study.

3.3 Percolation Theory

Simulation studies of cement hydration at NIST (Garboczi 1993) have shown that the application of percolation theory to the microstructure of cement paste can be very helpful in understanding relationships between degree of hydration and transport properties. Using the concept of *connectivity* from percolation theory, researchers have determined analytically under which conditions the capillary pores become discontinuous. At this point, transport of water through the paste would have to be through the much smaller gel pores. Based on the results of the simulations, it was concluded that pastes with a water-cement ratio of 0.6 or higher always contain a continuous capillary pore structure, or comprise a *percolated system*, regardless of the degree of hydration. At water-cement ratios less than 0.6, the simulations indicate that capillaries become discontinuous at different degrees of hydration. The lower the water-cement ratio, the less is the degree of hydration required to attain this desirable state.

In the NIST simulation studies, the microstructures of cement pastes at different water-cement ratios were examined at various degrees of hydration. In each case, the fraction of the total capillary pore volume that makes up interconnected paths through the paste was determined. Figure 3.7(a) summarizes the results of these simulations. For a given water-cement ratio, the fraction of connected pores decreases with increasing degree of hydration. For low water-cement ratio pastes, there are certain degrees of hydration at which no capillary pores are interconnected across the volume of paste. It is seen that for water-cement ratios of 0.6 or greater, there will always be connected capillary pores. The fraction

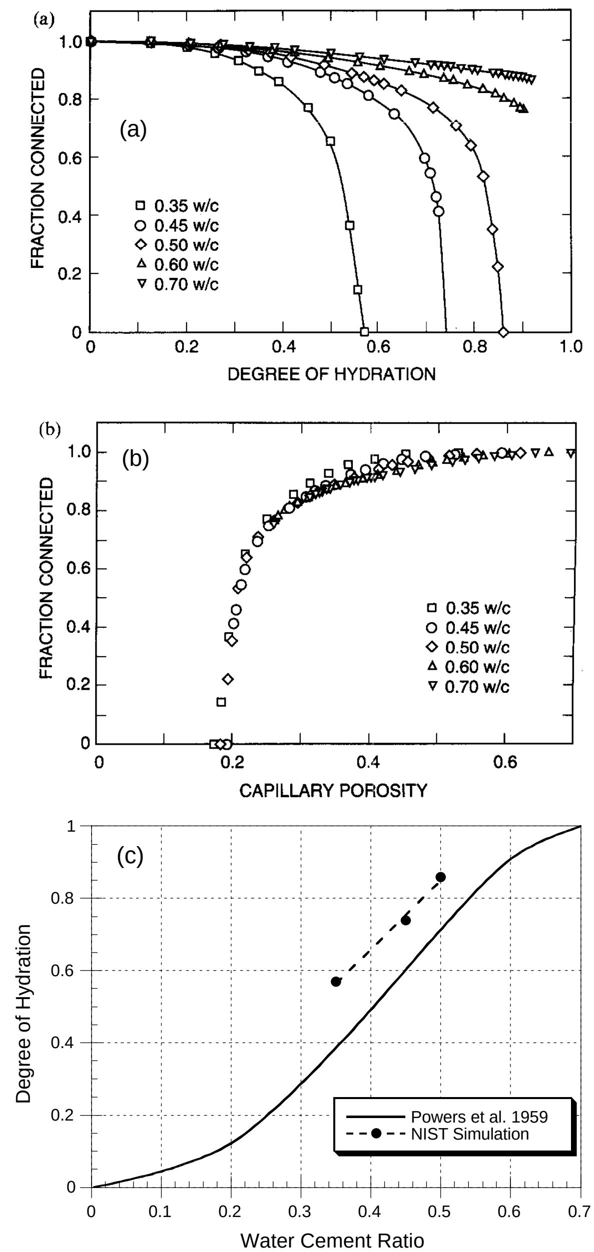


Figure 3.7 (a) Fraction of connected pores vs. degree of hydration and (b) fraction of connected pores vs. capillary porosity based on NIST computer simulation (Garboczi 1993); (c) comparison with results by Powers et al. (1959).

of connected pores was plotted as a function of the capillary porosity, as shown in Fig. 3.7(b). Capillary porosity is the volume of capillary pores divided by the total volume of hardened paste. Remarkably, all the results fall very close to a single curve.

These simulations demonstrate that capillary pores become discontinuous (percolation threshold) when the capillary porosity is reduced to about 18 %. This critical percolation threshold of 18 % porosity is independent of the water-cement ratio. However, it is expected to be somewhat affected by the cement particle size distribution and degree of dispersion (Garboczi 1993). Pastes with a water-cement ratio of 0.6 or greater will always have continuous capillaries because there is not enough cement per unit volume to produce porosity lower than 18 %, even when fully hydrated.

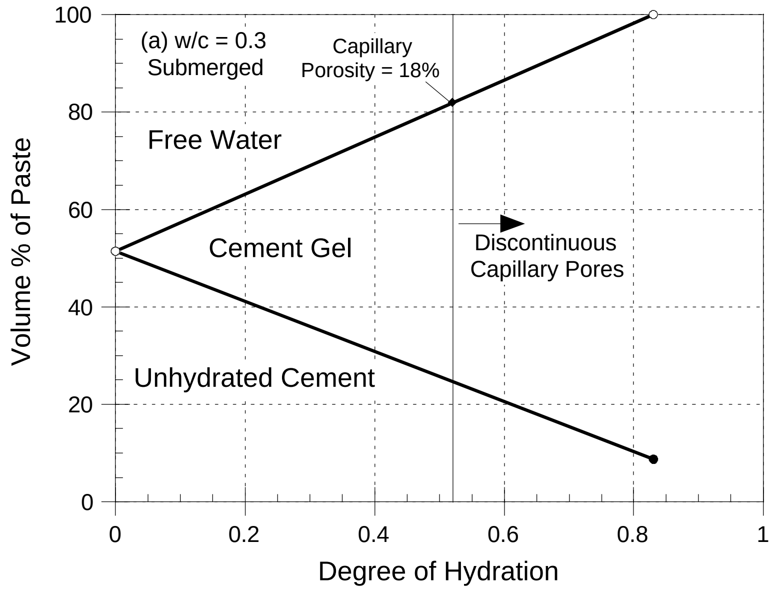
Figure 3.7(c) compares the results of the NIST simulations with the experimental findings of Powers et al. (1959). The agreement is reasonable. The major differences are that the simulations indicate higher degrees of hydration to achieve discontinuous capillary pores and a lower value of the water-cement ratio above which continuous pores cannot be eliminated.

Based on the simulation results, three distinct phases were identified with respect to the movement of water through cement paste.

- Early hydration—the capillary pores are fully connected (percolated), and any transport of water will be mainly by these spaces.
- Intermediate hydration—the capillary pores are decreasing in size, as the capillary porosity approaches the critical value. Flow of water will be occurring in both capillary pores and also within hybrid paths composed of isolated capillary pores linked by gel pores.
- Below critical capillary porosity—hydration has proceeded to the point of reducing connectivity of the capillaries to zero. All movement of water must include flow through the gel pores, using both the hybrid paths and the pure gel pore paths.

The results of these simulation studies provide additional insight into the microstructural changes that occur during hydration. It is now clear that there is a critical capillary porosity at which the transport of water through paste changes from the relatively unobstructed flow through continuous capillary pores to combined flow through capillary pores and gel pores. Flow through the gel pores is more difficult because of the much smaller dimensions compared with capillary pores. For moist curing conditions, the progress of hydration beyond the percolation threshold is expected to be controlled by two factors: the rate at which water in the discontinuous capillary pores is consumed and the rate at which external water can reach the discontinuous capillary pores. If the capillary water cannot be replaced as quickly as it is consumed, the likelihood of self-desiccation increases.

This additional information about microstructural changes can be used to re-examine the volumetric proportions of fully hardened paste presented in Figs. 3.1, 3.2, and 3.3. Take, for example, a paste with a water-cement ratio of 0.30. Figures 3.8(a) and (b) show the volumetric proportions of the unhydrated cement, cement gel, and free water as a function of the degree of hydration (fraction of the cement that has hydrated). Recall that the volumetric proportions of the different constituents of the paste are equal to the proportions of a vertical line that fall into the different regions. These figures were obtained by drawing straight lines between the initial volumetric proportions of cement and water and the



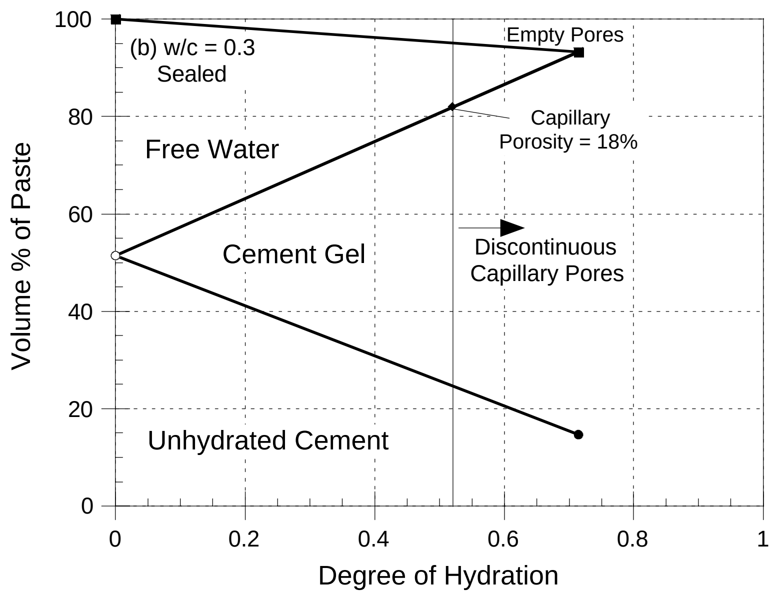


Figure 3.8 Volumetric proportions of materials in hardened cement paste with w/c = 0.30 as a function of the degree of hydration: (a) paste is submerged, and (b) paste is sealed

limiting values of unhydrated cement and cement gel when no further hydration is possible (Figs. 3.2 and 3.3). Vertical lines are drawn at the degree of hydration (which is approximately 52 %) that results in a capillary porosity of 18 %. Thus the portions of the figures to the right of the vertical lines represent conditions where the penetrability of the paste is drastically reduced and the possibility for self-desiccation is increased.

3.4 Influence of Low Penetrability and Self-desiccation on Curing Practice

Self-desiccation may be an important consideration in the study of optimal curing practices for high-performance concrete. Since most high-performance concretes will have low water-cement ratios, these concretes will be susceptible to self-desiccation. Proper moist curing at early ages is, therefore, needed to provide the supply of water to delay the onset of self-desiccation and enable the concrete to reach both its strength and durability requirements. In addition, self-desiccation also contributes to shrinkage of concrete, making high-performance concrete more prone to cracking under restrained conditions (Miyazawa and Monteiro 1996). Fundamental research is needed to ascertain precisely how self-desiccation affects the manner in which high-performance concrete should be cured, and for how long, particularly in the field.

Even when low water-cement ratio concrete is cured under water, self-desiccation can occur. This is possible because, as was discussed in Section 3.3, the paste becomes essentially watertight at an early age. As a result, it may not be possible for the external water to penetrate to the internal capillaries fast enough to offset the internal drying due to hydration (FIP-CEB Working Group 1990). Thus the curing of low water-cement ratio concrete presents a dilemma: a supply of excess water is necessary to counteract self-desiccation, but the low capillary porosity restricts the flow of water from the exterior supply to the interior dry pores. So there is a challenge to establish the optimal curing practices that will arrive at a balance between economy and achievement of the desired properties. It may turn out, as has been suggested by Hilsdorf (1994, 1996), that extended moist curing beyond a certain point will be of little additional benefit to the further development of the properties of the concrete.

3.5 Carbonation

Carbonation refers to the process whereby alkaline compounds (Na_2O, K_2O, and mostly $Ca(OH)_2$), created from the hydration of the cement paste, react with carbon dioxide from the surrounding environment. Carbon dioxide also reacts with the cement causing this material to decompose into calcium carbonate and an amorphous, porous silica gel (Kropp and Hilsdorf 1995). Carbonation lowers the pH of the pore water and, therefore, reduces the ability of the concrete to protect steel reinforcement from corrosion. Thus carbonation has serious consequences since the service life of reinforced concrete structures requires that integrity be maintained between the steel and the concrete.

Carbonation is normally discussed in concrete technology in terms of the rate of carbonation, that is, how fast the carbonation front penetrates into concrete. This rate is related to how readily carbon dioxide is able to penetrate the surface of the concrete by the process of diffusion. The two factors cited most often in relation to the rate of carbonation are the gas permeability of the concrete and the capillary absorption of water. Therefore, limiting the depth of carbonation becomes an important requirement to achieve the intended service life of a reinforced concrete structure. Using low water-cement ratio concrete and assuring sufficient curing to reduce capillary porosity increases resistance to diffusion of carbon dioxide.

As will be discussed in Chapter 6, some of the most recent technology on curing (Hilsdorf 1995) has proposed that controlling the rate of carbonation be considered as a criterion for curing duration of structural concrete. The required duration would be chosen to control the depth of carbonation of the concrete over its specified lifetime under the expected environmental conditions.

In Europe, carbonation has been a more serious problem than in the United States because:

- The concrete cover over reinforcing bars is often less than in the United States; and
- Greater quantities of pozzolanic materials are used in concrete, and longer curing durations are needed because these materials react more slowly than portland cement.

3.6 Summary

This chapter has reviewed our understanding of the characteristics of hardened cement paste, which is generally composed of cement gel, unhydrated cement, and capillary pores. The physical characteristics of the cement paste at different degrees of hydration are affected greatly by the initial water-cement ratio and whether curing takes place under sealed or submerged conditions. The capillary pores, that is, the spaces between hydrating cement grains, have profound effects on the proprieties of the hardened paste. By using known volumetric and mass relationships of hydrated cement, simple models can be used to predict the volumetric composition of a cement paste, with a given initial water-cement ratio, as a function of the degree of hydration. Such a model highlights the major differences between the characteristics of the low water-cement ratio pastes that are typical in high-performance concrete and the pastes in ordinary concrete. One of the major differences is the reduced capillary porosity as the water-cement ratio is reduced.

An important characteristic of cement paste is the connectivity of the capillary pores. When capillary pores become discontinuous, there is a large reduction in the penetrability of moisture through the paste, because water must pass through the relatively impermeable cement gel. The degree of hydration at which the capillary pores becomes discontinuous decreases with decreased water-cement ratio. For the low water-cement ratios associated with high-performance concrete, the capillary pores become discontinuous after a few days of hydration (at room temperature). While discontinuous capillary pores are desirable for improved durability, they may reduce the effectiveness of curing methods that supply excess moisture.

4. HISTORY OF ACI BUILDING CODE REQUIREMENTS FOR CURING

4.1 Introduction

This chapter summarizes the developments leading to the current American Concrete Institute code requirements for curing. It provides a look at the history of curing criteria for concrete from the early 1900s to the present. It will be seen that the current ACI-318 Code does not contain curing criteria specifically for high-performance concrete. As the body of knowledge on the optimal curing practices for high-performance concrete is developed, along with increased experience in the use of this class of concrete, future editions of the ACI Code may include specific curing requirements for high-performance concrete.

4.2 Early Years Under the National Association of Cement Users

During its very early years, the organization now known as the American Concrete Institute was called the National Association of Cement Users (NACU). Founded in 1904, this group provided the earliest regulatory guidance on curing. The following gives excerpts from six committee reports issued in 1907 to 1910.

4.2.1 *1907 (NACU), Report by the Committee on Laws and Ordinances*—This report included the following statements related to curing:

> *"When the concrete is exposed to a hot or dry atmosphere special precautions shall be taken to prevent premature drying by keeping it moist for a period of at least twenty-four hours after it has taken its initial set. This shall be done by a covering of wet sand, cinders, burlap or by continuous sprinkling or by some other method equally effective in the opinion of the Commissioner of Public Buildings.*
>
> *If during the hardening period the temperature is continually above 70 F, the side forms of concrete beams and the forms of floor slabs up to spans of eight feet shall not be removed before four days. The remaining forms and supports not before ten days from the completion of tamping.*
>
> *If during the hardening period the temperature falls below 70 F, the side forms of concrete beams and the forms of floor slabs up to spans of eight feet shall not be removed before seven days; the remaining forms and the supports not before fourteen days from the completion of the tamping. But if, during the hardening period, the temperature falls below 35 F, the time for hardening shall be extended by the time during which the temperature was below 35 F."*

Comments: This was the first document resembling a building code by NACU, and was contained in the 1907 Proceedings of the Association's Third Annual Convention. This was actually an ordinance that had been drafted and introduced previously in the Municipal Assembly in the City of St. Louis. A joint committee of structural engineers, architects, and concrete contractors had prepared this ordinance. It was presented to the NACU Convention for the members' information and consideration. As can be seen, the curing provisions were written in prescriptive language, and they address formwork removal as well as curing. It is seen that attempts were made to account for the effects of temperature on the duration of curing.

4.2.2 *1908 (NACU), Requirements for Reinforced Concrete or Concrete-Steel Constructed Buildings*—This document included the following statements related to curing:

> *"Concrete shall not be installed in freezing weather; such weather shall be taken to mean a temperature of 32 °F or lower; concrete shall not be allowed to freeze after being put in place, and, if frozen, shall be removed.*
>
> *The time at which forms and centering may safely be removed will vary from twenty-four hours to sixty days, depending upon temperature and other atmospheric conditions of the weather, the time for such removal to be determined by the commissioner of buildings."*

Comments: This language was proposed by the National Board of Fire Underwriters. It was recommended to NACU by the Committee on Laws and Ordinances, but it was never adopted by the Association. These provisions give specific requirements for cold weather concreting, rather than curing in general. However, they show clearly the dependence of safe formwork removal time on weather conditions.

4.2.3 *1909 (NACU), Suggested Standard Building Regulations for the Use of Reinforced Concrete*—The following statement was included on duration of curing:

> "The faces of concrete exposed to premature drying shall be kept wet for a period of at least seven days."

Comments: This provision was contained in a report submitted by the Committee on Insurance, Laws, and Ordinances. As can be seen, a minimum moist curing period of 7 d was specified, but the meaning of "premature drying" was not given. Here we see the first mention of the 7-day curing period that was to become a cornerstone of American curing practice. The report was discussed and revised at the annual convention of NACU.

4.2.4 *1909 (NACU), Proposed Standard Building Regulations for the Use of Reinforced Concrete*—Based on the discussion at the 1909 NACU convention, the following version of the previous curing requirement was proposed:

> *"The faces of concrete exposed to premature drying shall be kept damp for a period of at least seven days."*

Comments: Note that the wording for the revised curing requirement is the same as in the earlier proposal except that "wet" was replaced by "damp." In the proceedings of the convention, it was stated that members were not given enough time to properly review this proposed code prior to the convention. Therefore, it was decided to postpone a vote on acceptance to the next annual convention in 1910.

4.2.5 *1909 (NACU), Report of Committee on Reinforced Concrete*—This new committee report contained the following statement:

"Concrete should be frequently wet for several days to prevent too rapid drying out."

Comments: This was included in a report by the newly formed NACU Committee on Reinforced Concrete. The recommendations conformed, in general, to a report by the Joint Committee on Concrete and Reinforced Concrete representing the American Society of Civil Engineers, the American Society for Testing Materials, the American Railway Engineering and Maintenance of Way Association, and the Association of American Portland Cement Manufacturers. As can be seen, the proposed curing provision contained vague words, such as "frequently," "several," and "rapid." Such a loosely worded provision would probably have been difficult to enforce.

4.2.6 *1910 (NACU), Standard Building Regulations for the Use of Reinforced Concrete*—The following curing provisions were adopted by NACU:

"The faces of concrete exposed to premature drying shall be kept damp for a period of at least seven days."

Comments: This was presented in a report of the Committee on Building Laws and Insurance, and was the same document, with minor revisions, that was first proposed in 1909. No change was made to the curing provision. These standard regulations were adopted in February, 1910, and this can be considered to be the first national "code" on the use of reinforced concrete in buildings.

4.3 Early Years Under the American Concrete Institute

In 1913, the name of the National Association of Cement Users was changed to the American Concrete Institute. In the years from World War I through World War II, there was considerable progress in the development of national codes. Note, however, how little the curing requirements change during this period.

4.3.1 *1916 (ACI), Proposed Revised Standard Building Regulations for the Use of Reinforced Concrete*—The following modification of the curing provision adopted in 1910 was proposed:

"The face of concrete exposed to rapid drying shall be kept damp for a period of at least five days."

Comments: This was included in a report submitted by the ACI Committee on Reinforced Concrete and Building Laws. After a reading of this report at the convention, it was referred back to the Committee for reconsideration and for resubmission at a later date for formal adoption. Note that the minimum curing period was reduced from 7 d to 5 d, and the word "premature" was replaced by "rapid."

4.3.2 *1917 (ACI), Proposed Standard Building Regulations for the Use of Reinforced Concrete*—The curing provision remained unchanged:

"The face of concrete exposed to rapid drying shall be kept damp for a period of at least five days."

Comments: This was essentially the same document presented the previous year on which action had been delayed. It was presented by the Committee on Reinforced Concrete and Building Laws for approval at the 1917 convention. The proposed regulations differed in several ways from the recommendations of the Joint Committee on Concrete and Reinforced Concrete. Therefore, it was decided at the convention to defer adoption of these standard regulations until more data had been presented showing the reasons for the differences.

4.3.3 1919 *(ACI), Proposed Standard Building Regulations for the Use of Reinforced Concrete*—A proposal was made to modify the existing curing provision to read as follows:

"Newly placed concrete shall be protected from rapid drying and kept damp for a period of at least 5 days."

Comments: War conditions prevented the Committee on Reinforced Concrete and Building Laws from making substantial progress on addressing the differences between the recommendations of the Joint Committee and the proposed standard regulations that had been presented in 1917. At the convention this year, the Committee was only able to provide what it called a progress report. Some revisions had been made to the proposed regulations presented in 1917, but more work remained before it could be formally presented for adoption. As far as the curing provision is concerned, the committee proposed replacing the words "exposed to" with the words "shall be protected from."

4.3.4 1920 *(ACI), Standard Building Regulations for the Use of Reinforced Concrete*—The curing provision remained the same as presented in 1919:

"Newly placed concrete shall be protected from rapid drying and kept damp for a period of at least 5 days."

Comments: The proposed standard regulations were approved by vote of the members at the convention this year. They were adopted by letter ballot of the ACI membership on April 17, 1920. This can be considered as the second "code."

4.3.5 *1925 (ACI), Preliminary Draft of Proposed Standard Building Regulations for the Use of Reinforced Concrete*—A proposal was made to modify the curing provision of the 1920 regulations so it would read as follows:

> *"Exposed surfaces of concrete shall be kept moist for a period of at least 7 days after being deposited."*

Comments: This draft standard was presented at the convention by Committee E-1 on Design and Specifications. The Committee was requested by the ACI Board of Direction to prepare standard building regulations for reinforced concrete. The regulations were to be based on the provisions of the Standard Specifications for Concrete and Reinforced Concrete of the Joint Committee Report for 1924. The Committee, therefore, presented this preliminary draft. It was planned to present a Tentative Standard Building Regulations for the Use of Reinforced Concrete at the next convention. It is interesting to note this is the first document which has any resemblance in either form or content to modern codes. The proposed curing provision requires all exposed surfaces to be moist cured for at least 7 d, the same duration as had been proposed in 1909. The language is direct and unambiguous.

4.3.6 *1927 (ACI), Tentative Building Regulations for the Use of Reinforced Concrete*—The curing provision was identical to the one in the 1925 preliminary draft

> *"Exposed surfaces of concrete shall be kept moist for a period of at least 7 days after being deposited."*

Comments: These regulations were adopted by ACI at the 1927 convention.

4.3.7 *1928 (ACI), Tentative Building Regulations for Reinforced Concrete*—The curing provision in the 1927 tentative regulations was expanded to address curing during hot weather.

> *"Exposed surfaces of concrete shall be kept moist for a period of at least 7 days after being deposited. In hot weather, exposed concrete shall be thoroughly wetted twice daily during the first week."*

Comments: This document was the result of a joint effort between ACI Committee E-1 and the Committee on Engineering Practice of the Concrete Reinforcing Steel Institute. These regulations were adopted at the 1928 ACI convention. In the curing provision, the added statement related to hot weather seems redundant, as the requirement to keep the surfaces moist would dictate how often to rewet.

4.3.8 *1936 (ACI), Building Regulations for Reinforced Concrete (ACI 501-36T)*—The curing provision was modified by introducing a different requirement for "high early strength concrete:"

"In all concrete structures, provision shall be made for maintaining the concrete in a moist condition for a period of at least seven days after the placement of the concrete, and for high early strength concretes, special moist curing shall be provided for at least the first three days of the seven-day period after the placement of the concrete."

Comments: There was a major departure from the previous curing provisions with the addition of a separate minimum curing requirement for high early strength concretes. The definition of "high early strength concrete" was not given; presumably, it refers to concrete made with Type III cement. There was no explanation of "special moist curing," so the exact interpretation of the additional requirement is not obvious. These regulations were adopted as a tentative standard at the annual convention that year. As will be seen, these 7-day and 3-day provisions have survived until the present.

4.3.9 *1941 (ACI), Building Regulations for Reinforced Concrete (ACI 318-41)*—The 1936-curing provision was streamlined as follows:

"In all concrete structures, concrete made with normal portland cement shall be maintained in a moist condition for at least the first seven days after placing and high-early strength concrete shall be so maintained for at least the first three days."

Comments: The reference to "special moist curing" was deleted. Note that the requirement for maintaining a moist condition for at least the first 7 d after placing is applicable specifically to concrete made with normal portland cement.

4.4 Later Years Under the American Concrete Institute

From the end of World War II through the 1960s there was virtually no change in the ACI curing requirements. During this period, the word "code" was used for the first time in the title of the ACI regulatory document on the use of reinforced concrete for buildings.

4.4.1 *1947 (ACI), Building Code Requirements for Reinforced Concrete (ACI 318-47)*—The curing provision was identical to the one in the 1941 Regulations:

"In all concrete structures, concrete made with normal portland cement shall be maintained in a moist condition for at least the first seven days after placing and high-early-strength concrete shall be so maintained for at least the first three days."

Comments: This is the first time the word "Code" is used in the title of the ACI regulatory document.

4.4.2 1951 *(ACI), Building Code Requirements for Reinforced Concrete (ACI 318-51)*—The curing provision remained the same as in the 1941 Regulations:

"In all concrete structures, concrete made with normal portland cement shall be maintained in a moist condition for at least the first seven days after placing and high-early-strength concrete shall be so maintained for at least the first three days."

4.4.3 *1956 (ACI),* Building *Code Requirements for Reinforced Concrete (ACI 318-56)*—The curing provision in the 1941 Regulations was modified editorially by replacing the words "seven" and "three" by numbers.

"In all concrete structures, concrete made with normal portland cement shall be maintained in a moist condition for at least the first 7 days after placing and high-early-strength concrete shall be so maintained for at least the first 3 days."

4.4.4 *1963 (ACI), Building Code Requirements for Reinforced Concrete (ACI 318-63)*—Significant changes were made to the curing provisions:

"Concrete shall be maintained above 50 F and in a moist condition for at least the first 7 days after placing, except that high-early-strength concrete shall be so maintained for at least the first 3 days. Other curing periods may be used if the specified strengths are obtained."

Comments: The 1963 curing provision included two significant changes. First, there was a requirement that the concrete be maintained above 10 °C (50 °F) during the curing period. This is to assure that there is a reasonable rate of hydration during the moist protection period. Second, the provision permits other curing periods provided that specified strengths are obtained. This is the first evidence of what might be called a performance requirement. However, it was short lived. Unfortunately, no provisions were included for assessing when the specified strength had been attained.

4.5 Recent Years Under the American Concrete Institute

In the period from the early 1970s into the 1990s, the ACI Code added a requirement to verify the adequacy of the curing procedures. In addition, provisions were added to address accelerated curing, which is often used in the manufacture of precast concrete.

4.5.1 *1971 (ACI), Building Code Requirements for Reinforced Concrete (ACI 318-71)*—The 1963 curing provisions were augmented as follows:

"Unless cured in accordance with Section 5.5.2, concrete shall be maintained above 50 F and in a moist condition for at least the first 7 days after placing, except that high-early-strength concrete shall be so maintained for at least the first 3 days. Supplementary strength tests in accordance with Section 4.3.4 may be required to assure that curing is satisfactory."

Comments: The minimum durations of moist curing remained unchanged. Note that the sentence in the 1963 Code permitting other curing durations, provided specified strength was attained, was deleted. Paragraph 5.5.2 was added to accommodate accelerated curing, such as by steam at high pressure or steam at atmospheric pressure. The duration of accelerated curing was required to be as long as necessary for the concrete to attain the required design strength to safely resist imposed early-age loads, such as applied prestressing. There was also a requirement that accelerated-cured concrete should be as durable as concrete cured under ambient conditions.

The other significant addition related to curing was included in Section 4.3, titled "Evaluation and acceptance of concrete." Paragraph 4.3.4 contained the following requirement related to strength of field-cured cylinders:

> *"Strength tests of specimens cured under field conditions in accordance with ...(ASTM C31) may be required by the Building Official to check the adequacy of curing and protection of the concrete in the structure. Such specimens shall be molded at the same time and from the same samples as the laboratory-cured acceptance test specimens. Procedures for protecting and curing the concrete shall be improved when the strength of field-cured cylinders at the test age designated for measuring f_c' is less than 85 percent of that of the companion laboratory-cured cylinders. When the laboratory-cured cylinder strengths are appreciably higher than f_c' the field-cured cylinder strengths need not exceed f_c' by more than 500 psi even though the 85 percent criterion is not met."*

This is the first time that the ACI Code included a specific criterion for judging the adequacy of field curing. The criterion was based on earlier work by Bloem (1968) at the National Ready-Mixed Concrete Association. He demonstrated that under curing conditions representative of good construction practice, the in-place strength at 28 d, based on strengths of cores or cast-in-place-cylinders, was about 85 % of the strength of standard-cured cylinders.

4.5.2 *1977 (ACI), Building Code Requirements for Reinforced Concrete (ACI 318-77)*—The curing provisions of the 1971 Code, which were contained in two paragraphs, were revised so as to encompass five paragraphs:

> *"**5.5—**Curing*
> *5.5.1—Concrete (other than high-early-strength) shall be maintained above 50 F and in a moist condition for at least the first 7 days after placement, except when cured in accordance with Section 5.5.3.*
> *5.5.2—High-early-strength concrete shall be maintained above 50 F and in a moist condition for at least the first 3 days, except when cured in accordance with Section 5.5.3.*
> ***5.5.3—**Accelerated curing*
> *5.5.3.1—Curing by high pressure steam, steam at atmospheric pressure, heat and moisture, or other accepted processes, may be employed to accelerate strength gain and reduce the time of curing.*
> *5.5.3.2—Accelerated curing shall provide a compressive strength of the concrete at the load stage considered at least equal to required design strength at that load stage.*

5.5.3.3—Curing process shall be such as to produce concrete with a durability at least equivalent to the curing method of Section 5.5.1 or 5.5.2.
5.5.4—Supplementary strength tests in accordance with Section 4.8.3 may be required to assure that curing is satisfactory."

Comments: Note that there is no substantive change to the basic curing requirements of the 1971 Code. The changes were primarily editorial to improve the clarity of the provisions. Section 4.8.3 was titled "Tests of field-cured specimens" and it contained the same requirements as in Section 4.3.4 of the 1971 Code. However, the single paragraph in the 1971 Code was broken up into four paragraphs for improved clarity.

4.5.3 *1983 (ACI), Building Code Requirements for Reinforced Concrete (ACI 318-83)*—The basic curing provisions in the 1977 Code remained unchanged:

"5.5.1—Concrete (other than high-early-strength) shall be maintained above 50 F and in a moist condition for at least the first 7 days after placement, except when cured in accordance with Section 5.5.3.
5.5.2—High-early-strength concrete shall be maintained above 50 F and in a moist condition for at least the first 3 days, except when cured in accordance with Section 5.5.3."

Comment: The only significant change was a simplification of the last sentence in the paragraph dealing with evaluation of field-cured cylinder strength (see the previous 4.5.1). This change removed the ambiguity associated with the phrase: "When laboratory-cured cylinder strengths are appreciably higher than f_c'…" The revised wording was as follows:

"…The 85 percent may be waived if field-cured strength exceeds f_c' by more than 500 psi."

4.5.4 *1989 (ACI), Building Code Requirements for Reinforced Concrete (ACI 318-89)*—No changes, other than in section numbers, were made to the basic curing provisions in the 1983 Code:

5.11.1—Concrete (other than high-early-strength) shall be maintained above 50 F and in a moist condition for at least the first 7 days after placement, except when cured in accordance with 5.11.3.
5.11.2—High-early-strength concrete shall be maintained above 50 F and in a moist condition for at least the first 3 days, except when cured in accordance with 5.11.3."

Comment: A significant change, however, was made to the paragraph on the need for supplementary testing to evaluate adequacy of curing. The revised wording was a follows:

"5.11.4—When required by the engineer or architect, supplementary strength tests in accordance with 5.6.3 shall be performed to assure that curing is satisfactory."

This change permitted the engineer or architect to require supplementary tests to demonstrate adequacy of curing procedures. There was, however, an inconsistency because Section 5.6.3.1 stated the following:

> *"5.6.3.1—The Building Official may require strength tests of cylinders cured under field conditions to check adequacy of curing and protection of concrete in the structure."*

Thus there was ambiguity on who could require field-cured testing for checking the adequacy of curing.

Minor changes were made to remove non-mandatory language in the section dealing with field-cured cylinders. For example, the last sentence in the paragraph on evaluation of field-cured cylinder strengths was revised to read:

> *"…The 85 percent limitation shall not apply if field-cured strength exceeds f_c' by more than 500 psi."*

4.5.5 *1995 (ACI), Building Code Requirements for Structural Concrete (ACI 318-95)*—The title of the 1995 Code was changed because it was expanded to include provisions for plain concrete that were contained previously in a separate code. The basic curing provisions, however, remained the same as in the 1989 Code:

> *"5.11.1—Concrete (other than high-early-strength) shall be maintained above 50 F and in a moist condition for at least the first 7 days after placement, except when cured in accordance with 5.11.3.*
> *5.11.2—High-early-strength concrete shall be maintained above 50 F and in a moist condition for at least the first 3 days, except when cured in accordance with 5.11.3"*

Comment: A change was made to the introductory paragraph of Section 5.6.3 on field-cured specimens. The revised wording is as follows:

> *"5.6.3.1—If required by the building official, results of strength tests of cylinders cured under field conditions shall be provided."*

This change removed the ambiguity in the 1989 Code regarding who is permitted to require supplementary testing of field-cured specimens to check whether curing is satisfactory. The 1995 Code, clearly gives this authority to the architect / engineer.

4.6 Summary

The following observations are based on this comprehensive review of the historical development of the ACI Code provisions on curing:

- The basic, general requirements for the curing of concrete have changed very little since the first standard regulations were proposed in 1909.
- The basic requirement has been to cure concrete made with normal portland cement for a period of at least 7 d and to cure high-early-strength concrete for at least 3 d. While never defined, it is assumed that high-early-strength concrete refers to concrete made with Type III cement.
- In the 1971 Code, the requirement was added to maintain the concrete temperature above 50 °F (10 °C). This addition is to ensure that sufficient strength development will occur during the prescribed minimum curing periods. In addition, a provision was added for checking the adequacy of curing procedures on the basis of strength tests of field-cured cylinders. Both requirements have been carried over to the current ACI Code.
- Tests reported by Price (Price 1951) indicated that normal strength concrete that is moist cured for 7 d and then cured in air will attain approximately the same 28-day strength as if it had been continuously moist cured. These tests confirm the validity of the 7-day criterion in the ACI Code. Since high-early-strength concrete will gain strength more rapidly, the Code permits a 3-day curing period.
- There is no distinction between strength and durability considerations in the curing requirements of the ACI Code. The provisions are intended primarily to assure adequate structural capacity. The only explicit mention of durability is contained in the provisions dealing with accelerated curing.
- There are no curing requirements for concretes made with other cementitious materials besides portland cement. Since the nature of the cementitious system affects the early-age hydration characteristics, this is seen as a major deficiency in the current Code.
- The “performance-based” criterion in the 1963 Code was not carried over into subsequent versions of the Code. Without details on how to assess the adequacy of other curing periods, such a provision was probably unenforceable.

5. OTHER CURRENT CURING RECOMMENDATIONS, STANDARDS, AND CRITERIA

5.1 Introduction

This chapter examines currently existing regulatory requirements for the curing of concrete beyond the ACI Code provisions covered in the previous chapter. Currently, there are no standards that have been developed specifically for high-performance concrete. However, some existing requirements may be applicable to high-performance concrete. Historically, the curing requirements in standards, such as the ACI Code, have been based on strength considerations alone. Because high-performance concrete will often be specified for its enhanced durability, curing standards of the future will need to address durability as well as strength. The chapter begins with a review of various definitions of curing and then examines current curing provisions from various sources.

5.2 Definitions of Curing

There are numerous definitions of *curing* in relation to concrete technology, but most of them deal with basic principles and requirements that are similar in many respects. Some of these definitions are listed below.

Timms (1952):

> *"Curing as applied to the making of concrete, covers all the conditions both natural and artificially created that affect the extent and the rate of hydration of the cement. With respect to moisture content, in the art of concrete making, curing refers to the various means employed to control moisture content or temperature of the concrete or both."*

ACI Committee 612 (1958):

> *"Optimum curing is defined as the act of maintaining controlled conditions for freshly placed concrete for some definite period following the placing or finishing operations to assure the proper hydration of the cement and the proper hardening of the concrete."*

Neville (1996):

> *"Curing is the name given to procedures used for promoting the hydration of cement, and consists of a control of temperature and of the moisture movement from and into the concrete. [...] More specifically, the object of curing is to keep concrete saturated, or as nearly saturated as possible, until the originally water-filled space in the fresh cement paste has been filled to the desired extent by the products of hydration of cement."*

Cather (1994):

Cather proposed a materials science and an engineering definition of curing:

"Materials science: curing is the creation of an environment in which hydration reactions can proceed to help fulfill the aim of producing concrete of adequately low porosity."
"Engineering: curing is adequate when the resulting concrete achieves the expected service performance."

ACI Committee 308[5]:

"Curing is the process by which portland cement concrete matures and develops over time as a result of the continued hydration of the cement grains in the presence of sufficient water and heat."

ACI Committee 116 (1990):

"The maintenance of a satisfactory moisture content and temperature in concrete during its early stages so that desired properties may develop."

ASTM Committee C9 (ASTM C 125, 1995):

"The maintenance of moisture and temperature conditions in a cementitious mixture to allow its properties to develop."

Hilsdorf (1995):

"It is generally accepted that concrete has to be sufficiently cured i.e. protected from early moisture loss and unfavorable temperatures during its early state of hydration in order to assure sufficient strength and durability properties of the hardened concrete at a later stage."

In summary, the word *curing* has two levels of meanings: It may refer to maintaining concrete in a condition that will allow continued hydration, or it may refer to the field procedures used to assure that these conditions are achieved. Another common feature of some of these definitions is that they mention that the conditions for hydration should be maintained until the properties have developed to a desired level. This is important from an economical viewpoint because maintaining proper curing conditions is typically expensive, and it should not be required beyond the time needed to achieve required properties. Regarding the proper "conditions," most definitions refer to moisture and some also include temperature. Provided the temperature is maintained above freezing, the maintenance of a minimum temperature is technically not necessary. However, a minimum temperature is certainly of practical importance to assure a minimum rate of hydration so that the properties may develop over a reasonably short time. Note that Hilsdorf refers to protection from "unfavorable temperatures." This can be interpreted to mean a minimum temperature to prevent freezing. However, it could also refer to a maximum early-age temperature, because it is known that the early-age temperature may affect long-term properties.

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[5] "Curing Concrete - State-of-the-Art," 5th Draft 10/94, Reported by ACI Committee 308.

5.3 Basic Curing Requirements

The first time that an ACI committee issued a report dealing specifically with curing was in the late 1950s (ACI 612 1958). This report contained five basic requirements for curing as summarized below.

(1) *Adequate water content*—Proper curing involves maintaining suitable moisture content in the paste. This can be accomplished using many different curing methods, or combinations of various methods. All these methods, however, involve two basic concepts. Either the surface of the concrete is kept moist by supplying an external supply of water (water curing), or the loss of moisture is controlled by the use of impervious coatings, membranes, coverings, or by the concrete formwork.

(2) *Maintain adequate temperature*—Ideally the concrete temperature should be maintained above freezing at a relatively constant value throughout the period of curing. The concrete must also be protected from high temperatures. As stated, high temperatures at early ages may impair the long-term properties. Controlling the temperature can be a difficult matter since there are three potential sources of heat—the surrounding environment, absorption of solar heat, and the heat generated from the hydration reactions.

(3) *Preservation of reasonably uniform temperature throughout the concrete body*—The committee recommended curing at a constant temperature a few degrees below the average temperature to which the concrete in the field would be exposed during its lifetime. They also recommend that any drop in temperature during the first 24 h after curing not exceed 16.7 °C (30 °F) for mass concrete or 27.8 °C (50 °F) for thin sections. This is to reduce the chance of cracking due to temperature gradients.

(4) *Adequate protection from damaging mechanical disturbances during the early period of curing*—Any new concrete structure must be protected from heavy loads, large stresses, shock, and excessive vibration as the concrete gains strength during early curing. Concrete progresses to its hardened state as determined by the rate of hydration in the paste. This progress can be hampered if the concrete undergoes significant mechanical disturbances during the critical early-age period when the microstructure is beginning to develop. Damage at early age may prevent the concrete from attaining the desired strength and durability to perform satisfactorily.

(5) *Adequate time for sufficient hydration*—There must be enough time allowed for hydration to progress sufficiently to produce concrete having adequate properties for its intended use. The amount of time needed depends on a number of variables—curing temperature, type of cement, and moisture content of the paste.

These five requirements provide the foundation for acceptable curing practices. Among these, the requirement for protection against early-age mechanical disturbance is not completely understood. Some research has shown that a permanent loss in properties (specifically reinforcement bond strength) may occur (Altowaji, Darwin, and Donahey 1986). However, other studies have shown that there may not be permanent loss in properties due to excessive vibration during early ages. The latter conclusion is understandable because damage to the developing microstructure may be repaired (autogenous healing) by subsequent hydration.

5.4 Strength Considerations

Historically, curing requirements have been based on strength considerations. For example, in the 1940s, the ASTM standard specification for the curing of portland cement concrete contains the following requirement (ASTM 1945):

> "*The concrete shall be so cured that the compressive or flexural strengths of specimens of the concrete 28 days old, are not less than 90 % of the strengths of 28-day-old specimens of the same concrete cured in moist air at a constant temperature of 21 °C (70 °F).*"

Thus, at 28 d, the strengths of concrete specimens cured in the field were required to be at least 90 % of the strengths of standard-cured specimens. This specification can be considered a "performance" specification, because it states the required level of performance, but it does not dictate how to achieve the goal. The ASTM specification on curing was withdrawn in 1945, and the ACI code provided the basic curing requirements for concrete construction. As discussed in the previous chapter, the ACI requirements were (and still are) "prescriptive" because they specify fixed curing times, irrespective of actual in-place strength development (or degree of hydration).

Obviously for a given concrete, the curing conditions play a major role in the strength development of the concrete as it matures over time. The specified design strength of concrete, f_c', is the basis for the design and construction of reinforced concrete structures. Concrete being placed in the field is sampled and tested under standardized procedures to ensure that it meets the required strength criteria. However, this testing only assures that the concrete delivered to the site has the required strength potential. Curing requirements are established to provide the necessary moisture and temperature conditions in the field for adequate strength development after concrete placement. Currently, U. S. curing requirements are based on research and experiences with concrete having compressive strengths less than about 40 MPa (6 000 psi) (Carino and Clifton 1991). Studies on the low water-cement ratio concretes now in use are needed to determine if these curing requirements are appropriate for these new concretes.

The latest version of the report of ACI Committee 308 on curing of concrete (ACI 308 1992) simply states that curing is necessary for the development of both strength and durability. There are no special curing requirements for low water-cement ratio concretes. In the chapter on "Curing Methods and Materials," the report recommends the following minimum moist curing duration, at a concrete temperature greater than 10 °C (50 °F), for concrete made with different types of cement:

- Type I cement: 7 d
- Type II cement: 14 d
- Type III cement: 3 d

However, in the discussion of curing duration for different types of construction, the report includes the following requirement for pavements and slabs on ground:

> *"For daily mean ambient temperatures above 40 F (5 C) the recommended minimum period of maintenance of moisture and temperature for all procedures is 7 days or the time necessary to attain 70 percent of the specified compressive or flexural strength, whichever period is less."*

A similar recommendation is given for curing duration of concrete in "structures and buildings." Thus there is a performance-based recommendation for the curing duration that differs from the prescriptive fixed duration given in the ACI 318 Code.

