High-Performance Concretes, A State-Of-Art Report (1989-1994)

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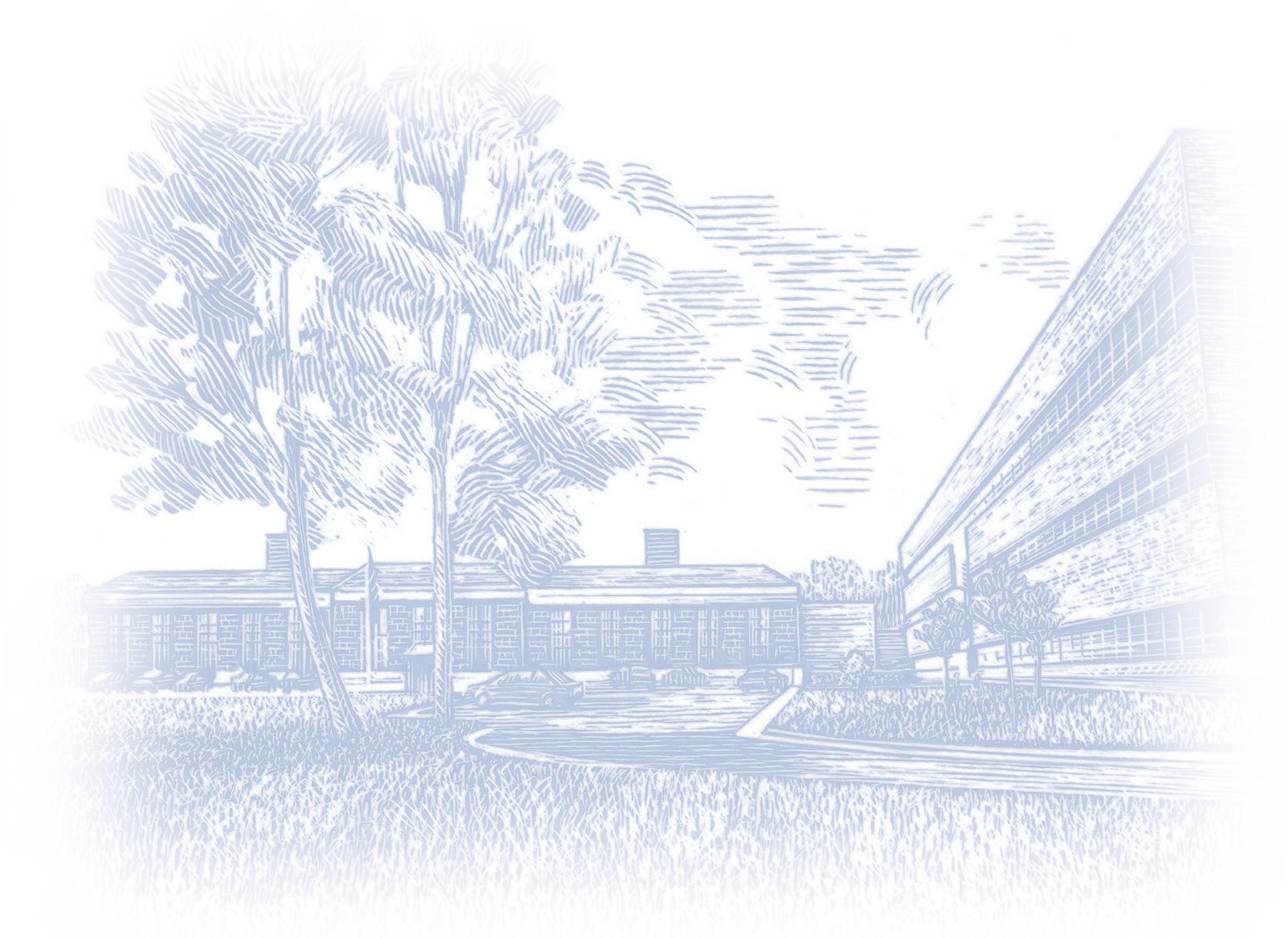

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Foreword

Paul Zia, Shuaib Ahmad, and Michael Leming

This report is a sequel to a previous state-of-the-art report published by SHRP in 1991 and is based on review of selected literature on high performance concrete with particular reference to highway applications. It covers primarily a six-year period from 1989 to 1994 for which the authors have recently compiled an updated annotated bibliography, which has been published by the Federal Highway Administration (Publication No. FHWA-RD-96-112). More recent information on the subject is obtained directly from a variety of publications including books, technical journals, conference proceedings, and research reports.

Included in the review and discussion are the behavior of plastic concrete as well as the strength and deformation characteristics of hardened concrete. Both short-term and long-term effects are considered. In addition, the behavior of both discrete and continuous fiber-reinforced concrete is covered. Furthermore, recent applications of high performance concrete for pavements and bridges, here and abroad, are summarized. Finally, recent activities of organized research and development programs on high performance concrete in different countries are described.

Based on the review of the available information, it is concluded that the growth of the amount of research and applications of high performance concrete has been phenomenal in the past several years. High performance concrete has become widely accepted practically on all continents. Much of the application of HPC remains in the areas of long-span bridges and high-rise buildings. Increasing emphasis is being placed on concrete durability than its strength. In many applications, high strength concrete is used only because of its high durability quality rather than the need for its strength.

Research progress on concrete durability, self-compactable concrete, and innovations in fiber-reinforced concrete are identified and additional areas of research needs are cited.

ACKNOWLEDGMENTS

The authors prepared in 1991 a state-of-the-art report as the first task of their research on high performance concrete sponsored by the Strategic Highway Research Program of the National Research Council. The report, covering the period from 1974 to 1989, was based on critical reviews of selected references from an annotated bibliography compiled by the authors.

Since the publication of the two previous documents, there has been a phenomenal increase in the development and use of high performance concrete. This report is a sequel to the previous state-of-the-art report and covers the six-year period from 1989 to 1994. The authors followed the same approach in preparing this publication by reviewing selected references from a second annotated bibliography, compiled by the authors and published recently by the Federal Highway Administration (FHWA) as a separate document. The authors are solely responsible for the content of the report and for any errors and omissions.

The authors would like to thank FHWA for the financial support provided for this project. They are indebted to Thomas J. Pasko, Jr. and Louis Colucci who served as the FHWA contacts. The draft of this report was critically reviewed by Thomas J. Pasko, Jr., Louis Colucci and Susan N. Lane. Their efforts are greatly appreciated.

EXECUTIVE SUMMARY

This report is a sequel to a previous state-of-the-art report published by SHRP in 1991 and is based on review of selected literature on high-performance concrete with particular reference to highway applications. It covers primarily the period from 1989 to 1994 for which the authors have recently compiled an updated annotated bibliography, which has been published by the Federal Highway Administration (Publication No. FHWA-RD-96-112). More recent information on the subject is obtained directly from a variety of publications including books, technical journals, conference proceedings, and research reports.

Much of the information contained in the previous state-of-the-art report is still current. Therefore this document should be viewed as a supplement to rather than a replacement for the previous report. For ease of cross-referencing, the organization and format of the present report have been kept nearly the same as the previous state-of-the-art report.

Included in the review and discussion are the behavior of plastic concrete as well as the strength and deformation characteristics of hardened concrete. Both short-term and long-term effects are considered. In addition, the behavior of both discrete and continuous fiber-reinforced concrete is covered. Furthermore, recent applications of high-performance concrete for pavements and bridges, here and abroad, are summarized. Finally, recent activities of organized research and development programs on high-performance concrete in different countries are described.

Based on the review of the available information, it is concluded that the growth of the amount of research and applications of high-performance concrete has been phenomenal in the past several years. high-performance concrete has become widely accepted practically on all continents. Much of the application of HPC remains in the areas of long-span bridges and high-rise buildings.

A generalized definition of high-performance concrete seems to have been accepted by the engineering community. Such a definition is based on achievement of certain performance requirements or characteristics of concrete for a given application that otherwise can not be obtained from normal concrete as a commodity product.

Based on the results of the SHRP projects, the Federal Highway Administration has developed a sensible classification of high-performance concrete according to different levels of performance requirements. Such a classification would enable design engineers to select appropriate performance criteria of HPC for different highway applications in different environmental conditions.

Increasing emphasis is being placed on concrete durability than its strength. In many applications, high strength concrete is used only because of its high durability quality rather than the need for its strength.

There is much understanding of selection of materials, proportioning methods and curing control for the production of high-performance concrete. However, much less control is exercised on the concrete placement. In this regard, the development of "self-compactable" concrete in Japan is a significant step toward achieving high-performance concrete through automation.

There has been an enormous amount of research performed on durabilty of concrete, but without much correlation largely because the property is "material specific" and dependent on test methods. There is an urgent need for new and improved test methods that would provide more consistent correlation between the laboratory and field results so that the data on durability can be better quantified. More research is needed to develop a rational design methodology for durability.

Much research continues to be focused on the mechanical properties of high- and very-high-strength concretes and their structural applications. The results of this research are being incorporated into various national codes of practice. However, more information is needed on the behavior of the concrete at its early age and its relationship to the long-term performance.

Comparatively speaking, there is a dearth of information on the mechanical behavior of controlled lower strength concrete. Similarly, the need exists for more research on high-performance lightweight concrete so that its use can be more widely accepted in practice.

The Slurry Infiltrated Mat Concrete (SIMCON) and the delivery system for non-metallic fibers developed by 3M Corporation are two significant recent developments in the area of high-performance fiber reinforced concrete.

There has been significant interest and development in the use of continuous fiber reinforcement for improving the behavior of cementitious composites and/or concrete. Fiber Reinforced Polymers (FRP) or sometime also referred to as Fiber Reinforced Plastic are increasingly being accepted as an alternative for uncoated and epoxy-coated steel reinforcement for prestressed and non-prestressed concrete applications.

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INTRODUCTION

1.1 Background

In 1989, a four-year investigation on the mechanical behavior of High-Performance concretes was initiated by the authors at North Carolina State University. The research was conducted under contract C-205 with the Strategic Highway Research Program (SHRP) which was a unit of the National Research Council. SHRP was authorized by section 128 of the Surface Transportation and Uniform Reloaction Assistance Act of 1987.

The first task of the research was to conduct an extensive literature search and review so that the existing knowledge about the mechanical properties of High-Performance concrete, with particular reference to highway applications, could be summarized and significant gaps in knowledge identified. Based on that literature search, an annotated bibliography containing 830 references published in the period of 1974-1989 was compiled and published by SHRP in 1990 as a reference document [Leming et al. 1990]. From that reference source, about 150 references were selected for critical review. The results of the review were summarized in a state-of-the-art report [Zia et al. 1991] published by SHRP in 1991.

Since the publication of these two previous documents, there has been a phenomenal increase in the development and use of High-Performance concrete. So a need exists to update the previous documents and summarize the significant developments during the past several years. This volume is a sequel to the previous state-of- the-art report and covers the six-year period from 1989 to 1994. A second annotated bibliography containing 776 references for the six-year period has also been compiled by the authors as a separate document [Zia et al. 1996].

It should be emphasized that, unless otherwise noted, much of the information contained in the previous state-of-the-art report is still current and will not be repeated in the present document. Therefore this document should be viewed as a supplement to rather than a replacement for the previous report.

1.2 Definition of High-Performance Concrete (HPC)

Any concrete which satisfies certain criteria proposed to overcome limitations of conventional concretes may be called High-Performance concrete (HPC). It may include concrete which provides either substantially improved resistance to environmental influences (durability in service) or substantially increased structural capacity while maintaining adequate durability. It may also include concrete which significantly reduces construction time to permit rapid opening or reopening of roads to traffic, without compromising long-term serviceability. Therefore it is not possible to provide a unique definition of HPC without considering the performance requirements of the intended use of the concrete.

Forster [1994] defined HPC as "a concrete made with appropriate materials combined according to a selected mix design and properly mixed, transported, placed, consolidated, and cured so that the resulting concrete will give excellent performance in the structure in which it will be exposed, and with the loads to which it will be subjected for its design life." In discussing the meaning of HPC, Aitcin and Neville [1993] stated that "in practical application of this type of concrete, the emphasis has in many cases gradually shifted from the compressive strength to other properties of the material, such as a high modulus of elasticity, high density, low permeability, and resistance to some forms of attack."

A more broad definition of HPC was adopted by the American Concrete Institute. HPC was defined as concrete which meets special performance and uniformity requirements that cannot always be achieved routinely by using only conventional materials and normal mixing, placing, and curing practices. The requirements may involve enhancements of (characteristics such as) placement and compaction without segregation, long-term mechanical properties, early-age strength, volume stability, or service life in severe environments. Concretes possessing many of these characteristics often achieve higher strength.

Therefore HPC is often of high strength, but high strength concrete may not necessarily be of High-Performance.

For the purpose of the SHRP C-205 project [Zia et al. 1993], HPC was defined in terms of certain target strength and durability criteria as shown in Table 1.1. In this definition, the target minimum strength should be achieved in the specified time after water is added to the concrete mixture. The compressive strength is determined from 4 x 8-in. (100 x 200-mm) cylinders tested with neoprene caps. The water-cement ratio (W/C) is based on all cementitious materials. The minimum durability factor should be achieved after 300 cycles of freezing and thawing according to ASTM C 666 (AASHTO T 161), procedure A.

Based on the results of SHRP C-103 and SHRP C-205 research, the Federal Highway Administration (FHWA) has proposed criteria for four different performance grades of HPC [Goodspeed et al. 996]. The criteria are expressed in terms of eight performance characteristics including strength, elasticity, freezing/thawing durability, chloride permeability, abrasion resistance, scaling resistance, shrinkage, and creep as shown in Table 1.2. Depending on a specific application, a given HPC may require different grade of performance for each performance characteristics. For example, a bridge located in an urban area with moderate climate may require Grade 3 performance for strength, elasticity, shrinkage, creep, and abrasion resistance, but only Grade 1 performance for freezing/thawing durability, scaling resistance, and chloride permeability.

1.3 Scope

This report focuses on more recent information and developments on mechanical behavior of High-Performance concretes to update the previous state-of-the-art report. Included in this report are discussions regarding selection of raw materials and production techniques, behavior of both plain and fiber reinforced concretes in fresh and hardened states, and applications of High-Performance concrete.

As was the case with the previous state-of-the-art report, several types of special materials are excluded from consideration for both economic and practical reasons. They include extremely high strength concrete, rapid setting patching compunds, roller compacted concrete, lightweight concrete, sulfur concrete, high alumina cement, polymer concretes, polymer impregnated concretes, and sealed concrete.

1.4 Organization of Report

This report contains eight chapters. Following this introductory chapter, a discussion on materials selections and production techniques for HPC is presented in Chapter 2. Chapter 3 describes the properties and behavior of HPC in its fresh state, followed by discussions of the behavior of HPC in its hardened state in Chapter 4. Behavior and applications of High-Performance fiber reinforced concrete (HPFRC) are covered in Chapter 5, and applications of HPC are described in Chapter 6. To give an indication of the trend of development, several current research programs on HPC are briefly summarized in Chapter 7. Finally, conclusions from this review of the state-of-the-art on HPC are drawn in Chpater 8.

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Table 1. Definition of HPC according to SHRP C-205 (Zia, et al. 1993)1

Category of HPC	Minimum Compressive Strength	MaximumWater/ Cement Ratio	Minimum Frost Durability Factor
Very early strength (VES)			
Option A (with Type III cement)	2,000 psi (14 MPa) in 6 hours	0.40	80%
Option B (with PBC-XT cement)	2,500 psi (17.5 MPa) in 4 hours	0.29	80%
High early strength (HES) (with Type III cement)	5,000 psi (17.5 MPa) in 24 hours	0.35	80%
Very high strength (VHS) (with Type I cement)	10,000 psi (70 MPa) in 28 hours	0.35	80%

Table 1.2 Definition of HPC according to Federal Highway Administration (Goodspeed, et al. 1996)

Performance Characteristics	Standard test method	FHWA HPC performance grade			
		1	2	3	4
Freeze-thaw durability (X = relative dynamic modulus of elasticity after 300 cycles)	AASHTO T 161 ASTM C 666 Procedure A	60%≤X<80%	80%≤X		
Scaling resistance (X = visual rating of the surface after after 50 cycles)	ASTM C 672	X=4, 5	X=2, 3	X=0, 1	
Abrasion resistance (X = avg. depth of wear in mm)	ASTM C 944	2.0>X≥1.0	1.0>X≥0.5	0.5>X	
Chloride penetration (X = coulombs	AASHTO T 277 ASTM C 1202	3000≥X>2000	2000≥X>800	800≥X	
Strength (X = compressive strength)	AASHTO T 2 ASTM C 39	41≤X<55 MPa (6≤X<8 ksi)	55≤X<69 MPa (8≤X<10 ksi)	69≤X<97 MPa (10≤X<14 ksi)	97 MPa≤X (14 ksi≤X)
Elasticity (X = modulus)	ASTM C 469	28≤X<40 GPa (4≤X<6x10^6psi)	40≤X<50 GPa (6≤X<7.5x10^6psi)	50 GPa≤X< (7.5x10^6psi≤X)	
Shrinkage (X = microstrain)	ASTM C 157	800>X≥600	600>X≥400	400>X	
Specific creep (X = microstrain per MPa)	ASTM C 512	75≥X>60/MPa (0.52≥X>0.41/psi	60≥X>45/MPa (0.41≥X>0.31/psi	45≥X>30/MPa (0.31≥X>0.21/psi	30/MPa≥X (0.21/psi≥X

MATERIALS AND PRODUCTION

2.1 Introduction

At least one type of HPC, that is, High-Strength Concrete (HSC), has been used in many locations for well over a decade. Since the completion of the Strategic Highway Research Program, other types of HPC have moved more and more from the research or limited field trial stage into full scale commercial use. However, significant new developments in the materials or production of HPC have been few since the publication of the SHRP C-205 State-of-the-Art Report (SAR) [Zia et al. 1991].