The 1992 report by ACI Committee 363 on high-strength concrete (ACI 363 1992) provides some general guidance and recommendations on curing to obtain desired strengths:

- *"Curing is extremely important in the production of high-strength concrete. To produce a cement paste with as high a solids content as possible, the concrete must contain the absolute minimum mix water. However, after the concrete is in place and the paste structure is established, water should be freely available, especially during the early stages of hydration."*
- *"Cement-rich mixtures frequently have very high water demands. Therefore, it is possible that special precautions may be necessary to provide adequate curing water, so that sufficient hydration can occur."*
- *"Water curing of high-strength concrete is highly recommended due to the low water-cement ratios employed."*

These represent recommendations that are directed specifically to one type of high-performance concrete. However, some of these recommendations are general and do not provide the specific guidance that would be of value when decisions have to be made about field curing procedures.

5.5 Durability Considerations

There may be fundamental differences between the curing requirements for strength and for durability. The strength of a structural element is affected by the concrete strength in the entire cross-sectional area that sustains the imposed load. Poor curing of the near-surface region would be expected to have a minor effect on the member strength, provided the core concrete experiences sufficient hydration. Since the "skin" provides a barrier to moisture egress, the core will continue to hydrate even if the surface dries below a critical level and hydration slows down. However, the durability of a concrete element is controlled primarily by the near-surface region, which provides a protective barrier against the ingress of harmful species. This protective quality is directly related to the degree of discontinuity of the capillary porosity. Thus it is expected that moist curing should be provided until the capillary pores become disconnected. The basic question that needs to be answered is whether different curing procedures are necessary for durability and strength. Standard curing requirements related directly to the durability of concrete are not established in the United States. Durability design standards will need to be developed first to define the performance levels for different exposure conditions. Meanwhile, researchers are working to develop rational curing requirements to achieve desired levels of durability-related properties[6].

Enhanced durability properties of high-performance concrete may be very important to the users, perhaps more important than high strength. Designers of structures with high-performance concrete should consider not just the strength requirements, but also the durability requirements for exposed surfaces (Bentz and Garboczi 1992). Many of the definitions of high-performance concrete cited earlier contain specific references to enhanced durability, as further testament to its significance. In Australia, the Roads and Transport Authority (RTA) has placed increased emphasis on the durability of concrete and incorporated durability considerations into the mixture proportioning provisions (Ho and Cao 1994). In a discussion of many of the issues concerning high-performance concrete, Aitcin emphasizes both the enhanced durability properties and the requirement for proper curing (Aitcin 1994). He states:

> *"When engineers will realize that concrete must no longer be specified in terms of compressive strength but rather in terms of its water-cement ratio, they can then begin to minimize durability problems. They will have, however, to make sure that this potentially durable concrete is adequately cast and cured."*

Durability is related to other important properties of concrete, besides strength, such as resistance to carbonation, porosity, permeability, and abrasion resistance. Studies to define curing requirements for required levels of durability must address the development of these durability-related properties. For example, if a given concrete attains a certain degree of impermeability, this may assure acceptable levels of durability in a particular application. The work of Hilsdorf (1995) deals with some of these other concrete properties, and Chapter 6 includes a discussion his work on curing.

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[6] Hilsdorf, H. K. and Burieke, F., "A Note on EN 206 -- Curing Requirements," Second Draft, March 1992.

5.6 Ideal Curing Conditions

No matter what curing requirement, standard, or other criterion is specified for a given project, rarely on a construction site is sufficient attention given to ensuring that proper curing procedures are followed. For structural concrete applications, Gilkey (1952) defined the following as ideal curing conditions:

- Continuously available free moisture at a sustained moderate temperature.
- Freedom from stress that approaches closely the strength developed at time of its application.
- Avoidance, upon termination of formal curing, of either rapid surface drying or of abrupt change of temperature (thermal shock) which would introduce potentially damaging differential volume changes within the concrete.
- Avoidance of exposure to freezing temperatures until curing has progressed far enough to partially empty the capillaries of their free moisture.

Even today, if these time-honored, ideal conditions were strictly enforced on the job sites, owners could be assured of high quality, well-cured structural concrete, whether it be ordinary concrete or high-performance concrete.

The basic obstacle is that proper curing (embodying the above principles) has a price. In today's competitive construction market, contractors are forced to cut costs wherever possible. Curing is often where cost reductions are taken. The situation will likely remain unchanged unless current practices are modified. Changes will probably have to be made in two areas:

- Rational performance-based criteria should replace current prescriptive criteria. These criteria should be well founded and supported by technical data.
- Standard procedures are needed that could be specified to measure in-place properties to assure that proper curing has been applied. These methods should be robust to simplify interpretation of results, and they need to address durability as well as strength.

These new approaches will undoubtedly increase construction costs, but they will provide assurance that the concrete will perform as expected. Therefore, the incentive for making these changes must come from the owners, who must be educated to recognize that these additional construction costs will provide long-term benefits, such as reduced maintenance costs and increased service life.

5.7 Accelerated Curing

As mentioned in Chapter 4, beginning with the 1971 ACI Code, there has been a section dealing with accelerated curing. The 1995 ACI Code states:

"Curing by high pressure steam, steam at atmospheric pressure, heat and moisture, or other accepted processes, shall be permitted to accelerate strength gain and reduce time of curing."

Although accelerated curing is used more frequently for precast members, this section also applies to cast-in-place concrete, if accelerated curing practices are used. A primary concern with accelerated curing is the potential for increased moisture loss during the curing process, as mentioned in the code commentary. Another concern is the possible detrimental effect on long-term properties from high temperatures. The ACI Code tries to address this latter concern with the following provision:

"Curing process shall be such as to produce concrete with a durability at least equivalent to the curing method of 5.11.1 or 5.11.2."

Sections 5.11.1 and 5.11.2 deal with the curing under ambient conditions.

There is limited information on how accelerated curing affects high-performance concrete. Some problems in strength gain have been noted in precast, silica-fume concrete members cured under accelerated conditions (Holland 1989). These problems were resolved, however, simply by allowing the concrete to attain initial setting prior to beginning the accelerated curing process. Accelerated curing has been shown to be effective in producing high-performance characteristics at early ages in silica-fume concrete (PCI Committee on Durability 1994). In this case, the additional heat is beneficial in enhancing the pozzolanic reaction of the silica fume. However, the heat greatly increases the moisture loss from exposed surfaces, which tends to cause more shrinkage problems. Therefore, when using accelerated curing with silica-fume concrete, the minimum amount of heat necessary for the required strength gain should be used, and the concrete should be allowed to attain initial setting prior to commencing accelerated curing.

5.8 Duration of Curing

Although the beneficial effects of good curing practices on any type of concrete are generally accepted by all, how long structural members should be cured is still open to question. As has been shown, the duration requirement in the ACI code for normal strength-gain concrete has historically been a minimum of 7 d, and at least 3 d for high early strength concrete. Certainly, the curing temperature and the kinetics of the hydration and pozzolanic reactions of the particular cementitious materials will affect the required duration of curing to attain a certain level of maturity. With respect to silica-fume concrete, some researchers have advocated prolonged curing to realize the full benefits of this material (Holland 1989; Ayers and Khan 1994). However, such recommendations are based more on professional judgement rather than data.

5.8.1 *Temperature effects*—The curing temperature affects the rate of hydration of the cement and the reaction of pozzolans, and, therefore, it affects strength development of the concrete. Thus temperature has an important effect on the curing duration required to achieve a specific strength or durability-related characteristic. The *maturity concept* is a simple tool to account for effects of time and temperature on property development. The maturity concept has been shown to be applicable to mixtures that are characteristic of high-performance concrete (Carino et al. 1992). This concept will be discussed further in Chapter 6 since it may have a role in establishing rational curing criteria for high-performance concrete. It may be feasible to determine required curing durations for high-performance concrete based on attaining a certain level of maturity. In this case, the duration of the curing period will depend on in-place temperature history.

Starting with the 1963 ACI Code, the requirement was added that concrete be cured at a temperature above 10 °C (50 °F). The minimum curing durations of 7 d for normal strength-gain concrete and 3 d for high early strength were based on this temperature requirement. It has remained an important element of the curing criteria in every code since 1963. However, there is no technical reason for requiring this minimum temperature provided the concrete is protected from freezing and the curing duration is adjusted based on the temperature of the concrete.

5.8.2 *Prolonged curing*—Prolonged curing of silica-fume concrete has been recommended to ensure optimum results. The term "over curing" is used in the literature, and it means that the concrete is cured for a longer period than would be the case with conventional concrete in an identical application. Over curing has been advocated because of the importance of curing in these silica-fume mixtures in which the water-cement ratio is typically quite low (Holland 1989). The Prestressed Concrete Institute specifically recommends over curing for high-performance concrete that contains silica fume (PCI Committee on Durability 1994). As mentioned, over curing has been recommended to assure that all the enhanced properties of the concrete will be realized. There has also been a tendency to err on the safe side, since the body of knowledge on how to cure silica-fume concrete most effectively and efficiently is limited. The safest course of action has been to advocate over curing (Ayers and Khan 1994).

5.8.3 *Rational parameters*—Recent work by Hilsdorf (1995) on the effects of curing on concrete properties lists the following key parameters as those which influence the duration of curing:

- Curing sensitivity of the concrete as influenced by its composition,
- Concrete temperature,
- Ambient conditions during and after curing, and
- Exposure conditions of the structure in service.

These very logical parameters encompass a modern view of the fundamental factors that are important. Hilsdorf's studies are discussed in more detail in Chapter 6, since they may be useful in the development of specific curing requirements for high-performance concrete.

5.9 Early-Age Curing of Silica-Fume Concrete

In 1996, ACI Committee 234 prepared a guide on the use of silica fume in concrete (ACI 234R 1996). A particular concern with silica-fume concrete is the prevention of plastic shrinkage cracking. These cracks develop on the exposed surfaces of newly placed concrete when the rate of evaporation exceeds the rate at which water appears on the surface due to bleeding. Due to the reduced tendency for bleeding, silica-fume concrete is more prone to plastic shrinkage cracking than ordinary concrete. Such cracking may develop any time between the completion of initial finishing and the time of final setting. In addition, cracking may occur under conditions less severe than implied by the evaporation chart in ACI 305R or ACI 308. Therefore, control of plastic shrinkage cracking is a critical aspect in the use of silica-fume concrete, and newly placed surfaces must be prevented from drying.

The ACI 234 guide offers the following recommendations related to curing and prevention of plastic shrinkage cracking in silica-fume concrete:

- To obtain the full benefits of silica fume, proper curing procedures must be followed. Extra attention is required because of typically low water-cementitious material ratios.
- Chlorinated-rubber based curing compounds have proven to be effective. Curing compounds should be added immediately after finishing to protect against plastic shrinkage cracking.
- An alternative to using curing compounds is to cover with wet burlap and plastic sheeting. However, there is a need for additional measures to protect against plastic shrinkage cracking (fog misting or evaporation retarder) until wet burlap and plastic sheeting can be applied. Maintain wet burlap and plastic covering for at least 3 d and preferably 5 d to 7 d.
- After form removal, exposed surfaces should be coated with curing compound or, if feasible, covered with wet burlap and plastic sheeting.
- To prevent drying of newly placed surfaces, a compressed-air water-misting device is recommended to apply a thin coating to produce a surface sheen. Do not use excess water from misting for finishing purposes.
- Cover concrete between initial screeding and subsequent finishing operations. Evaporation retarders can be used to prevent rapid evaporation.
- Avoid excessive use of set retarders, which may lead to the formation of a spongy crust that is highly susceptible to plastic shrinkage cracking.

In summary, formation of plastic shrinkage cracks can be prevented in silica-fume concrete by taking precautions to assure that the surface of the concrete is not permitted to dry at any time prior to application of final curing measures.

5.10 Curing for Different Types of Construction

Although the ACI Code requirements for curing do not address different types of concrete construction since the Code is intended primarily for building construction, some curing practices are better suited to one form of construction than another. Considerations of this nature can be thought of as best field practices in relation to the type of construction. The report of ACI Committee 308 gives guidance on best curing practices and curing durations for conventional concrete construction (ACI 308 1992). The report is called a "standard practice" but it is not written in mandatory language. Thus it can be used only as a guide in the preparation of project plans and specifications; it cannot be referenced in contract documents. In 1996, ACI adopted a greatly revised version of the standard specifications for structural concrete (ACI 301 1996). The standard specification includes curing requirements that can serve as alternatives to those in the ACI Code. Because the standard specification and is written in mandatory language, it can be referenced and cited directly by the Architect/Engineer in the project specifications.

5.10.1 *Pavements and slabs on ground*—Through the years, different methods of curing have been used for concrete pavement and slabs. Because pavements are horizontal structures, they are amenable to a variety of water-curing methods. Some of the popular materials for water curing have included saturated coverings of earth, hay, straw, jute mats, and multi-layered fabrics. The most effective methods for controlling moisture loss are sprinkling and ponding (Robinson 1952). Curing compounds have also been popular and are permitted in many jurisdictions in highway construction, as well as in building floor slabs. They are often the preferred method due to ease of application and overall economy compared with other curing methods.

Historically, the first ACI committee on curing recommended covering concrete pavements with two thicknesses of a woven fabric, a quilted fiber mat, or other absorptive material saturated with water (ACI 612 1958). Liquid curing compounds sprayed on the surface were also acceptable for the final curing periods (day 5 through day 10).

In more recent times, a survey of state highway departments found that the most common curing method for highway pavements and bridge decks is membrane-forming curing compounds (Senbetta 1988). This same survey confirmed that some state specifications for curing have serious deficiencies, providing further evidence that curing has not received the attention that it deserves in either horizontal or vertical construction.

Work on bridge-deck overlays by the Virginia Department of Transportation using silica-fume concrete found that insulating blankets were very effective in retaining the moisture and the heat of hydration, both of which enhance the pozzolanic reactivity of the silica fume (Ozyildirim 1991). Insulation blankets also reduce temperature differences between the surface and interior of the pavement or slab and reduce the likelihood of cracking due to thermal gradients.

The 1992 report of ACI Committee 308 recommends either sealing the surface with sheets or membranes, or continuous moist curing using wet burlap, cotton mats, rugs, or similar materials. For daily mean ambient temperatures above 5 °C (40 °F), the required minimum curing period is 7 d or the time required to attain 70 % of the specified strength, whichever is less. If the concrete is placed when the daily mean ambient temperature is 5 °C (40 °F) or lower, special precautions are necessary to protect the concrete against damage due to freezing.

5.10.2 *Structures and buildings*—Historically, the best curing methods for building construction have been those that maintain a steady supply of moisture to the structural member (water curing) while sustaining a moderate temperature (Gilkey 1952). More recently, the ACI 308 report provides recommendations based on current practices, and the ACI 301 specification gives specific requirements for curing and protection of building structures.

The report of ACI 308 recommends a wide range of curing methods for concrete building structures. All of the common water curing methods are mentioned—ponding, spraying, sprinkling, and saturated cover materials—as well as the use of sealing materials—plastic film, reinforced paper, and liquid membrane-forming compounds. The recommended curing duration is the same as for pavements and slabs on the ground given above, except for those key members, such as columns, designed for high strength of 41 MPa (6 000 psi) or greater. For these members, concrete strength is crucial for structural safety, and they may require longer curing periods of 28 d or more to assure that the design strengths are attained.

The ACI 301 standard specification provides the following requirements (in Section 5.3.6) related to curing and protection of cast-in-place structural concrete:[7]

5.3.6 *Curing and protection*

5.3.6.1 *General*—Immediately after placement, protect concrete from premature drying, excessively hot or cold temperatures, and mechanical injury. Protect concrete during the curing period such that the concrete temperature does not fall below the requirements of 4.2.2.7—Concrete temperature. Cure concrete in accordance with 5.3.6.2 or 5.3.6.3 for 7 days after placement. High early strength concrete shall be cured for 3 days after placement.

Alternatively, moisture retention measures may be terminated when:

- Tests are made on at least two additional cylinders kept adjacent to the structure and cured by the same methods as the structure, and tests indicate 70 percent of the specified compressive strength f_c' as determined in accordance with ASTM C 39 has been attained.
- Temperature of the concrete is maintained at 50 F or higher for the time required to achieve 85 percent of f_c' in laboratory-cured cylinders representative of the concrete in place.
- Strength of concrete reaches f_c' as determined by accepted non-destructive methods meeting the requirements of 2.3.4.2.

[7] Extensive quotations from specifications, codes, and other standards will be shown in **Arial font**.

When one of the curing procedures in 5.3.6.4—Preservation of moisture, is used initially, the curing procedure may be replaced by one of the other procedures when concrete is one day old, provided concrete is not permitted to become surface dry at any time. When specified in the Contract Documents, use a curing procedure of 5.3.6.4 that supplies additional water.

During and following curing, do not allow the surface of the concrete to change temperature more than the following:

- 50 F in any 24-hr period for sections less than 12 in. in the least dimension.
- 40 F for sections from 12 to 36 in. in the least dimension.
- 30 F for sections 36 to 72 in. in the least dimension.
- 20 F for sections greater than 72 in. in the least dimension.

The method of temperature measurement shall be acceptable.

5.3.6.2 *Unformed concrete surfaces*—Apply one of the procedures in 5.3.6.4—Preservation of moisture, after completion of placement and finishing of concrete surfaces not in contact with forms.

5.3.6.3 *Formed concrete surfaces*—Keep absorbent wood forms wet until they are removed. After form removal, cure concrete by one of the methods in 5.3.6.4—Preservation of moisture.

5.3.6.4 *Preservation of moisture*—After placing and finishing, use one or more of the following methods to preserve moisture in concrete:

a. Ponding or continuous fogging or continuous sprinkling.
b. Application of mats or fabric kept continuously wet.
c. Continuous application of steam (under 150 F).
d. Application of sheet materials conforming to ASTM C 171.
e. Application of a curing compound conforming to ASTM C 309 or Federal Specification TT-C-800. Apply the compound in accordance with manufacturer's recommendation after water sheen has disappeared from the concrete surface and after finishing operations. The rate of application shall not exceed 200 ft^2/gal. For rough surfaces, apply in two applications at right angles to each other, not to exceed 200 ft^2/gal. for each coat. Do not use curing compound on any surface where concrete or other material will be bonded unless the curing compound will not prevent bond or unless measures are to be taken to completely remove the curing compound from areas to receive bonded applications.
f. Application of other accepted moisture-retaining method.

Section 4.2.2.7 (concrete temperature) is referenced in regard to the minimum concrete temperature during the specified curing periods of 7 or 3 d. These temperatures are from the portion of ACI 301 that deals with concrete temperature immediately after placement during cold weather. The following is the wording in Section 4.2.2.7:

4.2.2.7 *Concrete temperature*—When the average of the highest and lowest temperature during the period from midnight to midnight is expected to drop below 40 F for more than three successive days, concrete shall be delivered to meet the following minimum temperature immediately after placement:

- 55 F for sections less than 12 in. in the least dimension
- 50 F for sections 12 to 36 in. in the least dimension
- 45 F for sections 36 to 72 in. in the least dimension
- 40 F for sections greater than 72 in. in the least dimension

The temperature of concrete as placed shall not exceed these values by more than 20 F.

These minimum requirements may be terminated when temperatures above 50 F occur during more than half of any 24 hr duration.

Unless otherwise specified or permitted, the temperature of concrete as delivered shall not exceed 90 °F.

These temperatures are based on the ACI standard specification for cold weather concreting (ACI 306.1 1990). The temperatures after placement are permitted to be lower for members that are more massive because a higher temperature rises would occur due to the heat of hydration. ACI 301 has taken these minimum temperatures immediately after placement and specified them as the minimum concrete temperatures during the specified curing periods of 7 or 3 d. The ACI 301 specification does specify how the temperature of hardened concrete is to be measured. Most likely, the intent is that the minimum concrete temperature during the curing period is the surface temperature of the member. However, there is no standard test method for measuring surface temperature of hardened concrete.

Section 2.3.4.2 is referenced in regard to techniques that are permitted for assessing the in-place strength. These are alternative methods to testing field-cured cylinders, and the language in 2.3.4.2 is as follows:

2.3.4.2 Alternatively, when specified, use of the following methods for evaluating concrete strength for formwork removal is permitted. Prior to using methods in 2.3.4.2.a through 2.3.4.2.d, submit sufficient data using job materials to demonstrate correlation of measurements on the structure with the compressive strength of laboratory cured molded cylinders or drilled cores. Correlation data for each alternative method for determining strength shall be submitted for acceptance.

2.3.4.2.a Tests of cast-in-place cylinders in accordance with ASTM C 873. This is limited to slabs with concrete depth from 5 to 12 in.

2.3.4.2.b Penetration resistance in accordance with ASTM C 803.

2.3.4.2.c Pullout strength in accordance with ASTM C 900.

2.3.4.2.d Acceptable maturity-factor procedure in accordance with ASTM C 1074.

Note that if field-cured cylinders are used to assess in-place strength, the required compressive strength of the cylinders is 70 % of f_c', but if the so-called nondestructive methods are used, the required strength is f_c'. This difference is probably to account for the uncertainty in the estimated compressive strength based on the nondestructive test methods.

5.10.3 *Mass concrete*—Internal curing temperatures will be high in massive structures, such as dams, because the heat of hydration cannot be easily dissipated. High-performance concrete is expected to attain higher internal temperatures than ordinary concrete because the cement content will be higher.

ACI 308 recommends the use of water curing to keep the surfaces of mass concrete structures continuously wet. Methods specifically mentioned include spraying, wet sand, or water-saturated fabrics. Liquid membrane-forming curing compounds are recommended if the surface is not a construction joint. If used at construction joints, the membrane has to be removed by sandblasting prior to placing the adjacent concrete. A curing duration of not less than two weeks is recommended for unreinforced sections containing no pozzolans.

When pozzolans are used, the minimum time for curing is three weeks. For heavily reinforced sections, the requirement is for a minimum of 7 d of continuous curing or until 70 % of the specified strength is attained.

ACI 301 requires that mass concrete be cured and protected as specified below:

8.3.2 *Curing and protection*

8.3.2.1 Preservation *of moisture*

8.3.2.1.a Cure mass concrete for the minimum curing period specified in 5.3.6—Curing and protection, unless Contract Documents require longer curing.

8.3.2.1.b When a specific curing method is not specified in the Contract Documents, preserve the moisture either by maintaining the forms in place or, for surfaces not in contact with forms, by applying one of the procedures specified in 5.3.6.4—Preservation of moisture.

8.3.2.2 *Cold weather concrete placement*—Protect the concrete from freezing and moisture loss for the required curing period in accordance with 5.3.6.1—General curing and protection. Do not use steam or other curing methods that will add heat to the concrete.

8.3.2.3 *Hot weather* concrete *placement*—Keep forms and exposed concrete continuously wet during the curing period whenever the surrounding air temperature is above 90 °F.

8.3.2.4 *Control of concrete surface temperature*—Unless otherwise specified, cool the concrete gradually so that the drop in concrete surface temperature during and at the conclusion of the specified curing period does not exceed 20 F in any 24-hr period.

5.10.4 *Specialized construction*—There are other types of concretes used in construction that require that special considerations be given to curing. These may include precast concrete; vertical slipform construction; shotcrete; refractory concrete; cement paint, stucco, and plaster; shell structures; insulating concrete; and concrete with a colored or hardened surface (ACI 308 1992). Due to the unique nature of some of these materials and their properties, they may have individual curing requirements somewhat different from conventional concrete. These requirements are described in various committee reports and guides in the ACI Manual of Concrete Practice, and are not reviewed further in this report.

5.11 ACI Standard Specification on Curing Concrete (ACI 308.1-98)

In 1998, the American Concrete Institute adopted a standard specification on curing of concrete (ACI 308.1 1998). This is a mandatory-language document that can be referenced in project specifications, and is intended to be modified by the specifier as necessary to suit the specific project. Although the specification was adopted after the bulk of this report was completed, some of the key features are summarized here for the sake of completeness.

The requirements in the ACI curing specification are similar, in many respects, to those in the ACI 301 specification discussed in the previous section. However, there are notable differences, for example, ACI 308.1-98 contains the following definitions:

Curing period—Duration of time in which continuous curing procedures are employed. (Note: The curing period includes initial and final curing stages.)

Curing, initial—Deliberate action taken between placement and final finishing of concrete to reduce the loss of moisture from the surface of the concrete.

Curing, final—Deliberate action taken between the final finishing and termination of curing.

By introducing the term "initial curing," the specification deals specifically with curing to reduce plastic shrinkage cracking. Another interesting feature is the use of the words "deliberate action." This makes it clear that curing involves doing something to the concrete during curing period.

Section 1 of ACI 308.1-98 gives the following general requirements for the curing period:

1.1.6 *Curing period*—Cure the concrete for the following time periods:

1.1.6.1 When testing is not performed to determine the curing period, cure concrete for at least 7 days provided that the concrete surface temperature is at least 10 C (55 F).

1.1.6.2 When strength basis testing is performed to determine the curing period, maintain curing procedures until test results meet or exceed requirements of Paragraph 1.6.4.2.

1.1.6.3 When durability basis testing is performed to determine the curing period, maintain curing procedures until test results meet or exceed Paragraph 1.6.4.3

The notes in the Specification Checklist portion of the standard that deal with paragraph 1.1.6.1 (no testing performed) instruct the Architect/Engineer to specify an alternative value of the minimum duration of curing if the default value of 7 d is not acceptable. The note further states that the Architect/Engineer needs to consider the mixture proportions, environmental conditions, and skill level of the contractor factors in selecting the minimum curing duration. The wording in 1.1.6.3 provides a specific requirement when durability is of concern rather than strength.

Section 1.6.4 supplements Section 1.1.6 by giving the following requirements related to tests for determining time for termination of curing methods:

1.6.4.1 *General*—Tests to determine time of termination for curing measures may be performed by a testing agency acceptable to the Architect/Engineer

1.6.4.2 Strength *basis*—When termination of curing measures is based on the development of strength, curing measures shall not be terminated before the compressive strength of the concrete has reached 70 percent f_c' as determined by one of the following methods:

1.6.4.2.a *Compressive strength basis*—Mold cylinders in accordance with ASTM C 31 and test in accordance with ASTM C 39. Maintain curing until tests of at least two cylinders, field-cured alongside the concrete they represent, have reached the compressive strength specified for termination of curing.

1.6.4.2.b *Maturity method basis*—Maintain curing methods until concrete attains the compressive strength specified for termination of curing, as estimated in accordance with ASTM C 1074.

1.6.4.2.c *Nondestructive test methods*—Maintain curing methods until testing indicates that the specified compressive strength has been reached.

1.6.4.3 Durability basis—Maintain curing methods until specified results are achieved.

The notes in the Specification Checklist dealing with paragraph 1.6.4.2 instruct the Architect/Engineer to specify the compressive strength that must be achieved prior to termination of curing if the default value of 70 % of f_c' is not acceptable. The notes to paragraph 1.6.4.3 instruct the Architect/Engineer to specify the durability-related properties, the test method(s) to be used, and the test results to be achieved.

The ACI 308.1-95 curing specification also addresses curing to minimize plastic shrinkage cracking. The section dealing with curing during cold weather states the following:

Use initial curing method or methods defined in Paragraph 1.8 to avoid plastic shrinkage cracks.

Paragraph 1.8 of the standard specification deals with curing during hot weather, and includes the following requirements:

1.8.1 This covers protection and additional curing requirements that are to be implemented during hot weather. Use initial curing method or methods to avoid plastic shrinkage cracks.

1.8.2 During the initial curing period use evaporation reducers, fogging, or shade (individually or in combination) to control the rate of bleed water evaporation and subsequent drying of the concrete.

...

1.8.4.4 Prevent drying of the concrete prior to the application of final curing methods by using the appropriate initial curing method. When necessary to prevent drying of the concrete surface, further reduce the loss of moisture from the concrete by shading concrete mixers, formwork, reinforcing steel, and concrete from direct sunlight, and by erecting windbreaks, or a combination of such methods. Place and finish concrete at night when loss of moisture from the concrete cannot be controlled by the above measures, and when evaporative conditions are less severe than in the daytime.

1.8.4.5 Use one of the two following methods for initial curing:

a. Use fog spray as specified in Section 5 of this Specification.

b. Use entrapment of the bleed water on the concrete surface under a uniform distribution of an evaporation-retardant film. Place the evaporative-retardant film between the different finishing operations. Do not work the liquid film material into the paste during subsequent finishing operations. Do not work water to the surface of the concrete in the finishing process.

1.8.4.6 Perform final curing methods immediately upon completion of the final finishing operation. Final curing may be performed using any of the methods described in this Specification

Section 2 through Section 5 of the curing specification provide requirements for products and procedures for different curing methods as follows:

Section 2—Moisture retention by using plastic sheets, plastic sheets bonded to water-absorbent fabric, or reinforced paper.
Section 3—Moisture retention by using liquid membrane-forming compounds.
Section 4—Addition of water by ponding or immersion.
Section 5—Addition of water by fog spray.

Section 6—Addition of water by sprinkling or soaker hoses.

Section 7—Addition of water by using water absorbent materials (sand, hay, burlap, cotton mats, rugs or earth).

The ACI 308.1-98 curing specification provides a framework for tailoring the curing period to suit the needs of the specific project. A curing period less than 7 d is permitted provided testing is performed to confirm adequacy of property development. However, the specification places a burden on the Architect/Engineer to specify alternative criteria for termination of curing. In addition, the specification addresses curing when durability is of concern instead of structural strength, but it does not provide for methods to assess whether durability-related properties are achieved. Again it is the responsibility of the specifier to establish the criteria for adequate curing for durability.

5.12 Applicability of Current Requirements to High-Performance Concrete

The current curing practices and standards have been developed based on studies primarily related to strength development characteristics of conventional (ordinary) concretes. As mentioned earlier, most high-performance concretes are fundamentally different from conventional concrete, because they will typically have a low water-cement ratio and one or more admixtures. Silica fume is a common additive for high strength and low permeability, and high range water-reducers are used to provide workability. Since the composition of high-performance concrete differs from conventional mixes, early-age characteristics of the hydrating paste will also differ. Therefore, existing curing may not be optimal for this class of concrete. A better understanding is needed of the role of an external supply of moisture and of the adequacy of membrane-forming compounds when low water-cement ratios are involved.

The effects of self-desiccation are important considerations in high-performance concretes with low water-cement ratios. To prevent self-desiccation during the curing period, water that is consumed by hydration needs to be replaced by the ingress of external moisture. Early-age, continuous curing may be critical for providing the necessary water. As hydration proceeds, the capillaries become discontinuous, thus effectively preventing the ingress of additional water into the concrete. When this state is reached, additional curing may be of little, or no, benefit, because the water may not be able to penetrate to the interior quickly enough to maintain saturation of the capillaries. This leads to early cessation of the hydration reactions within the paste. Current curing requirements, based on research on conventional concrete, do not consider these factors. Thus it may be necessary to develop new curing requirements for this new, modern class of concrete.

Tests on high-strength concrete with silica-fume have shown that the positive effects from the silica fume occur earlier after placement, within the timeframe of the first 28 d, than from other pozzolans. Silica-fume concrete with a low water-cement ratio has a slower rate of drying than normal concrete, and is able to form a dense microstructure after only

7 d of moist curing (Bentur and Goldman 1989). After about 56 d, silica-fume concrete will gain little additional strength, probably due to the effects of self-desiccation (Hooton 1993). Another key difference between silica-fume concrete and ordinary concrete is that the former has a lower tendency to bleed; therefore, finishing can be done immediately after placement, and curing can begin sooner. In ordinary concrete, the common practice is to delay finishing until the bleed water has disappeared or been removed. Since the characteristics of fresh silica-fume concrete are quite different from those of conventional concrete, there is a special need to examine the applicability of current curing practices to this popular type of high-performance concrete.

There is general agreement in the literature that proper curing is critical for high-performance concrete to develop its full potential (ACI 363 1992; Shah and Ahmad 1994; Zia, Leming, and Ahmad 1991; Neville 1996). However, there are some remaining questions, such as whether low water-cement ratio concrete is more sensitive to poor curing practices than ordinary concrete. Some of the recent research results are inconsistent (Hasni, Gallias, and Salomon 1994; Torii and Kawamura 1994). There is no agreement on what constitutes the optimum curing practice for high-performance concrete. In this case, the term “optimum” means striking a balance between economy and the development of concrete properties.

Chapter 6 provides an in-depth review of recent research on curing of high-performance concrete.

5.13 Curing Requirements in Other Countries

This section presents the curing requirements in the standards of other selected countries for comparison with those in the United States. The standards reviewed include those of Canada, United Kingdom, Norway, CEB-FIP, European Community, and Australia.

5.13.1 *Canada*—The Canadian Standard on concrete construction (CSA23.1-94) has the following curing requirements:

21.1 Curing

21.1.1 General

Freshly deposited concrete shall be protected from freezing, abnormally high temperatures or temperature differentials, premature drying, and moisture loss for the period of time necessary to develop the desired properties of the concrete.

Notes:

(1) *For information on curing, refer to ACI Standard 308.*

(2) *For structural concretes with slow strength gain characteristics at early ages, for high-strength concrete, or for other structural concretes requiring special curing conditions, the Owner should specify such conditions in the contract documents.*

21.1.2 Basic Curing Period

Concrete surfaces shall be cured for either 3 d at a minimum temperature of 10 °C or for the time necessary to attain 35 % of the specified 28 d compressive strength of the concrete.

Notes:

(1) *Concrete strength can be assessed by testing field-cured cylinders or by using nondestructive testing methods as covered in Clause 17.*

(2) *Following the cessation of moist curing, the development of strength continues for a short time, provided temperature conditions are favourable. Strength development will also be reactivated if moist curing is resumed.*

21.1.3 Additional Curing for Durability

Concrete for exposure classifications F-1, C-1, C-2, S-1, S-2, and concrete exposed to severe abrasion and to air pollution in heavy industrial areas as defined in Clause 15[8] shall, immediately following the basic curing period, be cured for either an additional 4 consecutive days at a minimum temperature of 10 °C or for the time necessary to attain 70 % of the specified 28 d compressive strength of the concrete.

Note: *At the end of the additional curing for concrete of C-1 and C-2 classes of exposure, a period of at least one month of air drying should elapse before the application of deicing chemicals on the concrete.*

21.1.4 Additional Curing for Structural Safety

The basic curing defined in Clause 21.1.2 shall be extended until the concrete has achieved sufficient strength for structural safety. The compressive strength level required for structural safety shall be specified by the Owner.

21.1.5 Additional Curing for Mass Concrete

For reinforced massive sections, the curing period specified in Clause 21.1.2 shall be extended for an additional 4 consecutive days. In unreinforced massive sections, the basic curing period shall be extended an additional 7 consecutive days.

Table 5.1 summarizes the curing requirements given in the CSA23.1 standard. In contrast with the ACI Code, the Canadian standard includes both prescriptive and performance requirements. Also, there is a specific requirement for durability, however, the performance criterion is given in terms of compressive strength. This reflects the fact that there is no simple, reliable standard test method to evaluate the in-place durability potential. Therefore, the measurement of compressive strength is chosen as the simplest method to show that a certain level of hydration has occurred. The requirement for structural safety is written in general terms, and it places the burden on the owner to establish the required strength at the end of the curing period. Since it is not stated specifically, the curing requirements for structural safety could be interpreted to cover both safety during construction, that is, during formwork removal, application of prestressing, etc., and during service.

Other sections of the Canadian standard that are applicable to curing address such areas as methods and materials of curing, curing under extreme temperatures, and curing for accelerated strength development. With respect to high-strength concrete, the note in Section 21.1.1 indicates that the Owner should specify the required curing conditions in the

[8] Specific exposure classifications and special provisions of Clause 15 are defined in another section of the standard.

Table 5.1 Summary of CSA23.1-94 Curing Requirements for Different Conditions

Condition	Requirements
Basic curing	3 d at T ≥ 10 °C
Durability	Additional 4 d at T ≥ 10 °C or Time to reach 75 % of specified 28-day compressive strength
Structural safety	Additional time until sufficient strength is attained; Owner specifies required strength
Mass concrete Unreinforced Reinforced	 Additional 7 consecutive days Additional 4 consecutive days

contract documents. Thus the burden is on the Architect/Engineer to provide the curing requirements for high-strength concrete. The question is: What information will the Architect/Engineer rely on to provide guidance on establishing optimal requirements?

5.13.2 *United Kingdom*—In the United Kingdom, British Standard BS 8110 (1985), the code of practice for design and construction of concrete structures, provides the following requirements for curing:

6.6 Curing

6.6.1 General

Curing is the process of preventing the loss of moisture from the concrete whilst maintaining a satisfactory temperature regime. The prevention of moisture loss from the concrete is particularly important if the water/cement ratio is low, if the cement has a high rate of strength development, if the concrete contains g.g.b.f.s. [ground granulated blast furnace slag] or p.f.a. [pulverized fuel ash], and when curing mortars. The curing regime should also prevent the development of high temperature gradients within the concrete.

The rate of strength development at early ages of concrete made with supersulphated cement is significantly reduced at lower temperatures. Supersulphated cement concrete is seriously affected by inadequate curing and the surface has to be kept moist for at least 4 days.

Curing and protection should start immediately after the compaction of the concrete to protect it from:

(a) premature drying out, particularly by solar radiation and wind;
(b) leaching out by rain and flowing water;
(c) rapid cooling during the first few days after placing;
(d) high internal thermal gradients;
(e) low temperature or frost;
(f) vibration and impact which may disrupt the concrete and interfere with its bond to the reinforcement.

Where members are of considerable bulk or length, the cement content of the concrete is high, the surface finish is critical, or special or accelerated curing methods are to be applied, the method of curing should be specified in detail.

6.6.2 Minimum periods of curing and protection

Surfaces should normally be cured for a period not less than that given in Table 6.5. Depending on the type of cement, the ambient conditions and the temperature of the concrete, the appropriate period is taken from Table 6.5 or calculated from the last column of that table. During this period, no

Table 6.5 Minimum periods of curing and protection (see also 6.6.1)

<table>
<tr><th rowspan="3">Type of cement</th><th rowspan="3">Ambient conditions after casting</th><th colspan="3">Minimum periods of curing and protection</th></tr>
<tr><th colspan="3">Average surface temperature of concrete</th></tr>
<tr><th>5 °C to 10 °C</th><th>Above 10 °C</th><th>T (any temperature between 5 °C and 25 °C)</th></tr>
<tr><td rowspan="2">OPC, RHPC, SHPC</td><td>Average</td><td>Days
4</td><td>Days
3</td><td>Days
$\frac{60}{T + 10}$</td></tr>
<tr><td>Poor</td><td rowspan="2">6</td><td rowspan="2">4</td><td rowspan="2">$\frac{80}{T + 10}$</td></tr>
<tr><td rowspan="2">All except RHPC, OPC and SRPC and all with g.g.b.f.s. or p.f.a.</td><td>Average</td></tr>
<tr><td>Poor</td><td>10</td><td>7</td><td>$\frac{140}{T + 10}$</td></tr>
<tr><td>All</td><td>Good</td><td colspan="3">No special requirements</td></tr>
<tr><td colspan="5">Note 1. Abbreviations for the type of cement used are as follows:
OPC Ordinary Portland cement (see BS 12)
RHPC Rapid-hardening Portland cement (see BS 12)
SRPC Sulphate-resisting Portland cement (see BS 4027)
Note 2. Ambient conditions after casting are as follows:
Good Damp and protected (relative humidity greater than 80 %; protected from sun and wind);
Average Intermediate between good and poor;
Poor Dry or unprotected (relative humidity less than 50 %; not protected from sun and wind).</td></tr>
</table>

part of the surface should fall below a temperature of 5 °C. The surface temperature is lowest at arrises and depends upon several factors, including the size and shape of the section, the cement content of the concrete, the insulation provided by the formwork or other covering, the temperature of the concrete at the time of placing and the temperature and movement of the surrounding air. If not measured or calculated, the surface temperature should be assumed equal to the temperature of the surrounding air.

Other sections of the British standard address methods of curing and special concreting conditions such as in cold weather and hot weather. The standard does not specifically address high-performance concrete curing requirements, nor does it distinguish between requirements for durability and for strength.

Examination of Table 6.5 of BS 8110: Part 1: 1985 shows that the standard tries to account for some of the main factors affecting curing duration. For example, mixtures containing fly ash or blast furnace slag are required to be cured for longer periods to account for the reduced rate of strength development of concrete containing these materials. Ambient conditions are taken into account by requiring a longer curing period with decreasing ambient relative humidity. Note that if the ambient relative humidity exceeds 80 % and the concrete is protected from sun and wind, no additional curing procedures are necessary. The standard accounts for the effect of temperature on the rate of hydration by specifying the curing duration as a function of the concrete surface temperature. The Nurse-Saul maturity function, with a datum temperature of -10 °C (Carino 1991), is used to establish the minimum duration of the curing period for surface temperatures between 5 °C and 25 °C. Thus the numbers 60, 80, and 140 in the last column of Table 6.5 represent *temperature-time factors* in °C·d. These temperature-time factors depend on the cementitious system and the exposure conditions.

5.13.3 *Norway*—In Norway, Norwegian Standard 3420 provides the following curing requirements:[9]

Curing

After setting, the concrete shall be kept moist for at least 3 days, either by water curing or by prevention of evaporation from the concrete.

For a water-cement ratio less than 0.4, water curing should be used.

Where there are special requirements for impermeability, absence of cracks and resistance to chemical attack, the concrete should be kept moist for at least 2 weeks unless otherwise specified. Subsequent drying should be at a slow rate.

Artificial drying of the concrete should not take place until the concrete has an in-situ strength of at least 70 % of specified 28-day strength.

Where silica fume or large amounts of fly ash or filler are used, special attention should be paid to proper curing in order to prevent early drying which will lead to plastic shrinkage. Where specific requirements for early strength are in place, it shall be verified that strength development is satisfactory.

If necessary, steps shall be taken to prevent temperature induced cracking.

In structures for exposure class *Very Aggressive* (MA), curing temperature shall not exceed 65 °C.

Other sections of the Norwegian standard address special conditions such as cold weather concreting and control of temperature until a certain strength level is attained.

The Norwegian curing requirements are rather concise but they address some of the issues related to curing low water-cement ratio concretes. Curing by “sealing” methods is not permitted when the water-cement ratio is less than 0.4. Undoubtedly, this is to mitigate the effects of self-desiccation at early ages. There is an early-age curing requirement to deal with the problem of plastic shrinkage cracking in mixtures with low bleeding, such as those containing silica-fume. While the minimum curing time is 3 d, there is a particularly stringent requirement for extended curing of at least 14 d when the concrete must meet special impermeability requirements. A unique requirement is the restriction on the maximum curing temperature for structures exposed to aggressive environments. Presumably, this is to control the formation of microcracking from thermally induced stresses. Another particular feature is the requirement to verify in-place strength when there are specific early-age strength requirements. In general, the curing requirements place an emphasis on durability rather than strength. It should be mentioned that Norway probably

[9] Mr. Per Fidjestol, Elkem Materials, provided translation of Section 08 of the Norwegian standard . Thus the wording shown here may not be the exact wording in that standard.

has the most experience in the construction of high-strength concrete structures exposed to severe environments, namely the North Sea oil platforms. To date there have not been any reports of major durability problems in these structures.

5.13.4 *Euro-International Committee for Concrete-International Federation for Prestressing (CEB-FIP)*—The CEB-FIP Model Code 1990 is intended primarily to serve as an aid and guide to drafters of national codes in the member countries. For this reason, it is written in more general terms than would be a specific national code. This model code is also intended to help standardize and harmonize basic criteria for the design and construction of concrete structures throughout the member countries. Appendix d, Concrete Technology, deals with curing and protection of concrete, and that information is given below.

d.12 Curing and Protection

d.12.1 General Considerations

In order to obtain the potential properties to be expected from the concrete, thorough curing and protection over an adequate period of time are vital. It is essential that curing and protection start immediately after compaction of the fresh concrete.

In this context curing is a measure to avoid premature drying of the concrete and to provide the cement paste in the concrete with a sufficient amount of water over a sufficiently long period of time to achieve a high degree of hydration within its mass and particularly in its surface layers. In addition, curing includes measures against the effects of sunshine or wind and the prevention of cracking due to early shrinkage.

Insufficient curing often has only a minor effect on the strength development of the concrete in the structure with the exception of thin sections because the core of a thicker concrete section maintains a sufficiently high moisture content for a prolonged period of time even without curing. However, lack of curing is detrimental to the durability of a concrete structure which is primarily controlled by the properties of the surface layers. Premature drying results in a permeable surface layer with little resistance to the ingress of aggressive media.

Protection is a measure against other external effects which may harm the young concrete such as leaching due to rain or flowing water, rapid cooling or freezing, thermal stresses due to heat of hydration, vibration or impact. Protection against freezing is dealt with in Section d.13.

d.12.2 Methods of Curing

The principle methods for the curing of the concrete are:

- Keeping the formwork in place.
- Covering with plastic films.
- Placing of wet coverings.
- Sprinkling with water (at temperatures above freezing).
- Application of curing compounds which form protective membranes.