Most of the research in HPC since the publication of the State-of-the-Art Report in 1990 has concentrated on increasing basic knowledge regarding HPC performance and properties rather than developing new types of HPC. HPC research and utilization continues to be concentrated in HSC or in improved durability, primarily using the materials and methods developed prior to the publication of the SAR. Changes have come primarily in modification of existing mix components and proportions.

While additional research on the mechanical properties of HSC has been conducted, much of the research of the last five years has concentrated on particular applications in buildings and bridges rather than the properties of the concrete itself. With time, data regarding the long term properties of HSC have been reported. Mechanical properties and durability of lightweight HSC have been reported, in many cases for use in extreme environments where concerns include abrasion resistance of the concrete to sea ice, as well as frost durability.

In the area of durability, substantial research has been conducted on the frost resistance of HPC. Much of the research has investigated the behavior of non-air entrained concretes with very low (<0.35) water to cementitious material ratio (W/CM), concentrating on HSC containing silica fume and high range water reducers. Research has also been conducted on the permeability, passage of chloride ions and carbonation of various concretes, many of which are HSC by virtue of their low W/CM ratios. The impact of high cement contents on heat of hydration, and the consequent effects on strength and durability in place have also been investigated. Some work on the abrasion resistance behavior of HPC concrete and its use to improve pavement durability have been reported, as has work on the fire endurance of HSC.

An important improvement in constructibility has been the further development of super-workable concrete. This has added another important, practical dimension to the concept of HPC. These concretes with improved constructibility have been developed using materials which, while not exotic, have not been routinely used, in some cases, and have not been used in the combinations and proportions reported. Other investigations concerned with constructibility issues have involved the relative sensitivity of HPC to curing conditions, including external temperature.

In this chapter, materials and methods used to produce HPC are reviewed which were not addressed previously or in which there has been a shift in focus in the last five years.

2.2 Raw Materials and Proportions

2.2.1 New Materials

Few new raw materials have been introduced into the technology in the last five years which promise to make a substantial difference in HPC production or use. However, some existing chemical admixtures have evolved. Modifications of high range water reducers to reduce slump loss have produced additional alternatives for the engineer, however, these HRWR's typically involve extended set times which may not be advantageous for early strength applications.

Another change in composition has been noted with air entraining admixtures. Air entraining agents are now frequently based on compounds other than "neutralized vinsol resin", in many cases producing finer and more uniform air void systems than were common earlier. Neeley et al. [1992] presented results of a new air entraining admixture intended to provide adequate frost durability at low air contents. Their research, not based on HPC, indicated that low air contents provided insufficient durability to rapid freezing and thawing.

Cements and combinations of cementitious materials have been the subject of continuing research but there have been few new materials used outside the laboratory. Some blended cements, specifically developed for niche markets, have been investigated, but their use in practice has been somewhat limited. Neeley [1995] reports promising results for one product based primarily on Class C fly ash. A key ingredient of the product was three admixtures used in various quantities, added at various times and in different sequences to regulate setting characteristics. Concrete made with this product had very low w/c ratios and setting times which varied from approximately a half hour to over three hours.

In preliminary testing at the U.S. Army Engineer Waterways Experiment Station, Ash Bonding Chemicals Corporation Cement was found to provide relatively high strengths at early ages with good frost resistance, low to very low permeability as measured by Chloride Ion Permeability tests and reduced shrinkage when compared to a conventional Type III portland cement based concrete containing water reducing and high range water reducing admixtures. However, the concrete was more variable and admixture effects are still neither completely understood nor completely predictable. In addition, costs and control issues due to the addition of the admixtures on the job site, possibly including air entraining agents, have apparently not been fully investigated.

2.2.2 Review of Other Selected Raw Materials

2.2.2.1 Cementitious Materials

Renewed interest in shrinkage compensating cements was generated several years ago in jointless slabs, including bridge decks. While concretes made with these cements have good strength characteristics and exceptional sulfate durability, there have been serious concerns with the durability of the concrete exposed to deicer salts. Reports presented in sessions at the 1995 Transportation Research Board indicated very different experiences in Ohio and New York with shrinkage compensating cement. Bridge members produced with shrinkage compensating cement in Ohio were sealed and have apparently performed well to date. Structures in New York which were not sealed exhibited some premature scaling and deterioration. There was speculation that differences in performance may be due to sealing of the concrete.

The blended cement Pyrament was investigated as part of the SHRP C-205 research. This material could be used to produce concretes with exceptional early strength characteristics and very good later age properties. Further research by Husbands et al. [1994] found that performance and durability were generally good. Concerns with alkali-silica reactivity have not been completely resolved but appear to be manageable. Unfortunately, Pyrament never gained broad market acceptance due to availability and cost, and its production has been suspended. Super-fine cements continue to be unavailable on a commercial basis. Regulated set cements are acceptable for early strength applications except where sulfate exposure is likely.

2.2.2.2 Mineral Admixtures

While additional research continues with mineral admixtures, especially silica fume, most of the new research involves different proportions rather than new materials. However, a few other mineral admixtures have also received attention. The use of zeolitic admixtures, a natural pozzolan, were examined by Feng et al. [1990] as was metakaolin, a reactive alumino-silicate pozzolan by Walters and Jones [1991]. The use of 5% and 10% metakaolin was found to be very similar to the use of similar percentages of silica fume in terms of permeability, frost durability and mechanical properties. The major differences noted were in color (the metakaolin was much lighter in color) and in HRWR dosages (concrete containing silica fume had a much higher demand).

A number of researchers have confirmed earlier reports that mineral admixtures typically reduce the permeability of concrete. Detwiler et al. [1994] have reported on this phenomenon in steam cured concrete. Geiker et al. [1991] noted that both silica fume and fly ash reduced the permeability of concrete to the penetration of chloride ion without changing the total porosity greatly. Bijen and van Selst [1991] found higher rates of carbonation in concrete with typical commercial quantities of ground granulated blast furnace slag compared with concretes containing typical quantities of fly ash. However, this study was not conducted with HPC.

Silica fume (also called condensed silica fume or microsilica) continues to be a popular element of high performance concrete, and especially high strength concrete. Not only does it provide an extremely rapid pozzolanic reaction, but researchers including Detwiler and Mehta [1989], and Goldman and Bentur [1993] found that its very fine size also appears to provide a beneficial contribution to concrete. Detwiler and Mehta, and Goldman and Bentur examined the effects of silica fume on mechanical behavior. Luther's [1989] review examined durability effects, while Fidjestol [1993], and Khayat and Aitcin [1993], have provided general reviews of the effects. These reports confirm findings that silica fume tends to improve both mechanical properties and durability.

However, St. John et al. [1994] report that deleterious expansion due to alkali silica reactivity is possible under wetting and drying conditions when particles of the densified form of silica fume admixtures are not sufficiently dispersed during mixing. In addition, a number of research efforts have attempted to answer important and unresolved questions in long term strength gain and frost resistance of silica fume concrete.

Maage et al. [1990] report that silica fume concretes continue to gain strength under a variety of curing conditions, including unfavorable conditions. They further indicate that concretes with silica fume appear to be more robust to early drying than similar concretes which do not contain silica fume.

A number of issues with frost resistance of concrete containing silica fume have been investigated, including the need for any entrained air when working with very low W/CM ratio concretes. Due to the dramatic reduction in permeability which accompanies the use of silica fume, concerns with frost durability in general, and with the usefulness of rapid freezing and thawing tests, have complicated the interpretation of research results. These issues are discussed in more detail in section 2.2.4, however it is useful to note here that ACI 318-95 [1995] limits the quantity of silica fume in concrete exposed to deicing salts to no more than 10 percent.

Attempts to improve the performance of systems of cementitious material in HSC have led researchers to examine mixes with multiple cementitious components. The use of ternary cementitious systems has received attention in recent years. Kashima et al. [1993] report on HSC produced with a blend of cement with large amounts of fly ash and ground granulated blast furnace slag in order to reduce heat of hydration. Their report is important because it reviews both experimental work and construction results.

Sarkar et al. [1991], examining the microstructural development in HSC using both silica fume and fly ash, found that strength at twelve hours was improved over similar mixes with silica fume alone. They state that this phenomenon may be related to the liberation of soluble alkalies from the surface of the fly ash. Baalbaki et al. [1993], reported on the properties of HPC produced with an extremely finely ground Type V cement with various mineral admixtures. Mixes with prolonged working times and very high strengths at ages out to one year were produced.

The use of cementitious systems with very high quantities of fly ash have also been investigated. Carette et al. [1993], and Bilodeau and Malhotra [1994], report that mixes have been developed which provide acceptable plastic and hardened properties, although strengths were not high, especially at early ages. In other studies, Bilodeau et al. [1994], and Malhotra [1990] report that performance in rapid freezing and thawing of concrete with high volumes of class F fly ash was adequate but that the concrete with very high quantities of fly ash performed poorly in deicer scaling tests. However, Nasser and Lai [1993] found that high volume, class C fly ash concrete was not frost durable even with a 6% air and after prolonged moist curing. They found that 20% fly ash mixes showed no difficulties in this respect. Kukko and Matala [1991] noted that frost resistance of non-air entrained concrete was reduced for very low W/CM ratio concrete produced with slowly hardening portland cement or containing slag, compared to rapid hardening portland cement with or without silica fume.

Naik et al. [1994] found that although concrete made with high volumes of class C fly ash passed ASTM C-944 for abrasion resistance, better abrasion resistance was obtained for concrete without the high fly ash content. Gjorv et al. [1990] also found that the abrasion resistance of conventional HSC pavements is exceptionally good. It would appear that high volume fly ash mixes have limited applicability to highway structures, although additional research appears warranted.

2.2.2.3 Aggregates

High Performance Lightweight Concrete (HPLC) has been extensively investigated for, among other applications, use in oil drilling platforms in severe environments. Hoff [1991], and Tachibana et al. [1990], have presented information on the behavior of HSLC in extreme conditions. Hoff has demonstrated that HSLC containing both lightweight aggregate and conventional weight stone is both frost resistant and resistant to abrasion by ice. Holm and Bremner [1991], have provided additional information on the long term durability of lightweight concrete, indicating that in general when well-known prophylactic measures are taken to insure durability, long term durability is good.

Zhang and Gjorv [1991a, 1991b, 1991c, 1991d] have investigated both the mechanical properties and the permeability of lightweight concretes with strengths ranging from 50 to 100 MPa (about 7,000 to about 14,500 psi). Elastic modulus and the tensile-compressive strength ratio were lower than would be expected with conventional stone concrete at the same strength levels. While permeability of the HSLC's was very low, it was noted that permeability might be higher with lightweight aggregate than with conventional aggregate at the same strength. This, of course, would depend on the porosity and permeability of the aggregate itself.

2.2.3 Proportioning Methods

Modifications to conventional proportioning methods have been proposed by several researchers. Mehta and Aitcin [1990], and ACI Committee 363 report [1990] provide an excellent review of proportioning considerations for HSC. Selection of the proper raw materials and adjustment of proportions based on experience, using mixes conducted both in the laboratory and in the field, have typically proven adequate to achieve the desired concrete characteristics, at least within the limits allowed by the available raw materials. With adequate control of production and placement, routine use of concrete with compressive strengths in excess of 70 MPa (10,000 psi) is practical in many areas.

Several articles have been published with suggestions on methods of optimizing the development of particular mixes by reducing the number of trial mixes necessary. Campbell and Detwiler [1993], for example, have provided guidance for proportioning and producing steam-cured concrete. While de Larrard [1990] has provided suggestions for HSC mixes based on rheological considerations, Domone and Soutsos [1994] have reexamined the maximum density theory for applicability to HSC.

Field trials of High Early Strength (HES), Very Early Strength (VES) and Very High Strength (VHS) concretes in SHRP C-205 and C-206 indicated that existing proportioning methods remain valid, with minor modifications, for these mixes. Routine precautions such as those regarding minimum water contents and appropriate quantities or combinations of chemical admixtures, contained in numerous publications and discussed in the previous State-of-the-Art Report, remain valid. Development or adaptation of new types of high performance concrete or combinations of raw materials are better served by engineering judgement than by more sophisticated proportioning techniques.

2.2.4 Air Entrainment

The need for any air entrainment at all in concrete with very low W/CM ratio has been questioned. This issue has been complicated by the problem of interpreting test results of one of the most commonly used test methods in practice. ASTM C 666 measures the resistance of concrete to rapid freezing and thawing. The rate of freezing is much higher in this test than is found in practice, and C 666 has been criticized in this respect even for conventional concrete.

Concretes with a low W/CM ratio, such as HPC and HSC, have a lower permeability than conventional concrete. A rapid freezing and thawing rate may induce additional damage to concretes with low W/CM ratio simply due to the lower permeability. On the other hand, the very low w/c ratio, for an adequately cured concrete, can reduce or even eliminate the amount of freezable water in the pores for practical temperature ranges. These mixes will also dramatically reduce the ingress of water during the test, therefore reducing the amount of damage due to physically freezing water in the concrete. The time required to achieve an internal moisture content sufficient to contribute to frost damage is less than the time required for the C 666 test for concretes with very low W/CM ratios. However, since hydraulic pressure due to freezing of water is only one of several mechanisms of frost damage, the use of very low W/CM may not be adequate in all cases. Experimental results have been mixed.

Damage to concrete specimens may be due to thermal shock or to disequilibrium between energy states during cooling rather than expansion associated with the presence of freezing water for the mixes of very low W/CM ratio. Hanson et al. [1993], found that the electrical impedance of air entrained concrete of very low W/CM ratio actually increased during ASTM C 666 testing while the dynamic elastic modulus decreased. The increase in impedance could only have come from internal drying associated with the loss of free moisture during curing. Therefore, the decrease in elastic modulus was not related to the formation of ice.

Many believe that C 666 testing is still a valid discriminant for frost durability even though it is extremely severe for concretes of low W/CM ratio. Others have chosen to rely on deicer scaling as a more useful and informative test, particularly for highway and pavement applications.

Hammer and Sellevold [1990] report that salt scaling resistance is acceptable for HPC with W/CM ratios below 0.37, even without entrained air but that rapid freezing and thawing is accompanied by deterioration for all non-entrained air concrete tested down to a W/CM of 0.25. However, they also note that calorimeter data indicates very little ice formation until -20 c. They state that this indicates that much of the deterioration may be due to thermal incompatibility of the components rather than the formation of ice.

In research conducted by Kashi and Weyers [1989], non-air entrained concrete with a W/CM ratio of less than 0.30 was found to be resistant to rapid freezing and thawing based on ASTM C 666, Method A. For concretes with a W/CM ratio of 0.32, the concrete was frost resistant only if silica fume was not used in the mix. Cohen et al. [1992] similarly found that non-air entrained concrete with a W/CM ratio of 0.35 and containing 10% silica fume were not resistant to rapid freezing and thawing when tested in accordance with ASTM C 666 (A), even when curing had been extended to 56 days. They also noted that although there was a dramatic drop in the elastic modulus, the reduction in compressive strength was much less severe. Li et al. [1994], on the other hand, found that a maximum W/CM ratio of 0.24 was necessary for adequate frost protection of non-air entrained concrete when based on ASTM C 666 (A) testing.

Tests conducted by Pigeon et al. [1991], and by Gagne et al. [1991], using both ASTM C 666 (A) and ASTM C 672 deicer scaling tests, indicated that non-air entrained concrete containing silica fume and good quality coarse aggregate, with a W/CM ratio of 0.30 generally, but not uniformly, had good resistance to deicer scaling. When tested using ASTM C 666 (A), the W/CM ratio required to provide acceptable performance ranged from less than 0.25 to over 0.30. The cement used was found to play a significant role in the performance of otherwise similar mixes. It was also noted that the air void system produced by the use of water reducing or high range water reducing admixtures, commonly used in all concretes with low W/CM ratio, may be contributing in a fashion not yet well documented.

Additional research in these areas, perhaps concentrating more on test methods such as resistance to deicer scaling and critical dilation test concepts rather than ASTM C 666 (A), would appear to be useful. At the present time, the use of at least minimal quantities of entrained air appear prudent for concrete exposed to severe freezing, especially when exposed to deicing salts, unless that concrete has a W/CM ratio less than 0.24.

2.2.5 Other Types of HPC

Another type of HPC has been developed for use in situations where vibration is difficult or impossible and where reinforcing steel is highly congested. "Super-workable" or "flowable" concrete has been developed in Japan and used in both bridge structures and buildings. Although not specifically developed for high strength, low water to "powder" ratios are common. Kuroiwa et al. [1993], report super-workable concrete with strengths in excess of 50 MPa, a marginally high strength concrete.

Low W/CM ratio, flowing concretes utilizing HRWR's are well known. Super-workable mixes are an extension of this concept which have been specifically formulated to resist segregation. Super-workable concretes also derive partly from concrete developed for underwater placement. Research has led in several different directions. Paste and aggregate volumes, and paste composition, admixture type and dosages, and testing methods have been investigated.

A number of different "powder" combinations are reported including portland cement, fly ash, GGBFS and silica fume. In addition, where a low heat of hydration was a concern, finely ground limestone powder was used as a partial replacement for cement. This material was reported [Tanaka et al. 1993] to have both a chemical and physical effect similar to that reported for silica fume. Low heat of hydration mixes will typically exhibit considerably extended set times, with final set at about twenty hours, and low early strengths. This is due both to large quantities of HRWR in conjunction with a blended cement composed of 30% low heat of hydration portland cement and 70% GGBFS. The aggregate paste ratio was somewhat lower compared to conventional concrete.

Cellulose based products have been known to improve the cohesion of concrete and to reduce segregation. Sogo et al. [1987] report that a polymer based on cellulose ether can be used to increase both water reduction and cohesiveness. Many long chain organic molecules have water reducing capability and, if air content and setting time can be controlled, can be successfully used in concrete [Mehta 1975] with various effects on cohesiveness. Kuroiwa et al. [1993] report on findings using a polysaccharide polymer. Ozawa et al. [1990] describe studies to optimize the combination of HRWR and other admixtures affecting the viscosity of this type of mix.

Self compacting capability and resistance to segregation were determined by so-called "slump-flow", by a modified grout cone flow test and by self-leveling flow through reinforcing bars, as well as by mock-ups of particular members. Self-leveling, non-segregating performance was reported with several different mixtures. Details of these tests are discussed in the next chapter.

The addition of materials to compensate for increased drying shrinkage due to the lower aggregate content has been suggested, although Kuroiwa et al. state that drying shrinkage is equal to or better than comparable, conventional concretes. Frost durability was also found to be adequate for super-workable concrete containing at least 4% entrained air. Testing was based on ASTM C 666, although it was not stated whether Procedure A or B was used.

It is significant to note that these types of mixes have been successfully used, without vibration, in field placements. Applications include the Akashi Kaikyo Bridge near Kobe, Japan, and in the heavily reinforced concrete core and shear wall of a 20-story building. Concrete delivery was by pump. The use of super-workable concretes in certain applications appears promising.

2.3 Production Considerations

For high strength concrete where early strength is not a critical consideration, the use of conventional production methods and facilities appears adequate, as long as well recognized practices for the production of good quality concrete are enforced. Howard and Leatham [1989] and Sanchez and Hester [1990] discuss the production and delivery of HSC noting the importance of a team approach. Kakizaki et al. [1993], note that the mixing sequence can affect the slump and compressive strengths of very high strength concretes. Leming et al. [1993], emphasize that pre-pour conferences and field trials are necessary with any HPC prior to actual use.

In situations where early strength is critical, particular care must be taken to insure that temperature is closely controlled and that high dosages of water reducing or high range water reducing admixtures are avoided, since these can extend the time of set. While strength after one day may not be significantly affected, strengths at less than twenty-four hours and particularly before twelve hours, can be significantly reduced. If mineral admixtures are included, caution must be exercised in dosing the entire quantity of cementitious material. When significant percentages of mineral admixtures are employed, the result may be to effectively overdose the portland cement, again resulting in extended set times.

Much of the VES and HES concrete used in field trials for SHRP C-205 and C-206 contained a corrosion inhibiting admixture as a non-chloride accelerator, added at the job site to mitigate rapid slump loss [Hanson et al. 1994]. The admixture was added either by hand or by pump typically from trailer-mounted tanks. In either case, provision for adding the admixture, adequate quality control and sufficient remixing time are necessary.

Due to the high water content of this admixture and the low W/CM ratio of the paving mixes investigated, it was necessary to employ a HRWR in the initial batching. Inadvertent use of large dosages of HRWR caused low strengths at ages up to and including one day. The low water content and rich paste of these mixtures required strict control of the batching sequence. Although only minor adjustments to conventional practice were required, some adjustment must be anticipated. Exact procedures will vary from one production facility to another, but there are several keys to a successful placement. The plant itself should be equipped with an automatic moisture indicator for the aggregate.

A pre-placement conference, including all parties who will be involved in the slab-on-grade placement, is required. A practice placement to adjust operations, if necessary, to develop estimates for slump and air loss in transit, and to acquaint the crews involved in the placement is highly recommended. Inspection of trucks, especially in a dry batch operation, is necessary to insure that only trucks with clean fins and adequate mixing speed are used. Trucks should carry no more than two-thirds of their rated mixing capacity to insure adequate mixing on the job site if there is to be any addition of admixtures on the job site. Trucks should discharge their entire load as soon as possible. A time limit of ten minutes after arrival on the job site should be used for planning purposes but may vary depending on the type and composition of the concrete being used. Special ready-mixed concrete trucks, intended for paving operations, should be used if discharge of very low slump is anticipated.

Since most HPC's are paste rich with low water content, bleeding is typically very low. This can potentially create difficulties with plastic shrinkage cracking. Therefore, it is necessary to apply curing compound or take other precautionary measures to reduce evaporation as soon as possible for slabs or members with large exposed surfaces. However, field trials for SHRP 205 [Leming et al. 1993] of HES and VES concretes in slabs found that cracks due to plastic shrinkage were rare. The concrete was apparently gaining strength faster than it shrank. Another consequence of the rapid setting and strength gain was that time prior to sawing the slabs was reduced. It was critical that joints be sawed as early as possible. Delays past eight hours were found to cause cracking of 20 cm depth (8 in.) pavements at approximately 7.5 m (approximately 25 ft) intervals.

2.4 Fiber Reinforced Concrete

Although there has been continued interest and research in the use of fiber-reinforced concrete (FRC), there have been few major innovations in proportioning or production of high performance fiber reinforced concrete (HPFRC) since the last State-of-the-Art Report. In addition, while research in FRC has examined the influence of modifications of existing fibers, fibers with larger aspect ratios, and higher fiber volumes, and there continues to be interest in non-metallic fibers or combinations of fibers, these researches were based on existing fiber materials.

One of the few new approaches in this area has been the development of SIMCON, or Slurry Infiltrated Mat Concrete, described by Hackman et al. [1992] and Krstulovic-Opara et al. [1994]. They noted that SIMCON is a different material from SIFCON, or Slurry Infiltrated Fiber-Reinforced Concrete, which is based on the use of prepacked discontinuous steel fibers. SIMCON, on the other hand, uses a manufactured continuous mat of interlocking discontinuous steel fibers, placed in a form, and then infiltrated with a flowable cement-based slurry. The use of continuous mats, typically made with stainless steel to control corrosion in very thin members, permits development of high flexural strengths and very high ductility with a reduced volume of fibers than SIFCON.

The use of SIMCON appears to be very promising for at least two reasons. First, the very high volume of fibers required to provide significant increases in mechanical properties such as SIFCON can create a problem with economic justification in a large number of practical applications. However, with SIMCON, direct tensile strengths of 15.9 MPa at 1.1% strain have been reported with only a 5% volume fraction of fibers. Secondly, in situations where normal FRC may be economically justified, such as in pavements, the addition of fibers to the mix and the placement of the fiber-reinforced mix required special care, and considerable extra time and expense. SIMCON overcomes many of these limitations since the fiber mat, normally delivered in large rolls, can be laid out by hand and the slurry simply pumped into place. The use of SIMCON permits fabrication of thin, complex shapes with very high ductility and flexural strength.

Another interesting and useful development in FRC construction has been to provide non-metallic fibers in small, cylindrical bundles, approximately 50 mm high (the length of the fiber) and 55 mm in diameter, wrapped in a water soluble compound. This permits the easy addition of the fibers, by hand, into the mixing drum of a truck mixer, either during charging or at the job site. The wrapper disintegrates, allowing the fibers to disperse into the concrete mixture with little balling or segregation. Quality control is improved by making the quantity of fibers added easy to determine and easy to check, and by minimizing problems in dispersion in the mixer. Further, production rates are maintained with little additional effort. Successful field applications in a full-depth pavement, a thin bridge-deck overlay, a Jersey barrier, and white-topping on scarified asphalt pavement have been reported from South Dakota [Ramakrishnan and Kakodkar 1995].

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BEHAVIOR OF FRESH CONCRETE

3.1 Introduction

The behavior of fresh HPC concrete is not substantially different from conventional concrete. While many HPC's exhibit rapid stiffening and early strength gain, others may have long set times and low early strength. Since setting and slump loss are not necessarily related, the specifics of each mixture must be analyzed individually. Workability is normally better than conventional concretes produced from the same set of raw materials, at least initially, and super-workable concretes can be used for difficult concrete placements where internal vibration is problematic. Curing is not fundamentally different for HPC than for conventional concretes, although many HPC's with good early strength characteristics may be less sensitive to curing.

However, the high volume of cementitious material and low bleeding characteristics of HPC's, particularly when combined with rapid slump loss, contribute to several characteristics which must be anticipated. Hard troweled surfaces may be difficult to attain in some cases. Plastic shrinkage problems may be exacerbated. While pumping is typically easy with HPC, breakdowns in pump operations may be more problematic. With adequate planning, these characteristics should pose no particular problem.

3.2 Workability

The workability of HPC is normally good, even at low slumps, and HPC concrete typically pumps very well, due to the ample volume of cementitious material and the presence of chemical admixtures, particularly HRWR. Page [1990] reviewed pumping operations and concrete mixtures used to successfully pump concrete on the 79 story South Wacker Tower in Chicago. One concern with pumping any concrete, including many of the HSC's is the development of a contingency plan for pump breakdown. Options include multiple pumps so that if one breaks down, concreting operations can be concentrated on one pump until resumption of pumping by all units.

While many HSC's respond well to pumping, once the movement of the concrete and vibration of the pump have stopped, it can be difficult to restart pumping operations due to the thixotropic behavior of these mixes. Contingency plans should therefore also address the issues of line clean-out and disposal of waste concrete. Again, detailed pre-construction and pre-pour meetings are very important.

The influence of internal vibration on air contents was investigated by Simon et al. [1992]. They concluded that air content at the point of insertion was dramatically changed but that away from the vibrator, the effect was minimal. HPC mixes have not been investigated in any depth in this respect but, due to the increase in consistency, should behave as well or better. Cursory examination of cores from C-205 HPC field placement in North Carolina using a bridge deck type of slip form paver with significant internal vibration showed no dramatic difference between cores and standard cylinders.

Hard, smooth, hand troweled finishes may be difficult to attain with some HSC's, with rapid slump loss and somewhat extended set times. The proper time to begin troweling operations can be difficult to judge. As moisture evaporates from the surface without being replaced, the surface will stiffen and appear ready for troweling. Since the concrete has not achieved true set, however, troweling may produce irregularities in the surface in addition to trowel marks. In addition, the slab must be adequately protected from plastic shrinkage during this time, otherwise severe plastic shrinkage cracks can develop. This is, of course, not a problem for most formed members.

The development of super-workable concretes has already been noted in Chapter 2. These concretes have the ability to fill heavily reinforced sections without internal or external vibration, without segregation and without developing large sized voids. These mixtures are intended to be self-leveling and the rate of flow is an important factor in determining the rate of production and placement schedule. It is also a useful tool in assessing the quality of the mixture. See section 3.5 for a further discussion of test methods. Flowing concrete is, of course, not required in all HPC and adequate workability is normally not difficult to attain.

3.3 Setting Time

Setting time can vary dramatically depending on the application and the presence of set modifying admixtures and percentage of the paste composed of portland cement. Concretes for applications with early strength requirements and concretes containing one of the many non-retarding HRWR's can lead to mixtures with rapid slump loss and reduced working time. This is particularly true in warmer construction periods and when the concrete temperature has been kept high to promote rapid strength gain.

Field trials using HPC intended for early strength applications in bridge decks or other transportation structures were conducted by SHRP C-205 and C-206. The concrete was easy to place, as long as the ready-mixed concrete trucks had adequate mixing capacity. The concrete was also easy to finish with bullfloat or highway straightedge, even though it was somewhat sticky. The early strength development of these mixes was enhanced by keeping the temperature near the maximum permitted by most specifications and then insulating the slabs after placement. This clearly also reduced setting times.

As noted in Chapter 2, the use of large quantities of HRWR or other water reducing admixtures can significantly extend setting times and therefore reduce very early strengths even though strengths at more than 24 hours may be relatively high. Dosage has to be monitored closely with mixtures containing substantial quantities of mineral admixtures so as to not overdose the portland cement if adding the chemical admixture on the basis of total cementitious material. The use of non-chloride accelerating admixtures was employed in mixtures tested by SHRP C-205 investigators [Zia et al. 1993a, 1993b, 1993c] to offset the retarding effects of the minimal dosages of melamine HRWR used. Extended set times are of value when using HPC which does not require very early strength, especially when transit and discharge times exceed about 30 minutes. This is easily attained through the use of retarding admixtures.

3.4 Curing

Since the last State-of-the-Art Report on High Performance Concrete, significant additional work has been conducted on early age characteristics and on curing requirements or sensitivity of low W/CM ratio concretes with various admixtures. Samarai et al. [1992], examined the influence of temperature, relative humidity and curing in hot climates on HSC properties. They report that the compressive strength of HSC is less sensitive to temperature and relative humidity than conventional strength concrete. However, tensile strength of HSC was found to be more sensitive. Swamy and Bouikni [1990] report that concrete containing very large quantities of ground granulated blast furnace slag require longer moist curing times to develop adequate strength and is more sensitive to drying than plain portland cement concretes, although the concrete used in the investigation was only marginally HPC.

Mak and Lu [1994] investigated HPC which contained GGBFS under non-standard curing conditions, including the effects of temperature rise due to heat of hydration. They found that lack of moist curing, such as might be found in actual, large section members, significantly reduced the compressive strength of concrete in which as much as 50% or more of the portland cement had been replaced by GGBFS, but that a mix with 30% replacement did not have a significant effect. However, they also noted that the reduction in heat of hydration of such high GGBFS mixes, with the associated reduction in microcracking, appeared to offset the extra sensitivity of the mixes to moist curing so that the results were comparable up to 91 days.

The higher initial curing temperatures associated with HPC with high cement contents require that slabs and pavements be sawed earlier than usual to prevent cracking. Cracks which form in the first 24 hours after placement have occasionally been mislabeled as shrinkage cracks. At these ages the concrete is too young to have undergone any significant drying shrinkage.

The higher internal temperatures frequently found with high early strength HPC can create a relatively large temperature change as the concrete cools. For concrete placed during the day and intended for use under early opening to traffic conditions, the temperature drop associated with a drop in the rate of reaction can coincide with cooling temperatures as evening approaches. As the concrete tries to contract, restraint by the base course creates tensile stresses.

A rapid strength gain in concrete is accompanied by a consequent gain in elastic modulus, although typically at a slightly slower rate. The large temperature change occurring with a stiffer concrete will create higher stresses and can cause more pronounced cracking than with conventional concrete pavements unless relieved by sawing. These cracks will occur, regardless of the method of curing, due to stress caused by temperature gradients, but can be minimized if the pavement is insulated on the surface until sawed.

The problem can be exacerbated with concrete mixtures containing large dosages of retarding admixtures. Although retarders, including extended set high range water reducers, delay setting and, typically, the onset of strength gain, increase in the temperature rise compared to non-retarded mixtures is common.

The increase in early strength associated with higher temperatures was not found to be problematic in 20 cm (8 in.) slabs by the SHRP C-205 research team [Leming et al. 1993; Schemmel and Leming 1993], using concretes with high cement factors. Burg and Ost [1992], and Cook et al. [1992], provide additional information on the effects of temperature rise on mechanical properties of HSC in large sections. They conclude that temperature rise is not a significant problem for members where HSC is appropriate. Sanvik and Gjorv [1992] draw the same conclusion for lightweight HSC. Detwiler et al. [1994], and Dhir et al. [1993] found that the use of fly ash in the concrete mixture also improved the resistance to chloride ion penetration of concrete at elevated temperatures. Silica fume was also reported to improve the resistance. Marzouk and Hussein [1990] reported on HPC properties at low temperatures. As expected, strength gain was slowed as temperature dropped. HPC did not appear to be significantly different from conventional concrete.

Early strength gain for transportation applications under field conditions has been reported by Schemmel et al. [1993], Leming et al. [1993], and Whiting et al. [1994]. The results of these field trials indicate that it is possible to reliably produce concretes which attain compressive strengths of 14 MPa (2,000 psi) at 4 hours up to 35 MPa (5,000 psi) at 24 hours, depending on mix composition, under summer and fall placing conditions.

3.5 Testing

Tests to measure the workability, resistance to segregation, self-leveling capability and filling capacity of super-workable concretes are not well established. However, substantial research has been conducted in this area. Super-workable concrete mixtures are self-leveling, making the slump test inappropriate. Test methods used by Kuroiwa et al. [1993] to determine the workability of super-workable concrete include the conventional slump cone test, measuring the increase in the diameter of the concrete base, or "slump flow", rather than the reduction in height of the settled concrete, for comparison. A measure of the rheological characteristics based on the rate of increase in diameter during the slump test, termed "flow speed", is also used for comparison. This is determined as the time, in seconds, required for the base diameter to exceed 50 cm (20 in.) [Kuroiwa et al. 1993]. Workability may also be gauged by time of flow through a funnel, similar to standard grout flow cone tests, adapted for use with concrete.