These methods may be used separately or in combination.

Not all methods of curing are equally effective. Therefore, it is essential that the effectiveness of the method chosen is controlled, e.g., by frequent inspection. In general, those methods where water is added result in a denser pore structure of the concrete than methods which only prevent the concrete from drying. Nevertheless, sprinkling of cold water on a concrete surface which is warm due to the heat of hydration may cause severe temperature stresses and early cracking of the concrete surface layers.

d.12.3 Duration of Curing

d.12.3.1 Parameters influencing the duration of curing

The concrete has to be cured until its surface layers are sufficiently impermeable. This generally means that it will attain the required strength. Therefore, the duration of curing depends on:

- *Curing sensitivity of the concrete as influenced by its composition*.

  The most important characteristics of concrete composition with respect to curing are the water/cement ratio, the type and strength class of cement, as well as type and amount of additions. Concrete with a low water/cement ratio and made of a rapidly hardening cement e.g. R- or RS-cements according to Section d.4.2.1 reach a required level of impermeability more rapidly and, therefore, need less curing than concretes with a higher water/cement ratio and made of cements which hydrate slower such as SL-cements according to Section d.4.2.1 or concretes containing higher amounts of Type II-additions. [Authors' note: R stands rapid-hardening cements; RS for rapid-hardening, high-strength cements; SL for slowly hardening cements; and N for normal hardening cements.]

  Also the resistance of the concrete to a particular exposure condition as influenced by the type of cement or additions should be taken into account. A lower resistance to e.g. carbonation which is characteristic for some of the blended cements CE II - CE IV according to section d.4.2.1 can be offset by the choice of a lower water/cement ratio or prolonged curing. [Authors' note: CE II are portland composite cements; CE III are blast furnace cements; and CE IV are pozzolanic cements.]
- *Concrete temperature.*

  Due to the heat of hydration generated by the reaction between cement and water the concrete temperature may increase, thus accelerating hydration. Therefore, the higher the temperature particularly of the surface layers of the concrete, the shorter the required duration of curing. The temperature of the concrete depends on the temperature of the ambient air, strength grade and amount of cement, the dimensions of the structural member and the insulation properties provided e.g. by the formwork. Thus thin concrete sections without thermal insulation exposed to low ambient temperatures during curing and made of cements with a low heat of hydration need particularly careful curing.
- *Ambient conditions during and after curing.*

  A low relative humidity of the ambient air, sunshine and high winds accelerate drying of the unprotected concrete at an early stage of hydration. Therefore, under such conditions prolonged curing is required, because after termination of curing the surface layers of the concrete dry out rapidly, and hydration will no longer continue. On the other hand when concreting in a humid environment at moderate temperatures curing will at least partially be provided by the surrounding atmosphere.
- *Exposure conditions of the finished structure in service.*

  The more severe the exposure conditions as given in Table d.1, the longer the required duration of curing. [Authors' note: portions of this table have been extracted and shown as Table 5.2].

Thus, an estimate of the required duration of curing is a complex problem. The best approach is to define limiting values of permeability of the concrete surface layers which have to be reached before curing can be terminated. These values should depend on the exposure condition of the structure in service as well as on the type of cement but not on water/cement ratio, strength class of the cement and concrete temperature. At this time neither methods to measure surface permeability nor limiting values of permeability are generally accepted. Therefore, the required duration of curing has to be estimated on the basis of the parameters given above.

Some of these parameters are interrelated particularly with regard to concrete temperature so that a reliable estimate of the required duration of curing necessitates preliminary experiments on the concrete in question and continuous measurements of concrete temperature on site. Such an

Table 5.2 CEB-FIP (1990) Exposure classes (summarized from Table d.1)

Exposure Class	Description
1	Dry environment
2a	Humid environment without frost
2b	Humid environment with frost
3	Humid environment with frost and deicing agents
4a	Seawater environment without frost
4b	Seawater environment with frost
5a†	Slightly aggressive chemical environment (gas, liquid or solid); aggressive industrial atmosphere
5b	Moderately aggressive chemical environment
5c	Highly aggressive chemical environment

†Severity of aggressive environments are defined in another section of CEB-FIP 1990.

approach which is based also on theoretical considerations is given in "Durable Concrete Structures - CEB Design Guide." Rules of thumb for a quick estimate are given in the following section.

d.12.3.2 Estimates of duration of curing

In Table d.10 minimum durations of curing are proposed for concrete members subjected to exposure condition 2a; 2b; 4a and 5a according to Table d.1. In Table d.10 distinction is made between different expected ambient conditions during curing and during the period immediately after curing and between the rate at which the concrete reaches a certain impermeability. This rate depends on the water/cement ratio and strength class of cement as proposed in Table d.11. There, reference is made to the general cement classification given in d.4.2.1.

Table d.10 is valid for concretes made of portland cements (CE I) and for a reference temperature of the concrete of 20 °C. For temperatures of the ambient air during curing < 10 °C the duration of curing should be increased. Then the required increase of curing time may be determined on the basis of the maturity concept given in clause 2.1.8.2 of this Model Code. As a rough guide for a concrete temperature of 10 °C the required duration of curing is about twice the required curing time for a concrete temperature of 20 °C. For a concrete temperature of 30 °C it is only about half the curing time at 20 °C. Therefore, thermal insulation of the concrete during curing may be an effective method to reduce curing times, particularly for concrete made of slowly hydrating cements. In such cases, however, attention has to be paid to thermal stresses which may be developed when the thermal insulation is removed. Refer to subsection d.12.4.

Concretes made of cements containing high amounts of constituents other than portland cement clinker (CE II 32.5, CE III 32.5, and CE IV 32.5) and concretes containing Type II-additions in amounts which approach the maximum values given in clause d.6.3.3.2 are more sensitive to curing than concretes made of Portland cements with the same water/cement ratio. Therefore, for such concretes the duration of curing should be increased by 1 to 2 days beyond the values given in Table d.10 if during curing the concrete is exposed to conditions II and III. For condition I the rate of drying after curing is so low, that even the surface layer of the concrete will continue to hydrate for some time after the termination of curing.

Depending on the type and use of the structural element (e.g. the intended finish) the minimum duration of curing given in Table d.10 should also be applied for exposure class 1. Where the concrete is exposed to severe abrasion or to severe environmental conditions (exposure classes 3,

Table d.10 Minimum duration of curing in days for $T > 10$ °C, exposure classes 2a; 2b; 4a and 5a, according to Table d.1 cement CE I

Rate of development of impermeability of concrete		Very rapid	Rapid	Medium	Slow
Exposed ambient conditions during and after curing	I No direct sunshine, rel. humidity of surrounding air $RH > 80$ %	1	2	3	4
	II Exposed to medium sunshine or medium wind velocity or rel. humidity RH: 50 % $<$ $RH < 80$ %	2	3	4	5
	III Exposed to strong sunshine or high wind velocity or rel. humidity $RH < 50$ %	3	4	6	8

Table d.11 Rate of development of impermeability of concrete

Rate of development of impermeability	w/c	Class of cement*
Very rapid	0.5 - 0.6 < 0.5	RS RS; R
Rapid	0.5 - 0.6 < 0.5	R N
Medium	0.5 - 0.6 < 0.5	N SL
Slow	All other cases	

* Refer to d.4.2.1

4b, 5b and 5c according to Table d.1) the length of curing given in Table d.10 should be increased by 3 to 5 days depending on the ambient conditions during curing according to Table d.10.

The CEB-FIP 1990 model code also contains a section on high-strength concrete that contains the following curing recommendations:

d.17 Production of High Strength Concrete
d.17.7 Curing

Because of the low water/cement ratio, early loss of water may stop hydration of high strength concrete and the low bleeding rates may cause plastic shrinkage. Therefore, the concrete has to be cured carefully, employing curing methods where water is added rather than using methods which only prevent moisture loss. Irrespective of clause d.12.3 [Duration of Curing] a minimum curing period of 3 days is recommended in all cases.

As can be seen, CEB-FIP Model Code 1990 provides a comprehensive treatment of curing requirements. The discussion makes it clear that curing is of paramount importance for durability, but it may not be critical for strength. The reason is because durability is controlled mainly by the quality of the surface zone, whereas the strength of a structural member is not sensitive to the surface strength. The core of a member will likely have sufficient moisture to sustain hydration even if the surface becomes dry.

According to CEB-FIP 1990, the main factors that affect the duration of the curing period include the following:

- the curing sensitivity of the concrete mixture (controlled by the cementitious system),
- the concrete temperature,
- the ambient conditions (humidity, solar exposure, wind) during and after curing, and
- the exposure conditions of the finished structures.

These factors have been taken into account in the preparation of Table d.10. It is seen that the minimum duration of the curing period ranges from 1 d to 8 d (these periods would be increased from 3 d to 5 d for more severe exposure conditions). The discussion in d.12.3.2 indicates that these curing periods are based on a *concrete* temperature of 20 ºC, and these durations are to be increased or decreased depending on the actual concrete temperature during the curing period. The maturity method, using the maturity function based on the Arhennius equation (Carino 1991), is recommended as a rational means for determining the duration of curing at lower or higher temperatures. As a rough guide, the model code states that the curing periods should be increased or decreased by a factor of two for every 10 ºC change in concrete temperature. This is based on the *rule of thumb* that the rate of an exothermic chemical reaction increases approximately by a factor of two for every 10 ºC increase in temperature. This approximation, however, may overestimate the influence of temperature on the rate of strength development of concrete. This can be shown by using the maturity function suggested in Section 2.1.8.2 of the model code. For the recommended activation energy of 33.2 kJ / mol, a better approximation is a two-fold change in the curing period duration for every 15 ºC.

The model code states that because of the complex interaction of several factors instead of specifying fixed durations of the curing period, the better approach is to define a limiting permeability value of the surface concrete. The concrete should be cured until that limiting value is attained. Unfortunately, there is presently no reliable test method for measuring the in-place penetrability characteristics of concrete.

With regard to curing of low water-cement ratio concrete, the model code recommends *careful* curing to avoid plastic shrinkage cracking and to assure an adequate supply of water for hydration. The use of curing methods that supply external moisture is recommended instead of *sealing* methods. In all cases, a minimum curing period of 3 d is recommended.

5.13.5 *European Committee for Standardization, Prestandard ENV 206*—The document ENV 206: Concrete Performance, Production, Placing, and Compliance Criteria is a 1990 European Prestandard developed by the European Committee for Standardization (CEN). CEN is comprised of the national standards organizations of 18 countries of the European Union and of the European Free Trade Association. ENVs are prepared as prospective standards when high level innovation is present, when there is an urgent need for guidance, and where safety is not a prime consideration.

ENV 206 gives the following provisions related to curing:

10.6 Curing and protection

10.6.1 General

In order to obtain the potential properties to be expected from the concrete especially in the surface zone, thorough curing and protection for an adequate period is necessary.

Curing and protection should start as soon as possible after the compaction of the concrete.

Curing is prevention against:

- premature drying, particularly by solar radiation and wind.

Protection is prevention against:

- leaching by rain and flowing water;
- rapid cooling during the first few days after placing;
- high internal temperature differences;
- low temperature or frost;
- vibration and impact which may disrupt the concrete and interfere with its bond to the reinforcement.

10.6.2 Methods of curing

The curing method shall be defined before the commencement of work on site.

The principal methods for curing concrete are:

- keeping the formwork in place
- covering with plastic films
- placing of wet coverings
- sprinkling with water
- application of curing compounds which form protective membranes

The methods can be used separately or in combination.

10.6.3 Curing time

The required curing time depends on the rate at which certain impermeability (resistance to penetration of gases or liquids) of the surface zone (cover to the reinforcement) of the concrete is reached. Therefore, curing times shall be determined by one of the following:

- from the maturity based on degree of hydration of the concrete mix and ambient conditions,
- in accordance with local requirements,
- in accordance with the minimum periods given in Table 12 [Authors' note: The exposure classes are the same as for the CEB-FIP model code as summarized in Table 5.2] In cases where the concrete is exposed to severe abrasion…or to severe environmental conditions…the curing times given in Table 12 should be substantially increased.

Table 12 Minimum curing times in days for exposure classes 2 and 5a

Strength development of concrete	Rapid			Medium			Slow		
Temperatures of concrete during curing above °C Ambient conditions during curing	5	10	15	5	10	15	5	10	15
I No direct sunshine, relative humidity of surrounding air not lower than 80 %	2	2	1	3	3	2	3	3	2
II Exposed to medium sunshine or medium wind velocity or relative humidity not lower than 50 %	4	3	2	6	4	3	8	5	4
III Exposed to strong sunshine or high wind velocity or relative humidity below 50 %	4	3	2	8	6	5	10	8	5

Table 13 Strength development of concrete

Rate of strength development	W/C	Cement strength classes
Rapid	< 0.5	42.5 R
Medium	0.5 to 0.6	42.5 R
	< 0.5	32.5 R and 42.5
Slow	all other cases	

Depending on the type and use of the structural element (e.g. the intended finish) the minimum curing time given in Table 12 should also be used for exposure class 1 [dry environment].

The strength development of concrete may be estimated using the information given in Table 13.

For cement types CEII [portland composite cements], CEIII [blast furnace cements] and CEIV [pozzolanic cements] longer curing times may be appropriate.

10.6.4 Protection against thermal cracking of the surface

The hardening concrete shall be protected against damaging effects due to internal or external restraint caused by heat generated in the concrete.

Where no cracking is permitted, adequate measures shall be taken to ensure that the tensile stresses caused by temperature differences are less than the instantaneous tensile strength.

To avoid surface cracking caused by heat generated in the concrete under normal conditions the temperature difference between the centre and the surface shall be less than 20 °C.

10.6.5 Protection against freezing

The period of protection against freezing may be calculated from the maturity of the concrete. Alternately protection is no longer needed if a compressive strength of 5 N/mm^2 is obtained.

A comparison of ENV 206 with CEB-FIP 1990 reveals the following:

- The strength development categories in ENV 206 are *rapid, medium,* and *slow*; the *rapid* and *medium* categories in ENV 206 correspond to *very rapid* and *rapid* in CEB-FIP.
- In ENV 206, there is no mention that the different curing methods (sealing versus supply of excess water) are not equally effective.
- The curing times in Table 12 of ENV 206 are for exposure categories 2 and 5a, whereas CEB-FIP also includes category 4.
- The minimum curing times at 10 ºC (50 °F) are the same in all cases except for two values.
- In ENV 206, the minimum curing times at 15 ºC (59 °F) are generally ½ of the values at 5 ºC (41 °F). Thus the *rule of thumb* appears to have been used.
- ENV 206 specifies that curing times can be based entirely on degree of hydration. However, there is no guidance for the required degree of hydration.

The minimum curing durations in ENV range from 1 d to 10 d, depending on environmental conditions, concrete temperature, and rate of strength development of the concrete.

The minimum curing durations in the 1990 version of ENV 206 have been criticized as being too arbitrary and uneconomical (Hilsdorf 1995). The curing requirements are under review as part of the revision to ENV 206. The version under review in 1995 did not contain the table of minimum curing durations in the 1990 version. Curing durations were specified in terms of the time to achieve specific fractions of the standard-cured 28-day strength. The required fractional strength depends on the ambient conditions and the demands placed on the concrete surfaces. This approach for specifying curing duration is an outgrowth of the work of Hilsdorf, which is discussed in the next chapter.

5.13.6 *Australia*—In Australia, the curing requirements are based primarily on exposure conditions during service. The continent of Australia is divided into three climatic zones: *temperate, tropical* and *arid*. These climatic zones and the exposure condition for the structure are used to define different *exposure classifications* (designated in increasing severity of exposure as A1, A2, B1, B2, C, and U). Curing criteria are specified for these different exposure classifications. The following are relevant excerpts from Australian Standard 3600-1994 (AS 3600, 1994):

19.1.5.1 Curing

Concrete shall be cured continuously for a period of time that ensures that the design requirements for strength, serviceability and stripping are satisfied. To satisfy durability requirements, the initial curing periods shall be not less than those given in Clauses 4.4 to 4.6 inclusive.

4.4 Requirements for Concrete for Exposure Classifications A1 and A2

Members subject to exposure classifications A1 and A2 shall be initially cured continuously for at least three days under ambient conditions, or cured by accelerated methods, so that the average compressive strength of the concrete at the completion of curing is not less than 15 MPa.

Concrete in the member shall have an f_c' not less than—

(a) 20 MPa for exposure classification A1; or

(b) 25 MPa for exposure classification A2.

4.5 Requirements for Concrete for Exposure Classifications B1, B2, and C

Members subject to exposure classifications B1, B2, or C shall be initially cured continuously for at least seven days under ambient conditions, or cured by accelerated methods so that the average compressive strength of the concrete at the completion of curing is not less than 20 MPa for exposure classification B1, 25 MPa for exposure classification B2, and 32 MPa for exposure classification C.

Concrete in the member shall have an f_c' not less than—

(a) 32 MPa for exposure classification B1;

(b) 40 MPa for exposure classification B2; or

(c) 50 MPa for exposure classification C.

In addition for special-class concrete, a minimum cement content and the cement type shall be specified.

If requirement (c) cannot be satisfied due to inadequate aggregate strength, concrete with f_c' not less than 40 MPa may be used for exposure classification C, provided that the cement content of the mix is not less than 470 kg/m^3 and the covers required by Clause 4.10.3 are increased by 10 mm.

4.6 Requirements for Concrete for Exposure Classification U

Concrete in members subject to exposure classification U shall be specified to ensure durability under the particular exposure environment.

Table 5.3 summarizes the curing period specified in the Australian standard. A significant difference between this standard and other standards that have been reviewed is the specification of a minimum grade of concrete strength for different exposure classifications (Ho and Lewis 1988). In other countries, the usual approach is to specify maximum values of water-cement ratio, depending on the exposure severity. However, it is difficult to control water-cement ratio in the field, and the specification of a minimum strength grade may be a more practical approach for quality control. The standard does not mention curing periods at low concrete temperatures, but this is probably because low temperatures are infrequent in Australia. The minimum compressive strengths in the third column of the table are target strengths at the end the curing period when *accelerated* curing methods are used. Presumably, these minimum in-place strengths can be used as alternatives to the minimum curing periods given in the second column when advantage is taken of temperature rise due to heat of hydration. The minimum strengths at the end of the curing period are within (60 to 70) % of the minimum specified strengths in the last column of the table. The exception is exposure classification A1 where this value is 75 %. For exposure category C, the minimum specified concrete strength is 50 MPa (7 250 psi). This can be considered a high-strength concrete, and its curing requirements are no different than for ordinary concretes of lower strength levels.

Table 5.3 Summary of Curing Requirements in AS 3600-1994

Exposure Classification	Curing Period		f_c', MPa
	Days[§]	Compressive Strength, MPa[†]	
A1	3	≥ 15	≥ 20
A2	3	≥ 15	≥ 25
B1	7	≥ 20	≥ 32
B2	7	≥ 25	≥ 40
C	7	≥ 32	≥ 50
U	To be specified by user		

[§]Curing under ambient conditions
[†]Compressive strength to be attained under accelerated curing methods

The Roads and Traffic Authority (RTA) of New South Wales, Australia, has been a leader in establishing performance criteria related to curing conditions in the field. RTA has made use of water sorptivity to determine the actual effectiveness of curing on the job site. Comparisons have been made between standard laboratory moist curing conditions and those occurring in actual building constructions. Attempts were made to determine the effects of the actual curing on concrete durability (Chirgwin and Ho 1995). Previous RTA project specifications treated curing in a prescriptive manner and did not account for different forms of curing and relationships between mixture proportions and curing. RTA has now recognized that the various methods commonly used for curing cannot be expected to give the same results. For example, plastic coverings will produce more permeable concrete than water curing methods.

RTA has taken steps to strengthen its curing specifications. Under these new standards, contractors will be allowed to select from curing schemes such as wet curing with water at the surface of the concrete, sealed curing of the surfaces, and heat accelerated curing with forms in place (including steam curing). Contractors will be responsible to demonstrate by sorptivity tests that the method selected and the concrete mixture that is used will produce concrete meeting the requirements for strength and exposure classification. The contractor's proposed concrete mixture must not result in concrete that is weaker or more permeable than those from the "deemed to comply" mixtures shown in Table 5.4 (Ho and Chirgwin 1996). The curing periods in Table 5.4 are "deemed to comply" periods if the contractor wishes to avoid having to measure sorptivity. The "sorptivity depth" refers to depth of water penetration after 24 h of wetting.

The RTA curing requirements for silica-fume concrete in harsh environments provide for only one approach, that is, 24 h to 48 h sealed in forms followed by 3 d to 6 d of water curing. It is interesting to note that the RTA requirements for bridge deck finishing and curing include a limit on crack width measured at 7 d after casting. This is another attempt

Table 5.4 Durability Requirements Adopted by the Road and Transport Authority, New South Wales, Australia (Ho and Chirgwin 1996)

Exposure		Minimum binder[a] content kg/m^3	Maximum water-binder ratio	Minimum period of standard curing, days	Maximum sorptivity depth, mm
Classification	Environment				
A	Dry climate, no industry, non-aggressive		0.56	7 (7)[b]	45
B1	Industrial areas, inland	320	0.56	7 (7)	35
B2	Close to coast or permanently in salt water	390	0.46	7 (14)	17
C	Tidal / splash zone	450	0.40	14 (21)	11

[a]Binder = cementitious materials
[b]Number of days in parentheses are for high slag cement

to improve the durability of bridge decks through better curing practices and quality control.

5.14 Summary

This chapter has reviewed the basic concepts related to curing of concrete. It has been made clear that curing is a complex process, and there are many factors that effect the duration of curing to ensure that the concrete will develop a sufficient level of its potential properties to perform as intended. A review has been presented of various curing requirements, including those from other countries, for comparison with the ACI 318 Code. The review has shown that the totally prescriptive requirements in ACI 318 are lagging those in other ACI documents and in other national standards. Modern curing requirements attempt to account for the complexity of the curing process by permitting more flexibility to account for the important factors that affect the required curing duration. It has also been shown that there are few curing recommendations directed specifically toward low water-cement ratio high-performance concrete. The next chapter reviews recent curing-related research that may provide additional understanding about the curing requirements of high-performance concrete.

6. RECENT RESEARCH ON CURING REQUIREMENTS

6.1 Introduction

This chapter discusses recent research in the United States and other countries that is either related directly, or indirectly, to the curing of high performance concrete. While some of this work has not been targeted directly to HPC, the results may turn out to be applicable to the development of new curing methods and standards for this class of concrete.

6.2 Research in the United States

6.2.1 *Silica-fume concrete curing requirements*—Silica-fume (SF) concrete is relatively new and there is still much to learn about the effects of various curing methods on the development of its properties. Most of the commonly used curing methods have been used to cure SF concrete—wet burlap, plastic sheeting, and curing compounds—with a recommended minimum duration of 7 d of wet curing, or the equivalent (Holland 1989).

> As discussed in Chapter 5, the report prepared by ACI Committee 234 provides guidance on the curing of silica-fume (ACI 234R 1996). Chlorinated-rubber based curing compounds that meet the requirements of ASTM C 309 have proven to be effective. Another curing method recommended for slabs is to cover with wet burlap and plastic sheeting and keep the concrete wet for at least 3 d. To achieve the full benefits of silica-fume concrete, however, the preferred curing duration is 5 d to 7 d.

Marusin studied the effects of various durations of moist curing on three concrete mixtures containing (2.5, 5, and 10) % silica fume as mass fraction of cement (Marusin 1989). The three mixtures used Type I portland cement and had air contents of about 7 %. The corresponding water-cement ratios were 0.37, 0.38, and 0.34, and the 28-day compressive strengths were 40 MPa (5 800 psi), 43 MPa (6 200 psi), and 60 MPa (8 700 psi), respectively. Moist curing was obtained by storing 100 mm (4 in) cubes in plastic bags for (1, 3, 7 and 21) d. Wet sponges were included in the bags to maintain a high humidity. After moist curing, the cubes were air dried for 21 d, and then placed in a salt solution for 21 d. After the soaking period, the cubes were air dried for another 21 d. Chloride ion penetration was determined using a potentiometric titration procedure on powder samples taken at four depth increments. The length of the curing period significantly influenced the measured concrete properties—water absorption, water vapor transmission, and penetration of chloride. Chloride ion penetration decreased in all concretes as the curing duration increased. All mixtures benefited from the maximum curing time of 21 d.

Khan and Ayers conducted studies to determine the minimum durations of curing for silica-fume concretes to achieve a prescribed level of strength development (Khan and Ayers 1995). They compared the minimum lengths of curing at 23 °C (73 °F) for SF concrete mixtures with that of a plain portland-cement concrete mixture. Four SF concrete mixtures

Table 6.1 Required Minimum Curing Period to Achieve 70 % of 28-day Strength (Khan and Ayers 1995)

Cementitious Materials	Curing Period, days at 23 °C (73 °F)
Portland cement (PC)	3.73
PC + 15 % Fly ash	6.40
PC + 5 % Silica Fume	3.04
PC + 10 % Silica Fume	2.90
PC + 15 % Silica Fume	3.33
PC + 20 % Silica Fume	3.81

were used, with (5, 10, 15, and 20) % silica fume as mass fraction of total cementitious material. In addition, one concrete mixture was made with 15 % fly ash. Type I portland cement was used, and all mixtures had a water-cementitious materials ratio of 0.38. Cylinder specimens (100 × 200 mm) were cured in their molds for the first day and then cured in a fog room for different durations. Compressive strengths were measured after (0, 1, 3, 7, 14 and 28) d of moist curing. The 0-day cylinders were tested after removal from their molds. Regression analysis was used to develop equations relating strength and curing duration. Using the ACI 308 guideline that structural concrete should be cured until it attains 70 % of the specified compressive strength, the minimum length of curing at 23 °C (73 °F) was determined for each mixture tested. The 28-day strength determined from the regression equations was used as the specified strength. The required minimum curing periods for the various mixtures to achieve 70 % of the 28-day strengths are shown in Table 6.1. Thus for concretes with 5 % and 10 % SF, the minimum curing period at 23 °C (73 °F) is approximately 3 d compared with approximately 3.75 d for the ordinary concrete. Khan and Ayers concluded that the lower curing period for SF concrete could be attributed to a higher rate of early strength gain compared with ordinary concrete. Earlier research credits this higher early-age strength gain primarily to the hydration of C_3S which is accelerated when SF is present (Roy 1989). The slightly higher curing period for the concrete with 20 % SF may be due to retardation caused by the high dosage of high range water-reducer in that mixture. Note that the mixture with fly ash requires a significantly longer curing period to achieve 70 % of the 28-day strength. This is probably related to the slower rate of the pozzolanic reaction of fly ash compared with that of silica fume.

6.2.2 *Application of the maturity method to high-performance concrete*—The maturity method is a procedure for estimating strength development using the measured temperature history of the concrete during curing. It uses a maturity function to account for the combined effects of time and temperature on strength gain (Carino 1991; ASTM C 1074). The maturity method may become the basis for some of the new curing guidelines to be developed for HPC, and certainly should be the focus of further study. Research at NIST, using low water-cement ratio mortars, confirmed that the maturity method can be used with low water-cement ratio mixtures to estimate strength gain during curing (Carino et al. 1992). This same study resulted in the surprising observation that the long-term strength of the low water-cement ratio mortars did not appear to be adversely affected by a high curing temperature. This contrasts with experiences with ordinary mortars and concretes, where a higher initial curing temperature reduces the long-term strength (Carino 1984).

Additional research at NIST developed a reliable model to quantify the effects of curing temperature on the strength development of ordinary concrete made with various cements and admixtures (Carino and Tank 1992). This research showed that the temperature dependence of strength development depends on the cementitious materials and the water-cement ratio. This model can be used with in-place temperature measurements to estimate the *relative strength* gain of the concrete. Note that the maturity method by itself is not a reliable estimator of absolute strength because it only accounts for temperature history, and the actual mixture proportions dictate the strength potential.

Research at Cornell University studied the use of the maturity method with high-strength concrete (Pinto and Hover 1996). The maturity function based on the Arrhenius equation (ASTM C 1074) was used to account for the effect of temperature on strength gain. This work involved five different curing conditions intended to cover a wide range of potential field conditions. The HSC mixture used Type I portland cement with 10 % mass fraction of silica fume. The superplasticizer was a naphthalene sulfonate-type, and the water-cementitious materials ratio was 0.30. Test cylinders were made using 102 mm × 204 mm (4 in × 8 in) plastic and steel molds. The steel molds were used for specimens to be tested at the lowest strength levels. The five curing conditions were:

- Isothermal curing at 55 °C (131 °F)
- Isothermal curing at 40 °C (104 °F)
- Isothermal curing at 25 °C (77 °F)
- Isothermal curing at 0 °C (32 °F)
- Variable curing temperature to simulate cold weather conditions

Compressive strength tests were conducted up to ages of 90 d.

One of the factors that must be known to use the maturity function based on the Arrhenius equation is the apparent activation energy. ASTM C 1074 describes how to estimate the apperant activation energy from the isothermal strength development of mortar specimens. Values of 40 kJ/mol to 45 kJ/mol are commonly assumed for mixtures with Type I cement and no admixtures.

Pinto and Hover considered three different functions to represent strength development as a function of the maturity index[10].

(1) The exponential equation suggested by Freiesleben-Hansen and Pederson (Carino 1991):

$$S = S_\infty e^{\left(\frac{\tau}{M}\right)^\alpha} \tag{6.1}$$

where:

M = maturity index,
S = strength at maturity index M,
S_∞ = strength at infinite maturity,
τ = a time constant, and
α = a shape parameter.

(2) A "linear-hyperbolic" function that incorporates the concept of "offset" maturity (Carino 1991):

$$S = S_\infty \frac{k(M - M_0)}{1 + k(M - M_0)} \tag{6.2}$$

where:

M_0 = the offset maturity index when strength development begins, and
k = a constant.

(3) A "parabolic-hyperbolic" equation as proposed by Knudsen (1984):

$$S = S_\infty \frac{\sqrt{k(M - M_0)}}{1 + \sqrt{k(M - M_0)}} \tag{6.3}$$

The apparent activation energy was calculated from the strength versus time relationships obtained for constant temperature curing. For each of the isothermal conditions, the rate constant, k_T, was estimated from the best fit curve of the strength versus time plots. The rate constant is plotted as a function of the inverse of the absolute temperature. A straight line having the following equation is fitted to the points:

$$\ln(k_T) = \ln(A) - \frac{E}{R\,T} \tag{6.4}$$

where:

A = constant
E = apparent activation energy,
R = universal gas constant (8.314 J/(mol K)), and
T = absolute concrete temperature (K).

[10] Maturity index is the general term for an indicator of maturity that is obtained from the measured temperature history using a maturity function, such as the exponential function. See ASTM C 1074 for the terminology associated with the maturity method.

Table 6.2 Apparent Activation Energy (Pinto and Hover 1996)

Strength-maturity Model	Apparent Activation Energy (kJ/mol)	95 % Confidence Interval (kJ/mol)
Linear-hyperbolic	33.4	(29.8 - 37.1)
Parabolic-hyperbolic	48.4	(41.7 - 55.2)
Exponential	25.8	(5.4 - 46.2)

Thus the slope of the best-fit line is the activation energy divided by the gas constant.

The apparent activation energy for this HSC mixture was calculated using each of the three strength-maturity functions presented above. The results are listed in Table 6.2.

Observations from these results are as follows:

- The parabolic-hyperbolic model resulted in a value close to the 45 kJ/mol suggested in ASTM C 1074.
- The exponential model did not give reliable results as evidenced by the wide confidence interval.

Using the above values of apparent activation energy, strength versus maturity index was plotted for each of the models. Pinto and Hover concluded that none of the models was able to compensate completely for the effects of temperature. The linear-hyperbolic model, however, gave the least deviation of data points from the fitted equation.

Pinto and Hover (1996) also considered the relationship of relative strength and maturity. It is recognized that concrete cured at different temperatures will achieve different final compressive strengths; however, the extent of the chemical reaction at a certain maturity should be very similar. Therefore, a more appropriate use of the maturity concept is to relate a fraction of the ultimate compressive strength (S/S_∞) to the maturity index. The limiting strength S_∞ was calculated by fitting the equations for the linear-hyperbolic, parabolic-hyperbolic, and exponential models using only the later-age strength values. The resulting calculated strengths at infinite maturity are shown in Table 6.3 (observed 90-day strengths are also shown).

The results summarized in Table 6.3 show a tendency for a decrease in the calculated limiting strength with increasing curing temperature. The magnitude of the strength reduction due to curing temperature differs for the three strength-maturity models. These results are in contrast to those of Carino et al. (1992), who concluded that curing temperature did not have a strong affect on the limiting strength of high-strength mortars. Thus additional research is needed to reach a consensus on whether the initial curing temperature affects the long-term strength of high-strength concrete.

Table 6.3 Calculated Limiting Compressive Strength Based on Strength-Maturity Models (Pinto and Hover 1996)

Curing Temperature	Model			Measured Strength at 90 d (MPa)
	Linear-Hyperbolic	Parabolic-Hyperbolic	Exponential	
0 °C	75.6	93.0	85.7	70.7
25 °C	68.6	74.7	69.7	64.2
40 °C	65.9	70.2	68.9	62.4
55 °C	63.1	64.2	61.3	63.6
Variable	84.2	101.7	99.7	77.6

With respect to the results from tests on specimens subjected to variable curing temperatures (designed to simulate actual field conditions), there was good overall agreement with the data from the isothermal curing conditions. This led Pinto and Hover to conclude that the exponential maturity function can be used with any curing temperature history, either variable or isothermal.

6.2.3 *Effect of different curing methods on durability properties*—One of the objectives of studies conducted by Senbetta and Malchow (1987) was to determine the effects of various curing methods, applied for 14 d, on durability-related properties. The test specimens were made with Type I portland cement with no admixtures or air entrainment. The water-cement ratio was approximately 0.67. Although the concrete was not HPC, it is interesting to note the effects of different curing methods. The curing methods used were:

- moist curing in a 100 % relative humidity (RH) chamber;
- completely sealing the surface using resin modified paraffin wax;
- applying a "fairly good quality" curing compound;
- applying a "relatively poor quality" curing compound; and
- covering with plastic sheet.

After 14 d, curing compounds and plastic sheeting were removed, and the moist-cured specimens were removed from the curing chamber. The specimens were allowed to air dry at temperatures between 22 °C to 28 °C (72 °F to 81 °F) and RH from 20 % to 60%. Air drying without the benefit of any type of curing method was the control condition. The specimens were tested to determine abrasion resistance, scaling resistance, resistance to corrosion of steel in concrete, chloride ion penetration, shrinkage, and absorptivity. In general, testing was performed in accordance with applicable ASTM standards. For the absorptivity test, a mortar mixture was used, rather than concrete, to make the test specimens. The chloride ion penetration was determined using powdered concrete samples drilled from specimens used in the corrosion tests. Abrasion resistance was determined in accordance with ASTM C 779 using the rotating disc type of machine and by sandblasting as described in ASTM C 418.

The conclusions were that good curing practices improves durability-related properties compared with air curing. The best methods were sealing with wax, covering with plastic sheets, and storage in 100 % RH chamber. The high water-cement ratio used in this study ensured a sufficient supply of water for hydration, so the curing practices based on sealing the concrete were just as effective as moist curing. Test results for the curing compounds were generally inconclusive due to equipment problems and difficulties in removing the compounds after the 14-day curing period.

Other major conclusions from this research were:

- Good curing practices can improve the abrasion resistance by at least 50 % compared with air curing or poor curing methods.
- Proper curing methods can significantly reduce drying shrinkage and cracking.
- The capillary porosity of a concrete specimen can be reduced by as much as 80 % through good curing methods. This has positive effects on the properties of absorptivity, corrosion resistance, and scaling resistance.

6.2.4 *Effects of common curing practices on high-strength concrete specimens* —Although all quality control procedures and specifications currently used for ordinary concretes may not apply directly to high-strength concrete, some of the practices for batching, mixing, transporting, and placing this class of concrete may need only minor modifications. Research at The University of Texas at Austin examined the applicability of some of the common concrete practices to HSC (Carrasquillo and Carrasquillo 1988).

This work involved over 1 000 specimens and 29 different HSC mixtures with 28-day compressive strengths ranging from 41 MPa (6 000 psi) to 100 MPa (14 500 psi). Some mixtures contained fly ash (25 % to 35 % mass fraction of cement). The study focused primarily on the preparation and testing of concrete cylinders. Among the variables considered in the investigation were curing conditions, the results of which are summarized below. Three curing conditions were examined for their effects on both flexural and compressive strength of the HSC specimens:

- Remove from molds at 24 h and curing in a moist room at 100 % relative humidity at a temperature of 23 °C ± 1.7 °C (73.4 °F ± 3 °F).
- Remove from molds at 24 h, application of a curing compound, and storage at ambient conditions.
- Remove from molds at 24 h and storage under ambient conditions without a curing compound.

The first condition was taken to represent ideal moist curing whereas the second and third conditions were to simulate field-curing conditions. During the study, the ambient temperature ranged from 27 °C to 38 °C (80 °F to 100 °F) and the relative humidity ranged from 30 % to 60 %. Compressive strengths were measured up to ages of 91 d.

With respect to compressive strength of 152 mm × 305 mm (6 in × 12 in) cylinders, the test results were as follows:

- On average, the specimens cured with curing compound were about 3 % stronger than those cured in the moist room for test ages up to 15 d.
- On average, the specimens had the same 28-day strength, irrespective of the curing conditions.
- Tests at ages of 56 d to 91 d showed that the compressive strength of specimens cured in the moist room were greater than those cured with the curing compound.

Carrasquillo and Carrasquillo concluded that the slightly higher strengths at early ages for cylinders with curing compound were due to storage at ambient temperatures that were higher than those in the moist room.

The influence of curing conditions on flexural strength (modulus of rupture) was found to be more pronounced than on compressive strength. Beams cured in the moist room were much stronger than those cured under simulated field conditions. At ages of (3, 7, and 28) d, the beams cured with curing compound were, on average, only 58 % as strong as those cured in the moist room. The large difference in strength is primarily caused by surface drying of the beams with curing compound, which has the detrimental effect of inducing surface tensile stresses. These tensile stresses are due to differential shrinkage strains in the inner and outer regions of the specimen because of non-uniform moisture distributions.

The overall, general conclusion of this research program is that both the compressive and flexural strengths of high-strength concrete specimens cured under simulated field conditions are lower than those cured under standard laboratory conditions.

6.2.5 *Microcracking behavior and strength gain of high-strength con*crete—In the late 1970s early 1980s, basic studies of the mechanical behavior of high-strength concrete were carried out at Cornell University. Some of the work focused on the development of microcracking in high-strength concrete cylinders loaded in compression. These studies revealed that significant mortar microcracking did not develop until about 90 % or more of the strain at maximum load (Carrasquillo, Slate, and Nilson 1981). In comparison, significant mortar microcracking in normal-strength concrete occurs at about 70 % of the strain at maximum load. Therefore, in a compression test, HSC experiences less microcracking at all stress levels than does normal-strength concrete. This study was conducted on normal weight concretes with strengths ranging from 31 MPa to 76 MPa (4 500 psi to 11 000 psi). “Normal” strength was considered to be about 31 MPa (4 500 psi), “medium” strength about 55 MPa (8 000 psi), and “high” strength about 76 MPa (11 000 psi). The corresponding water-cement ratios were 0.70, 0.47, and 0.32. Type I portland cement was the only cementitious material. The fracture surfaces in compression tests were quite different for the normal-strength and high-strength concrete specimens. Normal-strength specimens had irregular fracture surfaces, with fractures through the mortar and around most coarse aggregate particles. The high-strength specimens, however, behaved more like a homogeneous material, with fracture passing through both the mortar and the coarse aggregate particles.

In related studies on these same concrete specimens, results showed that at early ages (up to about 28 d) high-strength concrete gains strength at a higher rate than does normal-strength concrete. At later ages, there is negligible difference in the rate of strength gain (Carrasquillo, Nilson, and Slate 1981). This research involved three room-temperature-curing conditions:

- Continuous curing in a moist room at 95 % to 100 % relative humidity until 2 h before testing;
- Moist curing for 7 d followed by air storage at 50 % relative humidity until testing at 28 d, and
- Moist curing for 28 d followed by air storage at 50 % relative humidity until testing at 95 d.

Figure 6.1 shows the ratios of the strengths of specimens subjected to the drying conditions to the strengths of specimens continuously moist cured. Results for compressive strength and modulus of rupture (flexural strength) are shown for normal-strength concrete, having a 28-day strength of 23.0 MPa (3 330 psi), and for high-strength concrete, having a 28-day strength of 70.4 MPa (10 210 psi). Strength reductions due to the negative effects of drying are greater for the high-strength specimens than the normal-strength specimens. Strength reductions due to drying are greater for modulus of rupture than for compressive strength. In addition, the strength reductions due to drying from 8 d to 28 d are greater than the reductions due to drying from 29 d to 95 d.

6.2.6 *Method for monitoring adequacy of curing*—Senbetta and Scholer (1984) conducted an informative study on the use of absorptivity as an indicator of the adequacy of curing. Efficiency of a given curing procedure was determined based on differences in absorptivity at different depths in the test specimen. The concept was simply that the pore structure of a well-cured specimen should be essentially the same at the surface as it is within the specimen. Tests showed that this concept could be used with a high degree of confidence to distinguish between good and bad curing practices.

The test procedure involved using 89 mm (3.5 in) thick mortar slabs subjected to various curing conditions. Type I portland cement was used with two different types of sand, a natural masonry sand and a graded standard sand. An assessment was made of the difference in pore structure at various depths of the test slabs based on the absorptivity of the mortar. The test procedure for the absorptivity measurements was as follows (refer to Figs. 6.2(a) to 6.2(c)):

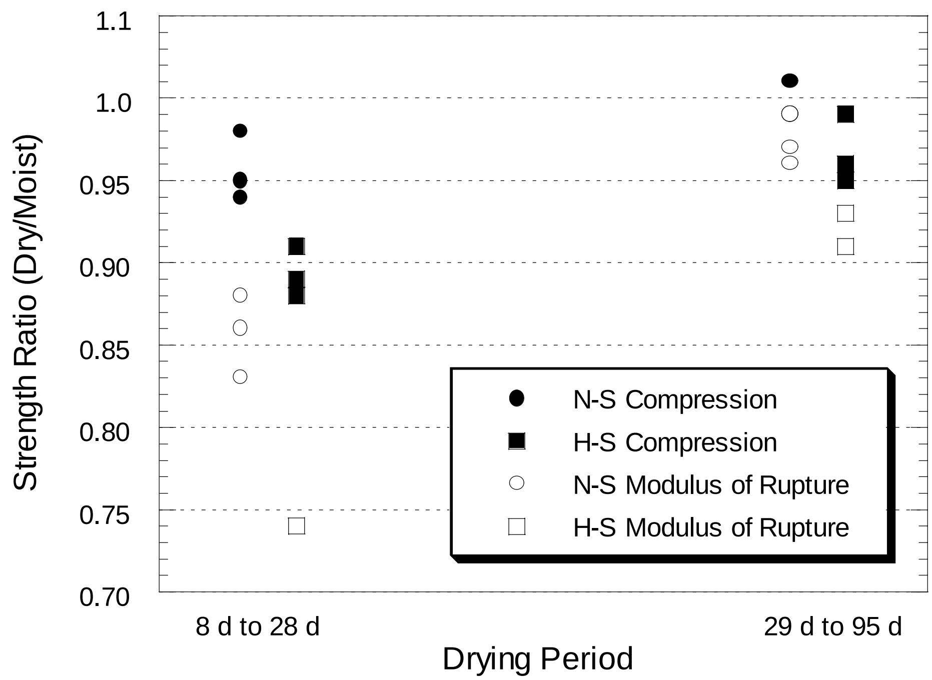


Figure 6.1 Ratio of strength (compression and modulus of rupture) after drying to strength for continuous moist curing for normal-strength (N-S) and high-strength (H-S) concretes (Carrasquillo, Nilson, and Slate 1981)

- Drill 25 mm (1 in) diameter cores through the mortar slabs after curing.
- Cut the cores into 10 mm (0.4 in) thick disks and dry them at room temperature under vacuum for 48 h. Measure the mass of the dried disks.
- Place the upper surface of the disks in contact with a free water surface for one minute.
- Measure the gain in mass of absorbed water. Convert the mass of absorbed water to volume of water.
- Calculate the "coefficient of absorptivity" according to the following formula:

$$K_a = \left(\frac{V}{A}\right)^2 \frac{1}{t} \tag{6.5}$$

where:

K_a = coefficient of absorptivity, cm^2/s,
V = volume of absorbed water during time t, cm^3,
A = cross-sectional area of disk, cm^2, and
t = duration of immersion of the disk, s.

Senbetta and Scholer (1984) studied the performance of the above procedure for slabs exposed to five curing conditions as follows:

- Covered with wet burlap,
- Covered with plastic sheet,

- Coated with curing compound (later found to be poor quality),
- Left exposed, and
- Left exposed in a windy environment.

The slabs were cured from 1 d to 5 d in a chamber maintained at a temperature of 27 °C (81 °F). Slabs were exposed to relative humidities of 22 %, 44 %, and 72 % during the curing periods. A fan created a windy environment in one part of the curing chamber.

Figures 6.2(d) through 6.2(f) summarize some of the results. These figures show the absorptivity coefficients plotted as a function of the depth from the slab surface. Results showed that the more effective curing methods (wet burlap and plastic sheet) produced smaller differences in absorptivity from the top to the bottom of the slab. Poorly cured slabs had large differences in absorptivity between the top (near surface) regions and the bottom, and absorptivity values were on the order of six times greater for poorly cured specimens. This large difference makes absorptivity a reliable measure of curing efficiency since differences are readily apparent. As shown in Fig. 6.2(g), there was good correlation between absorptivity and abrasion resistance. Abrasion resistance is enhanced and absorptivity is reduced as the curing conditions improve and durations increase.

Based on statistical analysis of the data for the 5-day old samples, Senbetta and Scholer established a quantitative dividing line to distinguish between adequate and inadequate curing. This dividing line was based on the differences in absorptivity at depths of 10 mm and 60 mm (0.4 in and 2.4 in) within the slab. A difference of $\leq 3.7 \times 10^{-6}\ \mathrm{cm^2/s}$ ($0.57 \times 10^{-6}\ \mathrm{in^2/s}$) indicated adequate curing; whereas, a difference of $\geq 5.5 \times 10^{-6}\ \mathrm{cm^2/s}$ ($0.85 \times 10^{-6}\ \mathrm{in^2/s}$) indicated inadequate curing.

Based on the results of this study, Senbetta and Scholer (1984) concluded that:

- Absorptivity can be used as a simple and accurate measure of cement hydration and pore structure development in cement paste.
- Poor curing practices may only affect the concrete surface to a depth of approximately 30 mm (1.2 in).
- The use of absorptivity differences between two depths, or zones, of a specimen to measure the effectiveness of curing may be applicable to any concrete mixture. This is because the quantity of interest is based on the difference between two zones of the same sample.
- More work is needed to confirm that the method can be used as a general method to confirm the effectiveness of curing.

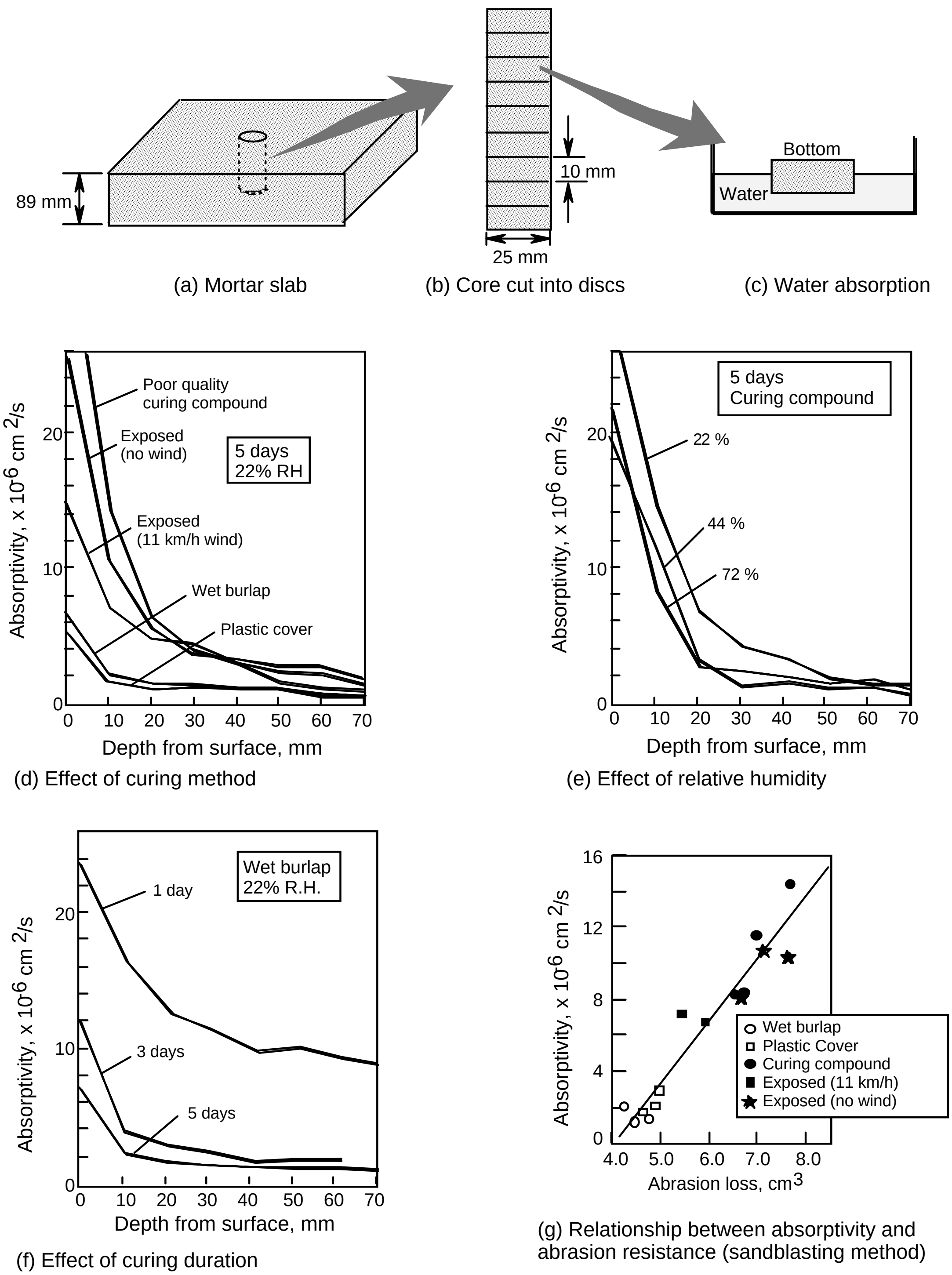


Figure 6.2 **Summary of study by Senbettra and Scholer (1984) on use of absorptivity to indicate curing efficiency**

In 1990, ASTM adopted a standard test method (ASTM C 1151) on the use of absorptivity difference as a measure of the efficiency of curing materials (membrane-forming compounds and sheet materials). Although the original research by Senbetta and Scholer (1984) that lead to ASTM C 1151 was not directed to high-performance concrete, the absorptivity approach could have merit as a tool for the evaluation of different curing conditions on high-performance concrete specimens.

6.3 Research in the United Kingdom

6.3.1 *Effectiveness of curing on mitigating corrosion of reinforcement*—It is difficult to link the actual curing used on a concrete structural member to the rate of deterioration from carbonation-induced corrosion of the reinforcement. A study of over 400 buildings in the United Kingdom found that concrete cover was the most important factor affecting the early deterioration of reinforced concrete structures (Marsh and Ali 1994). This work focused on in-place tests on concrete in structural members where corrosion of reinforcement due to carbonation was the primary potential problem. Members studied were cast in place using plywood forms, which remained in place for at least 16 h. Tests showed that after about 4 years, the depth of carbonation was the same for all curing methods used, from very good moist curing practices to no curing at all. Hence curing methods did not appear to have a significant effect on the long-term resistance to carbonation. This study found that the best approach for controlling carbonation would be to reduce the gas permeability of the concrete. It should be noted that this study did not involve HPC.

Some research on silica-fume concrete also focused on corrosion of steel reinforcement (Cabrera, Claisse, and Hunt 1995). Three curing methods were used to simulate conditions that might be encountered in offshore construction sites. The curing conditions were as follows:

- Maintained at 20 °C (68 °F) and 99 % RH until test age ("good" curing condition),
- Treated with aluminum pigmented curing agent and kept at 20 °C (68 °F) for 7 d and then in water at 5 °C (41 °F) ("fair" curing condition)
- In water at 5 °C (41 °F) until test age ("poor" curing condition)

Two basic mixtures were used with water-cementitious materials ratios of 0.3 and 0.46. For each of these, two types of concrete were prepared—one with silica fume and one without silica fume. Thus a total of four mixtures were used. Mortar and paste samples were also made for later testing. After being cast, the specimens were covered and maintained at 20 °C (68 °F) for 24 h; they were then cured using the three conditions described above.

Corrosion rate measurements were made on 75 mm (3 in) diameter concrete cylinders, each of which contained an embedded 12 mm (0.5 in) diameter steel bar. Corrosion was induced by immersing the cylinders in a salt solution after the curing period. Initial corrosion rates were measured using linear polarization resistance. After the steel bar was polarized to +100 mV relative to a standard calomel electrode for 28 d, another measurement was taken. The difference between the corrosion rates was taken as an indicator of the ability of the concrete to protect the concrete by preventing the ingress of chloride ions.

Other tests conducted on concrete specimens included chloride transport, depth of carbonation, oxygen transport, water vapor transport, lime content, and compressive strength Specimens were tested at ages of (3, 28, and 28) d after placement. Two results were obtained for each test, and average values were used in the statistical analysis, which included analysis of variance and regression.

Results confirmed that the corrosion rate was reduced for the mixtures with the lower water-cementitious materials ratio and for those specimens that received good curing. Another important finding was that silica-fume concrete is more sensitive to poor curing than concrete without silica fume. The decrease in corrosion resistance from the "good" curing condition to the "poor" curing condition for the silica-fume concrete was much greater than for the concrete without silica fume. For the "poor" curing condition, the resistance to corrosion was essentially the same for all mixtures except for the low water-cementitious materials ratio, silica-fume concrete, which had a significantly lower resistance to corrosion. For the other two curing conditions ("good" and "fair"), the silica-fume concretes showed significantly better corrosion resistance. The researchers concluded that the low temperature of the "poor" curing condition probably inhibited the hydration (and pozzolanic) reactions in all mixtures, and reduced the resistance to chloride ion penetration. The improved performance of the low water-cementitious materials ratio, silica-fume concrete, was likely due to self desiccation leaving the steel in a dry environment.

6.3.2 *Effects of curing on durability*—Researchers at the University of Leeds reviewed previous studies on the effects of different curing methods and curing durations on durability-related properties of concrete (Gowripalan et al. 1990). In addition, they reviewed the status of test methods to assess curing efficiency. The review focused on the effects of curing on porosity, permeability, and water absorption, and compared the differences when blended cements are used instead of just portland cement.

With regard to porosity, it was noted that the distribution of the larger pore sizes rather than the total porosity has the greatest influence on durability-related properties. Larger pores refer to those in the range of 0.09 μm to 0.15 μm (3.5×10^{-6} in to 5.9×10^{-6} in). The review indicated that blended cements could result in reduced pore sizes compared with portland cement alone. To attain this benefit, however, concretes made with blended cements require longer curing periods, or they need to be cured at higher temperatures for the same period, compared with concretes made with portland cement only.

Gowripalan et al. (1990) noted that gas permeability testing is more common than water permeability because of the difficulty in getting measurable quantities of water to flow through good quality concrete. A review of previous research indicated that initial curing is critical for reducing gas permeability of the outer zone of a concrete member. For example, it was reported that, for portland-cement concrete, increasing the moist-curing period from 1 d to 3 d would, on average, reduce oxygen permeability by a factor of 5. Increasing the curing period to 28 d, however, resulted in only a modest additional reduction of oxygen permeability. In addition, it was noted that the benefits of increased duration of curing were less for lower water-cement ratio mixtures.

It was reported that many researchers have attempted to use water absorption tests to assess the effects of curing. The results of absorption tests are indicative of the total capillary porosity and pore size distribution. Despite the existence of many such test methods, standardization of a reliable field method has been hampered by the problem of developing standardized moisture conditioning procedures prior to measuring absorption properties. Laboratory research at the University of Leeds on the effectiveness of different curing compounds involved an absorption testing procedure similar to the one used by Senbetta and Scholer (1984) that was discussed in Section 6.2.6.

In summary, the review by Gowripalan et al. (1990) provides the following important observations related to curing to ensure adequate durability:

- The initial curing period results in the most benefit.
- Low water-cement ratio concretes are less sensitive to initial curing.
- Concretes made with blended cements require longer durations of moist curing, but they will have improved durability-related properties compared with similar concretes made with portland cement only.
- Water absorption tests are useful for assessing curing efficiency, but the problem of moisture conditioning has hampered their standardization.

6.3.3 *Strength gain of silica-fume, high-strength concrete*—Sabir (1995) evaluated the strength gain of high-strength concrete with silica fume at two water curing temperatures, 20 °C (68 °F) and 50 °C (122 °F). The purpose of the research was to learn more about the influence of silica fume on the strength development of high-strength concrete. Six mixtures were investigated. A reference mixture was made with ordinary portland cement (OPC), and the other five included silica fume as a partial replacement of the cement. The silica fume replacement varied from 12 % to 28 % mass fraction of cement in the reference mixture. Superplasticizer was used in all the silica-fume mixtures to achieve workable concretes. The water-cementitious materials ratio for all mixtures was 0.35. Compressive strength tests were conducted on 100 mm (4 in) cube specimens at ages of (4 , 7, 14, 28, and 91) d. Figure 6.3 shows the strength development of the six mixtures for the two curing temperatures. Figure 6.4 shows the variation of compressive strength with silica fume content at the various test ages.

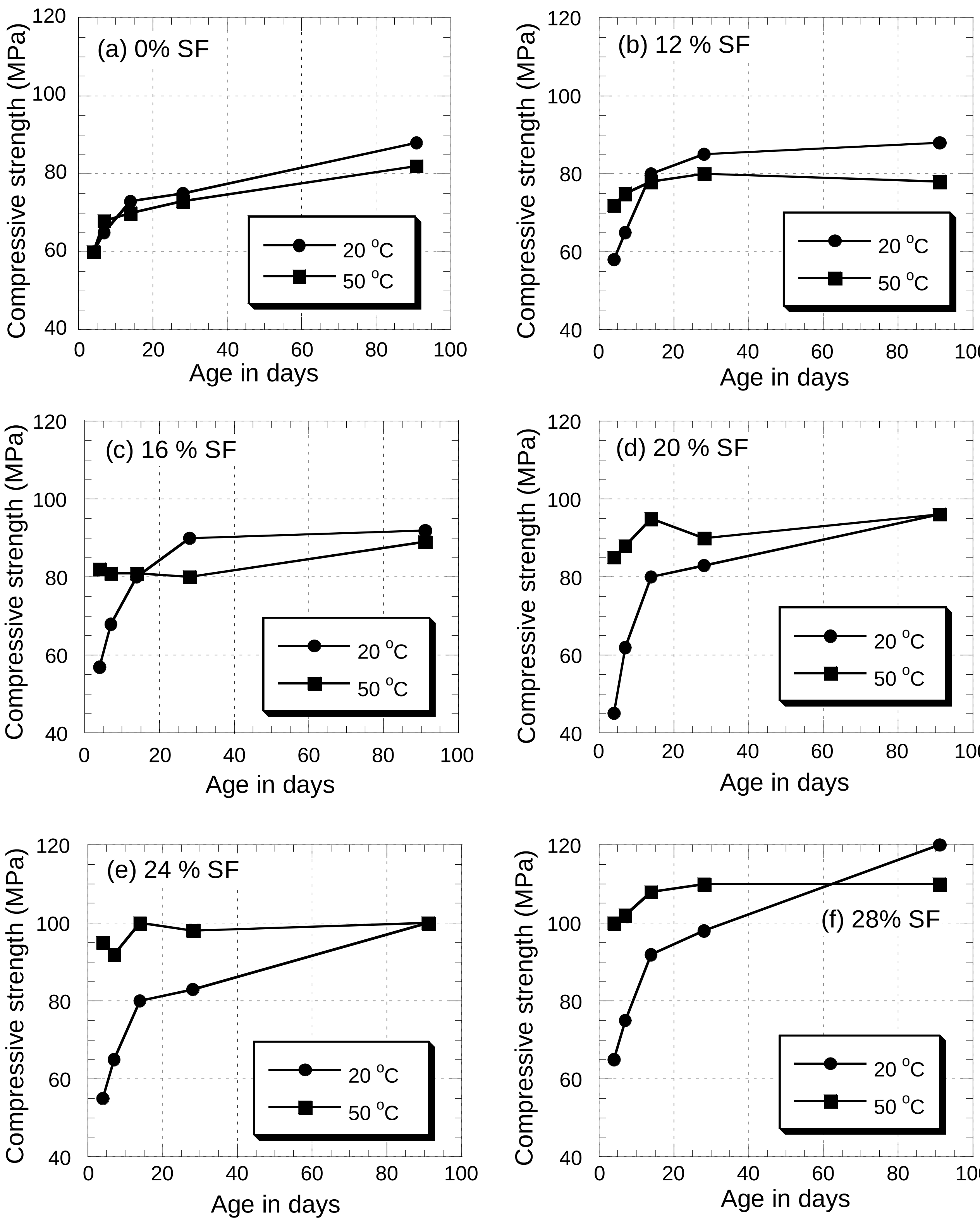


Figure 6.3 Strength development of concrete with various amounts of silica fume for water curing at 20 °C and 50 °C (adapted from Sabir 1995)

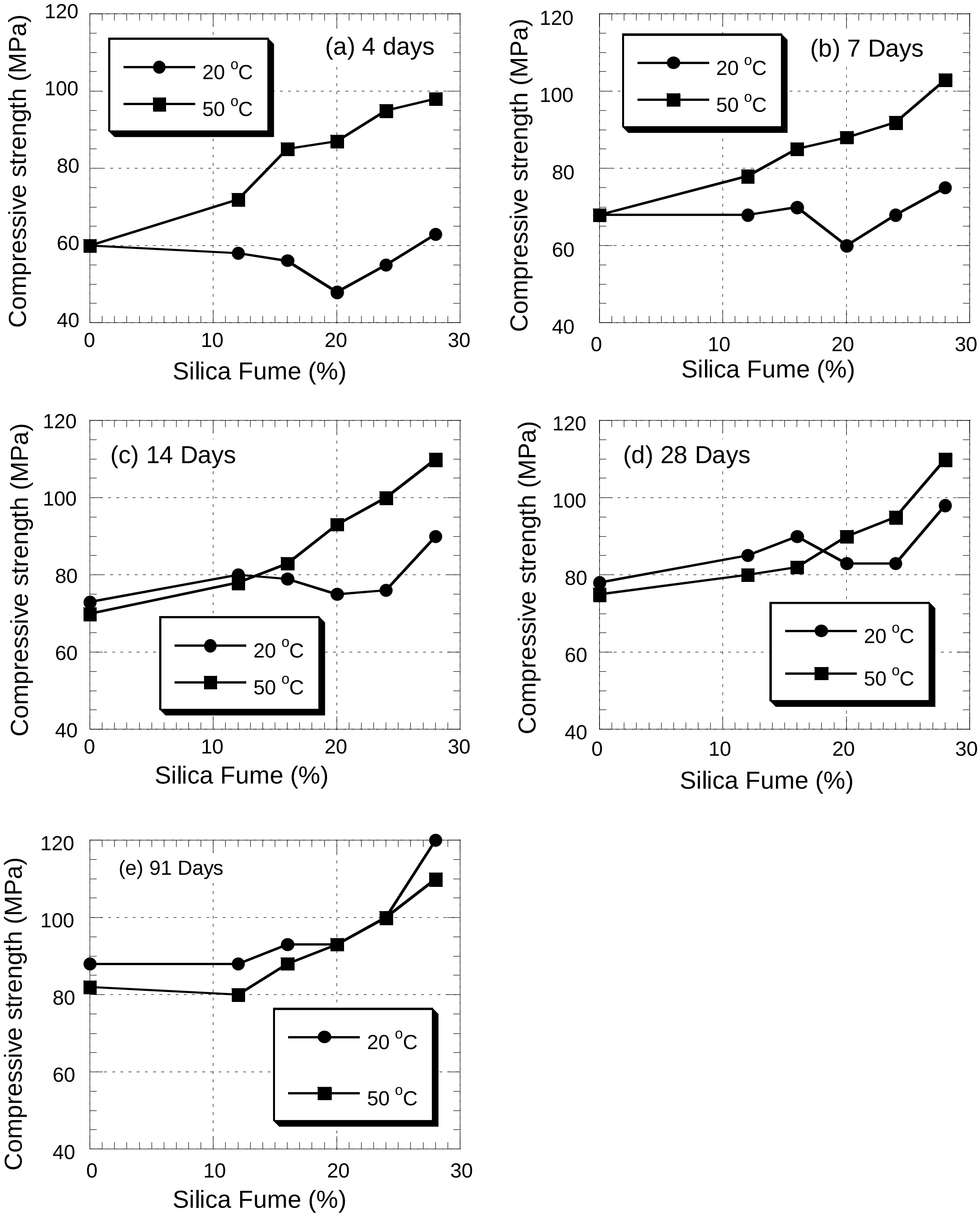


Figure 6.4 **Effect of silica fume content on strength after various periods of curing at 20 °C and 50 °C (adapted from Sabir 1995)**

The results showed that the higher curing temperature accelerated the strength gain of the silica-fume concrete. Beyond 28 d, however, there was less additional strength gain for the silica-fume concrete compared with the reference concrete. The effect of the curing temperature on the strength gain differed for the OPC and the silica-fume concretes. For the high-strength OPC concrete, 7-day strengths were 86 % and 93 % of the 28-day strength for curing at 20 ºC (68 ºF) and 50 ºC (122 ºF), respectively. For the six silica-fume concrete mixtures, the average ratios were 76 % and 97 % for curing at 20 ºC (68 ºF) and at 50 ºC (122 ºF), respectively. Thus it appears that the rate of strength gain of silica-fume concrete is more sensitive to the curing temperature than that of OPC concrete. While the higher curing temperature accelerated the early-age strength gain for the silica-fume concretes, at 91 d the strengths were equal to or less than those of the specimens cured at the lower temperature.

Although the addition of silica fume tended to increase strength, Fig. 6.4 shows that the beneficial effects depend on the age of the concrete and the curing temperature. At early ages, the benefits were minimal at the lower curing temperate, but were more pronounced at the higher temperature. At later ages, the benefits were significant only for higher silica-fume contents, and curing temperature was not a major factor. These results suggest that the early-age strength of silica-fume concrete will benefit by an elevated curing temperature.

6.3.4 *Monitoring concrete properties in the cover zone*—McCarter et al. (1995) studied techniques to monitor moisture movement within the *cover zone* of concrete. The cover zone is the portion of the concrete between the reinforcing steel and the exposed surface of the member. As has been mentioned previously, this region, which has sometimes been called the *curing affected zone*, determines the durability characteristics of the concrete. The research was directed toward the use of electrical methods to determine the distribution of moisture during absorption testing. A multi-electrode resistance probe, as shown in Fig. 6.5, was developed for this purpose. The probe has ten electrode pairs mounted on a plastic base. The paired electrodes have a center-to-center spacing of 5 mm (0.2 in), and electrode pairs are placed at 5 mm (0.2 in) depth increments. The 10 electrode pairs permit resistance readings at various depths, up to 50 mm (2 in), in the cover region. The probe indicates the degree of saturation at different depths based on the change in electrical resistance due to the presence of moisture.

The testing arrangement is shown in Fig. 6.6. The test specimens were small concrete slabs with dimensions of 300 mm × 300 mm × 150 mm (12 in × 12 in × 6 in). The probe was glued to the reinforcement bar at the center of a slab with 40 mm (1.5 in) of cover provided. A chamber was mounted to the top surface of the cured slabs, and the chamber was filled with water that penetrated into the slab under a 200 mm (7.9 in) pressure head. A horizontal capillary was used to measure the volume of absorbed water. Resistance readings were taken at 10-minute intervals for 24 h. The concrete was an ordinary portland cement mixture with a water-cement ratio of 0.5 and a 28-day compressive strength of 45 MPa (6 500 psi). The test specimens were prepared as follows:

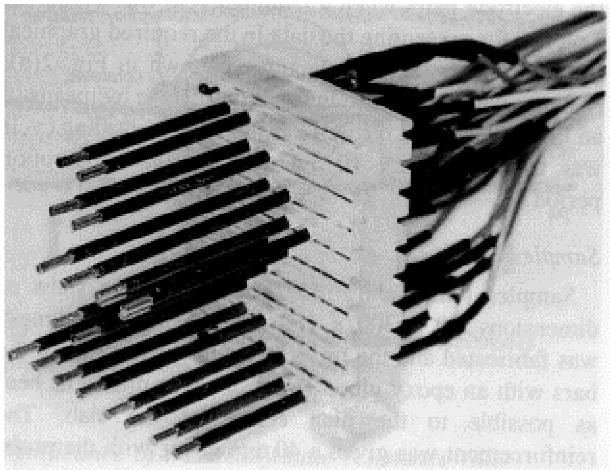


Figure 6.5 Electrical probe used to monitor moisture movement in cover zone; electrode pairs are offset from each other by 5 mm both vertically and horizontally (adapted from McCarter et al. 1995)

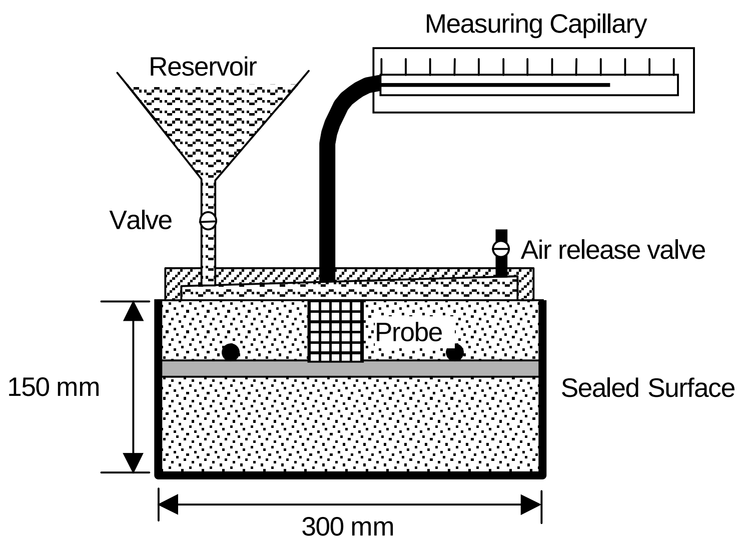


Figure 6.6 Testing arrangement for using electrical resistance probe to measure moisture distribution during absorption test (adapted from McCarter et al. 1995)

- Molds were removed after 24 h, and slabs were wrapped tightly in plastic sheeting for 7 d. Conditions under the plastic were monitored: the temperature was 22 °C (72 °F) and the relative humidity was 95 %.
- The plastic sheeting was removed after 7 d, and the slabs were sealed on three sides with an epoxy paint to provide for one-sided drying.
- The slabs were allowed to dry in the laboratory at 22 °C ± 1 °C (72 °F ± 2 °F) and 55 % to 60 % relative humidity for 16 weeks prior to testing.

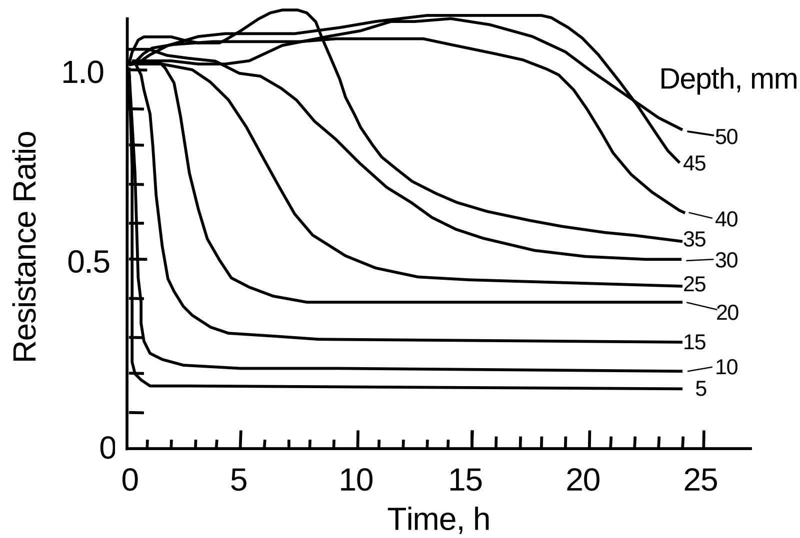


Figure 6.7 **Example of results using resistance probe (adapted from McCarter et al. 1995)**

Figure 6.7 is an example of results obtained with the resistance probe. The figure shows the "resistance ratio" as a function of elapsed time from start of water absorption. The resistance ratio is the resistance at any time after the start of the test divided by the resistance just before the start of the test. As the water penetrates to the depth of a probe, the resistance decreases. Once the water has moved beyond the probe, a constant resistance ratio is obtained. McCarter et al. chose the point of maximum slope of the resistance ratio-depth curve to signify when the moisture front arrived at the depth of the probe. It is observed that the steady-state resistance ratios increase with depth. This indicates that the degree of saturation at the start of the test was higher for the deeper probes. Thus McCarter et al. (1995) demonstrated the effectiveness of the multi-probe electrical resistance device in monitoring the moisture movement within concrete. An additional benefit is that the probe allows measurement of concrete resistivity at the level of the reinforcing steel, which is an important parameter in the corrosion process.

Parrott (1992) reported on a research program to examine the effects on the water absorption in the cover zone for different concretes subjected to indoor and outdoor exposures for 1.5 years. The study included the effects on water absorption of five exposure conditions, five water-cement ratios, three periods of moist curing, and four cements. Table 6.4 lists the factors considered in this study. Thirty combinations of these factors were included so that not all possible combinations were considered.

Compressive strength tests were conducted on 100 mm (4 in) cubes at ages of (1, 3, 28) d, and 1.5 years. Tests for absorption and carbonation were conducted on 100 mm (4 in) cubes that had been sealed against moisture exchange on five faces, so that one-dimensional moisture movement took place. Specimens were cured in their molds at 100 % relative humidity and 20 °C (68 °F) for the first 24 h. After removal of the molds, the specimens were

Table 6.4 Variables Considered in Study by Parrott (1992)

Exposure	Water-cement ratio	Moist-curing period, d	Cement
Laboratory	0.83	1	Ordinary portland cement
Indoor office	0.71	3	5 % limestone filler
Outdoor sheltered from rain	0.59	28	30 % pulverized fly ash (pfa)
Outdoor vertical exposed surface	0.47		50 % ground granulated blast-furnace (ggbs)
Outdoor horizontal exposed surface	0.35		

sealed and cured at 20 °C (68 °F) for the prescribed curing periods. At the completion of the curing period, the specimens were exposed to the five environments listed in Table 6.4. The quantity of water lost upon initial exposure to the different environments was measured. Conditions in the laboratory were relative humidity between 55 % and 61 % and temperature between 19 °C (66 °F) and 21 °C (70 °F). Outdoor conditions were in the ranges 58 % to 90 % relative humidity, 6 °C to 21 °C (43 °F to 70 °F) temperature, and 40 mm to 70 mm (1.6 in to 2.75 in) of rain each month. The inside office normally had a temperature range of 15 °C to 25 °C (59 °F and 77 °F), and the relative humidity was not controlled. At the end of the 1.5-year exposure period, the water absorption of the cubes was measured by placing the uncoated faces of the cubes in contact with water. The gain in mass after (1, 2, 4, 6 , 24 , and 30) h of wetting was determined and expressed in terms of kg/m^2. Some of the results of Parrott's study are summarized here (Parrott 1992).

Table 6.5 shows the effects of curing duration and portland cement replacement on water absorption after 1.5 years of laboratory exposure. Wetting periods of (1 , 4, and 24) h are shown. The water-cement ratio is 0.59. It can be observed that moist curing beyond 3 d reduced absorption levels for both the fly-ash and slag-cement concretes; however, curing

Table 6.5 Effects of Curing Period and Cement Replacement on Water Absorption (kg/m^2) After 1.5 years of Laboratory Exposure (w/c = 0.59) (Parrott 1992)

Curing Period, d	100 % OPC			30 % Fly Ash			50 % Slag		
	1 h	4 h	24 h	1 h	4 h	24 h	1 h	4 h	24 h
1	1.46	2.07	3.58	1.84	2.85	4.71	1.39	2.40	4.70
3	0.77	1.19	1.99	1.31	1.86	2.70	1.08	1.75	2.97
28	0.71	1.08	1.88	0.95	1.34	2.09	0.97	1.37	1.92

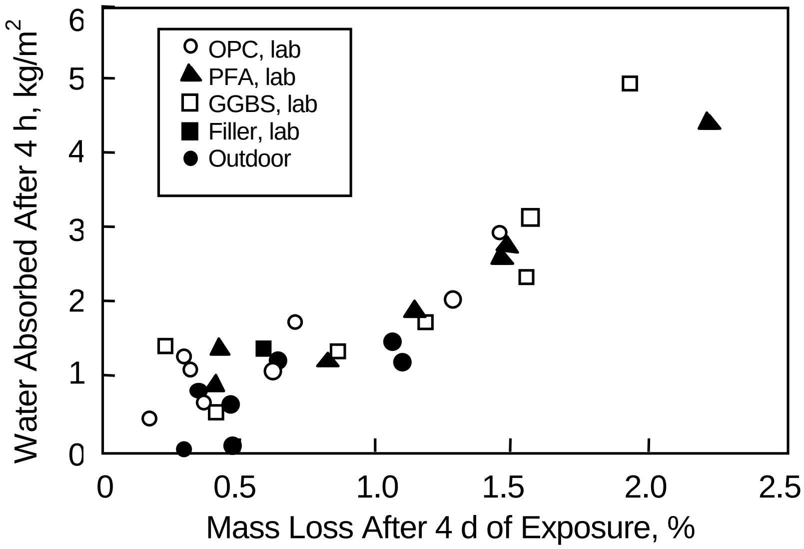


Figure 6.8 Relationship between water absorption after 1.5 years of exposure and water loss during first 4 d of exposure (Parrot 1992)

beyond 3 d had only a small effect on the absorption of OPC concrete. Thus concretes with partial replacements of portland cement require longer curing periods to achieve the same resistance to water absorption as concrete with 100 % portland cement. In this particular case, the concretes with cement replacements had to be cured 28 d to achieve similar absorption values as 3 d of curing for the OPC concrete. This clearly indicates that curing requirements for durability have to consider the type of cementitious system that is used.

Another objective of Parrott's study was to determine whether water absorption after the 1.5 years of exposure was related to concrete properties measured at early ages. Figure 6.8 shows the relationship between the water absorption after 4 h of wetting and the mass loss during the first 4 d of exposure after the curing period. It is seen that, with the exception of two points, there is a well-defined relationship between the initial water loss and the subsequent absorption. Both of these properties are related to the fineness of the pore structure. The two exceptions were the outdoor specimens exposed to rain. The pores of these specimens were nearly saturated due to rain and little additional water was absorbed during the wetting period.

Another comparison that was examined was between the 4-hour water absorption after 1.5 years of exposure and the cube strength at the end of the moist curing period. Figure 6.9 shows that, with the exception of the same two points as in the previous figure, there is a relationship between these two properties. Note that the relationship is less well defined for strength levels greater than about 20 MPa (3 000 psi). For the higher strengths at the start of exposure, there is little correlation between the 4-hour absorption and the cube strength.

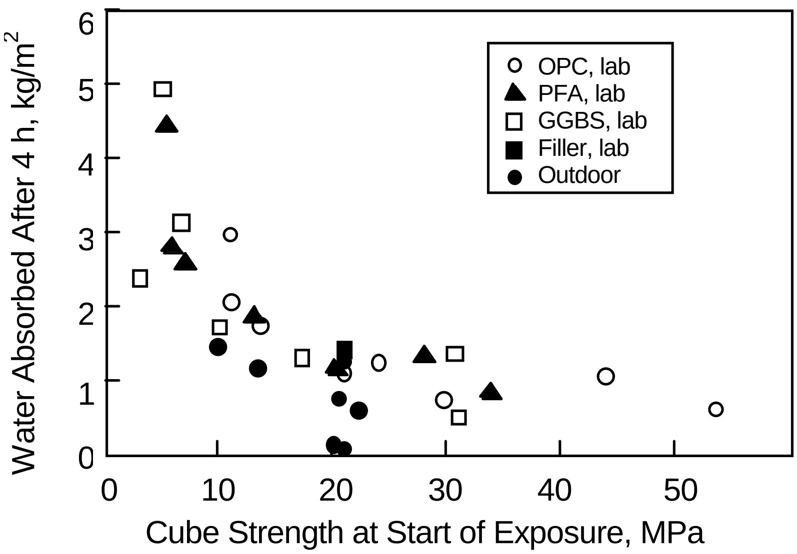


Figure 6.9 Relationship between 4-hour water absorption after 1.5 y of exposure and cube strength at time of initial exposure (Parrott 1992)

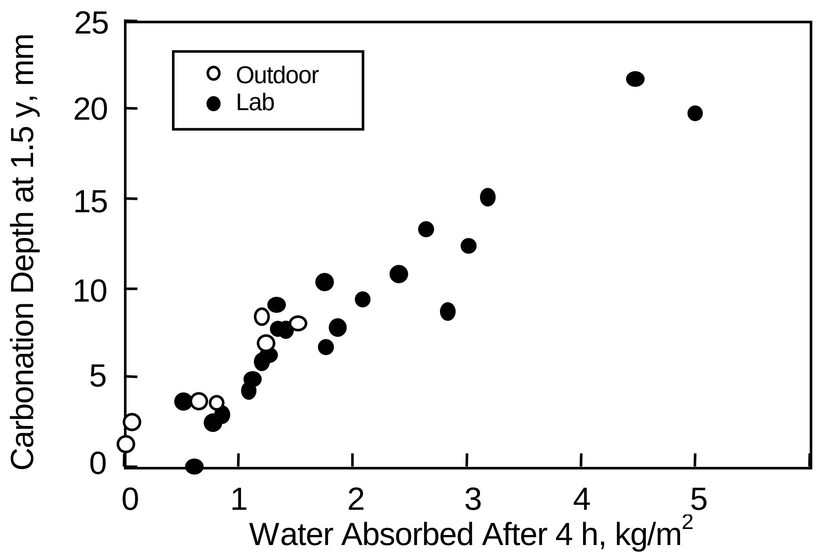


Figure 6.10 Relationship between 4-hour water absorption and depth of carbonation after 1.5 years of exposure (Parrott 1992)

The final comparison reviewed here is between the 4-hour absorption after 1.5 years of exposure and the depth of carbonation. Figure 6.10 shows that there is very nearly a linear relationship between carbonation depth and 4-hour absorption. The same relationship appears to be valid for both the concrete exposed indoors and outdoors. Thus Parrot (1992) noted that it may be possible to equate high water absorption with high depth of long-term carbonation.

Based on these studies, Parrott (1992) concluded that long-term water absorption performance could be estimated from measurements of properties at the time of exposure or from mass loss during initial exposure. He also suggested that field measurements of water absorption should not necessarily be preceded by a period of artificial drying, because absorption under naturally occurring conditions appeared "to be of greater significance for durability assessment."

6.3.5 *"Self-curing" concrete* —Some interesting research has investigated the feasibility of developing a "self-cure" concrete which would not need any externally applied curing (Dhir et al. 1994). This concept involves adding water-soluble chemicals to the concrete during mixing to reduce water evaporation as the concrete is exposed to air drying. Dhir et al. showed that chemicals are available which have a positive "self-curing" effect. The research was conducted on ordinary concrete; however, the concept may warrant further study to determine its applicability to HPC.

This research focused on finding a chemical additive that would enhance water retention in the concrete and thereby result in an increased degree of hydration, which would improve the physical properties. An ordinary portland cement concrete with a 28-day design compressive strength of 30 MPa (4 350 psi) was used. The two physical properties measured were compressive strength and permeability. Standard compressive strength tests were conducted on concrete cubes, and the initial surface absorption test (ISAT) was used to measure surface quality. The ISAT test involves measuring the initial rate of water absorption under a pressure head of 200 mm (7.9 in) of water. The degree of hydration was determined using x-ray powder diffraction and thermogravimetric methods.

The chemicals which tend to lower the water loss from concrete are water-soluble polymers that have hydroxyl (–OH) or ether (–O–) functional groups, or both. Hydrogen bonding occurs between these functional groups and water molecules. This bonding reduces the vapor pressure of the water and evaporation. The researchers tested six different chemicals, five of which were synthetic water-soluble polymers and one was a natural chemical. Due to the proprietary nature of the research the compositions of the chemicals were not revealed. Various concentrations for each chemical were tested to obtain optimum results. Control concrete without a chemical additive was also tested.

Two of the chemicals performed significantly better than the others and satisfied the requirements set out for a "self-cure chemical," that is, they resulted in significant improvements compared with concrete exposed to air drying. The main chemical characteristics of these chemicals are shown in Table 6.6.

The qualitative effects of these two chemicals on characteristics of the paste and concrete compared with the control concrete stored in air are shown in Table 6.7. The researchers noted that the increase in degree of hydration with chemical A1 is higher in relation to the control than the corresponding improvement in water retention. This indicates that chemical A1 is improving hydration beyond that which can be attributed to

Table 6.6 Characteristics of Successful "Self Cure" Chemicals (Dhir et al. 1994)

Chemical Designation	Type	Molecular weight	Maximum solubility at 20 °C (mass fraction %)	Functional group	
				Hydroxyl	Ether
A1	Synthetic	200	100	yes	yes
A3	Synthetic	5000	100	yes	yes

water retention alone. The researchers concluded that chemical A1 might reduce the concentration at which calcium hydroxide (CH) crystals begin to precipitate. As this concentration is lowered, the effect is to encourage the formation of additional CH, which improves the degree of hydration beyond that expected from water-retention alone.

Dhir et al. (1996) carried out additional research on the influence of microstructure on the physical properties of "self-curing" concrete. Further studies were performed to determine the effects of chemical additive A1 (identified as polyethylene glycol with an average molecular weight of 200) on the paste microstructure. Compressive strengths were determined for a range of concentrations of the admixture, for both paste and concrete. Comparisons were made with control specimens made without the chemical. Also, initial surface absorption tests (ISAT) were conducted and the results were compared with a water-cured control concrete.

Figure 6.11 shows the variations of cube compressive strength for paste and concrete specimens as function of the concentration of admixture A1. Notice that the strength of the paste increases rapidly as the admixture concentration increases, it reaches a maximum, and then decreases gradually with increasing concentrations. All the paste specimens with the admixture are stronger than the control (the strength at 0 concentration). For the concrete specimens, the strength is greater than the control at the low dosage, but at higher dosages, the strengths are lower than the control. The researchers infer that the likely reason for this difference in behavior is that the admixture affects the bond between the aggregate particles

Table 6.7 Effects of "Self-Cure" Chemicals on Paste or Concrete Attributes Compared with Air-stored Concrete (Dhir et al. 1994)

Chemical Designation	Attribute			
	Paste water retention	Degree of hydration	Compressive strength	Surface quality
A1	Very good	Large increase	Large increase	Extremely good
A3	Good	Increase	Considerable increase	Improvement

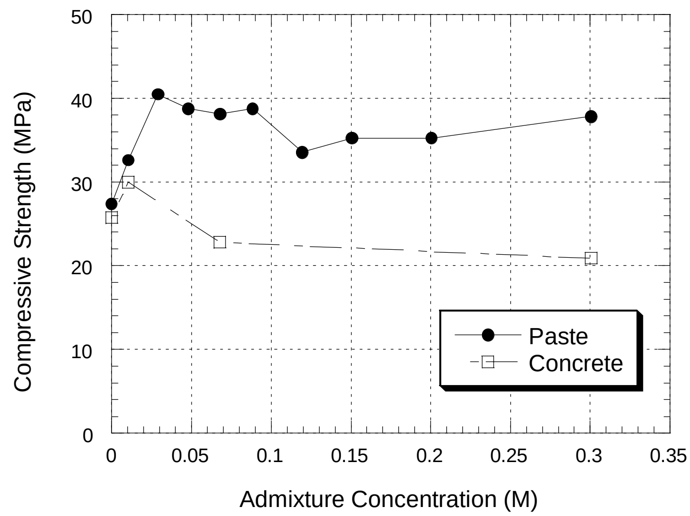


Figure 6.11 Compressive strength as a function of the concentration of "self-curing" admixture A1 (Dhir et al. 1996)

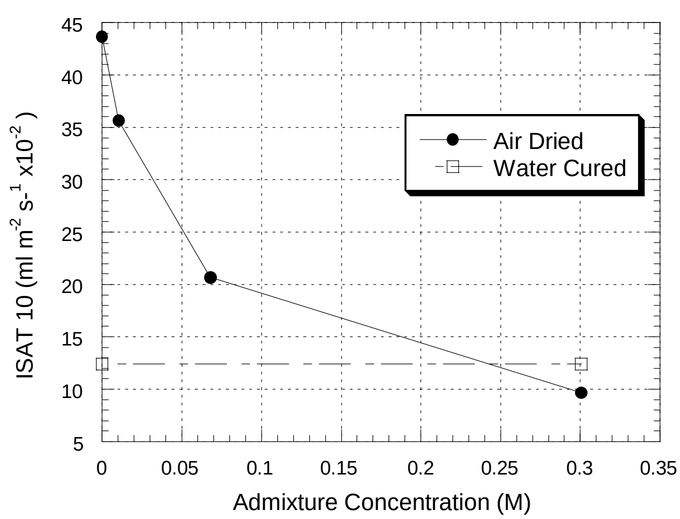


Figure 6.12 ISAT 10 values as a function of admixture concentration for air-dried and water-cured concrete specimens (Dhir et al. 1996)

and the cement paste. The paste-aggregate bond is probably adversely affected because the admixture alters the morphology of the CH crystals in the cement paste.