The test method used to evaluate resistance to both segregation and self-compaction simultaneously, termed "filling capacity", involved letting concrete flow down through one branch of a U-shaped tube, under a very slight pressure, and back up the other branch, past a mat of reinforcing bars. The height to which the concrete rose was used as a measure of the filling capacity of the mixture. Other tests include the use of mock-ups. Investigation may include examination of cores for indications of segregation.

Poor filling capacity was noted with some highly workable concrete due to segregation of the paste and aggregate. Filling capacity was found to be reasonably well correlated with the behavior of the concrete in the funnel test, in that concretes which segregated blocked the outlet of the funnel with aggregate, resulting in a longer flow-out time. This phenomenon is, of course, familiar to anyone who has tried to pump concrete which was too "wet" through several elbows.

Considerable research on the effects of different testing parameters of HSC was reported in the previous State-of-the-Art Report. Valuable additional research in this area has been provided by Carino et al. [1994], who concluded that cylinder size, cylinder end preparation, load rate, and testing machine capacity all had significant effects and all had significant interactions. Lessard et al. [1993] also report on the effects of testing methodologies for HSC. While the magnitude of the effects vary from researcher to researcher, the primary conclusions have not. The average measured strength of small cylinders is greater than that of large cylinders, stiffer machines typically provide higher measured strength and grinding ends is strongly recommended for HSC above at least 90 to 100 MPa (about 12.5 ksi to 14 ksi). Differences in careful end preparation are apparently minimal at lower strength levels.

Tests of in-place properties of HPC, particularly strength and permeability, have been reported. Haque et al. [1991], discussed the estimation of in situ strength of a variety of low, medium and high strength concretes compared to standard cylinders. They found that in situ strengths were 80% to 85% of cylinder strengths.

Air permeability tests, developed to provide a relatively quick measure of the permeability of concrete, particularly in-place, have increased in popularity. However, the sensitivity of the test to the moisture content of the concrete has limited its utilization due to concerns over interpretation of data. Concrete absorption tests, such as the Initial Surface Absorption Test, or ISAT, have also been used to characterize concrete permeability with some success. The low absorption of mature, low W/CM ratio concretes has reduced the use of this test on HPC. However, it appears well suited to lab studies, especially of deteriorated concrete.

One of the important applications of existing testing methodology was to the determination of early strength of HPC. Accurate determination of in-place strength is essential to insure that the pavement can be safely opened to traffic. Testing based on maturity concepts or using match cure systems to estimate the in-place strength of concrete was conducted in several early-to-open HPC pavements.

Both SHRP C-205 and SHRP C-206 successfully used conventional maturity methods to estimate the concrete strength in-place on several pavements constructed with HPC. SHRP C-206 used a match cure system which kept cylinders at the same temperature as the concrete in the pavement so that the compressive strength of the cylinder could be used to estimate the in-place strength. SHRP C-205 used a simpler, more cost effective system of keeping the test cylinders insulated until testing. This method was found to keep the cylinders at approximately the same temperature as the pavement so that the measured strength of the insulated cylinders provided a reasonable estimate of the in-place strength.

SHRP C-206 also investigated the use of pulse velocity in estimating the in-place strength of the pavements. Small access holes were left in the pavement to permit the use of conventional test apparatus. Another SHRP C-206 development was determination of the water content of concrete by microwave. However, this procedure proved highly variable.

Another w/c ratio gauge has been recently developed by Troxler, Inc., who have provided nuclear density gauges to the pavement construction industry for decades. This apparatus holds promise for accurately assessing both the water and cement content of mixes as batched. This product is still undergoing modifications and adjustments, but a substantial testing program was recently completed by the Highway Innovative Technology Evaluation Center (HITEC), which was established by the Federal Highway Administration and the American Association of State Highway and Transportation Officials. Preliminary evidence [HITEC 1996] suggests that the Troxler Model 4430 water-cement gauge can effectively estimate water and cement contents, when properly calibrated, within about 6 lbs/yd3 (3.56 kg/m3) for water and 20 lbs/yd3 (11.87 kg/m3) for cement, in approximately 10 minutes. Although at this time a separate and extensive calibration is required for each different concrete mix to be tested, additional work in this area is continuing. This device appears to provide a significant improvement in quality testing of concrete as delivered.

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BEHAVIOR OF HARDENED CONCRETE

The behavior of hardened concrete can be characterized in terms of its short-term (essentially instantaneous) and long-term properties. Short-term properties include strength in compression, tension, bond, and modulus of elasticity. The long-term properties include creep, shrinkage, behavior under fatigue, and durability characteristics such as porosity, permeability, freeze-thaw resistance, and abrasion resistance.

Comparatively speaking, information on the behavior of very early strength (VES) concrete and high early strength (HES) concrete is somewhat limited, whereas a substantial amount of information on the behavior of high strength concrete exists and additional information is being developed rapidly. Since high performance concretes typically have low water-cementitious materials ratios (W/CM) and high paste contents, their characteristics will, in many cases, be similar to those of high strength concrete.

A significant difference in behavior between the early strength and the high strength concretes is in the relationship of compressive strength to other mechanical properties. Typically, strength gain in compression is much faster than strength gain in aggregate-paste bond. This will lead to relative differences in elastic modulus and tensile strength of early strength concretes and high strength concretes, expressed as a function of compressive strength. Thus the relationships of mechanical properties to 28-day compressive strength of high strength concrete cannot be expected necessarily to apply to VES and HES concretes.

4.1 Strength

The strength of concrete depends on a number of factors including the properties and proportions of the constituent materials, degree of hydration, rate of loading, method of testing and specimen geometry.

The properties of the constituent materials which affect the strength are the quality of fine and coarse aggregate, the cement paste and the paste-aggregate bond characteristics (properties of the interfacial, or transition, zone). These, in turn, depend on the macro- and microscopic structural features including total porosity, pore size and shape, pore distribution and morphology of the hydration products, plus the bond between individual solid components.

Testing conditions including age, rate of loading, method of testing, and specimen geometry significantly influence the measured strength. The strength of saturated specimens can be 15 to 20 percent lower than that of dry specimens. Under impact loading, strength may be as much as 25 to 35 percent higher than under a normal rate of loading, i.e., 10 to 20 microstrains per second (me/sec). Cube specimens generally exhibit 20 to 25 percent higher strengths than cylindrical specimens. Larger specimens exhibit lower average strengths.

4.1.1 Constituent Materials and Mix Proportions

Concrete composition limits the ultimate strength that can be obtained and significantly affects the levels of strength attained at early ages. Two dominant constituent materials that are considered to control maximum concrete strength are coarse aggregate and paste characteristics.

4.1.1.1 Coarse Aggregate

The important parameters of coarse aggregate are its shape, texture and the maximum size. Since the aggregate is generally stronger than the paste, its strength is not a major factor for normal strength concrete, or for HES and VES concretes. However, the aggregate strength becomes important in the case of higher-strength concrete or lightweight aggregate concrete. Surface texture and mineralogy affect the bond between the aggregates and the paste as well as the stress level at which microcracking begins. The surface texture, therefore, may also affect the modulus of elasticity, the shape of the stress–strain curve and, to a lesser degree, the compressive strength of concrete. Since bond strength increases at a slower rate than compressive strength, these effects will be more pronounced in HES and VES concretes. Tensile strengths may be very sensitive to differences in aggregate surface texture and surface area per unit volume.

A. Effect of Aggregate Type The effect of different types of coarse aggregate on concrete strength has been reported in several recent articles. Sarkar and Aitcin [1990] conducted detailed petrological, petrographical and mineralogical characterization of twelve different coarse aggregates that have performed with variable success in very high strength concrete in Canada and the United States. They pointed out that the intrinsic strength of coarse aggregate is not an important factor if water-cement ratio (W/C) falls within the range of 0.50 to 0.70, primarily due to the fact that the cement-aggregate bond or the hydrated cement paste fails long before aggregates do. It is, however, not true for very high strength concretes with very low W/C of 0.20 to 0.30. For such concretes, aggregates can assume the weaker role and fail in the form of transgranular fractures on the failure surface. It was concluded that the minerals must be strong, unaltered, and fine grained in order to be suitable for very high strength concrete. Intra- and intergranular fissures, partially decomposed coarse-grained minerals, and the presence of cleavages and lamination planes tend to weaken the aggregate, and therefore the ultimate strength of the concrete.

Aitcin and Mehta [1990] also discussed the importance of evaluating the characteristics of coarse aggregates to be used in very high strength concrete. They tested four different types of coarse aggregates available in California in an identical concrete mixture proportion (W/CM = 0.275) to produce concrete strengths ranging from 85 to 105 MPa (12 to 15 ksi). The results showed that the compressive strength and elastic modulus were significantly influenced by the mineralogical characteristics of the aggregates. Crushed aggregates from fine-grained diabase and limestone gave the best results. Concretes made from a smooth river gravel and from crushed granite that contained inclusions of a soft mineral were found to be relatively weaker in strength. Their study suggested that the choice of coarse aggregate may be made by examining the stress–strain curves and the loading–unloading hysteresis loops for the high strength concrete mixtures made with the coarse aggregates under consideration.

Chang and Su [1996] found in a recent study that there existed a good correlation between the compressive strength of coarse aggregate and its soundness (weight loss) obtained by the ASTM C 88 test. They investigated four different types of aggregates with significantly different soundess values. Using the aggregates for the mix proportions of high strength concretes, they also found close correlations between the mean compressive strengths of the aggregates and the compressive strength of the concretes, ranging from 35 to 75 MPa (5,000 to 10,700 psi), at both 7 days and 28 days of age. The mean compressive strength of the aggregate is calculated as

$$\sigma_{22} = \frac{1}{V} Ph \qquad (4.1)$$

in which σ_{22} is the mean compressive strength of aggregate, V is the volume of a single aggregate determined using Archimedes's principle after the over dried weight is measured, P is the maximum load applied on a single aggregate, h is the distance between the two opposite load points of P. Obviously, to obtain statistically meaningful results, a large number of aggregates should be tested. In their investigation, Chang and Su [1996] testes seventy specimens for each of three types of aggregates and eight specimens for the fourth type of aggregate (crushed brick).

Leming [1990] also compared mechanical properties of high strength concrete made with four different types of aggregates (crushed shell-limestone, crushed granite, partially crushed gravel, and diabase) available in North Carolina. The 28-day concrete strength ranged from 51 to 81 MPa (7.35 to 11.57 ksi) with W/CM varying from 0.28 to 0.42. He observed that the mechanical properties varied significantly depending on the source and type of the coarse aggregates. The results also indicated that the W/CM ratio alone is not an effective predictor of strength for high strength concrete made with significantly different aggregates and paste composition.

In a study by Lindgard and Smeplass [1993] six aggregate types with different strength and rigidity were tested: dehydrated bauxite, quartzite, quartz-diorite (as reference), gneiss/granite, basalt and limestone. All the aggregates except gneiss/granite were crushed. Fig. 4.1 shows the effect of the aggregate type on compressive strength. The difference between the highest and the lowest strengths is approximately 40%. The authors noted, however, that the bauxite and the basalt aggregates were porous and capable of absorbing significant amounts of mixing water, thus reducing the effective W/CM from 0.30 to 0.24 and 0.27 respectively. If corrections were made for this effect, the compressive strength variation among the five strongest aggregates would be negligible. A similar study [Giaccio et al. 1992] with basalt, granite, and limestone aggregates and a constant W/C of 0.30 plus 2.5% naphthalene-based superplasticizer showed that the three different aggregates produced a compressive strength of 92 MPa (13 ksi), 80 MPa (11.5 ksi), and 62 MPa (8.86 ksi) respectively, using 100 x 200 mm (4 x 8 in.) cylinders.

Alexander and Addis [1992] studied the influence of aggregates and interfacial bond on the mechanical properties of high strength concrete, using different aggregates in South Africa. Based on the 28-day compressive (cube) strength, they showed that andesite and dolomite concretes were generally superior, with dolerite, quartzite and granite concretes following in descending order of strength. By using a special testing method in which cement paste was cast against half-cores from the rock core fracture tests, thereby creating an "artificial" paste/rock interface, they measured the interfacial work of fracture values (Rc) for paste/andesite and paste/dolomite interfaces. It was concluded that certain aggregates are more suitable for the production of high strength concrete because they bond better with cement paste as was the case with andesite.

B. Effect of Aggregate Size The use of larger maximum size of aggregate affects the strength in several ways. First, since larger aggregates have less specific surface area and the aggregate–paste bond strength is less, the compressive strength of concrete is reduced. Secondly, for a given volume of concrete, using larger aggregate results in a smaller volume of paste, thereby providing more restraint to volume changes of the paste. This may induce additional stresses in the paste, creating microcracks prior to application of load, which may be a critical factor in very high strength (VHS) concretes. Therefore, it is the general consensus that smaller size aggregates should be used to produce higher strength concrete.

The effect of the coarse aggregate size on concrete strength was investigated by Cook [1989] who used limestone of two different sizes: 10 mm (3/8 in.) and 25 mm (1 in.). A superplasticizer was used in all the mixes. In general, for a given W/C ratio, the smallest size of the coarse aggregate produced the highest strength; however, it was feasible to produce compressive strengths in excess of 69 MPa (10 ksi) using a 25 mm (1 in.) maximum size aggregate when the mixture was properly proportioned with a high-range water-reducing admixture.

In a similar study by de Larrard and Belloc [1992] using crushed limestone aggregates, portland cement, silica fume, and superplasticizer for eight different concrete mixtures, it was shown that better performances and economy could be achieved with 20 to 25 mm (3/4 to 1 in.) maximum size aggregates even though previous researchers had suggested that 10 to 12 mm is the maximum size of aggregates preferable for making high-strength concrete.

4.1.1.2 Paste Characteristics

It is generally accepted that the most important parameter affecting concrete strength is the W/CM ratio, sometimes referred to as the W/B (binder) ratio. Even though the strength of concrete is dependent largely on the capillary porosity or gel/space ratio, these are not easy quantities to measure or predict. The capillary porosity of a properly compacted concrete is determined by the W/CM ratio and degree of hydration. Most high performance concrete are produced with a W/CM ratio of 0.40 or less. The practical use of very low W/CM ratio concretes has been made possible by use of both conventional and high-range water reducers, which permit production of workable concrete with very low water contents [Fiorato 1989; Zia et al. 1991; Burg and Ost 1992].

A. Effect of Mineral Admixture Fly ash, slag and silica fume have been used widely as supplementary cementitious materials in high performance concrete. Although fly ash is probably the most common mineral admixture, on a volume basis, silica fume (ultra–fine amorphous silica, derived from the production of silicon or ferrosilica alloys) in particular, used in combination with high-range water reducers, has increased achievable strength levels dramatically [Ezeldin et al. 1989; Baalbaki et al. 1993; Zia et al.1993a, 1993b; Farny and Panarese 1994].

The effect of silica fume on the strength of concrete has been discussed in a comprehensive report [FIP 1988]. The contribution of silica fume to any property of hardened concrete may be expressed in terms of an efficiency factor, K. For compressive strength of concrete, K is in the range of 2 to 5, which means that in a given concrete 1 kg of silica fume may replace 2 to 5 kg of cement without impairing the compressive strength. This applies provided that the water content is kept constant and the silica fume dosage is less than about 20% by weight of cement.

Collepardi et al. [1990] studied the effect of combined addition of silica fume and superplasticizer on concrete compressive strength by taking into account such parameters as: (a) type and dosage rate of superplasticizer, (b) type and content of portland cement, and (c) way of silica fume utilization (as additional component or as cement replacement). They concluded that in the presence of silica fume, for both type I and type III portland cement, the melamine sulphonated polymer superplasticizer performs better than the naphthalene sulphonated polymer, particularly when a high dosage such as 4% is used. A change from 2 to 4% superplasticizer dosage rate in general does not modify or reduce compressive strength in the absence of silica fume, whereas significantly increases compressive strength in the present of silica fume.

High-reactivity metakaolin (HRM) is a more recently developed supplementary cementitious material. It is a reactive aluminosilicate pozzolan formed by calcining purified kaolinite at a specific temperature change. Chemically, HRM combines with calcium hydroxide to form calcium silicate and calcium aluminate hydrates. It has been shown that HRM in powder form is a quality-enhancing mineral admixture that exhibits enhanced engineering properties comparable to silica fume slurry [Caldarone et al. 1994]. At the present time, the supply of this material is limited and no practical cost data is available.

B. Effect of Chemical Admixture The performance of chemical admixtures is influenced by the particular cement and other cementitious materials. Combinations which have been shown to be effective in many cases may not work in all situations, due to adverse cement and admixture interaction. Substantial testing should be conducted with any new combination of cements, and mineral or chemical admixtures prior to large scale use.