Figure 6.12 shows the ISAT 10 values (rate of absorption at 10 minutes after starting the test) as a function of the admixture concentration for air-dried and water-cured (control) concrete specimens. Notice that the ISAT value for the air-dried concrete at the highest admixture concentration is lower than that of the control. This result is surprising since the degree of hydration of an air-dried specimen with the admixture should not exceed that of a continuously water-cured specimen. As mentioned above, this may be caused by the effect of the admixture on the CH crystals. This research also found some evidence that the admixture alters the CSH gel morphology, reducing the absorptivity of the concrete. Even though these results show some strength reduction using high concentrations of the admixture, it is clear that reductions in water absorption characteristics can be obtained. Although more studies are needed on the feasibility of formulating "self-curing" concrete, particularly as it relates to high-performance concrete, these studies appear to offer encouraging results.

6.4 Research in Canada

6.4.1 *Effect of curing on high-strength concrete*—The effect of curing conditions on the development of some of the mechanical properties of very high-strength concrete was studied in the late 1980s by Asselanis et al. (1989). The concrete was made with Type II portland cement, the water-cement ratio was 0.31, and it included both silica fume and a superplasticizer. Compressive strengths ranged from 81.2 MPa (11 780 psi) to 99.2 MPa (14 390 psi). The test specimens were 100 mm × 200 mm (4 in × 8 in) cylinders, which were cured under three different conditions after 1 d in the molds. The curing conditions were:

- Continuous moist curing at 100 % relative humidity in a moist room at 23 ± 2 °C,
- Continuous air curing in laboratory air at 20 °C ± 3 °C (68 °F ± 5 °F) and 49 % ± 24 % relative humidity, and
- Moist curing for 7 d followed by air curing until test age

Specimens were tested at ages of (1, 7, 28, and 56) d for mass changes, stress-strain behavior, and compressive strength.

Tests indicated that prolonged moist curing of high-strength concrete beyond 7 d is normally unnecessary to achieve desired levels of strength and durability. After 7 d of moist curing, the concrete is essentially impervious, which would be indicative of a high level of durability. Those specimens cured in air after 7 d of moist curing continued to show improvement in compressive strength and modulus of elasticity. This can be attributed to the concrete having retained sufficient moisture to sustain hydration, even after the surface is exposed to drying conditions.

6.4.2 *Long-term strength development of high-strength concrete*—Studies were conducted on the strength development of silica-fume concrete with compressive strengths greater than 80 MPa (11 600 psi). Long-term results up to two and a half years were reviewed in this research (Read et al. 1991). Several different curing conditions were used, some of which were designed to simulate exposure conditions in a typical multi-story building. Six concrete mixtures were tested as shown in Table 6.8. Note that mixtures 1 through 5 have the same total cementitious materials content of 485 kg / m^3 (817 lb / yd^3); however, they have different proportions of portland cement and other cementitious materials. Mixture 5 was a

Table 6.8 Mixture Proportions Used in Long-term Strength Study (Read et al. 1991)

Mixture Number	Water-Cementitious Materials Ratio	Quantities, kg/m³ †				
		Water	Cement	Slag	Silica Fume	Fly Ash
1	0.29	143	315	135	35	-
2	0.28	133	317	167	-	-
3	0.27	130	449	-	39	-
4	0.27	132	427	-	59	-
5	0.29	102	150	-	-	200
6	0.27	130	485	-	-	-

† To convert to lb/yd^3 multiply by 1.686

special new type of concrete being developed in Canada using a very low cement content and high proportion of fly ash. Type I portland cement was used in all the mixtures.

The concrete was produced by a commercial batching plant. Three types of structural elements were cast for each of the mixtures to simulate different structural elements as follows:

- A 500 mm (20 in) thick wall,
- A 250 mm (10 in) thin wall, and
- A 2.0 m × 1.2 m (79 in × 47 in) large column.

These structural elements were cured under conditions to simulate field curing. Formwork was removed between 18 h and 24 h after casting, then they were cured with wet burlap for 6 d. After the 6 d of moist curing, the elements were placed under a waterproofed plywood roof erected at the test site to afford them some protection from the weather. To simulate exposure conditions normally experienced by columns and walls at a construction site, the sides of the enclosure were left open. Thus the elements were exposed to natural seasonal variations in temperature and humidity.

A large number of 150 mm × 300 mm (6 in × 12 in) cylinders were also cast from each mixture. Half of these were cured under standard moist-room conditions and the rest under field conditions. Those cured in the moist room were removed from their molds after 24 h and stored in the moist room at 23 °C (73 °F) and relative humidity in excess of 95 % until time of testing. The other half were also removed from their molds after 24 h, stored under wet burlap in the laboratory for 7 d, and then stored alongside the structural elements under the enclosure.

The moist-cured and field-cured cylinders were tested for compressive strength at ages of (1, 3, 7, 28, 91, 182, 365, 548, 730, and 912) d. For the structural elements, three cores were taken at each test age from the top, middle, and bottom of each element. These cores were then tested for compressive strength.

Test results for the cylinders were as follows:

- Early-age strength development was not greatly influenced by the curing condition.
- Long-term strength development was clearly affected by the type of curing.
- For all six mixtures, the compressive strengths after the age of 6 months were higher for moist-cured cylinders than for air-cured cylinders.
- The silica-fume and slag-cement mixtures were least affected by the curing conditions.
- The fly-ash mixture was most affected by the curing conditions.
- Independent of the curing conditions, the mixtures with silica fume showed faster rates of strength development up to 28 d. After 28 d, however, strength gains were lower than for those without silica fume, up to 365 d. After one year, strength gains for all mixtures were relatively small.

These results confirm that the influence of silica fume on rate of strength gain occurs primarily at early ages.

Test results for the cores removed from the structural elements were as follows:

- Compressive strengths at later ages were higher for the cores from the thin-wall elements compared with the thick-wall elements.
- Compressive strengths at later ages were higher for the cores from the thick-wall elements compared with the column elements.
- The type of structural element did not influence compressive strength development for ages up to 28 d.
- Just as for the cylinders, the silica-fume mixtures showed higher early-age strength gains compared with those without silica fume—the higher the amount of silica fume, the greater the early-age strength gain. However, after 28 d the mixtures without silica fume had higher additional strength gains.

The researchers surmised that the higher early-age temperature rise in the thicker elements (the thick walls and columns) probably resulted in microcracking that adversely affected the compressive strengths at later ages. However, there were no microscopic studies to confirm this belief.

The results also showed that the lack of moisture, that is, air–curing, had the most detrimental effects on the strength development of the cylinders made from the high-volume fly ash mixture. However, this did not apply to the strength development of the structural elements as measured by the strengths of cores. Apparently, the availability of moisture from within the large members was sufficient to maintain strength development. These results show that conclusions about the effects of different curing conditions on strength development obtained from cylinders exposed on all sides may not be representative of strength development within a large concrete member.

6.4.3 *Influence of silica-fume content on physical properties*—In the early 1990s, Hooton (1993) conducted studies on the effects of different amounts of silica fume (5 % to 20 % mass fraction of cement) on strength and durability-related properties. The concrete had a water-cementitious materials ratio of 0.25. Compressive strength was measured using 150 mm × 300 mm (6 in × 12 in) cylinders that were moist cured at 23 °C (73 °F). Tests were conducted at ages of (1, 7, 28, 56, 91, 182, and 365) d, and also at (2, 3, and 5) years. The compressive strengths of all silica-fume concrete specimens tested after 7 d of moist curing were 34 % to 57 % higher than those without silica fume—the higher the silica fume content, the higher the strength. After 56 d of moist curing, the silica-fume concretes ceased to gain additional compressive strength, which, according to the author was probably, due to self desiccation. Long-term compressive strength (2 years to 5 years) test results showed some scatter and inconsistencies (typical of long-term tests); however, there was a trend of somewhat lower strength values for both the concretes with and without silica fume.

Mortar specimens were prepared to determine sulfate resistance and alkali-aggregate reactivity. Sulfate resistance was evaluated in accordance with ASTM C 1012, and alkali-aggregate reactivity was evaluated in accordance with ASTM C 441. Tests to evaluate freezing and thawing resistance were conducted on concrete specimens in accordance with ASTM C 666. Test results showed that silica fume enhances the resistance to sulfate attack, alkali-aggregate reactivity, and freezing and thawing. Hooton (1993) concluded that 10 % mass fraction replacement of portland cement by silica fume is adequate to provide enhanced levels of protection without detrimental effects of increased drying shrinkage and the resultant microcracking.

6.4.4 *Drying shrinkage of high-performance concrete*—A research program was initiated in the mid-1990s to learn more about the drying shrinkage of ready-mixed, high-performance concrete (Hindy et al. 1994). Tests were performed on two different HPC mixtures. The first mixture had a 28-day compressive strength of 92.6 MPa (13 430 psi) and contained 7 % to 8 % mass fraction of silica fume in a blended cement. The second mixture had a 28-day compressive strength of 73.0 MPa (10 590 psi) and did not contain silica fume. The respective water-cementitious materials ratios were 0.22 and 0.28. A high-range water reducer was used with both mixtures to give a slump of about 200 mm (8 in).

Two load-bearing columns were instrumented at Concordia University in Montreal, Canada, for this research study. They are 850 mm (33.5 in) diameter columns in the basement of a campus library constructed in 1990. Two unloaded, reference columns were also constructed near the load-bearing columns using the same concrete mixtures. A large number of control specimens were also cast from each of the two mixtures to determine various material properties, such as compressive strength, modulus of elasticity, creep and shrinkage, and freeze-thaw durability. The control specimens were cast on site, covered

Table 6.9 Curing Conditions in Study of Drying Shrinkage of HPC (Hindy et al. 1994)

Curing Condition	Specimen Type (number)
Air storage for 365 d	Cylinder (2)
	Prism (2)
Sealed for 4 d, then stored in air for 361 d	Cylinder (2)
Sealed for 7 d, then stored in air for 358 d	Cylinder (2)
Sealed for 28 d, then stored in air for 337 d	Cylinder (2)
Sealed for 365 d	Cylinder (2)
	Prism (2)
Water cured for 365 d	Cylinder (2)
	Prism (2)

with wet burlap overnight, and transferred to the laboratory the next day where the molds were removed. For the shrinkage studies, three curing conditions were used as follows:

- *Sealed curing*: 100 mm × 375 mm (4 in × 14.75 in) cylinders and 100 mm × 100 mm × 375 mm (4 in × 4 in × 14.75 in) prisms were sealed first with a plastic sheet and then with an aluminum foil. After different periods of sealed curing, the coverings were removed and the specimens were stored in the laboratory at 20 °C (68 °F) and a relative humidity of 50 %.
- *Water curing*: Specimens were placed in lime-saturated water at a temperature of 20 °C (68 °F).
- *Air storage*: Specimens were stored in the laboratory at 20 °C (68 °F) and a relative humidity of 50 %.

Table 6.9 summarizes the curing histories for the shrinkage specimens.

Shrinkage measurements in the laboratory were performed in accordance with ASTM C 157. For the reference columns, shrinkage measurements were taken with vibrating wire extensometers that were located at the column midheights.

Some of the key results from this research were:

- The longer the duration of sealed curing, the less was the drying shrinkage, for both concretes.
- For the curing conditions studied, drying shrinkage was less for the higher strength concrete which had a w / (c + SF) ratio of 0.22 compared with the concrete without SF and w / c = 0.28.

Table 6.10 Volume Change of HPC Specimens After 1 year (Hindy et al. 1994)

Specimen and Curing	Type of HPC	Volume Change	
		Shrinkage	Swelling
Air-stored cylinders	Without SF; w / c = 0.28	760×10^{-6}	
	With SF; w / (c+SF) = 0.22	650×10^{-6}	
Permanently-sealed cylinders	Without SF; (w / c = 0.28)	378×10^{-6}	
	With SF; w / (c+SF) = 0.22	322×10^{-6}	
Water cured-prisms	Without SF; w / c = 0.28		134×10^{-6}
	With SF; w / (c+SF) = 0.22		27×10^{-6}

- The duration of sealed curing had a greater influence on the shrinkage of the HPC without SF than on the HPC with SF.
- A comparison of the drying shrinkage for cylinders and prisms for both concretes showed that the cross-sectional shapes, which had the same ratio of volume to surface area, had no effect on the results.

Table 6.10 summarizes the drying shrinkage and swelling strains measured on the test specimens after one year. As expected, the effect of curing conditions on drying shrinkage is quite significant. As previously mentioned, drying shrinkage was less for the silica-fume concrete, which also had the lower water-cementitious materials ratio. It can also be observed that the silica-fume concrete was much less susceptible to swelling under saturated conditions.

The following expression, based on the hyperbolic equation recommended by ACI Committee 209, was used to represent the development of drying shrinkage:

$$\varepsilon(t-t_o) = \frac{(t-t_o)^b}{a+(t-t_o)^b}\varepsilon_\infty \tag{6.6}$$

where,

t = concrete age;
t_o = concrete age when drying shrinkage begins;
a and b = are coefficients determined by least-squares fitting; and
ε_∞ = ultimate shrinkage strain, also determined by least-squares fitting.

The parameter a is related to the initial rate of drying shrinkage. The researchers found that the best-fit values for the parameters a and b differed from the values normally assumed for ordinary concrete, as shown in Table 6.11.

Table 6.11 Values of Parameters in Hyperbolic Shrinkage-time Equation (Hindy et al. 1994)

Parameter	Value for Ordinary Concrete (ACI 209)	Value Obtained for HPC
a	35 d	10 d to13 d
b	1.0	0.5 to 1.0

Note: The values for HPC varied according to curing condition. The three curing conditions considered were permanently in air, 7 d sealed then in air, and 28 d sealed then in air. The permanently sealed curing condition is not included among these values because it gave a significantly larger value for *a*.

Hindy et al. (1994) concluded that the permanently sealed curing condition comes closest to simulating the drying behavior of high-performance concrete structures in the field. This is based on the dense nature of this concrete, and the fact that a large structural member, with a low surface-to-volume ratio, is not prone to lose much water in the field. So, the values of a = 80 d and b = 1, which were obtained for the permanently sealed condition, were recommended for use in estimating the development of drying shrinkage in actual high-performance concrete structures. The use of these values results in a lower rate of shrinkage than for the small laboratory specimens.

Shrinkage strains for the two unloaded reference columns were monitored. It was found that between 70 d and 330 d, the increase in drying shrinkage strain was negligible for the silica-fume concrete and it was only 20×10^{-6} for the concrete without silica fume. These same values for air cured laboratory specimens were, respectively, 162×10^{-6} and 227×10^{-6}. This large difference can be attributed to the much larger volume-to-surface ratio of the columns compared with the laboratory specimens. These results highlight the importance of considering member size when dealing with processes that involve moisture movement through high-performance concrete.

6.4.5 *Deicer salt scaling deterioration of concrete*—Proper curing is vital to develop sufficient surface durability to resist degradation by environmental actions, such as exposure to deicer salts commonly used on roads and bridges. Although water curing is generally felt to be the best, tests have shown that concrete cured with membrane-forming curing compounds have exhibited better resistance to deicing salts than those cured with an external supply of water. Air entrainment has also proven to be helpful in improving resistance to this type of scaling (Marchand et al.1994).

Marchand et al. provide a comprehensive summary of recent research on the deicer salt scaling of concrete. In addition, they discuss the influence of curing on the durability of concrete and its resistance to deicer salts. Their overview covers the basic concepts of frost action in concrete, freezing phenomena, and deterioration caused by deicing salts.

As mentioned above, recent research has confirmed that concretes cured with membrane-forming compounds exhibit better resistance to salt induced scaling than moist-cured concretes. This finding has been consistent in both laboratory and field experiments. Moist curing generally leads to the development of a denser concrete in the surface layer than does curing with a membrane-forming compound. This can actually be detrimental to the concrete's resistance to scaling. When freezing occurs, water movement from the concrete interior to the surface can be hindered, resulting in an increased likelihood of damage to the surface.

Research has also confirmed that early-age curing at high temperatures (common in precast plants) is detrimental to the scaling resistance of some concretes. These curing conditions cause a coarser pore structure than with normal curing practices. The coarser microstructure that results from rapid hydration at high curing temperatures is believed to be more susceptible to surface scaling.

Traditionally, air entrainment has been used in ordinary concrete to enhance its resistance to freezing and thawing and scaling by deicer salts. Since entrained air reduces bleeding and the permeability of the concrete, resistance to freezing and thawing in the surface layers is improved (Neville 1996). Air entrainment also densifies the concrete because it improves workability and allows a reduction in water content. High-performance concretes generally have low water-cement ratios in the range of 0.25 to 0.35. These concretes have little free water available for freezing, so air entrainment may not be required for protection against deicer salt scaling (Snyder, Clifton, and Knab 1994). Marchand et al. (1994) report that air entrainment is generally not required for scaling resistant high-performance concrete when these two conditions are met:

- Silica fume is used as a partial replacement for portland cement, and
- The water-cementitious materials ratio is 0.30 or less.

If concretes are made without silica fume, a water-cementitious material ratio of about 0.25 is recommended.

Curing significantly affects the characteristics of the surface zone, which is the crucial region in regard to resistance against deicer salt scaling. Poor curing practices can also cause plastic shrinkage cracking, which reduces surface durability. Although the complexities of the mechanisms involved in deicer salt scaling are not yet fully understood, it is clear that good curing practices are vital for resistance to the damaging effects of these phenomena.

6.5 Research in France

6.5.1 *Long-term strength losses of silica-fume, high-strength concrete*—Results have shown that there will be continuous increase in compressive strength of silica-fume, high-strength concrete when cured in water. Some researchers (Fouré and Bronsart 1989), however, have noted reductions in the long-term strength of these concretes when stored in air. de Larrard and Bostvironnois (1991) hypothesized that the observed strength reduction after extended storage in air was the related to the effects of drying of the test specimens. An experimental study was carried out to test this hypothesis. They conducted mechanical tests on three different concretes and three storage conditions. Three concretes were tested:

- An ordinary portland cement (control) concrete with w / c = 0.487,
- A high-strength concrete designed for a low heat of hydration with w / (c + SF) = 0.523, and
- A very high-strength concrete with w / (c + SF) = 0.242.

Three storage conditions were used:

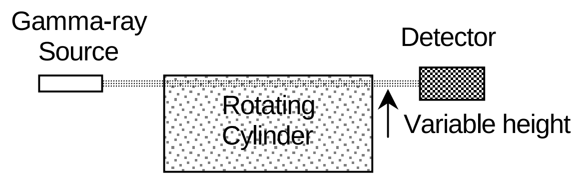
- Cured in water at 20 °C (68 °F),
- Stored in air at 20 °C (68 °F) and 50 % relative humidity, and
- Sealed with two layers of aluminum tape so that there was no exchanges of moisture.

Compressive strength tests were conducted at ages of 28 d, 3 months, and 4 years. Test cylinders were 160 mm × 320 mm (6.3 in × 12.6 in). The moisture distribution within the cylinders was measured as a function of time using a through transmission gamma-ray method. A cylinder was positioned so that the gamma rays penetrated along its length at a fixed distance form the axis. This allowed measurement of attenuation as function of radial distance, from which the variation in moisture content was determined. A schematic of the testing technique is shown in Fig. 6.13(a).

To measure the effects of uniform drying to the core, 40 mm × 40 mm × 160 mm (1.6 in × 1.6 in × 6.3 in) prismatic mortar specimens were used. The mortar specimens were removed from their molds after 1 d, sealed with aluminum sheets, and placed in an oven at 35 °C (95 °F) to accelerate hydration. After 14 d in the oven, half of the specimens were uncovered and oven-dried at 40 °C (104 °F) to constant mass, and the remainder were maintained sealed at 20 °C (68 °F). To reduce the drying gradients, small holes were drilled through the prisms at the third points. The effect of the holes on the measured compressive strength was considered. The mortar test results of the were as follows:

- Drying reduced the strength of the control mortar by about 7 %.
- Drying slightly increased the strength of the high-strength and very high-strength mortar by about 5 % and 2 %, respectively.

The conclusion drawn from the mortar tests is that the losses of compressive strength in high-strength concrete reported by some researchers are apparently not due to drying when there is a small drying gradient in the test specimen.



(a) Gamma ray density measurement

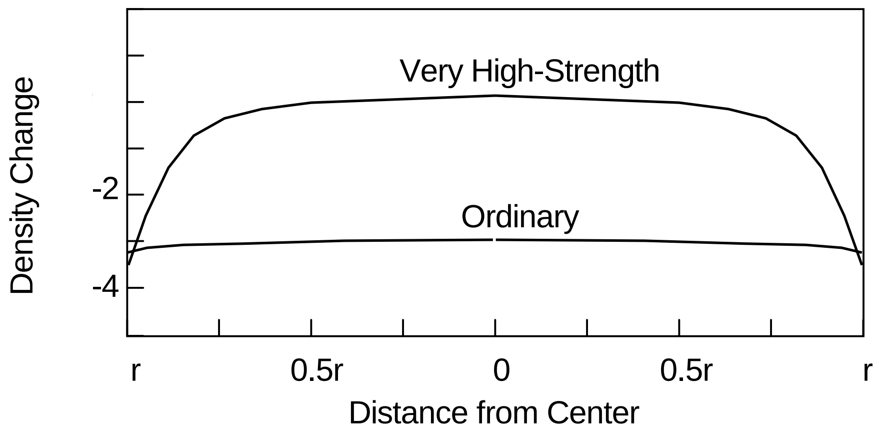


Figure 6.13 (a) Schematic of method to measure moisture distribution in drying cylinders and (b) schematic of density variation with radial distance after 4 years of drying (based on de Larrard and Bostvironnos 1991)

The gamma-ray tests of the concrete cylinders allowed comparison of the average moisture distribution across the diameter of the specimens. For the water-cured concrete, the moisture distribution was nearly uniform across the cylinders. For cylinders cured in air at 20 °C (68 °F) and 50 % relative humidity, the following observations were noted.

- For the ordinary concrete, drying occurred with a relatively high gradient at 3 months; however, the gradient had disappeared at 4 years.
- In the high-strength concrete mixture, drying occurred also with a low gradient
- In contrast to the above, the very high-strength concrete developed high moisture gradients with time. The drying was confined to the “skin” of the cylinders. After 4 years of exposure, only the first 3 cm (1.2 in) of the surface had shown the effects of drying.

Figure 6.13(b) is a schematic to illustrate the differences in the moisture content distribution for the ordinary and very high-strength concrete cylinders after 4 years of drying.

The authors measured strength up to only 3 months in their study, and they relied on the 4-year strengths from a previous study (using similar concrete) to arrive at an estimate of the reduction in strength between 3 months and 4 years due to drying. Compressive strength from the earlier study showed that the water-cured specimens of the very high-strength concrete gained strength continuously to an age of 4 years. As explained in their paper, the authors estimated the strength reduction expected between 3 months and 4 years for the air-dried specimens of the same concrete. This estimated strength reduction ranged between about 5 MPa (725 psi) and about 7 MPa (1 000 psi).

de Larrard and Bostivironnis (1991) theorized that air-dried specimens had lower strength compared with water-cured specimens because of shrinkage induced stress due to the non-uniform moisture distribution. Since the outer regions of the cylinders were much drier than the interior, the exterior region would tend to shrink and the interior region would restrain the shrinkage. Thus the outside of the cylinder would be in tension and the interior would be in compression. Based on the measured moisture distribution in the very high-strength concrete cylinders, de Larrard and Bostivironnis calculated the self-equilibrating stress distribution at the middle of the cylinder. The calculated maximum compressive stress at the center of the very high-strength concrete cylinder after 4 years of drying was approximately 5 MPa (725 psi). This compares favorably with the estimated strength reduction. Thus the long-term loss of compressive strength can be attributed to the internal drying stresses. These "self-stresses" result in a pre-compression of the core which reduces the magnitude of externally applied stress to reach ultimate strength. The magnitude of the compressive self-stress depends on the magnitude of the moisture gradient across the specimen cross section. By means of an innovative reasoning, the authors concluded that the maximum value of the compressive self-stress is twice the tensile strength of the concrete.

In summary, de Larrard and Bostivironnis (1991) demonstrated that it is plausible that the "strength retrogression" of very high-strength concrete observed by some investigators can be explained by the moisture gradients in the test specimens. According to the authors, this may help to explain why there have been no reported cases where the strength of cores taken from structures have shown a strength loss with time. The cores, being from the interior of the member would not have the same moisture gradients as cylinders allow to dry in the laboratory.

6.5.2 *Effects of curing method on the durability of high-performance concrete*—Hasni et al. (1994) evaluated the effects of three storage conditions on the strength and durability-related characteristics of three concrete mixtures—a high-performance concrete with silica fume, a high-performance concrete without silica fume, and a reference concrete with no additives and a water-cement ratio of 0.55. The water-cement ratio for the high-performance concrete without silica fume was 0.34, and the water-cementitious materials ratio for the one with silica fume was 0.33. The specimens were subjected to the following storage conditions (there is no mention of an initial moist-curing period):

- At 100 % RH and 20 °C (68 °F) for 50 d;
- Ambient laboratory conditions of 50 ± 10 % RH and 30 °C (86 °F) for 50 d; and
- Drying by subjecting the specimens to a flow of hot air at 30 °C to 35 °C (86 °F to 95 °F) and 20 % RH for 7 h / d, for 50 d.

Table 6.12 Results of Various Tests at Age of 50 d (Hasni et al. 1994)

Concrete Mixture	Exposure Condition	Porosity %	Compressive Strength MPa (psi)	Flexural Strength MPa (psi)
Reference Concrete	100 % RH	13.4	61.5 (9 000)	7.7 (1 100)
	Laboratory air	15.4	48.4 (7 020)	5.7 (830)
	Hot air, 7 h / d	14.7	47.2 (6 850)	5.8 (840)
HPC without SF	100 % RH	12.5	90.1 (13 080)	9.5 (1 400)
	Laboratory air	13.5	84.6 (12 280)	8.9 (1 300)
	Hot air, 7 h / d	13.9	81.3 (11 760)	8.2 (1 200)
HPC with SF	100 % RH	11.6	90.1 (13 080)	9.3 (1 350)
	Laboratory air	11.8	74.1 (10 760)	7.3 (1 060)
	Hot air, 7 h / d	11.2	62.0 (9 000)	6.9 (1 000)

The following durability-related characteristics were compared:

- Carbonation depth and reduction in alkalinity,
- Permeability to chloride ions under imposed electrical potential, and
- The nature of microcracking, porosity, and microstructure.

Compression and flexural strength tests, porosity, and depth of carbonation (after 28 d of exposure to a concentrated carbon dioxide mixture) were determined on 70 mm × 70 mm × 280 mm (2.75 in × 2.75 in × 11 in) prisms. A scanning electron microscope was used to examine the extent of microcracking and the nature of the microstructure on fracture planes near the surface and the interior of 50 mm × 50 mm × 20 mm (2 in × 2 in × 0.75 in) concrete specimens. Resistance to chloride ion penetration was evaluated in accordance with the electrical procedure given in AASHTO T 277 (similar to ASTM C 1202).

Table 6.12 shows the porosity, compressive strength, and flexural strength after 50 d of storage under the different exposure conditions.

Figure 6.14 shows the compressive and flexural strengths of specimens stored in the laboratory and exposed to hot air as a fraction of the strength of the specimens stored at 100 % RH. It is seen that the strength reductions for exposure to drying conditions are smallest for the high-performance concrete without silica fume. The concrete with silica fume had the greatest compressive strength reduction for the hot air exposure condition.

The microscopic study showed that the high-performance concrete with silica fume had more microcracking for the hot-air exposure than the concrete without silica fume. The porosity near the surface was also greater for the concrete with silica fume. The dryer the curing method, the deeper was the penetration of carbonation below the surface. The high-performance concrete without silica fume showed the least effects of carbonation, and the

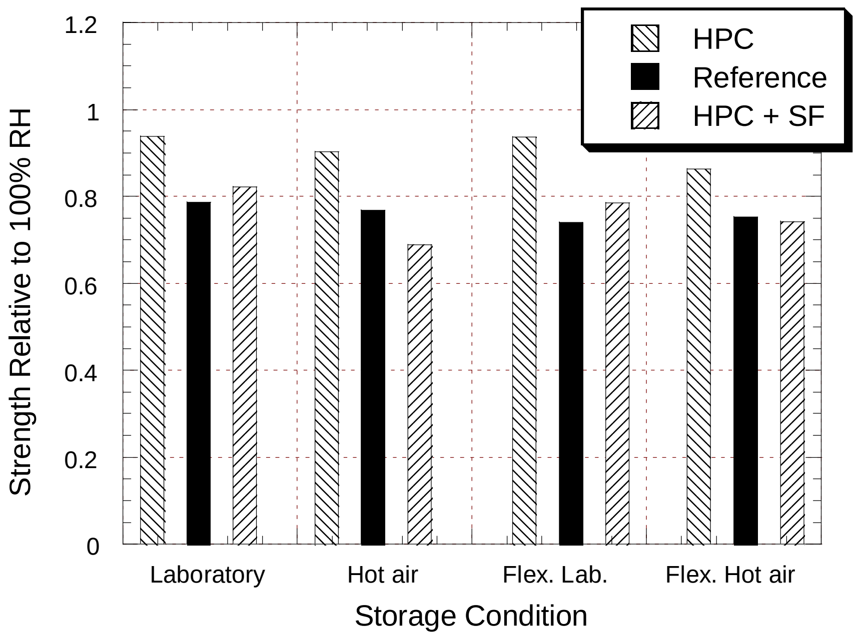


Figure 6.14 Strengths for specimens stored in the laboratory or exposed to hot air in relation to strengths of specimens stored at 100 % RH (Hasni et al. 1994)

concrete with silica fume experienced slightly more depth of carbonation (2 mm [0.08 in] to 3 mm [0.12 in] more) than the reference concrete after 28 d of exposure to a concentrated amount of carbon dioxide . A possible reason for this observation is that the concrete with silica fume had less calcium hydroxide and was more prone to carbonation.

The resistance to chloride ion penetration, as characterized by the electrical method of AASHTO T 277, was not affected by the exposure condition, and was in agreement with the relative porosities listed in Table 6.12. Thus the concrete with silica fume had the highest electrical resistance and the reference concrete had the lowest electrical resistance. Since details of the testing procedure were not provided by Hasni et al. (1994), it is not possible to explain why the exposure condition was not a significant factor. It may be that the tests were not done on the “skin” concrete.

In summary, these results showed that the effects of poor curing were more pronounced for the concrete with silica fume, even though both high-performance concretes had the same compressive strength after 50 d at 100 % RH, i.e., 90.1 MPa (13 080 psi). It was also concluded that the exposure conditions adversely affected the near surface concrete more than the interior concrete, and reinforced the need for good curing of the surface when maximum durability is desired.

6.5.3 *Effects of curing on carbonation*—Balayssac et al. (1995) studied the effects of duration of moist curing on the resistance to carbonation. The concretes that were used can

Table 6.13 Depth of Carbonation in mm as a Function of Age for Different Durations of Moist Curing (Balayssac et al. 1995)

Concrete	Properties	Curing duration	Age 90 d	180 d	360 d	540 d
B1	$C = 300\ kg/m^3$	1 d	6.5	11	13	15
	$w/c = 0.65$	3 d	4	6	9.5	13
	$f_{c28} = 25.1$ MPa	28 d	3	5	6	9
B2	$C = 340\ kg/m^3$	1 d	5.5	10	12	13
	$w/c = 0.61$	3 d	3.5	5	6	8
	$f_{c28} = 32.6$ MPa	28 d	2.5	4	4.5	6
B3	$C = 380\ kg/m^3$	1 d	4.5	9	10	11.5
	$w/c = 0.53$	3 d	3	5	5.5	7
	$f_{c28} = 37.8$ MPa	28 d	2	3.5	4	5
B4	$C = 420\ kg/m^3$	1 d	4	7.5	8.5	9.5
	$w/c = 0.48$	3 d	2.5	4	3.5	4
	$f_{c28} = 43.5$ MPa	28 d	1.5	3	3	3.5

1 in = 25.4 mm

not be classified as high-performance, however, the results point out the importance of water-cement ratio on the effects of moist curing.

Four concrete mixtures were made with cement composed of 75 % mass fraction portland cement clinker and 25 % limestone filler. The concretes were proportioned with different cement contents and water-cement ratios. Cylindrical test specimens were moist cured for (1, 3, and 28) d, after which they were stored in air at 20 °C (68 °F) and 60 % relative humidity for up to 18 months. The carbon dioxide content of the air was 0.03 %. At ages of (90, 180, 360, and 540) d, test specimens were split and the depth of carbonation was measured using phenolphthalein. Three replicate specimens were measured for each condition. Table 6.13 summarizes the characteristics of the four concrete mixtures and the average depths of carbonation as a function of age and duration of the moist-curing period.

As expected, the depth of carbonation at any age decreased as the water-cement ratio decreased or the duration of moist curing increased. However, it can be seen that when the moist-curing period increased from 3 d to 28 d, there was generally less improvement with decreasing water-cement ratio. This confirms the findings of others that high-performance concrete may be less sensitive to the duration of curing than ordinary concrete.

Balayssac et al. (1995) tested other concretes made with ordinary portland cement and with blended cement containing 65 % mass fraction of slag. They found good correlation between the carbonation depth at 18 months and the compressive strength at the end of a 3-day moist curing period. Thus it was concluded that strength at the termination of moist curing might be a good predictor of long-term resistance to carbonation. These findings are consistent with those of Parrott (1992), as summarized in Section 6.3.4.

6.6 Research in Australia

6.6.1 *Effects of curing on drying shrinkage in high-strength concrete*— Sri Ravindrarajah et al. (1994) investigated the shrinkage and swelling properties of high-strength concrete caused by moisture movements within the paste. The primary focus was to study the effects of binder type and duration of the initial water curing period on the development of drying shrinkage and subsequent swelling. Five different concrete mixtures were used containing from 500 kg/m^3 to 600 kg/m^3 (842 lb/yd^3 to 1 012 lb/yd^3) of cementitious material. Three of the mixtures contained silica fume ranging from 5 % to 15 % of total cementitious material, and the 5 % silica fume mixture also contained 5 % fly ash. One mixture contained blended cement with 35 % ground granulated blast-furnace slag. Water-cementitious materials ratios ranged from 0.23 to 0.35, and 28-day cube strengths for continuous moist curing were from 81.1 MPa (11 760 psi) to 107.0 MPa (15 500 psi). Specimens were cast in steel molds to use in testing for compressive strength, tensile strength, modulus of elasticity, chemical resistance, and drying shrinkage at various ages. The specimens for monitoring drying shrinkage were 75 mm $\times$ 75 mm $\times$ 280 mm (3 in $\times$ 3 in $\times$ 11 in) prisms. These were cured in water for either 3 d or 460 d, and then allowed to dry under laboratory conditions. Strains were measured on two opposite sides of the prisms over a 200 mm (8 in) gage length using a demountable mechanical gage. Specimens tested for swelling were subjected to wetting after 460 d of drying (initially moist cured for 3 d). Swelling strains were measured over a period of 55 d. Table 6.14 summarizes the test results, and Fig. 6.15 shows the shrinkage strains, after different drying periods, as a function of the water-cementitious materials ratio.

The following summarize the major observations and conclusions from this study:

- Drying shrinkage strain at 460 d for specimens initially water-cured for 3 d was between 545×10^{-6} and 775×10^{-6}, depending upon the cementitious materials and water-

Table 6.14 Drying Shrinkage and Swelling Strains for High-strength Concrete (Sri Ravindrarajah et al.1994)

	Mixture 1	**Mixture 2**	**Mixture 3**	**Mixture 4**	**Mixture 5**
Cementitious materials	100 % PC	8% SF	15 % SF	5% SF + 5% FA	35 % GGBFS
Water-cementitious materials ratio	0.30	0.35	0.23	0.28	0.30
28-d comp. Strength, MPa)	89.7	87.0	107.0	102.4	81.1
*Shrinkage @ 28 d; 3 d m-c	380	520	355	320	480
Shrinkage @ 100 d; 3 d m-c	590	685	440	435	615
Shrinkage @ 460 d; 3 d m-c	665	775	575	545	695
Shrinkage @ 100 d; 460 d m-c	395	350	280	245	240
Mass gain @ 55 d – 460 d dry, %	2.11	2.5	0.91	1.46	1.51
*Swelling @ 55 d – 460 d dry	410	410	260	415	305

*Shrinkage and swelling strains are given in units of $\times 10^{-6}$. The symbol "m-c" stands for moist curing.

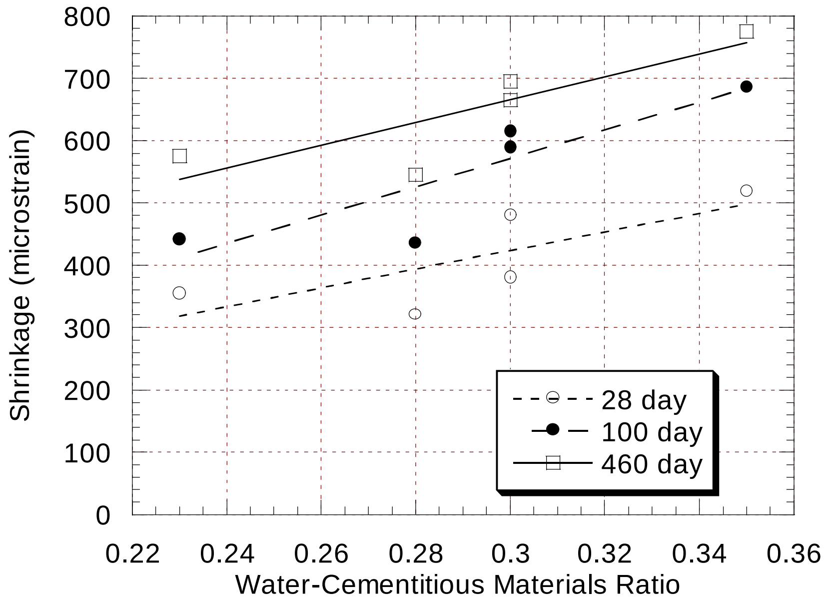


Figure 6.15 Drying shrinkage versus water-cementitious materials ratio for different durations of drying after 3 d of moist curing (Sri Ravindrarajah et al. 1994)

cementitious materials ratios. In general, the ultimate shrinkage strain was directly related to the water-cementitious materials ratio. The exception was the mixture with 5 % silica fume and 5 % fly ash, which had lower than expected shrinkage.

- Recoverable shrinkage (i.e., swelling) after 55 d of soaking was between 44 % and 76 % of the drying shrinkage at 460 d. The lowest recoverable shrinkage (as a percentage of the drying shrinkage) occurred in the mixtures with 15 % silica fume and the blended cement with 35 % slag content.
- Drying shrinkage after 100 d of air drying for concretes that had 460 d of water curing prior to drying was between 39 % and 67 % of the corresponding shrinkage for similar concretes that were initially water-cured for only 3 d. The smallest percentage was for the concrete with blended cement (Mixture 5).

The last of the above observations reinforces the notion that prolonged moist curing is beneficial in reducing the potential drying shrinkage in high-strength concrete. Because Ranvindrarajah et al. (1994) compared the drying shrinkage of mixtures having different water-cementitious materials ratios, it is difficult to draw conclusions regarding the effects of cement replacements. However, the water-cementitious materials ratios of mixture 1 (portland cement only), mixture 4 (5 % silica fume + 5 % fly ash), and mixture 5 (blended cement) are comparable. The results for the 460-day cured specimens from these mixtures show that the potential shrinkage of well-cured, high-strength concrete can be significantly reduced by using supplementary cementitious materials.

6.6.2 *Effectiveness of different curing methods*— Ho (1992) investigated the effectiveness of different curing techniques on durability related properties of concrete. Although this research did not specifically address high-performance concrete, it provides quantitative results on the performance of commonly used curing methods. The results of tests for water sorptivity, carbonation, and abrasion resistance, on both laboratory specimens and full-size columns, were used as indicators of surface quality due to different curing procedures. The author's objective was to develop the basis for improving project specifications to enhance the durability of concrete structures.

The curing methods included the following:

- Wrapping with plastic sheeting,
- Applying curing compounds,
- Keeping formwork in place,
- Covering with moist sand, and
- Using steam curing.

These are representative of methods commonly used in the field, and with the exception of steam curing, are typically called "water retaining" methods since their function is to reduce the loss of water incorporated into the concrete mixture. The development of durability related surface properties using these methods was compared with their development under standard moist-room curing.

Three grades of concrete (as given in Australian standards) were used, as shown below:

Concrete Grade	Water-Cement Ratio (mass fraction)
25	0.69
32	0.57
40	0.45

These grades correspond to the nominal compressive strength in megapascal. Normal portland cement (equivalent to ASTM Type I) was used, and the average slump for all concrete grades was 70 mm (2 ¾ in).

For the laboratory specimens, water sorptivity tests were performed on concrete slabs with dimensions of 400 mm × 170 mm × 60 mm (15.8 in × 6.7 in × 2.4 in). In theses tests, the depth of penetration of water was measured as a function of soaking time, and the slope of the straight line fitted to the data of depth versus square root of time was taken as the "sorptivity" value. Depth of penetration was estimated by visual observation of the moisture front after splitting the specimens. Carbonation tests were conducted on 75 mm × 75 mm × 210 mm (3 in × 3 in × 8.5 in) prisms stored in a chamber with air containing 4 % carbon dioxide and at 50 % relative humidity. Abrasion resistance was determined using the sand blasting method described in ASTM C 418.

Three columns were cast from three grades of concrete obtained from a commercial source. The column cross-sections were 800 mm × 800 mm (31.5 in × 31.5 in), and the columns were cast in 300 mm (12 in) segments separated by plastic sheets. Various control specimens (with the same dimensions as in the laboratory study) were also made for compressive strength, water sorptivity, and carbonation tests. The control specimens were subjected to standard moist curing. Cylinders with 100 mm (4 in) diameter were used for compressive strength tests. Column formwork was removed after 1 d, at which time the various curing methods were applied. In addition, a section of each column was removed at 1 d and moist-cured for 6 d. At ages of 7 d and 28 d, 100 mm (4 in) diameter cores were taken from the column segments and tested for water sorptivity and depth of carbonation.

Table 6.15 lists the average compressive strengths at different ages and for different curing conditions for the concrete mixtures prepared in the laboratory. Table 6.16 lists the average compressive strengths of cylindrical specimens made from the commercial concrete used in the columns. Standard curing refers to curing in a moist room (fog spray) at 23 °C (73 °F).

Table 6.15 Average Compressive Strengths of Cylinders Made From Laboratory Concretes, in MPa (Ho 1992)

	Type of Curing and Test Age					
Concrete Grade	**Unmolding after 1 d; 1-day strength**	**Unmolding at 1 d then 6 d standard curing; 7-day strength**	**Unmolding at 1 d then 27 d standard curing; 28-day strength**	**Steam cured at 70 °C for 8.5 h; 1-day strength**	**Steam cured then 6 d in water bath; 7-day strength**	**Steam cured then 27 d in water bath; 28-day strength**
25	5.5	20.5	28.0	13.5	16.5	21.5
32	8.5	27.5	35.0	20.0	23.0	30.5
40	15.0	37.0	45.0	27.5	30.75	41.0

Table 6.16 Average Compressive Strengths of Cylinders Made From Commercial Concretes Used in Columns, in MPa (Ho 1992)

	Type of Curing and Test Age		
Concrete Grade	**Unmolded after 1 d; 1-day strength**	**Unmolded at 1 d then 6 d standard curing; 7-day strength**	**Unmolded at 1 d then 27 d standard curing; 28-day strength**
25	6.5	21.0	31.0
32	7.0	27.0	39.5
40	9.0	41.5	50.5

For the laboratory concretes, the ratio of the 7-day to 28-day strength ranged from 73 % for grade 25 to 82 % for grade 40. The ratios of the 1-day to the 28-day strength were 20 %, 24 %, and 33 % for grade 25, grade 32, and grade 40, respectively. From these results, Ho (1992) suggested that higher strength concretes would be expected to be less sensitive to the lack of initial curing because of their higher rate of early-age strength gain. For the commercial concretes, the 7-day to 28-day strength ratios were 68 % for both grade 25 and grade 32, and 82 % for grade 40. The ratio of 1-day to 28-day strength was about 19 % for the three grades of commercial concrete. The generally lower rate of early-age strength gain for the commercial concretes compared with the laboratory concretes was attributed to differences in fineness and chemical composition of the cements that were used.

The results of water sorptivity, depth of carbonation, and abrasion tests for the specimens cured in the laboratory with *standard moist curing* were similar, and are summarized as follows:

- The quality of concrete improved with increased duration of moist curing.
- The greatest improvements in quality occurred when the curing duration was increased from 1 d to 7 d. Increasing the duration of curing from 7 d to 28 d resulted in marginal improvements in the measured properties.
- The properties of the laboratory-prepared and commercial mixtures were similaar.

The major of objective of Ho's work was to compare the results obtained under standard curing with those obtained under "practical curing conditions." Practical curing refers to the types of procedures that are used in construction, as opposed to standard laboratory moist curing. Curing of vertical and horizontal surfaces was considered.

Vertical surfaces—The practical curing procedures investigated for use on vertical surfaces (the most challenging condition to obtain good curing) included the following "water retaining" or "minimum water loss" techniques:

- Keeping the formwork in place for (3, 7, or 28) d;
- Sealing with plastic sheeting;
- Sealing with damaged plastic sheeting containing small holes;
- Keeping the formwork in place for 3 d, then sealing with plastic sheeting for 4 d;
- Using a curing membrane of wax emulsion (1 coat); and
- Using a curing membrane of 1 coat or 2 coats of chlorinated rubber.

Table 6.17 shows selected results for the sorptivity values of vertically cast surfaces for specimens made with the laboratory concretes. Vertical surfaces present the biggest challenge in providing adequate curing procedures. The results obtained with standard curing are compared with the results obtained by sealing the specimens with plastic sheeting. The specimens were kept in their molds for the first day, prior to application of the curing procedure. Multiple entries in Table 6.17 indicate replicate results. For standard moist-curing, the results are at (1, 7, and 28) d, while they are at (3, 7, and 28) d for curing

Table 6.17 Sorptivity in mm/h$^{0.5}$ for Different Curing Conditions on Vertical Surfaces of Specimens From Laboratory Concretes (Ho 1992)

Curing Method	Grade 25	Grade 32	Grade 40
Standard Curing:			
1 day in mold	18.5, 20.0, 18.0	10.0, 11.5, 10.5	6.5, 7.5
1 d mold + 6 d moist	5.5, 4.5, 4.0	2.5, 3.5, 3.0	2.0, 2.0
1 d mold + 27 d moist	3.5, 4.0, 3.0	2.5 , 1.5, 2.5	2.0, 1.5
Plastic Sheeting			
1 d form + 2 d sealed	11.0	7.5	
1 d form + 6 d sealed	6.5, 7.5	5.5, 6.5	5.5
1 d form + 27 d sealed	7.0	5.0	5.5

with plastic sheeting. A lower value of the sorptivity means that water penetrated into the concrete more slowly, and is indicative of higher quality concrete.

A review of all the sorptivity results for vertical surfaces lead Ho (1992) to make the following observations and conclusions:

- Low strength concretes are more sensitive to the lack of initial curing.
- The surface quality (in terms of sorptivity) achieved by practical curing procedures was equivalent to (at best) the quality achieved by 2 d to 3 d of standard moist curing.
- Use of plastic sheeting is effective in improving the surface quality of grade 25 concrete and grade 32 up to 7 d of protection. There was little improvement in surface quality by keeping the sheeting in place beyond 7 d. For grade 40 concrete, the use of plastic sheeting gave only slight improvement in surface quality over that achieved after removal of the formwork at 1 d.
- Extended curing in the formwork for grade 25 and grade 32 concrete gave good improvements in quality up to 7 d. Beyond 7 d, there was only slight improvement.
- For grade 25 and grade 32 concrete, the plastic sealed envelope was more effective than curing in the formwork or curing membranes; however, if the plastic envelope has small holes (punctured or torn), the benefits are negated.
- The chlorinated rubber membranes were less effective than the wax-based material. The wax-based membrane performed in a similar manner to the formwork.
- For grade 40 concrete, other than for wax membrane, all other initial curing procedures gave only a small improvement in quality compared with that achieved after removing the formwork after 1 d. Wax curing was by far the most effective procedure for this grade of concrete.

Similar conclusions about curing effectiveness were obtained from the rate of carbonation measurements. The surface quality achieved from the practical curing methods was generally equivalent to 3 d, or less, of that achieved by standard moist curing. The one exception was wax membrane that gave the equivalent of 4 d to 5 d of moist curing. Curing in the formwork for 7 d gave good results, with only a small decrease in quality if only 3 d was used. Curing beyond 7 d gave only slight improvements and would not be justified. The carbonation rate for the large columns was essentially constant for all the curing procedures. Practical curing produced a surface quality, as measured by carbonation, equivalent to 2 d to 4 d of standard moist curing.

Horizontal surfaces—Good curing practices can be applied more easily to horizontal concrete surfaces than to vertical surfaces. Sometimes horizontal surfaces can be ponded with water that will give results similar to standard moist curing in the laboratory. There are a number of other techniques that can be effective with horizontal surfaces. The practical curing procedures considered in this study included:

- Covering with plastic sheets at 24 h and maintaining for 6 d.
- Applying a 40 mm (1.6 in) layer of wet sand (moisture contents of 4 % or 8 %) at 24 h and maintaining for 6 d.
- Applying a curing membrane (chlorinated rubber) when surface water no longer existed.

In addition, Ho examined the effectiveness of moist curing after steam curing for 8.5 h at 70 °C (158 °F).

Based on the water sorptivity test results, the following conclusions were reached about the effectiveness of different curing methods for horizontal surfaces:

- Just as for vertical surfaces, all curing procedures gave improved results compared with only 1 d in the formwork.
- For each of the curing procedures, there were only small differences in the sorptivity values for grade 25 and grade 32 concrete. Sand with 8 % moisture content gave the best results. A single coat of chlorinated rubber membrane was slightly more effective than the use of plastic sheeting. Sand with 4 % moisture content gave about the same results as the chlorinated rubber membrane and the plastic sheeting.
- For grade 40 concrete, the plastic sheeting was ineffective. This may have occurred because the sheets were placed loosely over the concrete surface and were not sealed at their boundaries. The wet sands and the curing membrane were effective in improving surface quality. Water sorptivity values were equivalent to less than 3 d of moist curing with the exception of wet sand with 8 % moisture content which gave a quality equivalent to 7 d of moist curing.
- For the grade 25 and grade 32 concrete, steam curing gave sorptivity results generally similar to the other curing procedures. It was not as effective, however, as curing with the wet sand at 8 % moisture content.
- Steam curing for the grade 40 concrete gave sorptivity results similar to those for the curing compound and the 4 % wet sand. Just as for the other grades, it was not as effective as the wet sand with 8 % moisture.

- Using an additional 6 d or 27 d of water curing following the steam curing cycles was ineffective in improving the surface quality.

For the rate of carbonation measurements, Ho reached the following on the effectiveness of the various methods when applied to horizontal surfaces:

- All curing procedures improved the quality compared with that achieved after only 1 d in the formwork. Surface quality was generally equivalent to about 4 d of standard moist curing.
- For grade 25 concrete, the use of plastic sheets or moist sand (at both 4 % and 8 % moisture content) for 7 d gave similar results. For grade 32 and grade 40, the sand with 8 % moisture was more effective than either the plastic sheets or the sand with 4 % moisture. The curing compound gave the best results for the grade 25 concrete. For the grade 32 concrete, the curing compound was similar to using the sand with 8 % moisture, while for the grade 40 concrete it was similar to using the sand with 4 % moisture.
- Steam curing with the grade 25 and grade 32 concrete gave results similar to using wet sand and plastic sheets, but not as good as using the chlorinated rubber curing compound.
- Steam curing with the grade 40 concrete was more effective than the other methods with the exception of the sand with 8 % moisture.
- Additional water curing after steam curing gave negligible improvements for the grade 25 and grade 32 concrete, and only slight improvements with the grade 40 concrete.

Overall conclusions—For thin concrete sections, such as walls, the results from the laboratory specimens would closely simulate those from field conditions. Some of the major observations for these types of sections are:

- For the practical curing procedures considered, the surface quality achieved is generally equivalent to approximately 3 d of standard moist curing. From this, it is apparent that water-retaining curing methods (practical) are less effective than water-adding methods (moist curing).
- Most of the improvement in surface quality occurs in the first 3 d of curing. There is some improvement when the duration is extended to 7 d, but there is little improvement beyond this time.
- Based on the results with damaged plastic sheeting, good field supervision and oversight are required to ensure that there are no holes, tears, or punctures in the sheets.
- For vertical surfaces, the wax membrane gives an improved surface quality over the chlorinated rubber. For horizontal surfaces, the opposite is true probably because the effectiveness of the wax membrane is diminished by surface moisture diluting the water-based wax material.
- The various forms of practical curing had more effect on the lower grade concretes than on the grade 40 concrete.

- Steam curing is generally equivalent to about 3 d of standard moist curing—similar to other forms of practical curing.

For thick sections such as columns, the results from the large test columns will closely approximate actual field conditions. Some of the major results for thick elements are:

- Self-curing is an important consideration for columns—this occurs even after 7 d of air drying. The surface quality will be equivalent to about 2 d to 3 d of standard moist curing. This may not be the case, however, with very low water-cement ratio concrete where self-desiccation may be involved.
- Up to 7 d of practical curing resulted in concrete surface quality that was equivalent to 7 d of standard moist curing. Extended curing duration beyond 7 d provided little additional benefit.

6.6.3 *Strength development of high-strength concrete with and without silica fume*— Mak and Torii (1995) conducted studies on the strength development of high-strength concrete with and without silica fume under the influence of high curing temperatures up to 70 °C (158 °F). The primary objective of the research was to determine the effect of silica fume on concrete strength development under realistic curing histories.

The influence of a high curing temperature was studied by comparing strength development under standard conditions with strength development using a "temperature match conditioning" (TMC) system. The TMC system attempts to simulate the temperature history in an actual structural element. Figure 6.16 is a schematic of the TMC system, which consists of a heater-controlled water tank with a 500 L (132 gal) capacity. The tank can hold up to 40 cylinders of size 100 mm × 200 mm (4 in × 8 in). Computer software was developed to control the temperature of the water in the tank so that it matched a specific temperature history. It was reported that the control system was able to match the water temperature to within ±0.2 °C (±0.4 °F) of the input temperature history. The temperature histories that were used were obtained from experiments on columns cast in the laboratory. These laboratory columns were 1200 mm (47.2 in) long and 800 mm × 800 mm (31.5 in × 31.5 in) in cross section, which was the same as used in an actual building in Melbourne, Australia.

Two concrete mixtures were used: an 80 MPa (11 600 psi) ordinary portland cement (OPC) concrete and a 100 MPa (14 500 psi) silica-fume concrete. The amount of silica fume was 8.7 % of the mass of cement. The portland cement was a Type GP commonly available in Australia. The water-cementitious materials ratio for both concretes was 0.3, and a high range water-reducer (formaldehyde-based lignosulphonate type) was used to produce an average slump of about 140 mm (5.5 in).

Three curing methods were used for the 100 mm × 200 mm (4 in × 8 in) cylinders:

- Submerged in water at 23 °C (73 °F),

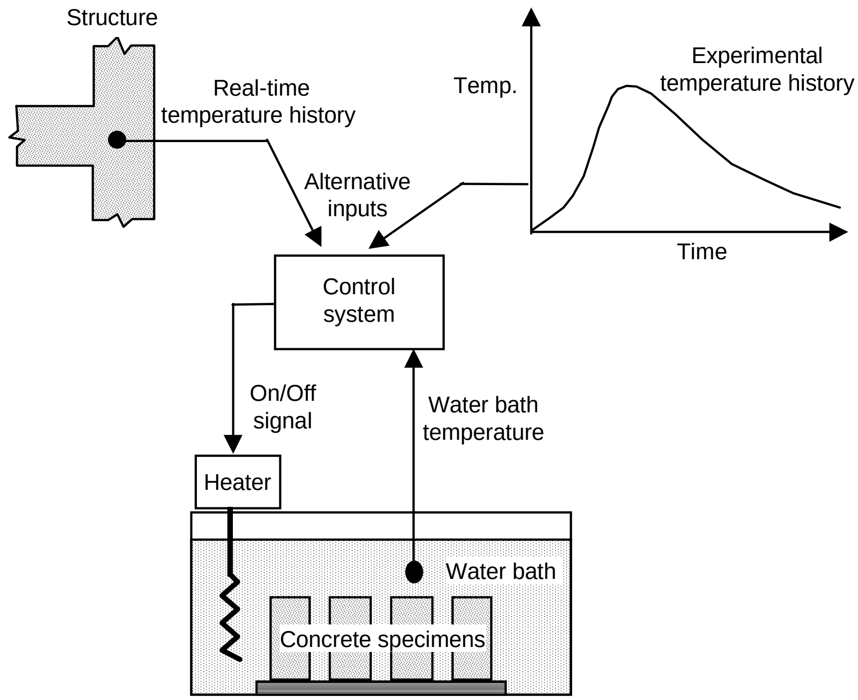


Figure 6.16 Schematic of temperature match conditioning (TCM) system used to simulate in-place curing temperature, adapted from Mak and Torii (1995)

- Sealed and stored at 23 °C (73 °F), and
- The TMC system

The sealed condition was obtained by using an adhesive backed plastic film. For curing with the TMC system, the cylinders were cured in the temperature-controlled water bath for 72 h and then were sealed and stored at 23 °C (73 °F).

Compressive strengths were measured at ages of (7, 28, 91, and 360) d. Figure 6.17 shows the compressive strength development of the cylinders subjected to standard curing under water at 23 °C (73 °F). It is seen that the strength for the silica-fume concrete and OPC concrete were similar at an age of 7 d. However, beyond 7 d, the silica-fume concrete was stronger than the OPC concrete.

Figures 6.18(a) and 6.18(b) compare the strength development of the ordinary portland cement concrete and the silica-fume concrete, respectively, for the three curing conditions. A comparison of the results for standard water-bath curing versus sealed curing shows that the OPC concrete benefited from the moist curing more than the concrete with silica fume.

The strength development characteristics of the two concretes were significantly different for the high curing temperatures obtained with the TMC system. The temperature histories that were used had a maximum temperature in the OPC concrete of 69.9 °C

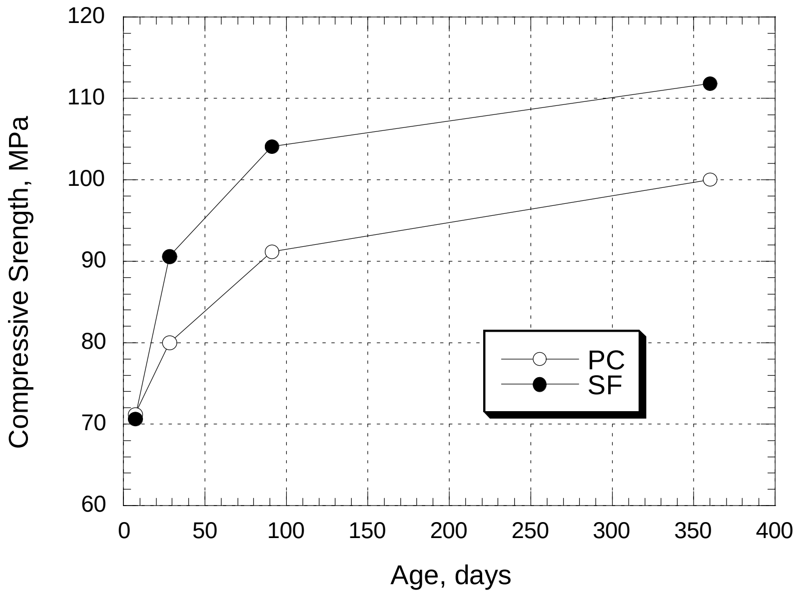


Figure 6.17 Comparison of compressive strength development of concrete mixtures with and without silica fume cured in a water bath at 23 °C (73 °F) (Mak and Torii 1995).

(158 °F) at 13.3 h after casting and the maximum in the silica-fume concrete was 66.5 °C (152 °F) at 15.3 h after casting. For the OPC concrete, the TMC cylinders were stronger than the standard water-cured cylinders up to 91 d. The TMC cylinders continued to develop strength up to 1 year, but, at 1 year, they were slightly weaker than the water-cured cylinders.

For the silica-fume concrete, the TMC cylinders were stronger at 7 d than the water-cured cylinders. However, beyond 28 d, there was little additional strength gain for the TMC cylinders compared with water-cured cylinders. At 1 year, the compressive strength of the TMC cylinders was 15 % lower than the standard water-cured cured cylinders. The failure of the TMC cylinders to undergo sustained strength development was attributed to self-desiccation in the silica-fume concrete. This lack of sustained strength development was consistent with supplemental measurements of non-evaporable water content and internal relative humidity (Mak and Torii 1995).

In summary, Mak and Torii (1995) showed that high curing temperatures, as might be obtained in actual structural elements, accelerate the 7-day compressive strength of silica-fume, high-strength concrete, but there is little additional strength gain thereafter. In contrast, the high-strength concrete without silica fume had higher subsequent strength development. This difference in strength development was attributed to self-desiccation in the silica-fume concrete. Mak and Torii noted that when silica fume is used in a low water-cement ratio concrete, as was investigated in this study, complete hydration cannot be achieved. It is, however, not necessary to have complete hydration for adequate high strength development.

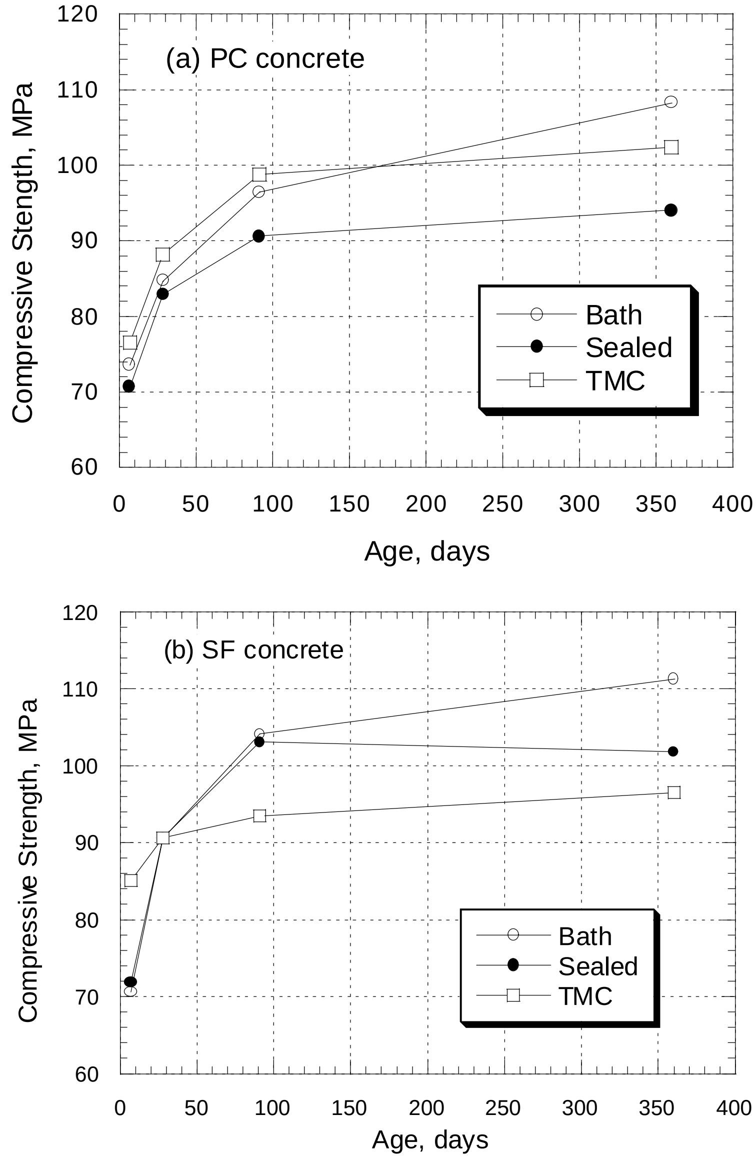


Figure 6.18 Comparison of compressive strength development of concrete mixtures (a) with and (b) without silica fume cured in a water bath at 23 °C (73 °F) (Mak and Torii 1995).

6.6.4 *Curing of concrete containing fly ash*—It is recognized that concrete in which fly ash replaces some of the cement may not develop its potential properties if proper curing is not applied and the concrete is exposed immediately to dry ambient conditions. Haque (1990) carried out a study to determine the minimum duration of curing so that the long-term properties of fly-ash concrete would not be not be affected adversely by exposure to drying conditions.

The research involved six concrete mixtures. Three mixtures were plain concrete made with only portland cement and three were made with fly ash replacing 30 % of the mass of cement. Test specimens included 100 mm × 200 mm (4 in × 8 in) cylinders for measuring compressive strength and 150 mm × 300 mm (6 in × 12 in) cylinders for measuring water penetration. The cylinders were stored in their molds in the laboratory for the first 24 h after casting, and then selected specimens were stored under the following conditions:

- Moist room at 23 °C ±2 °C (73 °F ±4 °F) and 95 % RH (condition F).
- Environmental chamber at 23 °C (73 °F) and 45 % RH (condition D).
- Environmental chamber at 45 °C (113 °F) and 20 % RH (condition WD).
- Water tank at 45 °C (113 °F) (condition WW).

Storage conditions D and WD were used to represent a temperate, dry exposure and a warm, dry exposure, respectively.

The following sequence was used in storing the specimens:

- At age 24 h, the molds were removed, some cylinders were placed in the moist room, and others were stored under conditions D, WD, and WW.
- After 7 d of storage in the moist room, some of the specimens were removed and stored under conditions D, WD, and WW. Some cylinders were kept in the moist room for additional curing.
- After 28 d of storage in the moist room, the remaining cylinders were removed and stored under conditions D, WD, and WW.

The result of the above process was to produce specimens with (0, 7, and 28) d of standard curing before being exposed to the other storage conditions. Specimens with 0 d of storage in the moist room were tested for compressive strength at ages of (7, 28, and 91) d. Specimens with 7 d of storage in the moist room were tested at ages of 28 d and 91 d; and those with 28 d of initial curing were tested at 91 d. The specimens for measurement of water penetration were stored under condition WD, and at 91 d they were immersed in water for 2 h. Depth of penetration was determined by splitting the 150 mm (6 in) diameter cylinders using the splitting tension technique, and the depth of water penetration that was observed visually was measured.

Table 6.18 gives the compressive strengths (average of two specimens) at ages of (7, 28, and 91) d for the different curing and storage conditions. The first number of the mixture identification indicates the nominal strength grade of the concrete in megapascals, and the second number indicates the percentage of cement replaced by fly ash (ASTM Class F). Thus all mixtures ending in the number 30 represent fly-ash concretes. Figure 6.19 shows the relationship between the depth of water penetration after 2 h of immersion and the 91-day compressive strength of the companion cylinders.

A careful study of the compressive strength values in Table 6.18 reveals the following observations (Haque 1990):

Table 6.18 Compressive Strength in MPa at (7, 28, and 91) d for Different Periods of Moist-room Curing Prior to Storage Under Different Conditions (Haque, 1990)

Mix-ture	0 days in Moist Room											
	7-day Strength				28-day Strength				91-day Strength			
	F	**WD**	**WW**	**D**	**F**	**WD**	**WW**	**D**	**F**	**WD**	**WW**	**D**
30-0	30.4	25.7	32.7	24.7	36.2	25.0	34.8	28.9	41.2	27.6	37.9	28.0
30-30	17.9	17.5	27.1	14.3	27.0	18.2	34.5	18.0	39.9	17.5	36.5	17.5
40-0	40.8	36.0	44.3	32.8	50.4	36.3	50.8	40.4	55.8	36.9	54.6	38.5
40-30	24.8	20.8	37.9	19.2	39.3	23.2	43.0	23.4	51.3	24.1	44.7	22.4
50-0	42.7	38.3	48.8	36.0	51.7	41.3	55.9	41.1	61.5	41.8	60.6	43.1
50-30	31.5	31.6	42.3	26.0	45.8	32.1	50.2	31.3	57.5	33.6	52.2	32.1
	7 days in Moist Room								**91 days in Moist Room**			
	28-day Strength				**91-day Strength**				**91-day Strength**			
	F	**WD**	**WW**	**D**	**F**	**WD**	**WW**	**D**	**F**	**WD**	**WW**	**D**
30-0	36.2	39.6	38.2	37.2	41.2	40.4	40.0	41.7	41.2	45.4	41.5	47.4
30-30	27.0	31.5	38.5	30.0	39.9	32.2	40.5	30.4	39.9	37.7	44.2	38.9
40-0	50.4	51.6	50.2	53.7	55.8	54.8	56.6	51.8	55.8	57.1	55.9	60.3
40-30	39.3	42.8	51.7	38.5	51.3	33.9	51.6	42.4	51.3	51.2	53.8	51.6
50-0	51.7	55.1	55.5	55.3	61.5	58.6	63.4	61.0	61.5	61.9	63.8	64.3
50-30	45.8	50.7	56.4	45.0	57.5	51.0	61.9	51.7	57.5	57.3	63.8	59.9

F = curing in moist room; WD = storage in air at 45 °C (113 °F) 20 %RH; WW = water bath at 45 °C (113 °F); D= storage in air at 23 °C (73 °F) and 40 % RH; 1 MPa ≈ 145 psi

- Strength development of the plain concretes and fly-ash concretes were seriously impaired when no curing (other than the 24 h in the molds) was provided prior to exposure to drying conditions (conditions D and WD). The strength development of the fly-ash concretes was impaired more than that of the plain concretes.
- When the concretes were subjected to 7 d of curing before exposure to drying conditions, the reduction in strength compared with the strength of the continuously moist-cured specimens was reduced greatly.
- At early ages, fly-ash concretes benefit from elevated temperature curing more than plain concretes.

The results of the water penetration tests (Fig. 6.19) showed the following:

- With no prior curing, the depth of water penetration (at an age of 91 d) was greater in the fly-ash concretes than in the plain concretes.
- With no prior curing, the depth of water penetration decreased in proportion to the increase in concrete strength for plain concrete and fly-ash concrete (observe the solid line in Fig. 6.18).
- With 7 d or 28 d of intial curing, the depth of water penetration was less in the fly-ash concretes than in the plain concretes for the same compressive strength (observe the two dashed lines in Fig. 6.19).

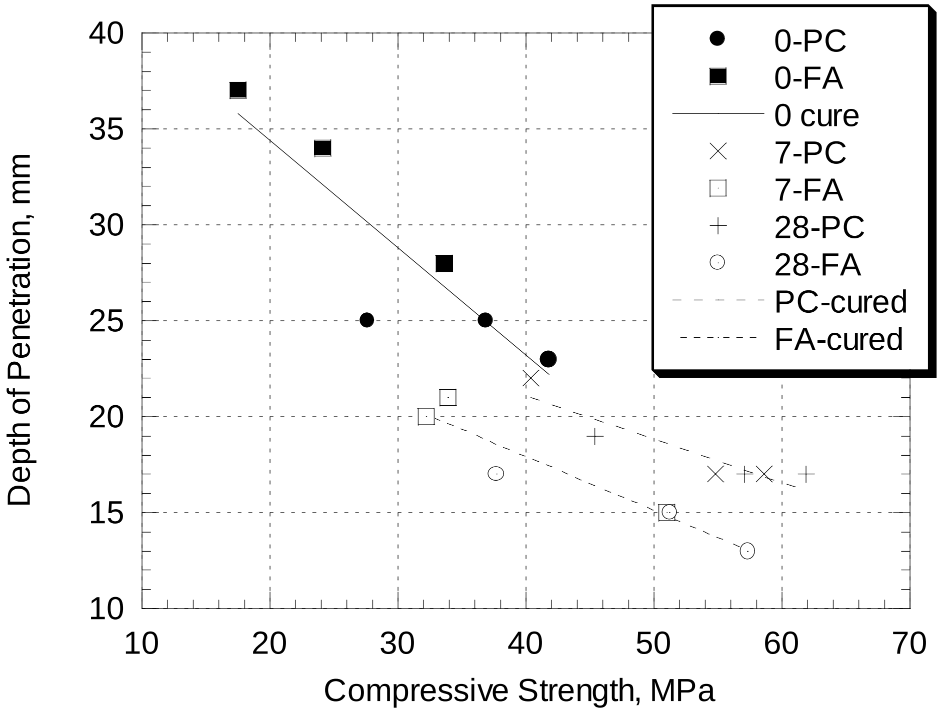


Figure 6.19 Depth of water penetration after 2 h of immersion versus compressive strength at age of 90 d for different periods of moist curing and subsequent storage under drying conditions (Haque 1990)

Based on these results, Haque (1990) concluded that curing of fly-ash concrete is critical if the concrete will be exposed to drying conditions immediately after mold removal at 24 h. A minimum curing period of 7 d was recommended. These findings are in agreement with those of Khan and Ayers (1995), which were summarized in Section 6.2.1. Since many high-performance concretes include supplementary cementitious materials, such as fly-ash and ground granulated blast furnace slag, which gain strength more slowly than portland cement at room temperature, these findings need to be considered in the formulation of rational curing guidelines.

6.7 Research in Japan

Some studies were conducted in Japan on the effects of silica fume on the properties of high-strength concrete with compressive strengths from 90 MPa (13 000psi) to 100 MPa (14 500 psi) (Torii and Kawamura 1994). The research included the influence of early-age curing conditions on durability-related properties. Detrimental effects of poor curing on pore structure were found to be more significant in normal-strength concrete than in high-strength concrete. Results showed that high-strength concrete with 8 % silica fume (mass fraction of cement) developed a dense pore structure at early ages regardless of the curing method used. This can be attributed to the low water-cementitious materials ratio and the effects of the rapid pozzolanic reaction of the silica fume. Poor curing practices also had

Table 6.19 Concrete Mixtures Studied by Torii and Kawamura (1994)

Mixture	Water-cementitious materials ratio	Cement, $kg/m^{3\dagger}$	Silica fume, $kg/m^{3\dagger}$
N0	0.55	300	0
N8	0.55	276	24
H0	0.30	490	0
H8	0.30	451	39

†To convert to lb/ft^3 multiply by 1.686

little effect on the resistance to chloride ion penetration (electrical method) of the high-strength concrete with silica fume.

The overall objective of this research was to investigate the effects of silica fume on both the mechanical and durability-related properties of high-strength concrete. The mechanical properties that were measured included compressive strength, splitting tensile strength, and modulus of elasticity; and the durability-related properties included pore size distribution, chloride ion permeability, resistance to freezing and thawing, and depth of carbonation. The concrete mixtures that were used included two "normal-strength" concretes and two high-strength concretes as shown in Table 6.19. High-early strength portland cement was used, and a high range water-reducer was used in all mixtures, except N0, to obtain slumps of 80 mm ±20 mm (3.2 in ±0.8 in). Silica fume was used in mixtures N8 and H8 to replace 8 % of the mass of portland cement.

Cylinders with dimensions of 100 mm × 200 mm (4 in × 8 in) were used to measure mechanical properties. These cylinders were cured in water for (3, 7, 14, 28, or 91) d, and then they were tested. The curing procedure for specimens tested for durability-related properties was as follows:

- Specimens, in their molds, were stored in a moist room at 20 °C (68 °F) for 24 h.
- After 24 h in the moist room, the molds were removed and the specimens were stored in water at 20 °C (68 °F) until ages of (3, 7, and 14) d.
- Specimens were removed from the water bath and subjected to air drying at 20 °C (68 °F) and 60 % relative humidity until an age of 28 d.
- In addition, specimens were stored continuously under water or exposed to air drying for 28 d.

Table 6.20 gives the results of mechanical property tests as a function of curing time under water. Figure 6.20(a) is a plot of compressive strength versus age; Fig. 6.20(b) shows the same strength-age data normalized by the corresponding strengths at 28 d; and Fig. 6.20(c) shows the compressive strength of the concrete with silica fume divided by the strength of the corresponding concrete without silica fume.

Table 6.20 Measured Mechanical Properties for Water-cured Specimens (Torii and Kawamura 1994)

Mixture	Curing Time in Water				
	3 d	**7 d**	**14 d**	**28 d**	**91 d**
	Compressive Strength (MPa)				
N0	26.8	34.7	35.5	41.6	42.4
N8	32.4	35.4	44.7	58.6	67.6
H0	49.0	53.5	59.3	71.4	75.0
H8	57.2	69.6	83.1	88.1	97.0
	Splitting Tensile Strength (MPa)				
N0	3.1	3.4	3.1	4.1	4.6
N8	3.3	3.8	4.2	4.7	5.7
H0	4.6	4.8	4.9	5.0	6.2
H8	5.0	5.6	6.0	6.5	7.1
	Static Modulus of Elasticity (GPa)				
N0	26.5	33.3	33.8	34.5	34.5
N8	31.4	33.8	36.0	39.5	44.1
H0	39.2	43.1	45.6	45.9	48.0
H8	39.7	45.9	47.5	48.0	50.0

1 MPa ≈ 145 psi

For continuous moist curing, the compressive strengths of the concretes with silica fume were consistently higher than the strengths of the concretes without silica fume (Fig. 6.20a)). The high-strength concrete with silica fume (mixture H8) had the most rapid early-age strength development (Fig. 6.20(b)). For the high-strength concrete with silica fume (H8), the strength improvement over the corresponding mixture without silica fume (H0) increased steadily between (3, 14, and 28) d, and beyond 28 d the improvement was relatively constant. Thus in the high-strength concrete, the positive effects of silica fume on strength gain were greatest during early ages.

Mercury intrusion porosimetry was performed on mortar samples (approximately 5 mm [0.2 in] cubes) that were taken from the interior of cylindrical specimens. The samples were freeze-dried in a vacuum, and were tested using intrusion pressures up to 414 MPa (60 000 psi). As expected, the porosity of the high-strength concrete without silica fume was significantly lower than the normal-strength concretes, both with and without silica fume. The addition of silica fume to the high-strength concrete reduced the porosity even further. The pore sizes became finer with the use of a lower water-cementitious materials ratio and with the addition of silica fume. It was found that the influence of poor curing practices had a greater effect on the pore structure of the normal-strength concrete than the high-strength concrete. The relative insensitivity of the pore size to the curing method may be related to the fact that the mortar samples were taken from the interior of the cylinders.

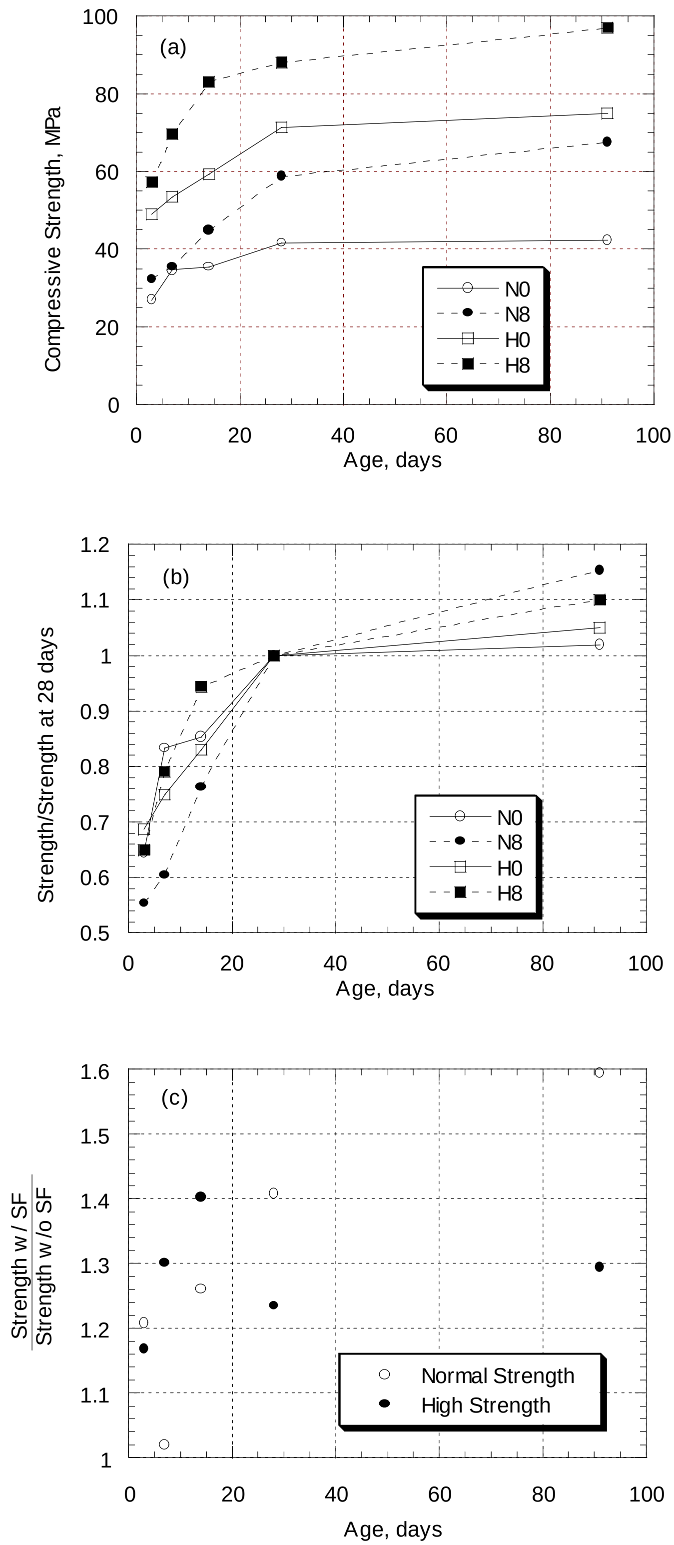


Figure 6.20 **(a) Compressive strength versus age for water-cured cylinders; (b) strength relative to strength at 28 days; (c) strength with silica fume relative to strength without silica fume (Torii and Kawamura 1994)**

The “rapid chloride permeability test” described in AASHTO T-277 (the same as ASTM C 1202) was used to indicate resistance to chloride ion penetration. Test specimens were prepared by cutting 50 mm (2 in) thick discs from the centers of the 100 mm $\times$ 200 mm (4 in $\times$ 8 in) cylinders. The specimens were tested at ages of 28 d. The charge passed after 6 h at a 60 V potential was taken as the indicator of the resistance to chloride ion penetration. The addition of silica fume reduced the charge passed for both normal and high-strength concretes. For the normal strength mixtures (N0 and N8), the charge passed was affected by the curing conditions. However, curing conditions had very little influence on the charge passed through the high-strength concrete with silica fume.

At ages of 28 d, specimens were exposed to a drying environment of 20 °C (68 °F), 60 % relative humidity, and 0.084 % carbon dioxide. After 1 year of exposure, the depth of carbonation was determined by spraying a 1 % phenolphthalein ethanol solution containing 10 % water on the surface of split specimens. For the normal-strength concretes (N0 and N8), the depth of carbonation increased with reduced duration of moist curing. Only the normal-strength specimens moist cured for 28 d had no measurable carbonation. On the other hand, all the high-strength concrete, both with and without silica fume, that had been water-cured exhibited no measurable carbonation. Only the specimens that were never water cured had measurable carbonation. The use of 8 % SF had no significant negative influence on the depth of carbonation for either the normal or high-strength concrete (some have argued that the consumption of calcium hydroxide by the pozzolanic reaction of silica fume makes the concrete more vulnerable to carbonation).

In summary, the study by Torii and Kawamura (1994) showed that for high-strength concrete 3 d of moist curing prior to exposure to air drying was sufficient to result in high resistance to chloride ion penetration (as measured by charge passed) and high resistance to penetration of the carbonation front.

6.8 Research in Norway

6.8.1 *The use of silica fume in concrete*—Extensive research programs have been undertaken in Norway on the use of silica fume in concrete. Large amounts of silica fume are produced and used in concrete construction. Silica fume contents greater than 10 % have been used along with high range water-reducers and very low water-cement ratios to produce high-strength concrete. Results of Norwegian studies on silica-fume concrete confirm many of the findings from other countries (Gjørv 1991):

- Silica-fume concrete is susceptible to plastic shrinkage cracking and moist curing must take place immediately after placement. The use of silica fume in high-strength concrete makes the concrete more sensitive to proper curing—early drying tendencies must be carefully controlled with proper curing procedures.
- The temperature of the concrete affects the age at which silica fume contributes to strength development. The effects of silica fume on enhanced strength development will

begin earlier as the temperature is increased. For example, increasing the concrete temperature from 30 °C (86 °F) to 40 °C (104 °F) and to 50 °C (122 °F) causes the effect of silica fume on strength development to commence at ages of (3, 2 , and 1) d, respectively. At 20 °C (68 °F), the major contribution of silica fume to strength gain occurs between 5 d to 7 d.

- Silica fume can be used to produce very durable concrete for structural applications in severe environments. The percentage of reduction in permeability of a given mixture with silica fume is greater than the percentage of strength gain, when compared with the same mixture without silica fume. Stated another way, silica fume has a proportionately greater effect on improving durability than it does on improving strength gain.

6.8.2 *The long-term strength of high-strength concrete with silica fume*—There were concerns about the long-term strength of high-strength concrete made with silica. Therefore, Maage et al. (1990) carried out a laboratory study to determine whether there were any long-term strength losses associated with the use of silica fume in high-strength concrete. High-strength concretes up to 10 years old were studied.

Various mixtures, all made with ordinary portland cement, were tested. The water-cementitious materials ratios varied from 0.37 to 0.61, and the silica fume content varied from 0 % to 20 % of the mass of cement. Laboratory specimens (100 mm [4 in] cubes) for measuring compressive strength were removed from the molds after 1 d, and then cured in water at about 20 °C (68 °F) or stored in air at about 20 °C (68 °F) and 50 % relative humidity until time of testing. Two replicate cubes were tested at each age. The strength development over the 10-year period for the various mixtures and the two curing conditions is shown in Fig. 6.21.

An examination by the authors of the results summarized in Fig. 6.21 leads to the following conclusions:

- For water-cured specimens, there was a consistent strength gain over the 10-year period. The concretes without silica fume had a strength gain from 28 d to 10 years of about 70 %. The concretes with silica fume had a corresponding strength gain of 30 % to 50 %, depending on the silica fume content. The primary benefit of the silica fume on strength gain occurs within the first 28 d. The effect of silica fume on strength gain is less significant beyond the initial 28-day period.
- For specimens stored in air, there was also a consistent strength gain over the 10-year period. From 28 d to 10 years, the strength gain was in the range of 0 % to 30 %. Theconcretes without silica fume attained the highest gains, and the concretes with the highest contents of silica fume had the lowest gains.

The lower strength gains for the specimens stored in air compared with the moist-cured specimens is not surprising, and can be attributed to the lack of sufficient moisture to sustain the hydration process. Figure 6.22 shows the normalized strength data obtained by dividing the cube strengths in Fig. 6.21 by the 28-day strength of the water-cured specimens.