Baalbaki and Aitcin [1994] conducted a research program to study the compatibility between three air-entraining agents, four water reducers, and one polynaphthalene sulfonate superplasticizer commonly used in Eastern Canada. Tests conducted on twelve differenct combinations of admixtures with a Type I cement showed that the addition of superplasticizer nearly always increased the air content without changing the bubble spacing. The only exception was when the air content of the concrete was lower than 4.5 percent 70 minutes after batching. In that case, the total air content decreased after the introduction of the superplasticizer and the spacing factor increased significantly. The tests were duplicated with another Type I cement and the results were not significantly different from the first set of test results.

4.1.2 Strength Development and Curing Temperature

The strength development with time is a function of the constituent materials and curing techniques. An adequate amount of moisture is necessary to ensure that hydration is sufficient to reduce the porosity to a level necessary to attain the desired strength. Although cement paste in practice will never completely hydrate, the aim of curing is to ensure sufficient hydration. In pastes with lower W/CM ratios, self-desiccation can occur during hydration and thus prevent further hydration unless water is supplied externally.

Many investigations have measured strength development (mostly compressive strength) with respect to time. In the previous state-of-the-art report [Zia et al. 1991], an in-depth discussion of the effects of silica fume, cement types, curing conditions (wet and dry), and curing temperatures on the strength development was presented. In general, a higher rate of strength gain is observed for higher strength concrete at early ages. At later ages the difference is not significant.

A recent study [Zia et al. 1993] on VES, HES, and VHS concretes reported the similar trend as shown in Fig. 4.2. It should be noted that insulation was used to trap the heat of hydration to accelerate the early strength development of the VES concretes. Since the VES concretes were kept moist for only the first 6 or 4 hours, to be followed by air curing in the laboratory, the strength development was very rapid during the first three days, and the subsequent rate of strength gain was greatly reduced. For the HES concretes, a large amount of Type III cement was used along with a fast-acting accelerator and a relatively low W/C ratio, thus the strength development of the concrete was much more rapid in the first 15 days, and then the rate of strength gain was greatly reduced. For the VHS concretes, a large amount of Type I cement plus fly ash or silica fume was used along with a relatively low W/CM ratio, so the strength development was more rapid in the first 7 days and the subsequent rate of strength gain was greatly reduced. These early strength gains are in substantial contrast to the prediction by the current ACI Committee 209 recommendation [1993] for conventional concrete.

A study by Aitcin et al. [1994] examined the effect of curing on measured compressive strength for different strength concretes. Cylinders of 100, 150, and 200 mm (4, 6, and 8 in.) diameter were cast from batches of ready-mixed concrete with target strengths of 35, 90, and 120 MPa (5, 13, and 17.5 ksi). The cylinders were air-cured, sealed, or water-cured up to one year. The beneficial effects of preventing moisture loss and water-curing were clearly observed from the test results. The increased apparent strengths with decreasing cylinder size were also noted. Based on the test results, the following equation was developed which is a modified form of the equation recommended by ACI Committee 209 [1993]:

$$f_c'(t) = A\frac{t}{t+B}$$

in which is the concrete compressive strength at an age of t days, A is the concrete compressive strength at an age of infinity, and B is a coefficient (in days) that determines the shape of the strength gain curve. The values of A and B are given in Table 4.1.

Oluokun et al. [1990] reviewed the maturity concept with respect to the prediction of early-age concrete strength and pointed out that the linear relationship between the strength development and maturity is inaccurate for concrete at very early age or at much later age. By conducting an experimental program involving three different concrete mixes and measuring the compressive strengths at ages of 6 and 12 hr, and 1, 2, 3, 7, and 28 days, they developed an exponential relationship between the early-age strength and maturity of concrete as follows:

$$f_{cx} = f_c'(1-e^{-rm})$$

in
which

f_{cx} = concrete strength in psi at a given maturity M in deg F-hr

f_c = 28-day compressive strength

m = M/10,000

Y = a constant as shown in the table below

Strength Range, psi	2500-5000	5001-7500	7501-10000
Y	1.028	1.457	2.826

Using pulse velocity method, Ravindrarajah [1992] investigated the development of compressive strength of high-strength concrete from the age of 5 hours with respect to curing conditions and types of cementitious material. Normal portland cement, blast-furnace slag cement and silica fume were used either individually or in combination. Test results indicated that silica fume in concrete increased the pulse velocity; both pulse velocity and strength were lower for air-cured concrete than for water-cured concrete; and the relationship between strength and pulse velocity could be expressed as an exponential curve which was less influence by curing or cementitious material type. Evaluation of in-situ concrete strength based on paste efficiency concept was examined.

Kawakami et al. [1989] reported a study on the rapid estimation of the 28-day compressive strength of concrete by 30-minutes curing of the wet-screened mortar in boiling water. It was demonstrated that the simple and rapid method is a possible way for estimating the 28-day compressive strength of concrete before the concrete is placed.

Several studies [Russell and Larson 1989; Aitcin and Laplante 1990; Bickley et al. 1991] have reported the long-term compressive strength of high strength concretes containing silica fume. Structures constructed of the concrete were monitored under the field conditions and cores were taken from the structures for testing one year to six years after the structures were constructed. These studies dispelled the suspicion of possible strength regression of some silica fume concretes.

The development of pavement concrete compressive strength, flexural strength, and splitting tensile strength as a function of time was reported by Lange [1994]. Significant strength gains beyond the 28-day strength were found. It is true for high- and normal-strength concretes, concretes with and without fly ash, and concretes cured in air and moist conditions. Although modern concretes have higher early compressive strength gains than concretes produced in 1940-56, both categories of concrete exhibit similar long-term trends in strength development when normalized to their 28-day strengths.

4.1.3 Compressive Strength

Concrete properties such as elastic modulus, tensile or flexural strength, shear strength, stress–strain relationships and bond strength are usually expressed in terms of uniaxial compressive strength of 150 x 300 mm (6 x 12 in.) cylinders, moist cured to 28 days. Compressive strength is the common basis for design for most structures, other than pavements, and even then is the common method of routine quality testing. The terms "strength" and "compressive strength" are used virtually interchangeably. The discussion above in sections 4.1.1 and 4.1.2 generally applies equally well to all measures of strength, although most results and conclusions were based either primarily or exclusively on compressive strength results.

Maximum, practically achievable, compressive strengths have increased steadily over the years. Presently, 28-day strengths of up to 84 MPa (12 ksi) are routinely obtainable. However, it has been reported [CEB-FIP 1994] that concrete with a 90-day cylinder strength of 131 MPa (19 ksi) has been used in buildings in the U.S., 143 MPa 20.5 ksi) cube strength for precast prestressed beams in Sweden, 131 MPa (19 ksi) cube strength for a slab on ground in Sweden, 110 MPa (15.7 ksi) cube strength for an overlay on bridge in Norway, 95 MPa (13.6 ksi) cube strength for highway pavement in Norway, and 100 MPa (14.3 ksi) for a pedestrain bridge in Japan. The trend for the future as identified by the ACI Committee 363 is to develop concrete with compressive strength in excess of 138 MPa (20 ksi) and identify its appropriate applications.

Testing variables have a considerable influence on the measured compressive strength. The major testing variables are: mold type, specimen size, end conditions, and rate of loading. The sensitivity of measured compressive strength to testing variables varies with level of compressive strength.

Since the compressive strength of VES and HES concretes are at conventional levels, conventional testing procedures can be used for the most part, although curing during the first several hours can affect test results dramatically. Testing of VHS concretes is much more demanding. However, in all concretes, competent testing is critical especially for high performance concrete.

4.1.3.1 Effect of Testing Variables

An extensive study to investigate the effects of testing variables on the measured strength of high-strength concrete cylinders was conducted by Carino et al. [1994]. The variables included: end preparation (sulfur capping versus grinding), cylinder size (100 versus 150 mm diameter), type of testing machine (1.33-MN capacity versus 4.45-MN capacity), and nominal stress rate (0.14 versus 0.34 MPa/s). Two levels of strength (45 and 90 MPa) were used, and three replicates were tested for each run. Specific gravities were measured to check on the consistency of cylinder fabrication. Statistical analyses indicated that all the factors had significant effects on the measured compressive strength. On average, the 100 mm cylinders resulted in about 1.3% greater strength, the faster stress rate produced about 2.6% greater strength, the ground cylinders were 2.1% stronger, and the 1.33-MN testing machine produced about 2.3% greater strength. There were significant interactions among the factors, so that the effects were greater than the average values for particular factor setting. For example, the effect of end preparation depended on the strength level. For 45-MPa concrete, there was no strength difference due to the method of end preparation, but for the 90-MPa concrete, grinding resulted in as much as 6% greater strength in certain cases. Analysis of dispersion indicated that the 100-mm cylinders had higher within-tests variability, but the differences were not statistically significant. Based on the results of this study, recommendations were made for modifications to testing standards with respect to the loading rate, the capping method, the test cylinder size, and the required time for removal of molds.

As a part of the Japanese 5-year R & D project on "the development of engineering technique for the construction of ultra-light and ultra-high-strength reinforced concrete buildings", Tanigawa et al. [1990] also conducted a study on testing method for compressive strength of high-strength concrete. Testing variables included three different concrete strengths: 58.8 MPa (8.5 ksi), 83.4 MPa (12.1 ksi), 107.9 MPa (15.6 ksi) at 28 days; eight end surface treatments (including ground surface); three different rigidities of testing machine (20 and 28 tf/mm) with a tilting platen (free versus fixed); four loading speeds (2~3 kgf/cm2/s, 1~1.5 kgf/cm2/s, 0.5~0.75 kgf/cm2/s, and 30~40 me/s); and two cylinder sizes (100 and 150 mm diameters). The test results indicated that the different treatments of the loading surface did not affect the compressive strength except for the case with friction reducing pad where a strength reduction of 9%

was observed. The compressive strength also tended to decrease as the flatness of the end surface decreased. No significant effect on the measured compressive strength was observed with the difference in the rigidity of the testing machine. The effect on the compressive strength was also negligible if the loading platen was free to tilt throughout the test or allowed to tilt initially but fixed when the load was at 44 kips (20 tf). On the other hand, the compressive strength is greatly reduced if the loading platen was fixed at the initial loading. The effect of loading speed on the compressive strength was greater than that of normal strength concrete. Under the slowest load speed, the strength decreased by about 6 to 9%. The effect of the specimen size was the same as that of normal strength concrete. On average, cylinders of 100 x 200 mm showed about 3% higher strength than cylinders of 150 x 300 mm.

4.1.3.2 Effect of Mold Type

The effect of mold type on strength was reported by Carrasquillo et al. [1988a] that use of 150 x 300 mm (6 x 12 in.) plastic molds gave strengths slightly lower than steel molds and use of 100 x 200 mm (4 x 8 in.) plastic molds gave negligible difference with steel molds. A recent report by French and Mokhtarzadeh [1993] also indicated that the compressive strength of cyliners cast in 150 x 300 mm (6 x 12 in.) heavygauge reusable steel molds was 2.5% higher than that of cylinders cast in flexible single-use plastic molds. It appeared that as long as the manual rodding method was used to consolidate the concrete, the effect of mold type on the compressive strength of concrete was insignificant. Carrasquillo et al. recommended that steel molds should be used for concrete with compressive strengths up to 103 MPa (15 ksi). For higher strength concrete, it seems logical that steel molds should also be used.

4.1.3.3 Effect of Specimen Size

Many studies [Tanigawa et al. 1990; Baalbaki et al. 1992; French et al. 1993; Leming 1993; Aitcin et al. 1994; Carino et al. 1994] have been conducted to investigate the specimen size effect on the compressive strength. Comparisons were usually made between the compressive strength of 100 x 200 mm (4 x 8 in.) cylinders and that of 150 x 300 mm (6 x 12 in.) cylinders. Generally, 100 x 200 mm (4 x 8 in.) cylinders exhibit higher strengths than 150 x 300 mm (6 x 12 in.) cylinders. The difference may vary from 2 to 10% with a common value being 5%, and the difference is lower for higher strength concrete. Burg and Ost [1992] reported, however, that their test data showed that the strength of 100 x 200 mm (4 x 8 in.) cylinders was within 1% of the strength of 150 x 300 mm (6 x 12 in.) cylinders. A contradition to this trend is the study reported by Carrasquillo et al. [1988a] which showed that the compressive strength of 100 x 200 mm (4 x 8 in.) cylinders were approximately 7% lower than 150 x 300 mm (6 x 12 in.) cylinders.

4.1.3.4 Strength of Concrete Core

The relationship between the compressive strength of 150 x 300 mm (6 x 12 in.) cylinders and cores from a column was studied by Cook [1989] for concrete with a strength of 69 MPa (10 ksi). It was concluded that the 85% criterion specified in the ACI Building Code (ACI 318-89) [1989] would be applicable to high strength concrete. The study also confirmed that job cured specimens did not provide accurate measurements of the in–place strength. The reason for lower core strength in the middle portion of the columns was attributed to temperature rise, i.e, 100oF (38oC) for high strength mixtures. In another study, Akers and Miller [1990] evaluated the relationship between 150 x 300 mm (6 x 12 in.) cylinders, 100 x 200 mm (4 x 8 in.) cylinders and drilled cores. The results showed that the strengths obtained from drilled cores were greatly influenced by three factors: their tested orientation relative to that in the structure; the elevation of the core in the structure; and the type of pre-test conditioning. A comparison of the core and cylinder compressive strengths indicated that the acceptance criteria of the ACI Building Code may have limited applicability at the higher strength levels. It was suggested that prior to core testing high strength concrete, the testing conditioning and acceptance criteria should be agreed upon in advance and be rigorously followed. Aitcin et al. [1990] also tested the strength of 100 mm (4 in.) cores taken from a mock column at two and four years after casting and found that it was nearly identical to that of cube specimens cured for 28 days in lime–saturated water at room temperature. The strength of the concrete tested was 85 MPa (12.3 ksi).

In an evaluation of engineering properties of six commercially available high-strength concrete mixes in the range of 69 to 138 MPa (10 to 20 ksi), Burg and Ost [1992] reported that the core strength tested at 91 days and 14 months was slightly lower than the strength of corresponding insulated cylinders and all but one concrete mix exceeded 85% of the specified design strength fc' of the concrete. They further reported that no significant strength difference was found between cores taken from near the surface and the center of large-sized cubes. This is in contrast to the findings of Cook [1989].

Recently, the compressive strength of concrete cores was extensively studied by Bartlett and MacGregor [1994a, 1994b, 1994c, 1994d]. By examining and statistically analyzing hundreds of core test data, they found that the core strengths were greatly affected by the variations of four critical factors: (1) the core size, 50 or 100 mm (2 or 4 in.) diameter; (2) the length-to-diameter ratio of the core (l/d); (3) the moisture condition (moisture content and moisture gradient) of the core; and (4) the damage sustained during drilling of the core.

4.1.3.5 Effect of End Condition

The preparation of the end conditions (cappings) of the concrete cylinder can significantly affect the measured compressive strength. Generally speaking, the standard sulfur mortar capping is suitable for concrete strength up to about 52 MPa (7.5 ksi). For higher strength concrete, different procedures are used to prepare the end conditions of cylinders for compressive testing. One procedure is the parallel grinding of the ends of the cylinder, thereby eliminating the need for end caps. While grinding is regarded as the best procedure, it entails expensive equipment and longer preparation time so that it is not practical for field applications. Another procedure is the use of an unbonded cap consisting of a restraining cap and an elastomeric pad as insert. The unbonded cap system is far more cost-effective and can be easily equipped by any laboratory and used in the field. A previous study [Carasquillo et al. 1988b] showed that for concrete strengths between 28 and 69 MPa (4 and 10 ksi), the use of polyurethane inserts with aluminum restraining caps produced average test results within 5% of those obtained using sulfur mortar caps. For concrete strengths below 76 MPa (11 ksi), the use of neoprene inserts with steel restraining caps yielded average test results within 3% of those obtained using sulfur mortar caps. For higher strength concrete, the use of either unbonded capping system became questionable.

More recently, Pistilli and Willems [1993] compared sulfur caps with unbonded polymer pads in compressive strength testing of concrete within the strength range of 20.7 to 124.2 MPa (3 to 18 ksi) and compared sulfur caps with ground and lapped surfaces within the range of 89.7 to 138 MPa (13 to 20 ksi). Similar tests were conducted by Ipatti [1993] comparing untreated mold surface, sulfur capping, and ground surface for cube specimens, and sawn surface, sulfur capping, and ground surface for cylindrical specimens. Sawing of the cylindrical specimens was accomplished with a water cooled cutting machine with a diamond saw blade of 625 mm diameter, capable of cutting cylinders up to 250 mm in diameter. The capping compound consisted of sulfur, quartz filler, and chalk powder in 1.25 : 0.25 : 0.75 ratio. The compressive strength of the sulfur compound, based on 40 mm cube, was 44 MPa (6.38 ksi). Four grades of concrete were used in the tests including 60, 80, 90, 110 MPa (8.7, 11.6, 13.1, 16 ksi). The test results indicated that for the cylindrical secimens, the highest average strengths were obtained with ground or sulfur capped surfaces. The coefficient of variation averaged 1% for ground surfaces, 1.5% for sulfur capped surfaces and 7.9% for sawn surfaces. For the cube specimens, the highest average strength were obtained with ground surfaces, the second highest with untreated mold surfaces, and the lowest with sulfur capped surfaces. The corresponding coefficients of variation were 0.8%, 1.9%, and 1.3% respectively.