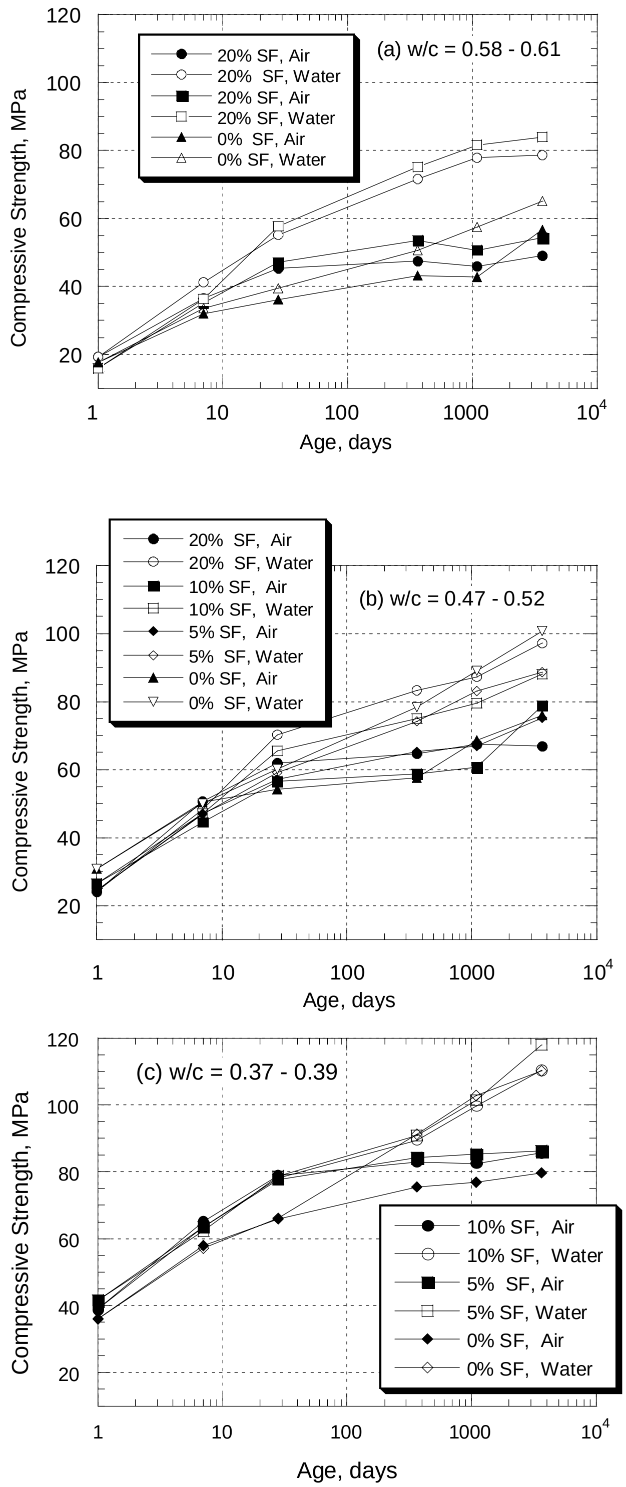


Figure 6. 21 Strength development of various concrete mixtures made with and without silica fume and subjected to moist curing or stored in air (Maage, et al. 1990)

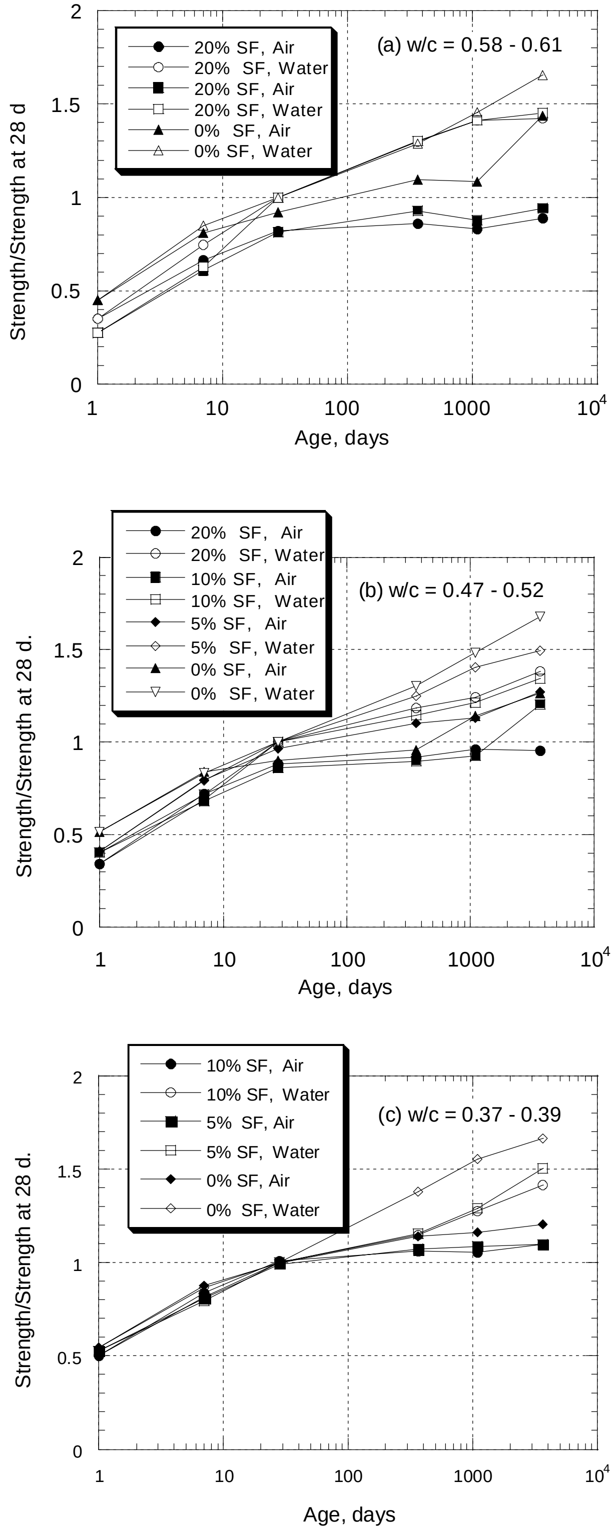


Figure 6.22 Compressive strength divided by strength after 28 days of moist curing (Maage et al. 1990)

A comparison of the results for the moist-cured specimens with those for the specimens stored in air indicates that the mixtures with lower water-cementitious materials ratios were not more sensitive to drying than the mixtures with higher ratios. Likewise, the mixtures with silica fume were also not more sensitive to drying than the mixtures without silica fume. In fact, Fig. 6.22(c) shows that the mixtures with the water-cementitious materials ratios between 0.37 and 0.39 had similar strengths up to 28 d, irrespective of the curing condition.

In summary, this laboratory study employing 100 mm (4 in) cubes did not reveal any long-term strength loss in high-strength concrete containing silica fume. In addition, there was no strong evidence that high-strength concretes, with or without silica fume, were more sensitive to drying than normal strength concretes. On the contrary, the high-strength concrete appeared to be less sensitive to drying with regard to compressive strength development.

6.9 Research in Israel

Work in Israel in the late 1980s focused on the effects of curing on the development of properties in high-strength concrete with and without silica fume (Bentur and Goldman 1989). Tests were conducted to evaluate the effects of the following curing methods:

- Water curing for 7 d, then exposure to air drying under "mild" conditions, 20 °C (68 °F) and 60 % RH, until tested.
- Wetting twice a day and exposed to hot and dry conditions to simulate poor field practice in a "harsh" environment. Specimens were immersed in water for 5 minutes twice a day, then exposed to 30 °C (86 °F) and 40 % RH. This wetting and drying treatment was continued for durations of (1, 2, 3, or 6) d. Specimens were then stored at 30 °C (86 °F) and 40 % RH until tested at 28 d. Specimens were kept sealed in their molds for the first day after casting.
- Water immersion for 6 d (after being sealed in the molds for the first day after casting), then exposed to 30 °C (86 °F) and 40 % RH. This condition was intended to simulate good water curing practice in a harsh environment.
- Continuous water curing until tested.

Three high-strength concrete mixtures were investigated, and they all used portland cement similar to ASTM Type I. The two reference concretes did not contain silica fume, the third mixture contained silica fume at a cement mass fraction of 15 %. All mixtures contained a high range water-reducer to provide workable concretes with slumps of approximately 100 mm (4 in). Characteristics of the mixtures are given in Table 6.21. The specimens for compressive strength measurement were 70 mm (2.75 in) cubes. Cubes that were not continuously water cured were soaked in water for 48 h prior to strength testing. In addition, 70 mm × 70 mm × 280 mm (2.75 in × 2.74 in × 11 in) prisms were used to

Table 6.21 High-strength Concrete Mixtures Used by Bentur and Goldman (1989)

Concrete Mixture	Water-Cementitious Materials Ratio	Silica Fume Content (by mass)	Compressive Strength at 1 day (MPa)*	Compressive Strength at 28 days (MPa)*
Reference I	0.40	—	16.3	62.7
Reference II	0.33	—	20.9	77.9
SF Concrete	0.33		23.2	107.6

*Compressive strength for continuous water curing. 1 MPa ≈ 145 psi

evaluate resistance to carbonation after 28 d of exposure to accelerated conditions (namely, 30 ºC (86 ºF), 50 % RH, and 5 % CO_2).

It is observed that the 28-day compressive strength of the silica-fume concrete is much greater than that of reference concrete II without silica fume (despite having equal water-cementitious materials ratios). The researchers attributed this enhanced strength to improved aggregate-paste bond in the silica-fume concrete. Since the 1-day strength was also greater in the silica-fume concrete, the positive effects of silica fume began at an early age.

Figure 6.23 compares the 90-day compressive strengths of the specimens that were continuously water cured with the strengths of the specimens that were cured in water for 7 d followed by storage at 20 °C (68 °F) and 60 % RH. Similar relationships were observed

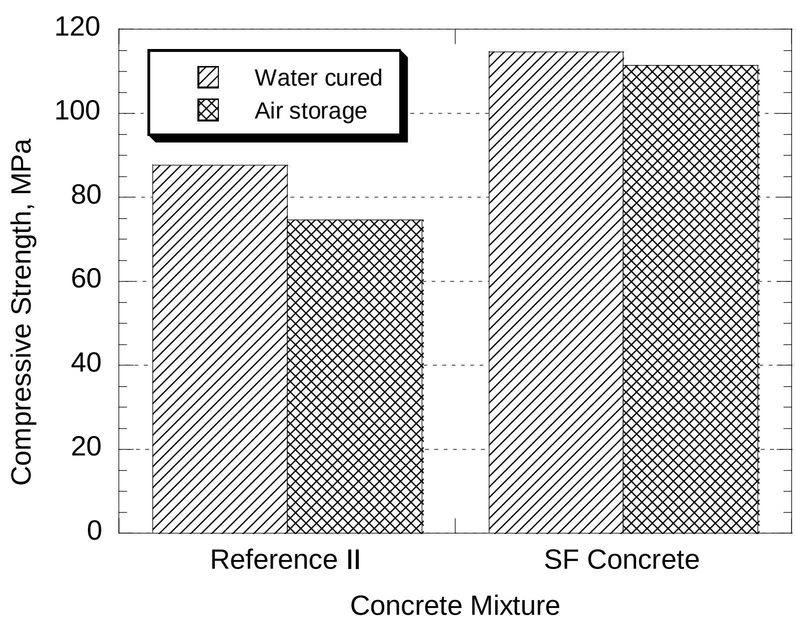


Figure 6.23 Comparison of 90-day cube compressive strength for Reference II and silica-fume concretes (Bentur and Goldman 1989)

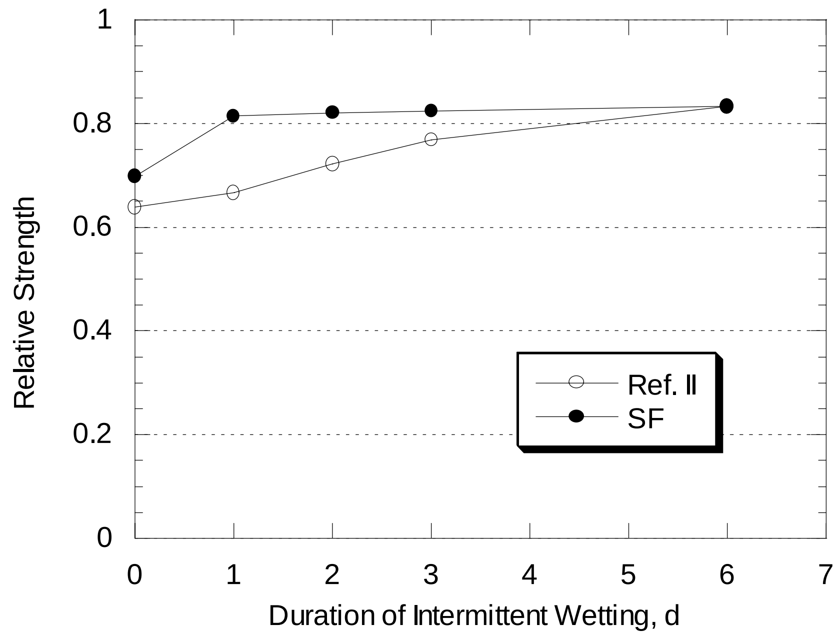


Figure 6.24 Compressive strength at 28 d for specimens subjected to wetting twice daily relative to strength of specimens moist cured for 6 d (Bentur and Goldman 1989)

for the 28-day compressive strengths except that the strengths were generally about 10 % lower. As expected for the reference concrete, storage in air after 7 d of moist curing resulted in a somewhat lower 90-day strength than the continuous water curing. For the silica-fume concrete, however, the specimens stored in air after 7 d of water curing were almost as strong as those cured in water continuously. This behavior can be attributed to several characteristics of high-strength concrete with silica-fume:

- The effects of strength gain from silica fume occur quite early—within the first 28 d.
- Silica-fume concrete develops a dense microstructure after only about 7 d of water curing, and this reduces the rate of drying when exposed to air.

Figure 6.24 shows the 28-day strengths of specimens subjected to the intermittent wetting procedure relative to the strength of specimens that were water cured for 6 d and exposed to the “harsh” environment. The latter treatment was taken to be representative of good curing practice. The intermittent wetting procedure resulted in reduced strength compared with water curing. The relative strength reduction, however, was similar for both the reference II concrete and the silica-fume concrete. For the short durations of intermittent wetting, the silica-fume concrete seemed to perform slightly better than the reference concrete. Note that for the silica-fume concrete, there appeared to be no benefit to increasing the duration of intermittent wetting from 1 d to 6 d.

Figure 6.25 shows the depths of carbonation as a function of the duration of intermittent wetting and these are compared with the depths in the specimens that were

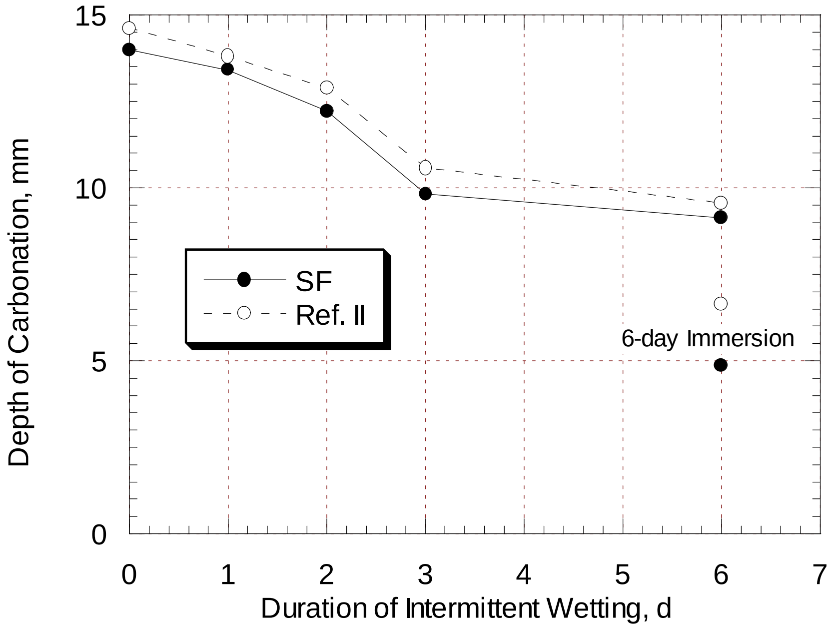


Figure 6.25 Depth of carbonation after 28 days of exposure to accelerated conditions for specimens subjected to wetting twice daily compared with specimens moist cured for 6 days (Bentur and Goldman 1989)

water cured for 6 d. The effect of intermittent wetting was similar for the reference and silica-fume concretes. This is further evidence that the use of silica fume does not make concrete more susceptible to carbonation. The researchers note that for 3 d to 6 d of intermittent wetting there was about a 20 % reduction in strength compared with the 6 d of water curing. In contrast, the depth of carbonation was about 100 % higher for the intermittent wetting. Thus, regardless of the composition of the cementitious material in high-strength concrete, the adverse effect of intermittent water curing is of greater importance to the "skin" properties (as represented by the depth of carbonation) than to the compressive strength.

6.10 Research in Germany

6.10.1 *The use of lightweight aggregate to enhance the curing of high-strength concrete*—Some research in Germany focused on the replacement of a portion of normal weight aggregate by lightweight aggregate to provide a supply of water within the concrete to sustain the curing process (Weber and Reinhardt 1996). This technique has proven to be effective in offsetting some of the effects of self desiccation in low water-cement ratio concrete. The concept is relatively simple—to store water for curing inside the concrete by using lightweight aggregate with high moisture content. This research investigated the trade off between the benefits of additional moist curing versus the potential strength loss from using lightweight aggregate.

The research program involved testing high-strength concrete made with normal weight aggregate (the reference concrete) and a number of different high-strength concrete mixtures containing a blend of normal weight aggregate and lightweight aggregate. The reference concrete had a water-cementitious material ratio of 0.33 and contained 7 % silica-fume as mass fraction of the cement. The 28-day compressive strength of 100 mm (4 in) cubes that were wet cured for 7 d and stored for 21 d at 20 °C (68 °F) and 65 % relative humidity was 104 MPa (15 000 psi).

Eleven different high-strength concretes with blended aggregates were tested. Prior to use, the lightweight aggregate (an expanded clay) was wetted either by sprinkling with water for ½ h or immersion in water for 24 h. Aggregate moisture contents ranged from 7.8 % to 20.8 % for the various mixtures. The volume of lightweight aggregate ranged from 10 % to 25 % of the total aggregate volume. For all the concretes, the amounts of cement and water were kept constant, and the amounts of high range water-reducer and silica fume were varied to produce workable mixtures. Most of the concretes contained the same amount of silica fume as the reference concrete (7 %), with only two of them containing 10 % silica fume. Compressive strength tests at an age of 28 d were conducted on 100 mm (4 in) cubes that had been cured by one of the following methods:

- NK – Stored 6 d under water and then in air at 20 °C (68 °F) and 65 % RH (standard curing)
- KR – Stored in air at 20 °C (68 °F) and 65 % RH
- SK – Stored in air at temperatures varying from 15 to 25 °C (59 to 77 °F) and RH varying from 40 % to 45 %
- KL – Sealed with aluminum foil and plastic sheeting

After casting, the cubes were compacted with a vibrating table for 30 seconds, then stored in a moist room for 24 h. The cubes were then removed from their molds and stored for one hour in the laboratory prior to beginning the curing conditions described above. Cubes were removed from their curing conditions 1 h before being tested for 28-day compressive strength.

Figure 6.26 shows the cube strengths for the different mixtures and curing methods. The results indicated that the curing method had little effect on the 28-day compressive strength of a given mixture. Many of the concretes with lightweight aggregate had lower compressive strengths than the reference concrete made without lightweight aggregate, which had a standard-cured strength of104 MPa (15 000 psi).

The researchers modified one of the most promising mixtures (mixture 7 in Fig. 6.26) by increasing the amount of silica-fume and lightweight aggregate. Characteristics of the resulting mixture are shown in Table 6.22. This mixture contained a retarder and a high range water-reducer which resulted in a good, workable concrete with a uniform distribution of the lightweight aggregate. The same type of cement was used as with the reference concrete—a rapid hardening portland cement. The 28-day standard-cured

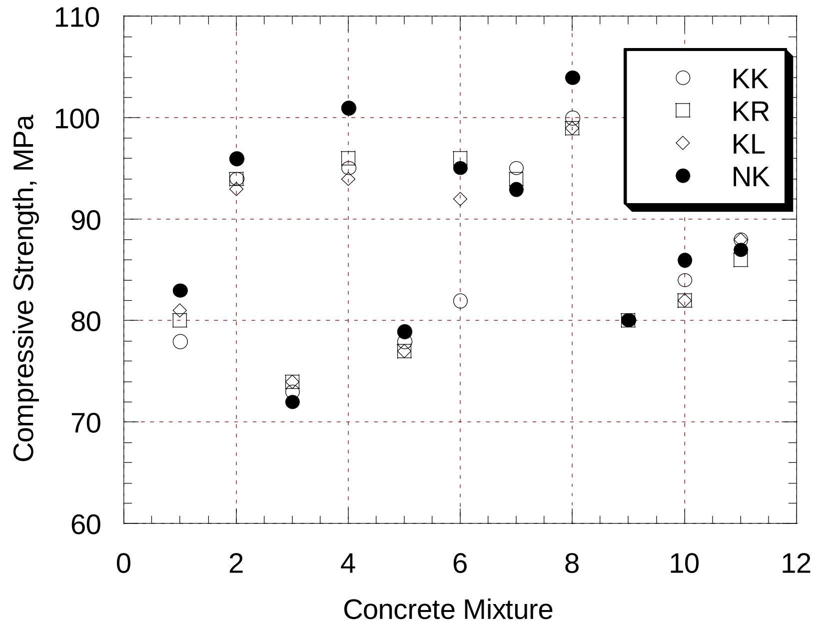


Figure 6.26 Compressive strength at 28 days for different curing conditions (Weber and Reinhardt 1996)

compressive strength of the modified mixture equaled that of the reference concrete containing normal weight aggregate, that is, 104 MPa (15 000 psi). Compression tests were conducted for each of the four curing conditions at ages of (7, 28, 91, and 180) d on 100 mm (4 in) cubes. Figure 6.27 shows the resulting relative compressive strengths for the different curing conditions and for the concretes with and without lightweight (LW) aggregate. The strengths are relative to the 28-day standard-cured strengths of 104 MPa (15 000 psi) obtained for the two types of concrete. Based on the results shown in Fig. 6.27, the following conclusions were drawn:

- The modified high-strength concrete mixture, made with 25 % by volume lightweight aggregate, gave compressive strengths equivalent to that of the reference concrete with all normal weight aggregate.
- The modified high-strength concrete mixture with lightweight aggregate had a higher

Table 6.22 Modified Concrete Mixture with Lightweight Aggregate Used by Weber and Reinhardt (1996)

Water-cementitious materials ratio	0.33
Silica-fume	10 % of mass of cement
Lightweight aggregate*	25 % of volume of total aggregate
28-day compressive strength**	104 MPa

* Moisture content = 20.2 %

**For 100 mm (4 in) cubes after 7 days wet curing and 21 days storage at 20 °C (68 °F) and 65 % RH .

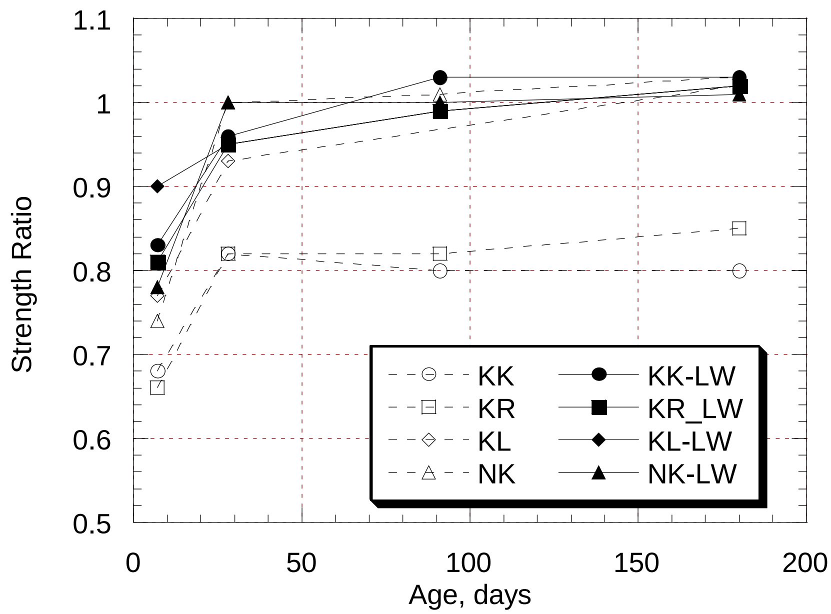


Figure 6.27 Relative strength development (in terms of standard-cured reference strength) for different curing conditions for concretes with and without lightweight aggregate (Weber and Reinhardt 1996)

compressive strength at 7 d than the concrete with normal weight aggregate for all four curing conditions. The 28-day compressive strength for the lightweight aggregate concrete (about 99 MPa [14 350 psi] to 100 MPa [14 500 psi]) was also higher than the strength of the normal weight aggregate concrete for curing conditions KR, KK, and KL shown above. The 28-day strength for the lightweight aggregate concrete for these curing conditions was slightly less than the standard-cured compressive strength of 104 MPa (15 000 psi).

- After 180 d, the compressive strength of the high-strength concrete with lightweight aggregate was essentially the same for all curing conditions and slightly exceeded the reference compressive strength of 104 MPa (15 000 psi). In comparison, the 180-day compressive strength for the concrete with normal weight aggregate was sensitive to the curing condition. For example, considering the two poorest curing conditions (KK and KR as shown above), strengths were 15 % to 20 % lower than the reference compressive strength.

The insensitivity of the high-strength concrete with lightweight aggregate to the type of curing was a significant benefit when compared with the concrete with normal weight aggregate. This can be attributed to the availability of internal moisture from the lightweight aggregate for the continuation of curing, which was independent of the external curing method. Within the lightweight aggregate concrete, curing can continue even after the surface becomes impervious, thus reducing the need for additional moist curing.

6.10.2 *Efficiency of curing methods*—A series of studies on curing was conducted at Darmstadt University. Kern et al. (1995) reported preliminary results of a study to evaluate the effectiveness of various curing methods. Although this work was not specifically directed at high performance concrete, some of the concepts and results may be applicable. The basic premise of these studies was that the degree of hydration largely determines the durability of concrete, and the degree of hydration can be determined from the amount of chemically bound water. The authors defined curing efficiency as, "...*the ability to keep the water in the concrete to guarantee high quantities of chemically bound water and thus to guarantee a high degree of hydration*." Quantitatively, Kern et al. (1995) suggested expressing curing efficiency as the ratio of the degree of hydration for a given curing condition to the degree of hydration produced by curing under water for 7 d at 20 ºC (68 ºF).

The following curing methods were investigated:

(a) Water retaining methods:
- Covering the concrete surface with vapor-proof sheets
- Keeping the formwork in place
- Using curing compound

(b) Water adding methods:
- Ponding the surface
- Placing wet coverings on the surface and protecting these coverings against drying by using vapor-proof sheets
- Keeping the surface wet by spraying with water

(c) No curing procedures after removal of formwork

Test specimens were 200 mm (8 in) cubes, and they were prepared to simulate curing conditions in walls and slabs. To simulate walls, four sides of the cubes were sealed so that only two opposite faces were exposed to the curing environment. To simulate slabs, five faces were sealed so that only the top surface of the cube was exposed to the curing environment. These treatments were intended to simulate one-dimensional moisture movement as would be expected to occur within the mass of a concrete member.

The test procedure was as follows:

- Cubes were kept in molds for 1 d, then the curing procedure was begun.
- After removal from their molds, specimens were cured for an additional 2 d or 6 d prior to testing.
- The amounts of chemically bound water were analyzed at (1, 3, and 7) d after casting.
- Concrete powder samples were obtained from various depths using a drill.
- Acetone was used to stop hydration, and the amounts of free water and gel water were determined by drying first at 60 °C (140 °F) and then at 105 °C (221 °F) for 1 h.
- The amount of chemically bound water was determined by ignition.

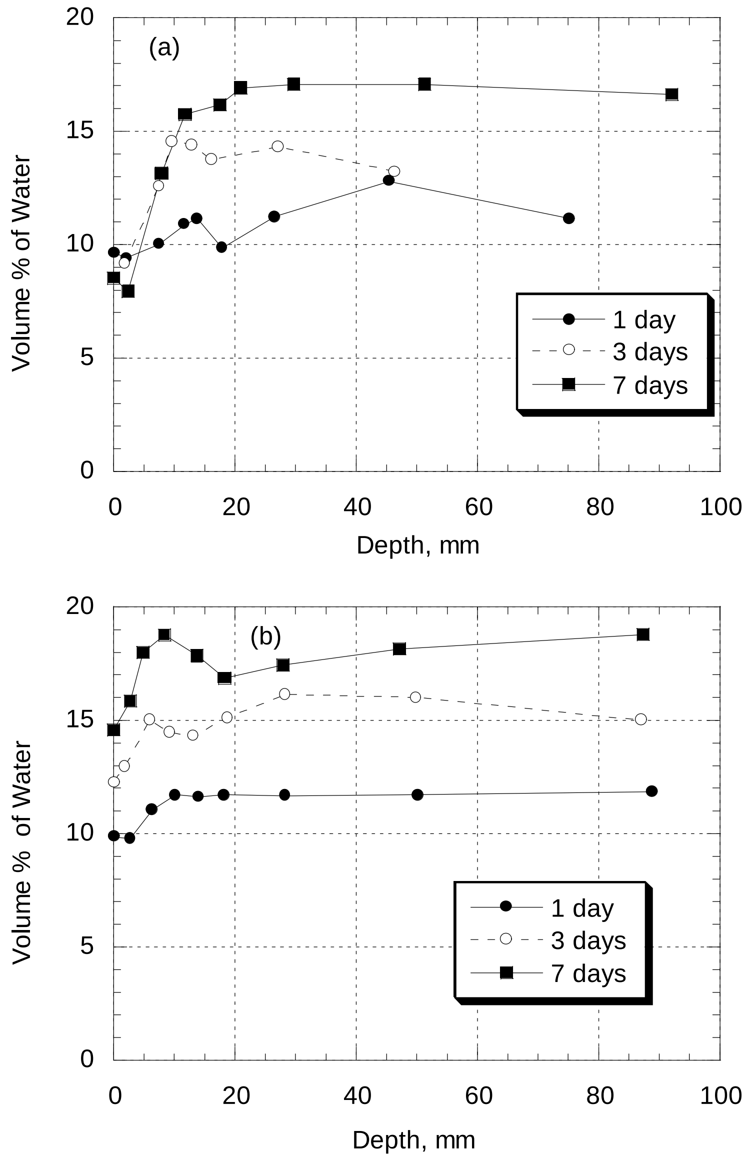


Figure 6.28 Chemically bound water at different ages as a function of distance from exposed surface: a) one day in mold and then exposed to air; b) 1 day in mold and then kept under wet covering

The brief report by Kern et al. (1995) provides only preliminary results. Figures 6.28(a) and 6.28(b) show results for two conditions—no curing and cured for 1 d in the molds and then protected with wet coverings and vapor-proof sheets. The concrete had a water-cement ratio of 0.60. The amount of chemically bound water is expressed in terms of the volume of cement in the powder samples. For the 1-day tests, the results are similar in both figures since the specimens were kept in their molds for that period. Thus the results were repeatable.

In Figs. 6.28(a) and 6.28(b), the steep gradients in chemically bound water within the first 10 mm (0.4 in) for the 3-day and 7-day curves for the "no curing" condition are obviously due to evaporation occurring near the concrete surface. For the curing with a wet covering, the gradient in less steep and only the outer 5 mm (0.2 in) appears to be affected. Note also that for no curing. the quantity of chemically bound water in the near surface layer remains at less than 10 %, while it increases in the case of curing with the wet covering. Although only preliminary results were reported, the researchers concluded that the degree of hydration, as indicated by the amount of chemically bound water, can be used to determine the curing efficiency of the various curing methods normally used for concrete.

In a second summary paper related to the same project, Grübl et al. (1996) provided further information on the efficiency of eight curing methods, which included nine different curing compounds. Figure 6.29 is a schematic of the different patterns that were observed for the curves of relative amounts of chemically bound water versus depth. In this case, the amount of chemically bound water at a given depth was expressed relative to the average over depths of 16 mm to 100 mm (0.6 in to 4 in). Curve "b" represents the case where the curing condition has no effect, and the amount of chemically bound water is constant. Curve "a" represents the case where evaporation leads to a reduction in the degree of hydration in the surface zone. Curve "c" represents the case where the degree of hydration close to the surface is low, but it increases below the surface and exceeds the value within the interior of the concrete. No explanation was provided for this behavior.

Grübl et al. (1996) proposed three criteria to describe curing efficiency for those conditions that result in a curve similar to "a" in Fig. 6.29. These criteria are shown in Fig. 6.30, and they are described as follows:

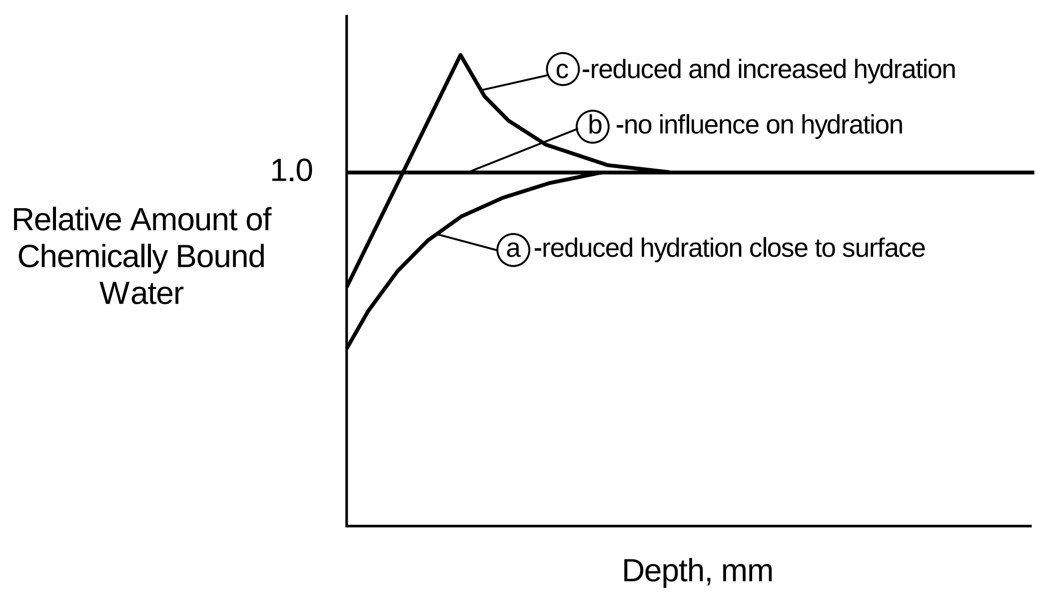


Figure 6.29 Schematic of typical curves showing relative amount of chemically combined water versus depth for different curing conditions (Grübl et al. 1996)

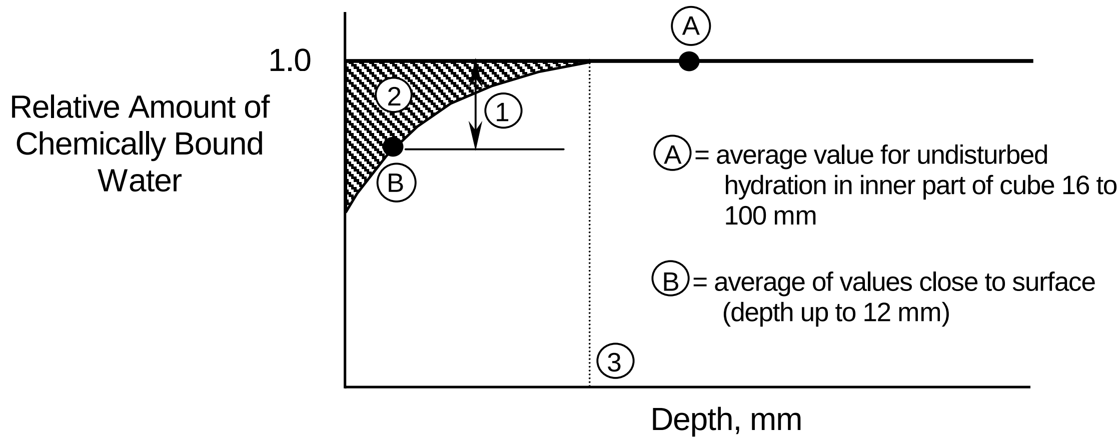


Figure 6.30 Proposed criteria 1, 2, and 3 to evaluate efficiency of curing methods (Grübl et al. 1996)

1 Difference between the average chemically bound water within 12 mm (0.5 in) of the surface and the average at depths between 16 mm (0.6 in) and 100 mm (4 in).
2 The area between the curve and the line representing the average chemically bound water at depths between 16 mm (0.6 in) and 100 mm (4 in).
3 The maximum depth that is influenced by the curing condition.

Grübl et al. (1996) chose criterion 1 to evaluate the curing efficiency of the various procedures that were studied. The distributions of chemically bound water at 7 d were used in the analyses. The results are given in Table 6.23, and examination of the values in the last column of the table reveals the following:

- Most of the curing methods showed a reduced amount of chemically combined water in the outer 12 mm (0.5 in) of the specimens compared with the interior.
- In most cases, extending the curing duration from 2 d to 6 d reduced the difference between the outer zone and the interior concrete.
- There was wide variation in the efficiency of the various curing compounds.

In a later study, Breier and Kern (1997) extended the research on efficiency of curing methods to high-strength concrete. The study involved two concrete mixtures with water-cementitious materials ratios of 0.22 and 0.33, and they included 7 % silica fume (mass fraction of cement). Four curing methods were involved, as follows:

- Specimens kept in their molds for 3 d, then stored at 20 ºC (68 ºF) and 65 % RH until testing.
- Specimens kept in their molds for 7 d, then stored at 20 ºC (68 ºF) and 65 % RH until testing.
- Specimens kept in their molds for 1 d, water ponding for 6 d, then stored at 20 ºC (68 ºF) and 65 % RH until testing.

Table 6.23 Efficiency of Curing Methods Evaluated at 7 d (Grübl et al. 1996)

Curing method	Duration of curing, d*	ID	Curing Efficiency**
Under water	2	W16	0.92
	6	W17	0.95
Spraying surface with water at 12 h intervals	2	W27	0.85
	6	W28	1.08
Wet covering plus vapor-proof barrier (no rewetting of cover)	2	W6	0.95
	6	W7	1.01
Cover with water-proof sheets	2	W14	0.94
	6	W15	1.01
Retain in steel formwork	2	W8	0.90
	6	W9	0.98
Retain in wooden formwork	2	W24	0.94
	6	W25	1.00
Exposed to air at 20 ºC and 65 % RH	6	W26	0.87
	6	W1n	0.88
Curing compound	6	W10	0.82
	6	W11	0.89
	6	W12	0.96
	6	W13	0.86
	6	W18	0.99
	6	W19	0.95
	6	W21	0.98
	6	W22	0.89
	6	W23	0.99

* Specimens were kept in molds for first day then subjected to curing conditions

** Evaluated at age 7 days using Criterion 1, i.e., ratio of degree of hydration within outer 12 mm to degree of hydration at 16 mm to 100 mm

- Specimens kept in their molds for 1 d, then stored at 20 ºC (68 ºF) and 65 % RH until testing.

Four properties were used to evaluate the curing efficiencies of these four curing methods:

- Tensile strength 50 mm (2 in) from the surface.
- Capillary water absorption after a soaking period of 144 h.
- Depth of carbonation measured at an age of 56 d.

- “Open” porosity, which was defined as the difference in the average open porosity within the outer 10 mm (0.4 in) and the average open porosity in the interior of the specimen.

Unfortunately, the summary paper by Breier and Kern (1997) does not provide details of the testing methods to measure the tensile strength and “open” porosity. Test specimens were 150 mm (6 in) and 200 mm (8 in) cubes that were sealed on four sides to simulate curing in a wall as explained above. Two replicate specimens were used for each concrete property measurement.

In this study, Breier and Kern (1997) used a different approach to define the *relative efficiency*, *E,* of a curing method for a particular concrete and age, which is defined as follows:

$$E = \frac{X - X_{\min}}{X_{\max} - X_{\min}} \tag{6.7}$$

where,

X = measured property for a particular curing method,
$X_{\min}$ = worst value of property obtained by the least efficient curing method, and
$X_{\max}$ = best value of property obtained by the most efficient curing method.

Thus curing efficiency ranges from 0 to 1.0, where 0 signifies that the curing method resulted in the lowest level of the property and 1.0 signifies that it resulted in the highest level of the property. The authors argue that this relative efficiency factor allows averaging of the relative efficiency of a particular curing method across different concrete mixtures, different test ages, and different concrete properties. Figure 6.31 shows the relative curing efficiency values for the four curing methods, as obtained for the different concrete properties and the different mixture/age combinations. Two findings are evident: (1) the relative rankings of the curing methods are not the same when evaluated at ages of 7 d and 28 d; and (2) the relative rankings are not the same for different concrete properties.

The researchers calculated average relative efficiencies for each curing method across the different properties and concrete mixtures. These are shown in the third and fourth columns of Table 6.24. The results for the “open” porosity measurements were not included in the averaging, because of “ambiguous tendencies” (Breier and Kern 1997). Finally, the relative efficiencies were averaged across the two testing ages to obtain the overall average values shown in the last column of Table 6.24. Based on these results, the authors reached the following conclusions for the high-strength concrete that was used:

- Water curing was the most effective curing method .
- Storage in air after 1 d of storage in molds was the least efficient curing method.
- The relative efficiency of water curing was higher when evaluated at an age of 28 d than at 7 d.

Table 6.24 Average Values of Curing Efficiencies for Different Curing Methods (Breier and Kern 1997)

Curing Method	Age at Start of Testing		Overall Average
	7 days	28 days	
3 d in mold	0.79	0.54	0.66
7 d in mold	0.68	0.68	0.68
1 d in mold, 6 days water	0.67	0.97	0.82
1 d in mold, in air at 20 ºC and 65 % RH	0.10	0.28	0.19

6.11 Contributions of Hilsdorf's Group

Prof. H. K. Hilsdorf has led some of the most extensive and informative work on concrete curing. He has been involved in the effort to revise the European standard for curing of structural concrete (see Section 5.13.5). This effort has included experimental studies as well as theoretical considerations in the search for more rational, better-defined curing requirements. Hilsdorf's studies have focused mainly on different criteria for the duration of curing, certainly an area of interest and controversy in the use of high-performance concrete. This section summarizes the findings, conclusions, and recommendations of Hilsdorf's group (Hilsdorf and Burieke 1992; Hilsdorf 1995).

According to Hilsdorf and Burieke (1992) concretes can be distinguished by their *curing sensitivity*, which is defined as the curing duration needed to reach some specified level of durability or an applicable mechanical property, such as strength. For example, for concrete with low curing sensitivity, the long-term properties would not be affected significantly by the duration of the curing period. Factors that determine curing sensitivity include the properties of the cementitious materials, mixture proportions, and the environment to which the concrete is exposed when curing is terminated. The latter factor affects the rate of moisture loss at the end of the curing period.

The water-cement ratio has a significant influence on the curing sensitivity of a particular concrete. Concretes with low water-cement ratios, as commonly found in high-performance concretes, will gain strength faster and attain high levels of impermeability sooner than those with higher water-cement ratios. This is an important characteristic since it may mean that curing duration can be reduced in accordance with the water-cement ratio, resulting in significant economic advantages during construction.

As has been stated often, increasing the duration of moist curing improves the mechanical and durability properties of concrete. The challenge is to determine the minimum duration of curing that is necessary to achieve the required level of performance for the specific application, taking into account all pertinent parameters. Prolonged curing

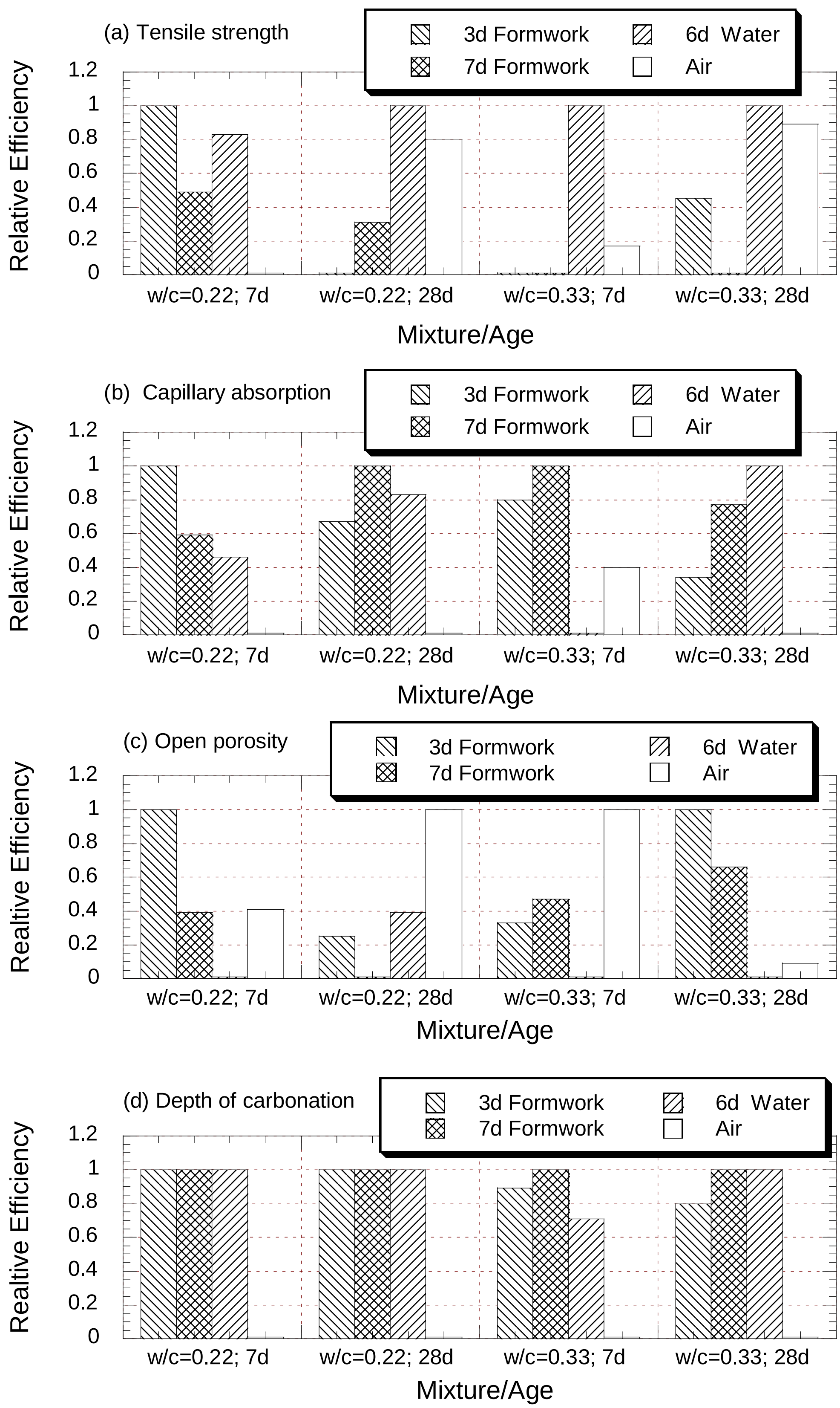


Figure 6.31 Relative efficiency for curing methods (Breier and Kern 1997)

adds to construction costs and unnecessary delays on the job site, and, therefore, it should be avoided, unless it is necessary to assure required performance.