A new technique for the unbonded cap system has been developed recently in France [Boulay et al. 1992; Boulay and de Larrard 1993] in which the neoprene insert is replaced by dry sand (the sand box). The strength of the confined sand seems to have no limit and when used as the capping system, the results it produces are comparable to those obtained with grinding for concrete strength between 50 to 80 MPa (7.25 to 11.6 ksi).

Lessard et al. [1993] has found a high strength capping compound (testing 60 to 70 MPa using 50-mm cubes) to be effective when used for testing high strength concrete up to 120 MPa due to the confinement of the capping compound between the platen of the test machine and the speciment, but the capping layer has to be less than 3 mm thick. Under such conditions, the results are similar to those obtained when the cylinder ends are faced by grinding.

Another recent method developed by Johnson and Mirza [1993] is to provide a confining ring to the standard sulfur mortar capping. The method employs standard concrete laboratory equipment and an inexpensive customized capping apparatus for preparing the cylinder ends, see Figs. 4.3 and 4.4. The technique ensures that confinement is provided to the cap without having to place tight controls on cylinder end roughness prior to capping and on the cap thickness itself. Using a capping compound with the manufacturer's specified ultimate compressive strength of 35 MPa after 5 minutes and 55 MPa after 48 hours (based on 50-mm cubes), they tested cylinders (one hour after capping) with compressive strength of over 100 MPa. The results compared closely to those with ground ends, see Fig. 4.5.

4.1.3.6 Effect of Loading Rate

It is generally understood that the measured compressive strength of concrete increases with increasing rate of loading. Many studies of the subject have been conducted over the past seven decades, covering a wide range of strength of concrete (17.4 to 60.4 MPa), strain rate (10-6/s to 10/s), specimen size and shape (cylinder, cube, and prism), curing and testing conditions (wet and dry), and loading mechanism (electrohydraulic servosystem, pressure-activated piston system, ballistic pendulum and drop hammer apparatus).

An excellent review of the literature has been prepared by Fu et al. [1991] and their general conclusions are as follows:

1. Both compressive strength and stiffness increase with increasing strain rates.
2. Increasing the rate of strain has not resulted in consistent increase or decrease in ultimate strain and strain at maximum stress. The degree of change depends on the constitutive model used as much as on the strain rate itself.
3. Higher strain rates appear to have a more profound effect on low to moderate strength concrete than on high-strength concrete.
4. Wet concrete is relatively more sensitive to a change in loading rate than dry concrete.
5. The failure of conrete at very high strain rates can be explosive.
6. Slope of the descending branch in the stress–strain diagram increases with increasing rate of straining.

The review by Fu et al. also presented four different dynamic constitutive models to represent a complete stress–strain curve for plain and reinforced concrete under compression. It should be noted that the proposed models have been based on axial compression and the effect of strain gradient has not been accounted for. Scott et al. [1982] had pointed out that strain gradient due to eccentric compression may lead to an increase in the strain at failure and a decrease in strength.

Virtually no information is available on the effect of loading rate for concrete with strengths in excess of 69 MPa (10 ksi).

4.1.3.7 Effect of Temperature

The strength properties of concrete are not affected by normal changes of temperature. Under extreme temperature conditions, however, the behavior of concrete may be substantially different. Castillo and Durrani [1990] reported a study on the effect of transient high temperature on the strength and load-deformation behavior of high-strength concrete. The concrete strength ranged from 31.1 to 89 MPa (4.5

to 12.9 ksi) and the temperature exposure was in the range of 23 to 800 C (73 to 1,472 F). Before exposure to elevated temperature, the test specimens were preloaded to simulate the presence of loads in real structures. It was observed that the compressive strength of high-strength concrete decreased by 15 to 20% when exposed to temperatures in the range of 100 to 300 C (212 to 572 F) With temperatures in the range of 300 to 800 C (752 to 1,472 F), the compressive strengh of concrete decreased to about 30% of its strength at room temperature.

The effect of exposure to low temperature on the mechanical properties of high-strength concrete containing silica fume and fly ash was reported by Marzouk and Hussein [1990]. Test cylinders were exposed to cold ocean water with temperature varying from -10 to 20 C (14 to 68 F) after the cylinders being cured for only 24 hours. For the specimens exposed to cold ocean water, the strength at 28 and 91 days was only 40 and 54% respectively of the 3-day strength at room temperature. However, for the specimens exposed to temperatures of -10 C, the strength at 28 and 91 days was 97 and 91% respectively of the 3-day strength at room temperature. The low rate of maturity was attributed to slow hydration process due to low temperature, which caused the rate of evolution of calcium hydroxide to decrease and the secondary pozzolanic reaction to stop.

Lee et al. [1989] also conducted studies of the basic mechanical properties of concrete under low temperature in the range of -70 to 20 C (-94 to 68 F). Their results indicated that the compressive strength increased as the temperature decreased and the rate of increase for high-strength concrete at different low temperatures was generally lower than that for normal strength concrete.

4.1.4 Tensile Strength

The tensile strength governs the cracking behavior and affects other properties such as stiffness, damping action, bond to embedded steel, and durability of concrete. It is also of importance with regard to the behavior of concrete under shear loads. The tensile strength is determined either by direct tensile tests or by indirect tensile tests such as flexural or split cylinder tests.

4.1.4.1 Direct Tensile Strength

The direct tensile strength is difficult to obtain. Due to the difficulty in testing, only limited and often conflicting data is available. It is often assumed that direct tensile strength of concrete is about 10% of its compressive strength.

An account of several earlier studies using different specimen sizes and geometries, examining the effects of curing, loading rate, sustained and cyclic loadings as well as impact, has been given in the previous report [Zia et al. 1991]. It was concluded that the uniaxial tensile strength of concrete can be estimated by the expression 6.5f_c' and that data were not available for higher strength concrete with f_c' greater than 55 MPa (8 ksi).

Among more recent studies on tensile strength of high strength concrete, very few involved direct tensile testing. One such study was reported by Marzouk and Jiang [1994] regarding the effects of freezing and thawing on the tension properties of high-strength concrete. Flat direct tension speciments of 20 x 75 x 300 mm (0.8 x 3 x 12 in.), with sawed notches (11 mm in depth and 3 mm in width) on both edges, were attached to a pair of special wedge-type frictional grips and subjected to direct tension test in a closed-loop servo-hydraulic universal test machine. An electromechanical extensometer (gage length of 25 mm) was used to control the loading. The concrete contained both silica fume and fly ash with W/CM of 0.3 and was cured in water for 28 days. The 28-day compressive strength of the concrete was 74.4 MPa (10.8 ksi). Each freezing and thawing cycle consisted of alternately lowering the temperature of the specimen from 18.3 to -17.8 C and raisingit from -17.8 to 18.3 C in 3.4 hr. The direct tensile strength was found to be 4.2 and 3.4 percent of the compressive strength before freezing-thawing cycling and after 700 cycles, respectively. The average value of cracking stain was found to be 115 me before cycling and 65 me after 700 cycles of freezing and thawing. A set of typical tensile stress–strain curves is shown in Fig. 4.6.

4.1.4.2 Indirect Tensile Strength

The most commonly used tests for estimating the indirect tensile strength of concrete are the splitting tension test (ASTM C 496) and the third-point flexural loading test (ASTM C 78). Both the splitting tensile strength (fct) and the flexural strength or modulus of rupture (fr) are related to the compressive strength by the following general expression:

$$f_{ct} \quad \text{or} \quad f_r = k\sqrt{f_c'}$$

ACI Committee 363 [1993] recommended that for concrete strength up to 83 MPa (12 ksi), the coefficient should be taken as 7.4 for and as 11.7 for . Other investigators have proposed slightly different values for , or suggested variations to Eq. (4.4). More details can be found in the previous report [Zia et al. 1991].

The results of the recent SHRP studies [Zia et al. 1993] indicated that for the splitting tensile strength the recommendation of ACI Committee 318 is equally acceptable as that of ACI Committee 363. However, for the flexural strength (modulus of rupture), the recommendation of ACI 318 is a better representation than that of ACI Committee 363. See Figs. 4.7 and 4.8.

In a two-year study to obtain "optimum" high performance concrete based on cement dosage, dosage of superplasticizer, addition of silica fume, and selection of size and shape of aggregates, Charif et al. [1990] found that the tensile strength could be increased by 40 to 60% over the normal strength concrete. Regarding the effect of freezing and thawing, Marzouk and Jiang [1994] reported that after 700 cycles, the modulus of rupture of high strength concrete was reduced by 15% whereas the reduction was 60% for normal strength concrete.

The study by Burg and Ost [1992] showed that the average modulus of rupture and splitting tensile strength in comparison with the compressive strength were similar to the recommendation of ACI Committee 363. The moist cured specimens consistently produced higher strength than air cured specimens. See Figs. 4.9 and 4.10.

4.1.5 Bond

Two types of bond strength are of interest in pavement and bridge deck applications: bond strength of concrete to concrete and bond strength of concrete to reinforcing steel.

4.1.5.1 Concrete-Concrete Bond

New concrete is placed against existing concrete in many circumstances. Often an attempt is made to bond the two concretes together. The stress states that develop at the bonding surface will vary considerably depending on the type and the use of the structure. For example, the bond on a bridge deck overlay may be subject to shear stress in conjunction with tensile or compressive stresses induced by shrinkage or thermal effects, in addition to compression and shear from service loads.

Bonding agents are often used with the intention of producing a bond that is as strong as the components being joined. A wide variety of bonding agents have been used in practice, including epoxy resin, acrylic latex, styrene butadiene rubber (SBR) latex, copolymer polyvinyl acetate (PVA), and portland cement mortar. The latter two are used widely as inexpensive general purpose agents for bonding concretes.

No single method of test can replicate all in-service state of stresses in bond. Several earlier research studies describing test methods and evaluating bonding materials were summarized previously [Zia et al. 1991]. Generally, there are three different methods to measure the interfacial bond strength between two concrete surfaces being joined [Hindo 1990]. They include split shear (slant shear) test, direct shear test, and direct tensile test as shown in Fig. 4.11. In the slant shear test, the bonding plane is inclined at 30 degrees from the longitudinal or loading axis and it is subjected to normal compression and in-plane shear. The test specimen may be in the form of a cylinder or a rectangular prism. If failure occurs by shearing of the bonding plane, the bond strength is then determined by dividing the maximum load by the area of the shearing plane. For the direct shear test, there are three altenatives — push-off test using a specimen of two L-shaped parts, double direct shear test, and double notched shear test as illustrated in Fig. 4.12 [Horiguchi et al. 1988]. In all these three cases, the bond strength is determined by dividing the maximum shear load by the area of the shear plane. In the direct tensile test, the bond strength of the joint is actually measured by the tensile strength of the test specimen. The direct tensile test has been adapted to field application where a partially cored specimen is pulled in-place through a steel plate epoxied to the top of the core [Hindo 1990; Petersen et al. 1993].

In the SHRP study on VES and HES concretes, Zia et al. [1993c, 1993d] used push-off tests to assess the bond strength between VES(A), VES(B), HES and the standard North Carolina Department of Transportation (NCDOT) pavement concrete. They found that the VES(A) concrete with crushed granite developed a nominal bond strength at 6 hours ranging from 0.83 MPa (120 psi) to 1.03 MPa (150 psi). The VES(B) concrete with crushed granite developed a 4-hour nominal bond strength of 1.55 MPa (225 psi). These values were much lower than the corresponding value of 2.28 MPa (330 psi) obtained from the control specimen tested at 7 days using the standard NCDOT pavement concrete. The differences reflect primarily the effect of test age. For the HES concretes tested at 24 hours, the nominal bond strength was 1.9 MPa (275 psi) with crushed granite as aggregate and was 2.4 MPa (350 psi) with marine marl as aggregate. These values compared well with the bond strength of 2.3 MPa (330 psi) of the control specimen tested a 7 days.

Kudlapur and Nawy [1990] also used push-off tests to evaluate the early-age bond strength between high-strength regular portland cement concrete and cold weather high-strength concretes for the rehabilitation of bridge decks. One half of the push-off specimen was made of a regular high-strength concrete which, after 28 days of moist curing, produced compressive strengths of 83 MPa (12 ksi) to 100 MPa (14.5 ksi). This part of the test specimen was cured for 30 days before the other half of the push-off specimen was cast with magnesium phosphate concrete or methylmethacrylate (MMA) polymer concrete in the cold room at a temperature of 15o to 20o F (-9.4o to -6.7o C). The push-off tests were conducted at 1, 3, and 7 days after the cold weather concrete was cast. The test results indicated that the magnesium phosphate concrete developed a bond strength of 1.71 MPa (248 psi) in one day and increased to 1.92 MPa (278 psi) in 7 days. For the MMA polymer concrete, the bond strength was 6.14 MPa (890 psi) in one day and increased significantly to 7 MPa (1,015 psi) in 7 days.

Using slant shear tests, Wakeley et at. [1991] evaluated the bonding characteristics of three commercially available rapid-setting materials as spall-repair concretes for rapid runway repair. The three materials were a methyl methacrylate binder (Silikal R17AF), a magnesium phosphate mortar mix (Set-45), and a high-performance blended cement mortar mix (Pyrament 505). Each was extended 50 percent by mass with coarse aggregate for the tests. The specimens used for the slant shear tests were 100 x 200 mm (4 x 8 in.) cylinders rather than the 75 x 150 mm (3 x 6 in.) cylinders as specified in the ASTM C 882-87 test method. The bond strength was calculated by dividing the load carried by the specimen at failure by the area of the bonding surface which was 162 cm2 (25.13 in.2). The substrate concrete had a 28-day compressive strength of 41 MPa (5.9 ksi) and the bonding area was sandblasted lightly before the repair concrete was cast. The tests were conducted 24 hours after the repair concrete was cast and at two temperature levels — a warm temperature of 22.8oC (73oF) for all three materials and a cold temperature of -15oC (5oF) for Silikal, -9.4oC (15oF) for Pyrament, and 7.2oC (45oF) for Set-45. The three different cold temperatures were used because of the difference in strength gain of the three materials. Furthermore, both dry and wet aggregates were used for the tests at warm temperature. The test results are summarized in Table 4.2.

4.1.5.2 Steel-Concrete Bond

Methods recommended by the current ACI Building Code (ACI 318 – 89) for estimating the development length and anchorage of tensile steel are based on bond tests generally using concrete with compressive strength not greater than about 28 MPa (4,000 psi). It is uncertain that these empirical equations for estimating the steel–concrete bond are applicable for higher strength concrete, and research on bond strength characteristics of higher strength concrete has been identified as one of the research needs by the ACI Committee 363 [1987].

Pull-out test is commonly used for the evaluation of bond strength between steel and concrete. However, this test does not represent the stress conditions which exist in the concrete around the reinforcement in a flexural member. To overcome this shortcoming, a flexural test is often used in which both the steel and the concrete are in tension.

Several earlier researches [Zia et al. 1991] showed that higher rate of loading would cause more rapid deterioration of anchorage bond and that the bond characteristics of deformed bars were affected significantly by the age of concrete. It was also concluded that the use of superplasticizer did not seem to affect the bond strength and the increased addition of silica fume up to 16% showed an improved effect on the pull-out strength, especially in the high compresssive strength range of concrete, because it "densifies" the interfacial zone between the steel and the surrounding concrete [Gjorv et al. 1990].

Since 1990, several studies have been conducted to investigate specifically the bond strength of reinforcement in high strength concrete. de Larrard et al. [1993] evaluated the bond strengh between high strength concrete and reinforcing bars using the RILEM beam test. A high strength concrete with 28-day compressive strength of 95 MPa (13.6 ksi) was used along with a normal strength concrete of 42 MPa (6 ksi) as control. Three different sizes of deformed bars (10, 16, 25 mm) and one smooth bar (25 mm) were used. Based on several preliminary tests, the RILEM recommended bond (anchorage) length of 10 times bar diameter had to be reduced to 3 times to 2.5 times bar diameter for high strength concrete to ensure bond failure rather than yielding of reinforcement. Transverse reinforcement was used in the test specimens so the lateral confinement of concrete was provided. The average bond strength along the bond length was calculated corresponding to the free end slip of the bar at 10 and 100 mm. The test results are shown in Table 4.3. It was concluded that the effect of bar size on bond strength was very significant as one would expect. The increase in bond strength with high strength concrete (as compared to normal strength concrete) was approximately 80% for 10 mm deformed bars and 30% for 25 mm deformed bars. The improvement of bond is attributed to the increase in concrete tensile strength and confinement due to both concrete shrinkage and transverse reinforcement.