6.11.1 *Parameters affecting curing duration* —Hilsdorf lists four parameters which must be considered when establishing minimum curing durations (Hilsdorf 1995):

- Curing sensitivity of the concrete as influenced by composition—water-cement ratio and strength development properties of the cement are the most important factors affecting this parameter.
- Concrete temperature—the rate of hydration (and, therefore, strength development and reduction in porosity) is affected greatly by this parameter.
- Ambient conditions during and after curing—this parameter affects the severity of drying of the surface layer.
- Exposure conditions of the structure in service—these affect the required "skin" properties for adequate service life.

6.11.2 *Curing criteria*—In defining the requirements for the minimum curing duration, Hilsdorf considered the following four criteria.

- *Rate of carbonation—The C-Concept*
  In this case, the curing duration is sufficient to ensure that the depth of carbonation does not exceed a specified value at a specified time (the service life).
- *Permeability—The P-Concept*
  The required duration of curing is based on the condition that a specified impermeability has been reached at the end of curing.
- *Maturity or Degree of Hydration—The M-Concept*
  The duration of curing is sufficient to ensure that the concrete attains a certain maturity or degree of hydration at the end of the curing period.
- *Compressive Strength—The R-Concept*
  Within this concept, two different approaches are possible. The first approach is to cure the concrete until it reaches a certain minimum compressive strength. The second approach is to cure the concrete until it attains a certain fraction of its standard 28-day compressive strength.

These criteria are discussed further in the following sections.

6.11.2.1 The C-Concept—This criterion would be applicable in situations where there is concern about corrosion of reinforcing steel due to the environmental exposure. Corrosion can occur when the cover concrete becomes carbonated and no longer provides the high alkalinity needed to maintain the reinforcement in a passive state. The curing duration is based on the acceptable rate of carbonation of the specific concrete being used and the specific exposure conditions. The desired service life of the structure and the depth of cover are additional input parameters. The minimum curing duration, according to the

C-Concept, ensures that, at the end of the specified service life, the depth of carbonation is less than the depth of cover.

The procedure to establish the curing duration for the C-Concept is summarized below. Additional details may be found in Hilsdorf and Burieke (1992) and Hilsdorf (1995). In developing the curing durations for the C-Concept, Hilsdorf assumed a service life of 50 years and a depth of concrete cover of 25 mm (1 in). Long-term, laboratory, carbonation studies established the following relationship between depth of carbonation at 20 °C (68 °F) and 65 % RH and air permeability measured (according to a specific test method) at 56 d:

$$d_c^c(t) = d_{c0}\sqrt{\left(\frac{K_{56}}{K_{056}}\right)^m \frac{t}{t_0}} \tag{6.8}$$

where,

$d_c^c(t)$	=	the depth of carbonation at time t under standard laboratory conditions, mm,
K_{56}	=	air permeability (m ^{2} /s) of the concrete at an age of 56 d,
K_{056}	=	a constant equal to 2×10^{-10} m^2/s or 1×10^{-11} m^2/s depending on the type of cement used,
m	=	0.5,
t	=	duration of carbonation, in years,
t_0	=	1 year, and
d_{c0}	=	1 mm

This expression is based on the assumption that a $\sqrt{t}$ relationship is valid for the development of carbonation depth with time.

According to Hilsdorf, vertical surfaces exposed to rain result in the most severe carbonation in environments typical of Northern and Central Europe. In an attempt to take into consideration the outdoor exposure conditions on the rate of carbonation, the following factor is used:

$$\alpha_c = \frac{d_c^{RV}}{d_c^c} \tag{6.9}$$

where,

α_c	=	a conversion factor with a value of 0.44 that has been determined experimentally,
d_c^{RV}	=	depth of carbonation of vertical surfaces exposed to rain (assumed to be 25 mm at 50 years), and
d_c^c	=	depth of carbonation measured under standard laboratory conditions.

Thus the depth of carbonation for a realistic exposure that is representative of Northern and Central Europe is slightly less than one-half of that obtained under standard laboratory conditions.

By combining Eqs. (6.8) and (6.9) and solving for K_{56}, one obtains the following expression:

$$K_{56} = \left[\left(\frac{d_c^{RV}}{\alpha_c d_{c0}} \right)^2 \frac{t_0}{t} \right]^{\frac{1}{m}} K_{056} \tag{6.10}$$

Equation (6.10) represents the required 56-day air permeability of the concrete so that the depth of carbonation is limited to d_c^{RV} after t years of exposure. If we assume that d_c^{RV} = 25 mm (1 in) and t = 50 years, the value for K_{56} can be calculated that will limit the depth of carbonation to 25 mm (1 in) after 50 years of exposure.

To find the required curing time to attain the permeability K_{56}, Hilsdorf used three relationships, as follows:

1. *Permeability-porosity relationship:*

$$K_{56} = \frac{K_{056}}{(1 - v_p)^n} \tag{6.11}$$

where,

v_p = capillary porosity and
n = coefficient equal to 4.9 determined experimentally

2. *Porosity-degree of hydration relationship:*

$$(1 - v_p) = \frac{0.68m}{0.32m + w/c} \tag{6.12}$$

where,

m = degree of hydration (expressed as a fraction) and
w/c = water-cement ratio

3. *Degree of hydration-time relationship:*

$$m = 1 - 10^{-\alpha \left(\frac{t}{t_0} \right)^r} \tag{6.13}$$

where,

α and r = coefficients determined experimentally that depend on type and strength class of cement,

t = time in d, and

t_0 = 1 d.

Equations (6.11), (6.12), and (6.13) can be combined to obtain an expression for the 56-day air permeability as a function of hydration time.

$$K_{56} = K_{056}\left[\frac{0.32\left[1-10^{-\alpha\left(\frac{t}{t_0}\right)^r}\right]+{}^{w}\!/_{c}}{0.68\left[1-10^{-\alpha\left(\frac{t}{t_0}\right)^r}\right]}\right]^n \tag{6.14}$$

If it is assumed that hydration stops at the end of the curing period, Eq. (6.14) can be used to determine the required duration of curing to obtain the 56-day air permeability required according to Eq. (6.10).

It is important to remember the assumptions and limitations for Hilsdorf's calculated minimum curing times (Hilsdorf and Burieke 1992, Hilsdorf 1995) to achieve adequate resistance to carbonation:

- They are based on a thickness of concrete cover of 25 mm (1 in) and a duration of exposure of 50 years.
- They are based on long-term carbonation data from Central and Northern Europe and would not be applicable to drier locations because a lower relative humidity would increase the rate of carbonation.
- They assume that the $\sqrt{t}$ relationship is valid for carbonation depth.
- Some of the values and coefficients were determined empirically from experimental results.

As will be shown in a subsequent table, the C-Concept will not normally be the controlling criterion for curing duration. Tests have indicated the C-Concept will control only for concretes made with blended blast furnace slag cements with high slag content. This concept could also control in a situation where lower thicknesses of cover are used, that is, less than 25 mm (1 in). Other conditions where this criteria might control would be environments where sustained periods of hot, dry weather are followed by a season of rainy, cooler weather (Hilsdorf 1995). An environment such as this would be conducive to more rapid penetration of the carbonation front, and could lead to carbonation-induced corrosion even if adequate cover had been provided. These conditions would indicate the need for longer curing periods to provide the necessary resistance to carbonation.

To summarize, the following information is needed to estimate minimum duration of curing using the C-Concept:

- Exposure conditions of the structure in service,
- Carbonation-time relationships,
- Assumed service life of the structure,
- Thickness of concrete cover, and
- Carbonation-permeability relationships or carbonation-strength relationships.

6.11.2.2 The P-Concept—According to the P-Concept, curing is maintained until the concrete reaches a prescribed level of impermeability. This is a logical criterion when adequate durability is desired, as opposed to adequate mechanical properties. However, several factors make it difficult to implement the P-Concept:

- There is no widely accepted method for measuring in-place permeability. While a number of air permeability methods have been proposed, each provides a measure that is specific to that method.
- There are no well-defined relationships between the level of impermeability, as measured by a specific test method, and durability.
- Air permeability test results depend on the method of preconditioning, and incorrect inferences can be reached if the in-place moisture conditions are not uniform from test point to test point.

As a result of these obstacles, it is difficult to select the level of impermeability, as measured by a specific test method, that will assure adequate durability for the anticipated exposure conditions. Nevertheless, it has been demonstrated how this procedure could be applied if a maximum permeability value were specified (Hilsdorf and Burieke 1992; Hilsdorf 1995). The method for calculating the minimum duration of curing is similar to that used in the C-Concept. In the C-Concept, the maximum permeability is based on the required resistance to carbonation. As was explained, once this critical value of permeability is established, a series of relationships (Eqs. (6.11) to (6.14)) can be used to estimate the curing time to achieve that level of permeability. These same relationships can be used to obtain the curing duration to achieve any other level of impermeability specified in the P-Concept.

6.11.2.3 The M-Concept—This approach requires that curing be maintained until a prescribed degree of hydration has been achieved. Since degree of hydration is defined in terms of the amount of cement that has reacted, this term becomes uncertain when the concrete contains blended cements that include other cementitious materials. Thus Hilsdorf and Burieke (1992) suggest that *maturity* be used as an alternative to degree of hydration. In this context, maturity is defined in terms of the relative amount of heat of reaction that has evolved at a particular age of the concrete.

The basic problem in implementing the M-Concept is selection of the target value of maturity (or degree of hydration). The properties of a given concrete depend on the mixture proportions as well as the extent of hydration. Thus, if a minimum level of an applicable concrete property were the objective, the minimum level of maturity at the end of the curing period would depend on the specific concrete mixture. However, if a level of maturity were specified, a relationship such as Eq. (6.13) could be used to determine the minimum duration of curing, at a specific temperature, to attain that maturity.

6.11.2.4 The R-Concept—According to the R-Concept, curing is maintained until the concrete reaches a prescribed level of compressive strength. Two approaches are proposed (Hilsdorf 1995):

1. *R1-Concept*: The concrete is cured until it attains a specified minimum strength. As an example, a suggested minimum strength is the strength after 7 d of moist curing that would be obtained by a reference concrete with a water-cement ratio of 0.6 and made with the same materials as the concrete to be cured (Hilsdorf 1995). A water-cement ratio of 0.6 corresponds closely to the highest value for which capillary pores can become segmented with good curing.
2. *R2-Concept*: The concrete is cured until the in-place compressive strength f_{cc} reaches a prescribed fraction of the 28-day standard-cured compressive strength, f_{c28}.

The R1-Concept offers the advantage that using mixtures with low water-cement ratios or having rapid early strength development can reduce the curing period. This criterion may be applicable when durability is of concern, because it has been established that, for a given concrete, there is "reasonably reliable" correlation between compressive strength and other durability-related characteristics, such as resistance to carbonation, permeability, and abrasion resistance (Hilsdorf and Burieke 1992; Ho and Lewis 1988).

In the R2-Concept, the curing duration is independent of the water-cement ratio, but it would depend on the rate of strength development, which would be affected by the mixture constituents and the concrete temperature. The R2-Concept is appropriate when structural strength is of concern. The basic notion is that the concrete should be cured long enough so that the in-place strength at some depth below the surface attains the specified strength used to design the structure. This is illustrated schematically in Fig. 6.32, where the solid curve represents strength development of the concrete under standard curing and the dashed curve represents strength development of the in-place concrete. When curing is terminated, drying of the surface occurs and hydration ceases when the moisture content falls below a critical value. However, it will take time for the drying front to penetrate into the core of the concrete member. As result, the interior concrete will continue to gain strength after curing is terminated. A longer curing period will reduce the rate of movement of the drying front into the interior concrete, and the strength will continue to increase at depths closer to the surface.

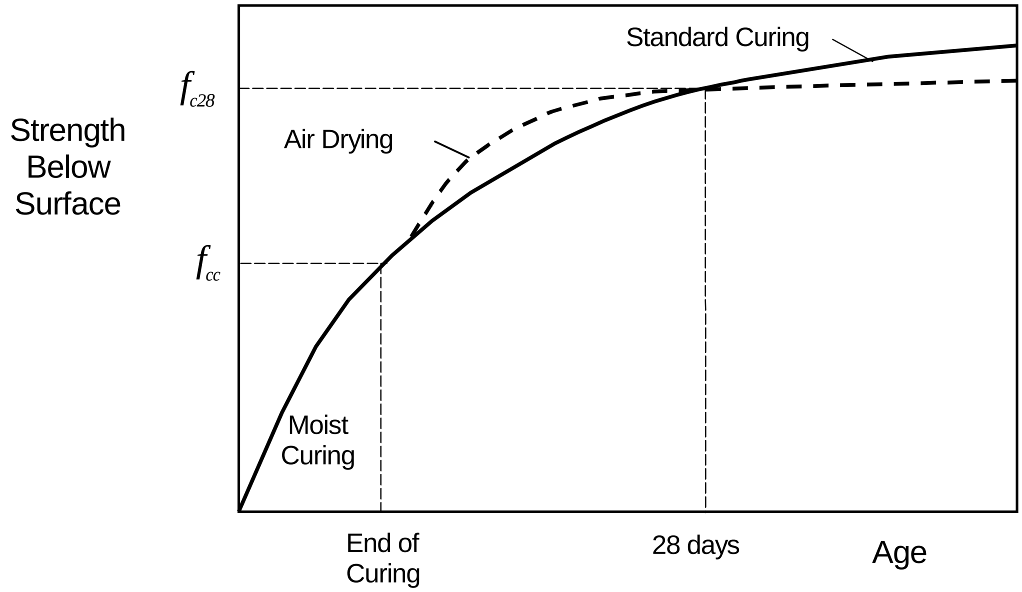


Figure 6.32 Schematic of strength development below surface for standard curing and for moist curing followed by air drying.

The question to be answered to implement the R2-Concept is as follows: What fraction of the standard-cured strength has to be attained at the end of the curing period to assure that the design strength is attained in the interior of the member? As was mentioned in Section 5.4, ACI Committee 308 suggests that the strength at the end of the curing period should be at least 0.7 of the design strength. Hilsdorf (1995) notes that this value is based on data obtained in the early 1950s, and those results may not be applicable to modern concretes. Hilsdorf suggests that a value of 0.7 may be conservative, and that research is needed to understand the strength development of different types of modern concretes after curing is terminated.

Hilsdorf (1995) discussed a method for estimating the fraction of the standard-cured strength that should be attained at the end of the curing period to assure that the interior concrete will achieve the design strength. The method is based on the requirement that the curing duration should be long enough so that at 28 d (or other applicable age) the concrete strength at the depth of the first layer of reinforcement will equal the design strength. The rationale for this requirement is to assure that the bond strength (or development length) of the reinforcing steel will attain the value assumed in the design. Diffusion theory was used to model the drying of the concrete from the exposed surface. It was assumed that the rate of hydration was not affected until the moisture content dropped below the value that is in equilibrium with a relative humidity of 90 %. The calculations were carried out for a concrete with a 28-day strength of 40 MPa (5 800 psi), for cements with different hardening rates, and for different ambient relative humidities (ambient temperature was 20 °C (68 °F)). The cover depth was taken as 25 mm (1 in). The results of the calculations are shown in Fig. 6.33.

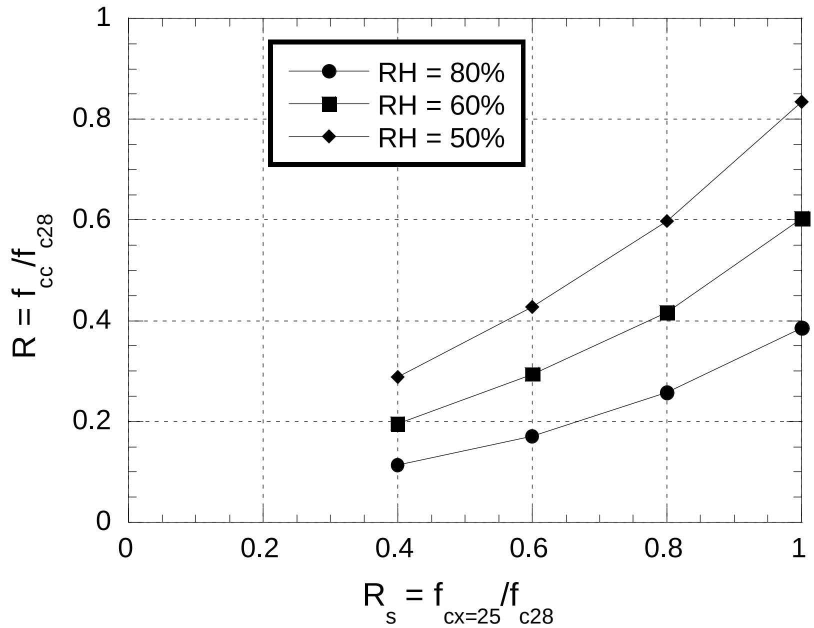


Figure 6.33 Relationships between strength ratio at depth of 25 mm (1 in) and ratio of strength at end of curing period (based on figure provided by H.K. Hilsdorf)

The vertical axis in Fig. 6.33 represents the fraction of the standard-cured, 28-day strength when curing is terminated. The horizontal axis represents the strength at a depth of 25 mm (1 in) expressed as a fraction of the 28-day design strength. The effects of different cement types were minor (see Hilsdorf 1995), and so the results of the calculations are shown as three curves. Based on these calculations, for an ambient relative humidity of 60 %, curing may be terminated when the concrete has attained 0.6 of the standard-cured, 28-day strength. If the ambient relative humidity is 50 %, curing has to be maintained until 0.85 of the standard-cured strength is attained. On the other hand, if the ambient relative humidity is 80 %, only about 0.4 of the standard-cured strength has to be attained. The time required to achieve these fractional strengths at a specific temperature can be estimated from the strength development characteristics of the cement.

6.11.3 *Summary*—Table 6.25 summarizes the calculated minimum curing durations for the different curing criteria proposed by Hilsdorf and co-workers. The cement types are those specified in European standards (see Section 5.13.4). Additional information about the assumptions and parameters used to obtain these values are given in Hilsdorf and Burieke (1992).

The curing durations based on the C-Concept are for a cover depth of 25 mm (1 in) and a service life of 50 years. The underlying data involved relationships between strength and air permeability. It is seen that the duration of curing to protect against carbonation at the depth of 25 mm (1 in) is relatively short, except when blended cement containing blast furnace slag is used. Cement type CEM III/A contains 36 % to 64 % slag and type CEM III/B contains 66 % to 80 % slag.

Table 6.25 Summary of Minimum Curing Periods based on Different Criteria (adapted from Hilsdorf and Burieke 1992; and Hilsdorf 1995)

		Duration of Curing in Days for Different Cements				
		CEM I		CEM II-S	CEM III/A	CEM III/B
Criterion[†]	**w/c**	**42.5R**	**32.5R**	**32.5R**	**42.5**	**32.5**
C-Concept	0.4	<1	<1	<1	1.5	2
	0.5	<1	<1	<1	2	3
	0.6	<1	1	1.5	3	5.5
P-Concept	0.4	1	2	2	2	3
	0.5	1.5	3	3	3	5
	0.6	2.5	5.5	5.5	5.5	9
M-Concept	—	2.2	5.2	5.2	3.7	8
R1-Concept	0.4	1.5	2	3	2	4
	0.5	3	3.5	4	3.5	5
	0.6	7	7	7	7	7
R2-Concept	—	2	4	4	3	7

[†]Refer to text in this report for assumptions used to obtain tabulated values

The curing durations based on the P-Concept were calculated for a permeability value that is lower than that required to hinder penetration of the carbonation front. Consequently, the calculated curing durations are longer than those from the C-Concept.

For the M-Concept, curing times were calculated for an arbitrary degree of hydration of 0.75.

For the R1-Concept, the reference strength was taken as the strength of a mixture made with the same materials at a water-cement ratio of 0.6 and cured for 7 d.

For the R2-Concept, it was assumed that the concrete would be exposed to an ambient relative humidity of 60 % after termination of curing, and the 28-day strength at a depth of 25 mm (1 in) would equal the design strength. The concrete was assumed to have a standard-cured 28-day strength of 40 MPa (5 800 psi).

In all of the calculations it was assumed that the concrete temperature during curing was 20 °C (68 °F). Thus the actual required curing durations could be longer or shorter than the values in Table 6.25 if, during the curing period, the concrete temperature were below or above 20 °C (68 °F), respectively. The maturity method could be used to calculate required curing periods at different temperatures.

A study of Table 6.25 reveals that, in most cases, the longest curing duration is associated with the R2-Concept. The exceptions are when high durability is required from a concrete with a high water-cementitious materials ratio, which would typically not be a practical condition.

The key aspect of the work of Hilsdorf and his co-workers is the methodology that has been developed for arriving at rational curing requirements. Application of the methodology has demonstrated clearly that the minimum curing period, at a specific temperature, may depend on:

- The primary performance requirement (strength or impermeability);
- The mixture proportions (primarily the water-cementitious materials ratio);
- The strength gaining characteristics of the cementitious system; and
- The ambient conditions after curing is terminated.

Thus it may be concluded that curing requirements that do not consider these factors may be inadequate, or they may result in unnecessarily prolonged curing periods. Another important aspect of Hilsdorf's work is the affirmation that a strength-based curing criterion may be adequate even when durability is the key performance requirement. However, this requires that there be a correlation between strength and durability for the concrete to be used in construction. This suggests that measurement of in-place strength may be a satisfactory means of assuring that adequate curing practices are used during construction.

6.12 Summary

This section summarizes some of the major findings of the reviewed research programs that have dealt either directly or indirectly with the curing of high-performance concrete. In this context, high-performance concretes are concretes with low water-cementitious materials ratios and usually include silica fume.

6.12.1 *Strength development of high-performance concrete*—The strength development characteristics of high-performance concrete are quite different from those of normal concrete. Tests by Sabir (1995) showed that high-strength concrete with a water-cement ratio of 0.35 (without silica fume) had a 7-day compressive strength that averaged 86 % of the 28-day strength when cured at 20 °C (68 °F). This same ratio for normal-strength concrete is in the range 60 % to 70 %. When silica fume was added to the high-strength concrete in the range 12 % to 28 % mass fraction of cement, the average ratio of the 7-day to the 28-day strengths was 76 % when cured at 20 °C (68 °F). When the curing temperature was increased to 50 °C (122 °F), this ratio increases significantly to 97 %, indicating that high curing temperatures can be very beneficial to early strength development in silica-fume high-strength concrete.

The use of silica fume affects the strength development characteristics of the concrete. Regardless of the curing methods used, high-strength concrete with silica fume will gain strength faster during the first 28 d than a similar high-strength concrete mixture without silica fume. Compressive strengths of high-strength concrete with silica fume replacements of 5 % to 20 % of the mass of cement and after 7 d of moist curing were 34 % to 57 % higher than high-strength concrete without silica fume (Hooton 1993). The higher the silica fume content (up to 20 %), the higher is the compressive strength after 7 d of moist curing. Beyond 28 d, the strength gain of concretes with silica fume is somewhat slower than concretes without silica fume. Beyond 56 d, high-strength concrete with silica fume gains additional strength very slowly, probably due to the effects of self-desiccation. There is general agreement among researchers that the positive influence of silica fume on the strength gain of high-strength concrete occurs mostly during the early age of the concrete, i.e., the first 28 d after placement.

As expected, higher than normal curing temperatures accelerate strength gain of silica-fume high-strength concrete (Sabir 1995). High early-age curing temperatures will increase significantly the 7-day compressive strength, but have only limited effect thereafter. The compressive strength at 3 months has been found to be independent of the curing temperatures used. So, where accelerated early-age strength gain is required, high curing temperatures may be very beneficial to high-strength concrete with silica fume. In high-strength concrete without silica fume, high early-age curing temperatures result in enhanced strength development up to an age of about 100 d (Mak and Torii 1995).

Mak and Torii (1995) also mention that the effects of self-desiccation must be considered with respect to the strength gain of high-performance concrete. Beyond the first 7 d after placement, self-desiccation is detrimental to the strength development of low water-cement ratio concretes. In these concretes, the hydration reactions will slow down at an early age as a result of the reduction in relative humidity caused by self desiccation. It should be noted, however, that hydration of all the cement is neither possible or necessary for the development of adequate strength in these concretes (Mak and Torii 1995).

There is some evidence that the long-term strength development of low water-cementitious materials ratio (0.29 and 0.36) mixtures may not be adversely affected by the use of high curing temperatures, as is the case with normal concrete mixtures. There are not sufficient independent data, however, to validate this potentially important finding by Carino et al. (1992).

High-performance concretes containing silica fume are susceptible to plastic shrinkage cracking; however, this can be controlled largely with proper curing practices. Plastic shrinkage cracks can be prevented, for example, by providing a supply of excess moisture, such as by using a fine mist spray, immediately after placement of the concrete. This requires changing the usual construction sequence where external moisture is supplied typically after final finishing.

There is agreement that high-strength concrete with silica fume will continue to gain strength over a long period when cured in water (data are available up to 10 years). However, researchers have reported instances of apparent strength reductions in the early years when concrete is stored in air. Some have dismissed this possibility of relatively small strength losses as being of little consequence, and perhaps caused by the difficulty of obtaining accurate strength data on actual structures because of limited sampling (Maage et al. 1990). It has been hypothesized that these apparent losses can be attributed to drying effects that cause self-induced compressive stresses near the core of cylindrical test specimens (de Larrard and Bostvironnois 1991).

6.12.2 *Curing methods applied to high-performance concrete*—Most of the commonly used curing methods for normal concrete have been successfully applied to high-strength silica-fume concrete, including wet burlap, plastic sheeting, and curing compounds (Holland 1989). The best curing practices for high-strength concrete, however, are the water-adding methods since they will tend to counter the effects of self-desiccation and lead to improved strength and durability properties. Water curing is effective in preventing surface drying that can cause shrinkage cracking in the cover concrete. As is the case with normal concrete, ponding or immersion are the best water curing methods. When these methods are not practical or possible (such as with vertical surfaces), other methods of water curing can be used effectively, such as covering with wet burlap, cotton mats, rugs, or other absorbent materials that will hold water on the surface of the concrete (ACI 363 1992).

It is generally agreed that good curing of silica-fume concrete with a low water-cementitious materials ratio is necessary to assure adequate durability. A dense, low-porosity microstructure with a minimum of microcracking is essential for an impermeable protective “skin.”

A novel approach for curing low water-cementitious materials ratio concrete containing silica fume was proposed by Weber and Reinhardt (1995). They investigated the replacement of a portion of the normal weight aggregate with saturated lightweight aggregate to provide a supply of water within the concrete. Their results indicate that this technique can offset the tendency for self-desiccation in concretes with low water-cementitious materials ratios. Ordinarily, high-strength concrete made with lightweight aggregate has lower strength compared with concrete of the same water-cementitious materials ratio but made with normal weight aggregate. The replacement of some of the normal weight aggregate by saturated lightweight aggregate proved to be effective in enhancing the curing of the high-strength concrete without reducing strength.

An important area of additional research is to establish unequivocally the sensitivity of various properties of high-performance to different curing. Some researchers have reported that high-performance concrete is more sensitive to proper curing than normal concrete; whereas, others have found the opposite to be true, at least for some properties. These differences may be attributed to the different experimental procedures that have been used. For example, Hasni et al. (1994) reported that the use of silica fume makes high-performance concrete more sensitive to different curing methods when considering both strength and durability properties. In addition, high-performance concrete with silica fume is more sensitive to different curing methods than is normal concrete for characteristics such as compressive and flexural strength, depth of carbonation, and microcracking. Comparison of high-performance concrete without silica fume with normal concrete showed that normal concrete was more sensitive to the curing method for these same properties. With respect to resistance to penetration of chloride ions, results showed that high-performance concretes with and without silica fume, as well as normal concrete, were insensitive to the curing method. The high-performance concretes had greater resistance to chloride ion penetration than the normal concrete.

Results of work in Norway summarized by Gjørv (1991) generally agree with the findings by Hasni et al. (1994). Gjørv reported that the use of silica fume makes concrete more sensitive to proper curing compared with normal concrete. Silica-fume concrete is more vulnerable to plastic shrinkage cracking than normal concrete, which necessitates good, early-age, curing practices to control this tendency. Another reason cited by Gjørv for why silica-fume concrete is more sensitive to proper curing is related to its strength properties. Good curing practices must be used to prevent early drying, which can reduce tensile and flexural strengths of silica-fume concrete more than for normal concrete.

Torii and Kawamura (1994) also reported on the effects of curing on mechanical and durability-related properties of concrete, and some of their results do not agree with those above. Their results indicated that the detrimental effects of poor curing practices on pore structure are more significant in normal-strength concrete than in high-strength concrete with silica fume. In their studies, high-strength concrete with 8 % replacement of cement by silica fume apparently developed a dense pore structure at early ages regardless of curing method. This independence of the curing method is attributed to the use of a low water-cementitious materials ratio (0.30) and the early pozzolanic reactions of the silica fume. Tests for resistance to chloride ion penetration and carbonation depth also showed that high-strength concrete, both with and without silica fume, was less affected by poor curing conditions than normal concrete. This can be attributed to the fact that concrete with a low water-cementitious materials ratio may attain a low porosity paste at a lower degree of hydration than concrete with a higher water-cementitious materials ratio. Comparisons between the high-strength concrete with and without silica fume revealed that the concrete with silica fume was less affected by the changes in curing method than was the concrete without silica fume, when considering resistance to chloride ion penetration and carbonation depth. These results differ from the findings of Hasni et al. discussed above. Thus additional studies are needed to explain these conflicting conclusions regarding the sensitivity of low water-cementitious materials ratio concrete to the curing method.

6.12.3 *Duration of curing*—There is probably not enough conclusive information available at this time to develop definitive guidelines for the curing duration of high-performance concrete. Curing duration requirements have been a topic of much debate and discussion—particularly in Europe. There does seem to be general agreement within the international community that new criteria and standards are needed for this modern class of concrete. Traditional curing requirements, such as those in the ACI Code, need to be replaced by modern criteria that account for major factors affecting curing duration. What is probably the most progressive standard for curing duration is under development in Europe (ENV 206). It takes into account factors such as water-cement ratio, cement type and strength class, and exposure conditions in determining curing durations. The work of Hilsdorf in Germany, as discussed Section 6.11, has greatly contributed to this proposed standard. Because of the different cement types used in Europe, the European standards can not be applied directly in the Western Hemisphere. The challenge is to develop curing guidelines that are rational but, at the same time, practical for adoption in American codes.

Holland (1989) recommended that silica-fume concrete be given a minimum of 7 d of wet curing, or the equivalent at non-standard temperatures. Khan and Ayers (1995) inferred that perhaps silica-fume concrete does not have to be moist cured as long as normal concrete. However, they caution that further studies are needed to better define the curing requirements of silica-fume concrete. They determined the minimum curing durations, at laboratory temperature, to achieve a certain level of relative strength. The results were 3 d for the silica-fume concrete and 3.75 d for the ordinary portland cement concrete. On the other hand, the minimum length of curing for a concrete with Class F fly ash was 6.5 d. Thus there is evidence that high-performance concrete with supplementary cementitious materials may require prolonged curing periods.

Asselanis et al. (1989) also investigated the curing duration requirements for a low water-cement ratio concrete with silica fume, and concluded that prolonged moist curing beyond 7 d is normally unnecessary to achieve desired levels of strength and durability. After 7 d of curing, the concrete is sufficiently impervious so that any further moist curing is ineffective in improving the concrete's properties.

The most extensive recent research on curing duration requirements has been conducted by Hilsdorf (1995). Although this work was not directed specifically toward high-performance concrete, the underlying approach appears to be applicable to all classes of concrete. He states that four parameters must be considered to establish curing duration requirements:

1. Curing sensitivity of the concrete as influenced primarily by the cementitious system
2. Concrete temperature
3. Ambient conditions during and immediately after curing
4. Exposure conditions of the structure in service

To establish minimum curing durations, Hilsdorf emphasized that compressive strength is not the only criterion that must be considered. Other criteria that should be considered include:

- Depth of carbonation

- Permeability
- Maturity or degree of hydration

The depth of carbonation must be controlled to assure that the reinforcing steel is surrounded by an alkaline environment and remains in a passive state. The minimum duration of curing for adequate resistance to carbonation depends on the depth of cover, the desired service life, the relationship between time and depth of carbonation, and the relationship between concrete permeability and carbonation. Given this information, additional relationships between permeability, water-cement ratio, and time can be used to estimate the minimum duration of curing.

The permeability criterion is a more general form of the carbonation criterion. In this case, the minimum curing duration is based on achieving a certain level of impermeability as measured by a specific test method. One difficulty in using the permeability criterion is the selection of the critical level of impermeability because there is insufficient knowledge of the relationships between measured permeability values and long-term durability.

In using the maturity, or degree of hydration criterion, the minimum duration of curing is based on the concrete reaching a specified maturity (defined in terms of heat of hydration) or degree of hydration. Once the specified value is defined, empirical relationships between time, temperature, and degree of hydration (or maturity) can be used to estimate the minimum curing duration. The empirical relationships would be expected to be affected by the characteristics of the cementitious system used in the concrete. As is the case with the permeability criterion, there is insufficient knowledge to relate the minimum degree of hydration (or maturity) at the end of the curing period with long-term performance.

A compressive strength criterion may involve one of two approaches:

1. Curing the concrete until a specified minimum strength is reached. Hilsdorf proposed that the minimum strength be the strength of a reference concrete made with the same materials with a water-cement ratio of 0.6 and cured for 7 d (R-1 concept).
2. Curing the concrete until a specified ratio of the in-place compressive strength to the compressive strength after 28 d of standard curing is attained (R-2 concept). Traditionally, a value of 0.7 has been used for this ratio. Hilsdorf proposed a rational approach to determine what this ratio should be based on the requirement that, at a specified age, the compressive strength at the depth of the reinforcing steel equals the design strength.

Hilsdorf's studies showed that in most cases the critical curing duration was controlled by the compressive strength criteria (see Table 6.25). This is an important aspect of the research because it tends to affirm that strength-based curing criteria may be the most practical approaches, possibly even when durability is a primary concern. Assuming that preliminary testing of the specific concrete mixture used in construction gives a reliable correlation between strength and durability, in-place strength measurements would be a suitable method of assessing curing in the field.

The maturity method is a viable method for determining curing durations under different temperature conditions. Several studies have shown that the maturity method can be used to estimate strength gain during curing with high-performance concrete.

6.12.4 *Verification of curing effectiveness* —Senbetta and Scholer (1984) have demonstrated that absorptivity can be used as a simple and accurate means to evaluate pore structure development in cement paste. Absorptivity differences between two levels, or zones, of a concrete element can be used to measure the effectiveness of curing. Well-cured concrete will have only small differences in absorptivity at different levels because the pore structure should be essentially the same throughout the paste. In using this approach, laboratory data for a specific concrete can be used to establish a quantitative dividing line between adequate and inadequate curing, based on differences in absorptivity at different depths. The researchers discussed ASTM C 156, which is the most used test procedure to evaluate membrane-forming curing compounds and sheet materials. This procedure has been found to be unreliable in assessing the effectiveness of a specific curing material. They suggested that more reliable procedures are needed to assess how well concrete is cured and to evaluate the efficiency of various curing methods. An approach based on absorptivity differences considers the quality of the concrete itself, as opposed to just an acceptance specification for the curing compound or other curing material being used. In 1991, ASTM adopted Test Method C 1151 based on the work of Senbetta and Scholer. The method, however, has not found widespread acceptance because of the relatively high cost associated with the absorptivity tests of the discs cut from the drilled cores

Kern et al. (1995) have shown how curing effectiveness can be related to the degree of hydration, which, in turn, determines the strength and durability characteristics of the concrete. Their approach is based on evaluating the amount of chemically bound water in the paste to determine the degree of hydration.

As previously discussed, in-place strength measurement can be used to monitor curing sufficiency for either strength or durability requirements. The curing period would be based on achieving a required strength. The strength requirement to assess adequacy of curing might be in terms of a minimum strength when durability is important, and it might be in terms of a fraction of the standard-cured 28-day strength when structural strength is important.

7. CONCLUDING REMARKS AND RESEARCH NEEDS

7.1 Introduction

The previous chapters have summarized our level of understanding regarding curing requirements of high-performance concrete. In this context, high-performance concrete has been taken to be concrete with a lower than normal water-cementitious materials ratio, with or without supplementary cementitious materials, such as silica fume, fly ash, or ground granulated blast furnace slag. Although a significant amount of research has been carried out on this subject, there is no consensus on curing requirements of HPC, and there is no agreement on whether it requires special considerations compared with normal concrete. Some conclusions have been contradictory. Possible reasons for these contradictions by different investigators include the use of different materials and techniques to study the influences of curing methods. This chapter concludes the state-of-the-art report by providing a discussion of critical research needs to arrive at definitive conclusions about curing requirements for high-performance concrete.

The long-term goal of the proposed research is to provide the basis for modifying current ACI standards related to curing so that structures in service will perform as required. Curing requirements should consider economy of construction and not place undue demands on the construction team. On the other hand, curing requirements should assure the owner that the potential properties of the concrete in the structure are realized. To achieve these goals, emphasis should be placed on developing a system to verify the adequacy of curing on the job, such as, for example, by requiring the measurement of in-place properties of the at the end of the curing period.

7.2 Laboratory Techniques to Simulate In-place Curing

More research effort should be devoted to the evaluation of curing on full-scale models in laboratory conditions that closely simulate actual field conditions and environments. Laboratory testing offers the advantage of controlling the factors that affect curing so that definite conclusions can be reached. When full scale model testing is not feasible, curing simulations need to be developed that will approximate the moisture transport conditions within actual structural members. For example, standard test cylinders that are allowed to dry from all surfaces in the laboratory do not reflect the drying characteristics of the concrete in an actual structural member. Because drying occurs from all surfaces of a cylinder, it is not representative of the one-dimensional drying that occurs in a structural member. A better approximation of the field conditions would be to seal all but one end face of each cylinder so that moisture moves in one direction.

Another important consideration is the thermal history of the concrete. The primary value of full-scale model testing is that the temperature rise and thermal history can be accurately portrayed. The thermal history is greatly affected by the specimen size and geometry. One approach that can be used in the laboratory is to cover surfaces with insulation. The insulation restricts heat flow in the specimen so that it represents more closely the conditions in a large member in the field.

To evaluate, in an economic manner, the factors affecting the curing of high-performance concrete in the field, novel techniques are needed that permit the use of small test specimens in the laboratory, which simulate actual conditions in larger structural members. Consideration should also be given to studies that compare the results from full-scale models in the laboratory with results from actual structures in the field.

7.3 Efficiency of Moist Curing

All methods of curing can be grouped into two main categories: curing practices where water is added, and curing practices that prevent or retard the loss of moisture. For normal concrete with a water-cementitious materials ratio greater than about 0.45, these two curing practices will give similar results because there is water in excess of that needed for hydration and pozzolanic reactions. However, for high-performance concrete, it is generally accepted that the “water-added” methods are more effective. Most high-performance concretes involve low water-cementitious materials ratios in which the quantity of mixing water is insufficient to maintain the water-filled capillaries that are needed to sustain hydration and pozzolanic reactions. Hence curing practices in which water is added will provide the supply of moisture needed to sustain hydration and pozzolanic reactions.

While water-added curing practices may be effective in countering the effects of early-age self-desiccation, the effectiveness at later ages is uncertain. Capillaries become disconnected at different degrees of hydration depending on the water-cementitious materials ratio. Therefore, the use of water-adding methods after the capillaries become discontinuous may not be justified. There are indications that prolonged moist curing of high-performance concrete may not be productive since it tends to become impermeable sooner than normal concrete; hence, it may not need to be cured as long as conventional concrete. Research is needed to establish practical curing durations for high-performance concrete, taking into account the improvement in properties and additional cost when prolonged curing is applied.

7.4 Strength Development Characteristics

Curing criteria have traditionally been based on strength development of the concrete. Codes have made no distinction between the curing required to develop a given strength as opposed to curing required to assure durability. Research is needed to consider both strength and durability with respect to what type and how much curing is necessary to realize the full potential of the high-performance concrete. There seems to be general agreement within the concrete community that strength can no longer be the only criterion for curing.

As stated in the review of Hilsdorf's work, the strength-gaining characteristics of a particular concrete mixture is a key factor establishing the required minimum duration of the curing period. Since high-performance concrete can be made from a variety of cementitious materials, it is necessary to understand how they affect the rate of strength development (resulting from hydration and pozzolanic reaction). Ideally, it would be desirable to be able to predict the rate of strength development from knowledge of the composition of the cementitious system. Hilsdorf's methodology for estimating the minimum curing duration appears to be both reasonable and practicable and should be expanded to address high-performance concrete mixtures. Special attention needs to be paid to concrete mixtures with supplementary cementitious materials (fly ash, blast furnace slag) that have slower reaction rates that portland cement. Often these materials are used to enhance durability-related properties, and longer curing periods may be needed to assure that the desired properties develop.

The maturity method has been shown to be applicable to high-performance concrete. This method accounts for the effects of temperature on strength development. It is necessary to understand the temperature sensitivity of the rate of strength development of different high-performance concrete mixtures in order to use this method to account for the effects of temperature on the minimum required curing duration.

7.5 In-place Assessment of Curing

Up to now, the curing requirements in the ACI building code (ACI 318) have been prescriptive. On the other hand, Standard Specification ACI 301 (1996) and Standard Specification ACI 308.1 (1998) contain alternative performance-based curing requirements, as mentioned in Sections 5.10.2 and 5.11 of this report. These alternatives are based on measurement of the strength of field-cured cylinders, on an estimate of in-place compressive strength based on in-place tests, or measurement of an in-place property related to durability. While the use of in-place strength tests is permitted, there are no standards on how to implement in-place testing to verify adequacy of curing. A sound statistical practice is needed before in-place tests can be used effectively. The procedures given in ACI 228.1R (1995) could provide the basis for such a standard practice.

Current curing requirements in the ACI code are strength based. However, there may be applications where durability is the principal performance requirement. In such cases, curing requirements have to assure that the structure will satisfy its durability requirements. Ideally, an in-place property related to durability should be measured to verify whether adequate curing has been used. In fact this has been incorporated into the new curing requirements specified by ACI 308.1-98. However, due to many practical difficulties, no standard test methods exist for this purpose. As noted by Hilsdorf, a practical alternative is to use in-place strength to verify the adequacy of curing for durability. To permit this approach, relationships between durability properties and a measured strength parameter are required. Standard procedures for establishing these relationships need to be developed.

7.6 Development of Proposed Rational Curing Requirements

Over the past 60 years, there has been little change in the ACI code requirements for curing. Current codes and specifications do not address curing requirements that are specific to the high-performance concretes being used in construction. Until such curing requirements are developed, professionals must rely on the conflicting results of research studies in writing job-specific specifications. Information from the above-mentioned research areas can be used to develop new rational curing standards. The new provisions should be applicable to all types of concrete, should address durability as well as strength, and should account for the major factors affecting the required duration of curing. The type of curing (water adding or water retaining) might have to be specified to assure optimal properties for high-performance concrete.

7.7 Future of High-performance Concrete

As new standards and criteria are developed for high-performance concrete (to include curing requirements), builders and designers will have a better appreciation for the enhanced performance of this type of concrete. Use of high-performance concrete within the construction industry during the past few years has grown. This is an indication that this industry is gaining a better understanding of the improved properties that are possible with high-performance concrete. This trend is expected to continue.

One of the most challenging tasks facing the United States in the next few years is the rebuilding of the infrastructure. Much of it is constructed of concrete, so high-performance concrete can be expected to play a major role in this massive undertaking. There has been a lot of interest in using more high-performance concrete in the nation’s highway construction programs. The Federal Highway Administration (FHWA) is attempting to stimulate more interest in the use of high-performance concrete throughout the country. The FHWA plans to conduct state highway demonstration projects over the next few years featuring the use of high-performance concrete. The FHWA is also developing different classes for high-performance concrete to assist local highway agencies in specifying different levels of performance (Goodspeed et al. 1996).

The potential for high-performance concrete appears to be almost unlimited. It is being used in more and more types of construction each year, such as core walls and columns of tall buildings, long-span bridges, and many different types of marine structures. Some have predicted that advances in concrete technology will provide the capability to design and construct facilities with useful lives of a century or more.

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