Azizinamini et al. [1993] and Azizinamini [1992] reported a study on tension splice of #11 and #8 bars embedded in high strength concrete. Nominal concrete strength varied from 45 MPa (5 ksi) to 105 MPa (15 ksi). Different splice lengths were used for two-bar and three-bar specimens. The concrete cover was equal to one bar diameter. The tension splice was tested in the constant moment region. The test results indicated that the nomalized bond strength utest/Öfc' decreased as concrete strength increased and that the rate of decrease increased as splice length increased. With constant concrete strength, the nomalized bond strength also decreased with increasing splice length. Furthermore, top-cast bars in normal strength concrete showed approximately 8% reduction in bond capacity compared to bottom-cast bars, whereas top-cast bars in high strength concrete showed slightly higher bond capacity in comparison with bottom-cast bars. It was concluded that over the splice region the bond stress distribution at the ultimate stage might not be uniform for high strength concrete and the nonuniform bond stress distribution could be more pronounced for increased splice length or decreased concrete cover. With high strength concrete and small concrete cover, it is not an efficient approach to increase bond capacity by increasing the splice length. Instead, the use of some minimum amount of transverse reinforcement over the spliced region would be a better approach to prevent splitting of concrete cover so as to increase the bond capacity and to provide for more ductility.

Kaku et al. [1992] tested 26 simply supported beams to investigate bond splitting strength of tensile reinforcement in a shear span. Test variables included concrete strength (40, 60, 80, 100 MPa), development length and spacing of reinforcement, amount and detail of transverse reinforcement, and two cross sections of test specimens. The test results indicated that (1) the bond splitting strength is proportional to Öfc' or, more conservatively, (fc')0.6, (2) the use of transverse reinforcement with supplementary ties significantly increases the bond splitting strength, (3) without transverse reinforcement, bond strength decreases with increase in development length, and (4) bond strength ratio of top bars to bottom bars increases to unity with increase of concrete strength. Based on the test results, a bond strength equation was developed which acounts for the concrete strength, the development length and spacing of reinforcement, and the amount and detail of transverse reinforcedment. The proposed equation is slightly more conservative than the recommendation of the Architectural Institute of Japan.

Using pull-out tests of 62 specimens, Teng and Ye [1992] investigated the bond-slip behavior of deformed bars in high strength concrete. Compressive strength of concrete varied from 60 MPa to 100 MPa based on 150 x150 mm cubes. Bar diameter, embedment length, and confining reinforcement were other variables considered. The test results indicated that with increasing concrete strength, both the bond strength and the stiffness of the bond-slip relationship increased. For high strength concrete, the splitting failure of specimens without spirals was more brittle. For a given embedment length, the bond strength was proportional to the tensile splitting strength of concrete and to square roots of relative concrete cover. With increasing embedment length, the bond strength decreased.

In a recent statistical analyses of 133 splice and development specimens with no transverse reinforcement and 166 specimens with transverse reinforcement to develop design criteria for development length of conventional and high relative rib area reinforceing bars (with fc' varying from 2,500 to 16,000 psi or 17 to 110 MPa), Darwin et al. [1996] found that the effect of concrete strength on bond strength was more accurately represented by (fc')1/4 than (fc')1/2 for the full range of concrete strength.

Steel-concrete bond in high-strength lightweight aggregate (LWA) concrete was reported by Mor [1992] based on pull-out tests. By varying silica fume contents (0 and 13 to 15%) and W/CM (0.25 to 0.34), four different concrete mixes were produced with similar high strength of 69 MPa (10 ksi). The use of silica fume doubled the ratio of bond strength to compressive strength at 0.25 mm (0.01 in.) slip for LWA concrete, while having no significant effect on normal weight aggregate (NWA) concrete. Silica fume reduced porosity and thickness of the transition zone adjacent to the steel, thus improving the adhesion-type bond at small slip levels. The lower Ec of LWA concrete, combined with its compatible aggregate and cement paste matrix, results in better utilization of the bond adhesion, allowing larger stress and strain levels.

The bond characteristics along a high-strength beam reinforcement within an interior beam-column joint panel under monotonic loading was studied by Kitayama et al. [1991]. They found that the bond strength reached the maximum value for the joint constructed with high-strength concrete and steel until a diagonal shear crack occurred across the beam reinforcement. On the contrary, with lower strength concrete and steel, the bond deterioration of the joint panel was caused by yielding of the beam reinforcement. A study of anchorage of beam reinforcement within a typical high-strength concrete interior beam-column joint under load reversals was conducted by Lee et al. [1991]. Based on their results, it was concluded that the design criterion of bond performance recommended by the Architectural Institute of Japan (AIJ) can not be applied to the high-strength reinforced concrete.

The bond characteristics of prestressing strands in both normal strength and high strength concretes is an important issue relative to prestressed concrete design. Mitchell et al. [1993] investigated the influence of high-strength concrete on the transfer and development lengths of pretensioning strands, using concrete strengths varying from 31 to 89 MPa (4.5 to 12.9 ksi) and three different strand diameters of 3/8, 1/2 and 0.62 in. (9.5, 12.7 and 15.7 mm). Expressions for transfer length and development length were proposed to account for the influence of concrete strength, a concept also suggested previously by Zia and Mostafa [1977]. It should be mentioned that a major research program on transfer and development lengths of prestressing strands has been underway for some time at the Turner and Fairbank Laboratory of the Federal Highway Administration and the final report is expected in 1996.

4.2 Deformation

The deformation of concrete depends on short-term properties such as the static and dynamic modulus, as well as strain capacity. It is also affected by time dependent properties such as shrinkage and creep.

4.2.1 Static and Dynamic Elastic Modulus

The modulus of elasticity is generally related to the compressive strength of concrete. This relationship depends on the aggregate type, the mix proportions, curing conditions, rate of loading and method of measurement. More information is available on the static modulus than on the dynamic modulus since the measurement of elastic modulus can be routinely performed whereas the measurement of dynamic modulus is relatively more complex.

4.2.1.1 Static Modulus

It is generally agreed that the elastic modulus of concrete increases with its compressive strength. The modulus is greatly affected by the properties of the coarse aggregate, the larger the amount of coarse aggregate with a high elastic modulus, the higher would be the modulus of elasticity of concrete. The modulus also increases with concrete age. Regardless of the mix proportions or curing age, concrete specimens tested in wet conditions show about 15% higher elastic modulus than tested in dry conditions. This is attributed to the effect of drying of transition zone between the aggregate and the paste. As strain rate is increased, the measured modulus of elasticity increases.

Much more research data have been presented in the past few years and they generally confirm the above fundamental understandings. The effect of aggregate type was considered by Baalbaki et al. [1991], Giaccio et al. [1992], Nilsen and Aitcin [1992]; the effect of curing conditions by Asselanis et al. [1989] and Cabrera and Claisse [1991];and the size effect of test specimens by Baalbaki et al. [1992]. The effect of cold temperature on the elastic modulus was investigated by Lee et al. [1989] and their results indicated that the elastic modulus increased as the concrete is subjected to very low temperatures.

Four commonly used empirical equations for the elastic modulus in terms of the compressive strength were presented in the previous state-of-the-art report [Zia et al. 1991]. They include the ACI Code equation, the equation recommended by the ACI Committee 363, the equation suggested by Ahmad et al., and the equation proposed by Cook. In a recent study by Alfes [1992], he proposed that

$$E_c = 0.20 w^{1.09} (f_c')^{0.84}$$

in which is the elastic modulus in GPa, is the unit weight of concrete in kg/m3, and is the 28-day compressive strength of concrete. It is noted that this equation is quite similar to the equations recommended by Ahmad et al. and by Cook.

Many authors [Baalbaki et al. 1992; Setunge et al. 1990; Nilsen and Aitcin 1992a] have found that their test data were overestimated by the ACI Code equation and the equations in other national codes. Some indicated a better representation by the equation recommended by the ACI Committee 363. However, many other authors have found that the ACI Code equation was quite acceptable for prediction [Zia et al. 1993a]. It seems that there is no consensus on this issue at the present time. If an accurate knowledge of the elastic modulus is desired, the best approach is to obtain the value from direct testing of the specific concrete being used for construction.

A valuable record of long-term performance of the high strength concrete used in Water Tower Place has been reported by Russell and Larson [1989]. This record covers a period of 13 years. Another set of valuable test data on the engineering properties of commercially available high-strength concretes, including the elastic modulus, can be found in the report presented by Burg and Ost [1992].

4.2.1.2 Dynamic Modulus

As stated previously, much less information is available on the dynamic modulus than on the static modulus of HPC. In the past few years, very little has been published in the literature. What has been summarized in the previous state-of-the-art report [Zia et at. 1991] still represent the current knowledge of the subject.

Generally speaking, the measurement of dynamic modulus corresponds to a very small instantaneous strain. Therefore the dynamic modulus is approximately equal to the initial tangent modulus which is appreciably higher than the static (secant) modulus. The difference between the two moduli is due in part to the fact that heterogeneity of concrete affects the two moduli in different ways. For low, medium, and high strength concretes, the dynamic modulus is generally 40%, 30% and 20% respectively higher than the static modulus of elasticity [Mehta 1986]. Recently, Nilsen and Aitcin [1992b] used pulse velocity test to predict the static modulus of elasticity of high-strength concrete.

4.2.2 Strain Capacity

The strain capacity of concrete can be measured either in compression or in tension. In the compression mode, it can be measured by either concentric or eccentric compression testing. In the tensile mode, the strain capacity can be either for direct tension or indirect tension. The behavior under multiaxial stress states is outside the scope of this report, and only the behavior under uniaxial stress condition will be discussed.

4.2.2.1 Stress-Strain Behavior in Compression

The stress-strain behavior is dependent on a number of parameters which include material variables such as aggregate type and testing variables such as age at testing, loading rate, strain gradient and others. Higher strength and corresponding strain are achieved for crushed aggregate from fine-grained diabase and limestone as compared to concretes made from smooth river gravel and from crushed granite that contained inclusions of a soft mineral.

Many investigations have been conducted to obtain the complete stress-strain curves in compression with compressive strengths up to 20,000 psi (140 MPa). For concrete of higher strength, the shape of the ascending part of the curve becomes more linear and steeper, the strain at maximum stress is slightly higher, and the slope of the descending part becomes steeper. This is true whether the aggregate is normal weight or lightweight.

To obtain the descending part of the stress-strain curve, it is necessary to avoid specimen-testing machine interaction. One approach is to use a closed-loop testing system with a constant rate of axial strain as a feedback signal for closed-loop operation. For very high strength concretes it may be necessary to use the lateral strains as a feedback signal rather than the axial strains. Another successful approach is to test high strength concrete cylinders in parallel with two or more instrumented auxiliary high strength steel tubes as reported by Banthia and Sicard [1989].

A comprehensive and simple way of characterizing the stress-strain response of concrete in compression is the fractional equation which has been described thoroughly in the previous state-of-the-art report [Zia et al. 1991].

Since high strength concrete is increasingly being used in members subjected to high compressive stress, the question of its ductility has become an issue of considerable interest. Several studies [Muguruma et al. 1989; Hatanaka et al. 1990; Hatanaka et al. 1991a; Hatanaka et al. 1991b; Koike and Hatanaka 1991; Sun and Sakino 1993] of high strength concrete with different degrees of lateral confinement have been conducted and the results were used to modify the previously proposed stress-strain models for confined high strength concrete. Due to the effective lateral confinement, the descending part of the stress-strain curve rises, becoming less steep, and the ultimate limiting strain is also increased.

4.2.2.2 Stress-Strain Behavior in Tension

The direct tensile stress-strain curve is difficult to obtain. Due to difficulties in testing concrete in direct tension, only limited and often conflicting data are available. Other than the several studies described in the previous report [Zia et al. 1991], no new developments can be found in the recent literature.

4.2.2.3 Flexural Tension

While the information on the stress-strain behavior in tension is severely limited, virtually no data is available regarding the strain capacity in flexural tension. In the recent SHRP C-205 studies, Zia et al. [1993c, 1993d, 1993e] developed a special mounting device which was utilized to measure the flexural strain capacity of HPC under flexural tension tests. As expected, there was a wide range of scatter of the test data, roughly varying from 120m to 200m with 150m being a reasonable average value. This remains an area for which research is sorely needed to provide a basis for design where flexural cracking is an important consideration.

4.2.3 Poisson's Ratio

Poisson's ratio under uniaxial loading conditions is defined as the ratio of lateral strain to strain in the direction of loading. In the inelastic range, due to volume dilation resulting from internal microcracking, the apparent Poisson's ratio is not constant but is an increasing function of the axial strain.

Very limited data on the values of Poisson's ratio for high strength concrete is available. In general, Poisson's ratio of higher strength concrete in the elastic range appears comparable to the expected range of values (0.15 to 0.20) for lower-strength concrete. Slightly higher values (~0.22) are given by ultrasonic tests. In the inelastic range, the relative increase in lateral strains is less for higher-strength concrete than for concrete of lower strength. That is, higher-strength concrete exhibits less volume dilation than lower-strength concrete. This implies less internal microcracking for concrete of higher strength.

Other than the data referenced to in the previous report [Zia et al. 1991], a more recent study on high strength concrete with strength up to 120 MPa (17,000 psi) by Setunge et al. [1990] indicated that the Poisson's ratio of very high strength concretes increased with an increase in compressive strength. An empirical equation was proposed for the Poisson' s ratio as a function of the square root of the compressive strength.

4.2.4 Shrinkage and Creep

Shrinkage and creep are time-dependent deformations that, along with cracking, provide the greatest concern for designers because of the degree of uncertainty associated with their prediction. Concrete exhibits elastic deformations only under loads of short duration, and due to additional deformation with time, the effective behavior is that of an inelastic and time-dependent material.

4.2.4.1 Shrinkage

Shrinkage is the decrease of concrete volume with time. This decrease is due to changes in the moisture content of the concrete and physicochemical changes, which occur without stress attributable to actions external to the concrete. Swelling is the increase of concrete volume with time. Shrinkage and swelling are usually expressed as a dimensionless strain (in./in. or mm/mm) under given conditions of relative humidity and temperature. Concrete immersed in water does not shrink but may swell.

Shrinkage of high performance concrete may be expected to differ from conventional concrete in three broad areas: plastic shrinkage, drying shrinkage, and autogenous shrinkage. Plastic shrinkage occurs during the first few hours after fresh concrete is placed. During this period, moisture may evaporate faster from the concrete surface than it is replaced by bleed water from lower layers of the concrete mass. Paste-rich mixes, such as high performance concretes, will be more susceptible to plastic shrinkage than conventional concretes. Drying shrinkage occurs after the concrete has already attained its final set and a good portion of the chemical hydration process in the cement gel has been accomplished. Drying shrinkage of high strength concretes, although perhaps potentially larger due to higher paste volumes, do not, in fact, appear to be appreciably larger than conventional concretes. This is probably due to the increase in stiffness of the stronger mixes. Data for VES and HES mixes is limited. Autogenous shrinkage due to self-desiccation is perhaps more likely in concretes with very low W/CM ratio, although there is little data outside indirect evidence with certain high strength concrete research [Aitcin et al. 1990]. Shrinkage should not be confused with thermal contraction which occurs as concrete loses the heat of hydration.

Shrinkage is a function of the paste, but is significantly influenced by the stiffness of the coarse aggregate. The interdependence of many factors creates difficulty in isolating causes and effectively predicting shrinkage without extensive testing. The key factors affecting the magnitude of shrinkage are:

- b. Aggregate The aggregate acts to restrain the shrinkage of cement paste; hence concrete with higher aggregate content exhibits smaller shrinkage. In addition, concrete with aggregates of higher modulus of elasticity or of rougher surfaces is more resistant to the shrinkage process.
- B. Water-cementitious material ratio The higher the W/C ratio is, the higher the shrinkage. This occurs due to two interrelated effects. As W/C increases, paste strength and stiffness decrease; and as water content increases, shrinkage potential increases.
- C. Member size Both the rate and the total magnitude of shrinkage decrease with an increase in the volume of the concrete member. However, the duration of shrinkage is longer for larger members since more time is needed for shrinkage effects to reach the interior regions.
- D. Medium ambient conditions The relative humidity greatly affects the magnitude of shrinkage; the rate of shrinkage is lower at higher values of relative humidity. Shrinkage becomes stabilized at low temperatures.
- E. Admixtures Admixture effect varies from admixture to admixture. Any material which substantially changes the pore structure of the paste will affect the shrinkage characteristics of the concrete. In general, as pore refinement is enhanced, shrinkage is increased.

  Pozzolans typically increase the drying shrinkage, due to several factors. With adequate curing, pozzolans generally increase pore refinement. Use of a pozzolan results in an increase in the relative paste volume due to two mechanisms; pozzolans have a lower specific gravity than portland cement and, in practice, more slowly reacting pozzolans (such as Class F fly ash) are frequently added at better than one-to-one volume replacement factor, in order to attain specified strength at 28 days. Additionally, since pozzolans such as fly ash and slag do not contribute significantly to early strength, pastes containing pozzolans generally have a lower stiffness at earlier ages as well, making them more susceptible to increased shrinkage under standard testing conditions. Silica fume will contribute to strength at an earlier age than other pozzolans but may still increase shrinkage due to pore refinement.

Chemical admixtures will tend to increase shrinkage unless they are used in such a way as to reduce the evaporable water content of the mix, in which case the shrinkage will be reduced. Calcium chloride, used to accelerate the hardening and setting of concrete, increases the shrinkage. Air-entraining agents, however, seem to have little effect.

- F. Cement Type The effects of cement type are generally negligible except as rate-of-strength-gain changes. Even here the interdependence of several factors make it difficult to isolate causes. Rapid hardening cement gains strength more rapidly than ordinary cement but shrinks somewhat more than other types, primarily due to an increase in the water demand with increasing fineness. Shrinkage compensating cements can be used to minimize shrinkage cracking if they are used with appropriate restraining reinforcement.
- G. Carbonation Carbonation shrinkage is caused by the reaction between carbon dioxide (CO2) present in the atmosphere and calcium hydroxide (CaOH2) present in the cement paste. The amount of combined shrinkage varies according to the sequence of occurrence of carbonation and drying process. If both phenomena take place simultaneously, less shrinkage develops. The process of carbonation, however, is dramatically reduced at relative humidities below 50 percent.

A widely used predictive equation for shrinkage strain is that given by the ACI Committee 209 [1993]. It should be noted that if the usual laboratory-sized test specimens are used for determining the shrinkage properties of a concrete mix, then the predicted behavior of a concrete structure such as bridge, could well be in error unless correct allowances for curing and size effect are used. Surface area to volume and shape effect correction factors are used to accommodate physical differences affecting drying rates. Humidity and composition effect parameters can also be applied, however, these corrections are necessarily broad. Differences in empirical data can be large. Given a choice, casting a trial section is much preferred than relying on generalized predictive equations.

The shrinkage properties of concretes with higher compressive strengths are summarized in an ACI State-of-the-Art Report [1993]. The basic conclusions were:

1. Shrinkage is unaffected by the W/C ratio but is approximately proportional to the percentage of water by volume in concrete,
2. Laboratory and field studies have shown that shrinkage of higher strength concrete is similar to that of lower-strength concrete,
3. Shrinkage of high strength concrete containing high range water reducers is less than for lower strength concrete,
4. Higher strength concrete exhibits relatively higher initial rate of shrinkage, but after drying for 180 days, there is little difference between the shrinkage of high strength concrete and lower strength concrete made with dolomite or limestone.

Collins [1989] conducted an experimental study on shrinkage of high-strength concrete in which five test mixes were prepared based on historical data. These mixes varied in paste content, aggregate size, and the use of high-range water-reducing admixtures. The results of the test program showed that shrinkage deformations were somewhat less for concrete mixtures with lower paste contents and larger aggregate size.

In a more recent study, Alfes [1992] examined how shrinkage was affected by the aggregate content, the aggregate modulus of elasticity, and the silica fume content. Using W/C ratio in the range of 0.25 to 0.3 with 20% silica fume by weight of cement and varying amount and type of aggregates (basalt, LD-slag, and iron granulate), he produced concretes with 28-day strength in the range of 102 to 182 MPa (14,600 to 26,000 psi). The test results showed that there is a direct and linear relationship between the shrinkage value and the modulus of elasticity of the concrete.

Carette et al. [1993] reported a study of high performance concretes with high volume fly ash from sources in the U. S. The concretes had low bleeding, satisfactory slump and setting characteristics and low autogenous temperature rise. These concretes also had excellent mechanical properties at both early and late ages with compressive strength reaching as high as 50 MPa (7,000 psi) at 91 days and the drying shrinkage of the concretes was relatively low.

Field measurements of surface shrinkage strains on a mock column, fabricated with high strength concrete, after two and four years were made by Aitcin et al. [1990] and the measurements were compared with results on specimens under laboratory conditions. It was shown that the surface shrinkage strains under the field condition were considerably lower than those measured under the laboratory conditions.

In a similar study by Hindy et al. [1994], measurements of dry shrinkage were made on concrete specimens as well as on instrumented reference columns made with two different ready-mixed high-performance concretes. One had a 91-day compressive strength of 98 MPa (14,200 psi) and the other had a 91-day compressive strength of 80 MPa (11,600 psi). The first contained silica fume but the second did not. The effects of curing time, curing conditions, silica fume content, and W/CM ratio were considered. It was found that the longer the curing time the lower the dry shrinkage, and that the lower the W/CM ratio the lower the dry shrinkage. Dry shrinkage of small specimens measured by the conventional laboratory test was found to over-estimate shrinkage of the concrete in the real structure. The ACI 209 predictive equation was found to be valid for the high performance concretes only if new values for the parameters were introduced.

In the recent SHRP C-205 studies, Zia et al. [1993c, 1993d, 1993e] evaluated the shrinkage behavior of VES, HES, VHS concretes with different aggregates (crushed granite, marine marl, rounded gravel, and dense limerock). Shrinkage measurements were made for three to nine months in different cases. The observed behavior followed the general trend of conventional concrete except for the two cases of VES concrete using a special blended cement (Pyrament) with marine marl and rounded gravel as aggregates. In these two cases, the specimens exhibited an expansion of approximately 140 microstrains, rather than shrinkage for the entire period of 90 days. The expansion was attributed to the lack of evaporable water in the concrete because of its very low W/C (0.17 for marine marl, and 0.22 for rounded gravel).

It should be noted that a valuable collection of 13-year shrinkage data on the concretes used for Water Tower Place in Chicago has been reported by Russell and Larson [1989] and the shrinkage data of a group of commercially produced high strength concretes can be found in the report by Burg and Ost [1992].

4.2.4.2 Creep

Creep is the time-dependent increase in strain of hardened concrete subjected to sustained stress. It is usually determined by subtracting, from the total measured strain in a loaded specimen, the sum of the initial instantaneous strain (usually considered elastic) due to sustained stress, the shrinkage, and any thermal strain in an identical load-free specimen, subjected to the same history of relative humidity and temperature conditions.

Creep is closely related to shrinkage and both phenomena are related to the hydrated cement paste. As a rule, a concrete that is resistant to shrinkage also has a low creep potential. The principal parameter influencing creep is the load intensity as a function of time; however, creep is also influenced by the composition of the concrete, the environmental conditions, and the size of the specimen.

The composition of concrete can essentially be defined by the W/CM ratio, aggregate and cement types and quantities. Therefore, as with shrinkage, an increase in W/CM ratio and in cement content generally results in an increase in creep. Also, as with shrinkage, the aggregate induces a restraining effect so that an increase in aggregate content and stiffness reduces creep.

Numerous tests have indicated that creep deformations are proportional to the applied stress at low stress levels. The valid upper limit of the relationship can vary between 0.2 and 0.5 of the compressive strength. This range of the proportionality is expected due to the large extent of microcracks in concrete at about 40 to 45% of the strength.

A widely used predictive equation for creep strain at time t days after loading is that given by the ACI Committee 209 [1993]. As with shrinkage, use of standard-sized laboratory test specimens for determining creep properties of a concrete mix will require adjustment for size and shape effects to reasonably predict behavior of a concrete structure.

Earlier studies on creep of concrete containing fly ash, silica fume, and high-range water reducer have been summarized in the previous report [Zia et al. 1991]. It was observed that, with comparable strength, there was no apparent difference between the specific creep of silica fume concrete and that of portland cement concrete, or of fly ash concrete. However, creep strains, creep coefficient, and specific creep were all smaller for high strength concrete than for concretes of medium and low strengths at different stress levels and at any time after loading.

Collins [1989] investigated the effect of mix proportions on creep characteristics in a study in which five mixes with 28-day design strengths ranging from 8,700 psi to 9,300 psi (60 MPa to 64 MPa) were used. The results indicated that creep was somewhat less for concrete mixtures with lower cement paste and larger aggregate size. It was also shown that the use of high-range water-reducing admixture did not show a significant effect on the creep deformations.

Carette et al. [1993] reported a study of high performance concretes with high volume fly ash from sources in the U. S. The concretes had low bleeding, satisfactory slump and setting characteristics and low autogenous temperature rise. These concretes also had excellent mechanical properties at both early and late ages with compressive strength reaching as high as 50 MPa (7,000 psi) at 91 days and the creep of the concretes was relatively low.

In a study of long-term deflection of high-strength concrete beams, Paulson et al. [1991] pointed out that a large body of experimental evidence was available confirming that the creep coefficient of high-strength concrete under sustained axial compression was significantly less than that of ordinary concrete. Thus the ratio of time-dependent deflection to immediate elastic deflection of high-strength concrete beams under sustained loads should likewise be lower. With nine beams made of nominal concrete strengths over a range to 13,000 psi (91 MPa) and loaded over a 12-month period, they found significant differences between the deflection of high-strength concrete beams and that of nornal strength concrete beams. It was suggested that the long-term deflection multipliers of the ACI Code should be modified to account for the reduced creep deflection with high strength concrete.

Since high-strength concrete is increasingly being used in compression members with reinforcement and creep data are generally obtained with small unreinforced specimens in the laboratory, Yamamoto [1990] conducted a creep test of a reinforced concrete column of 25 x 25 x 100 cm (9.84 x 9.84 x 39.4 in.) containing 2.44% longitudinal reinforcement and 0.79% lateral ties. The concrete strength was 582 kgf/cm2 (57 MPa or 8,280 psi) at 31 days. When the concrete was 33 days old, a sustained load of 198 kgf/cm2 (19.4 MPa or 2,815 psi) was applied on the column. After 170 days of loading, the creep coeffficient of the concrete was determined to be 0.57 and the ultimate creep coefficient was estimated to be 1.4 which suggested that the creep deformation of high-strength reinforced concrete columns would be much smaller than that of normal strength concrete columns.

In the recent SHRP C-205 studies, Zia et al. [1993e] evaluated the creep behavior of VHS concretes with different aggregates (crushed granite, marine marl, and rounded gravel). Creep strain measurements were made for 90 days in each case. The observed creep strains of the different groups of VHS concrete ranged from 20% to 50% of that of conventional concrete. The creep strains were especially low for concretes with a 28-day strength in excess of 10,000 psi (70 MPa). The specific creep of the concrete with marine marl was much higher than that of the concrete with either crushed granite or washed rounded gravel.

It should be noted that a valuable collection of 13-year creep data on the concretes used for Water Tower Place in Chicago has been reported by Russell and Larson [1989] and the creep data of a group of commercially produced high strength concretes can be found in the report by Burg and Ost [1992].

4.2.5 Thermal Properties

The thermal properties of concrete are of special concern in structures where thermal differentials may occur from environmental effects, including solar heating of pavements and bridge decks. The thermal properties of concrete are more complex than for most other materials, because not only is concrete a composite material whose components have different thermal properties, but its properties also depend on moisture content and porosity.

Data on thermal properties of high performance concrete is limited, although the thermal properties of high strength concrete fall approximately within the same range as those of lower strength concrete, for characteristics such as specific heat, diffusivity, thermal conductivity and coefficient of thermal expansion [Farny and Panarese 1994].

Several earlier studies on the effects of extreme temperature on the behavior of concrete were summarized in the previous report [Zia et al. 1991], including the study by Castillo and Durrani [1990] on high-strength concrete (12,900 psi or 89 MPa) under temperatures ranging from 23oC to 800oC, and the study conducted by Diederichs et al. [1989] on high-strength concretes of 12,300 to 16,000 psi (85 to 111 MPa) subjected to temperatures up to 350oC. Also included was the study by Lee et al. [1989] on normal and high-strength concretes exposed to temperatures ranging from +20oC to -70oC.

In a more recent study, Olsen [1990] evaluated the explosion risk of heat induced high strength concrete as compared to normal strength concrete. Cylinders of 100 x 200 mm with a compressive strength in the range from 30 MPa to 90 MPa were cured in two different ways: (a) Seven days in water followed by 21 days in laboratory atmosphere (20oC and 60%RH). (b) Seven days in water followed by 21 days sealed with plastic aluminum foil. A total of 36 cylinders were heated in an electrical oven at a heating rate of 2.5oC per minute until reaching a temperature of 600oC. After 2 hours at this temperature, the cylinders were cooled at a rate of up to 1oC per minute. The tests showed that the explosion risk depended on the curing conditions and that, in the case of high strength concrete, the explosion risk is not higher than for normal strength concrete especially for concrete cured under condition (a).

In a study of deterioration of lightweight fly ash concrete due to gradual cryogenic frost cycles, Khayat [1991] monitored longitudinal thermal strains of water-saturated and air-dried concretes between 65o and -250oF. Cumulative drops in compressive and splitting tensile strengths were measured after each of 5 gradual freeze-thaw cycles ranging from a high of 65oF to two low temperatures of -40o and -100oF. That was done to evaluate the concrete's frost durability at liquified petroleum and natural gas temperatures, respectively. As expected, moist concrete exhibited larger dilation and residual strains than air-dried concrete. Burg and Ost [1992] measured the coefficient of thermal expansion of five commercially available high-strength concrete in the Chicago area and they found that the coefficient varied between 5.2 to 6.8 microinches/in./oF.

4.3 Fatigue

Under repeated loads, concrete suffers damages resulting from progressive growth of internal microcracks. After a sufficient number of load repetitions, concrete fails at a load less than its static strength. The fatigue strength of concrete is therefore a fraction of its static strength that the concrete can support repeatedly for a given number of load cycles. At fatigue failure, concrete exhibits increased strains and reduced modulus (i.e. slope of its stress-strain curve) due to the progressive internal damages from microcracking.

As the static strength of concrete increases, it becomes increasingly more brittle and its ultimate strain capacity does not increase proportionately with the increase in strength. Therefore very high strength concrete could be vulnerable to fatigue loading. However, in high strength concrete, the elastic modulus of the paste and that of the aggregate are more similar, thereby reducing stress concentrations at the aggregate paste interface, which would make high strength concrete less susceptible to fatigue loading.

The basic concepts of S–N curve, Goodman diagram, and Miner's Rule regarding fatigue strength have been described in the previous report [Zia et al. 1991]. In addition, earlier fatigue studies examining the various effects of loading variations (such as effect of resting, effect of loading rate, effect of stress gradient, effect of stress reversal, effect of impact load, and effect of air entrainment) have also been summarized in the same report. In the following, only a few of the special fatigue studies reported recently will be described.

Bazant and Schell [1993] reported an experimental study of fatigue fracture of geometrically similar high-strength concrete specimens of very different sizes. Notched beams were subjected to three-point cyclic loading. The number of cycles to failure ranged from 200 to 41,000. It was found that Paris law for the crack length increment per cycle as a function of the stress intensity factor, which was previously verified for normal strength concrete, was also applicable to high-strength concrete. However, for specimens of different sizes, an adjustment for the size effect needs to be introduced, of a similar type as previously introduced for normal strength concrete. A linear regression plot estimating the size-adjustment parameters was derived. A linear elastic fracture mechanics type calculation of the deflections under cyclic loading on the basis of the size-adjusted Paris law produced correct values for the terminal phase but grossly underpredicted the initial deflections. The results underscore the importance of considering fatigue fracture growth in the case of high-strength concrete structures subjected to large, repeated loads, and taking into account the very high brittleness under fatigue loading.

Based on a deformation formulation and utilizing the monotonic force-deflection curve and the fatigue creep curve, Daerga and Pontinen [1993] developed a hypothesis for fatigue failure of concrete structures. The hypothesis was applied to flexural fatigue tests of notched beams of a plain high performance concrete. The experiments comprised monotonic loading in deformation control and constant amplitude loading at three different load levels in flexural tension. The test results essentially confirmed the hypothesis. Furthermore, the hypothesis provided an estimation of accumulated damage and a prediction of remaining service life, which accounted for the nonlinear nature of damage development.

Do et al. [1993] conducted tests to provide data on the response of high-strength concrete to cyclic loading. The test results were compared with those on normal strength concrete and also with those obtained in other studies with high-strength concrete. It was found that fatigue life results are scattered, but may be predicted reasonably well using a probabilistic approach based on the McCall model and assuming a probability of failure. Longitudinal strain development in high-strength concrete is similar to normal strength concrete.

The results of low-cycle fatigue testing of high-strength concrete were reported recently by Mor et al. [1992]. In their study, lightweight and normal weight aggregate concretes were tested under reversible loading under both submerged and air-dry conditions. Fatigue capacity of lightweight aggregate concrete was similar to or better than that of normal weight aggregate concrete of similar strength properties. Submersion of high-strength concrete in water did not affect its fatigue capacity. Fatigue capacity of high-strength reinforced concrete was found to be directly related to the bond between concrete and reinforcement and not related to any other strength property. The addition of silica fume to lightweight aggregate concrete improved its bond by 100 percent and its fatigue life by over 60 percent. No significant improvement was observed when silica fume was added to normal weight aggregate concrete.

Flexural fatigue loading is common in concrete pavements and bridge decks. Fatigue fracture characteristics of Florida concrete subjected to flexural loading was investigated by Tawfiq [1993]. The objectives of the investigation were (1) to investigate the effect of several parameters on the fatigue strength of Florida concrete under constant amplitude loading, and (2) to obtain a relationship between these parameters and the fatigue life represented by the number of cycles of loading endured. Three parameters were used in the study: (1) stress ratio, (2) number of cycles of loading to failure, and (3) the compressive strength of concrete. Additionally, the strain accumulation, hysteretic behavior, and stiffness degradation were also studied. Based on the test results, the author suggested four empirical formulas to predict the fatigue strength of Florida concrete. Three of the four formulas are intended for plain concrete elements with pre-existing cracking (notched) and the fourth formula is used to estimate the total fatigue life for uncracked elements (unnotched). The test results showed that high strength concrete could have about 22% shorter fatigue life than lower strength concrete. This implies that to prolong the fatigue life of a high strength concrete element subjected to 70% loading ratio, the total applied loads should be multiplied by a factor of 0.78. The rate of stiffness deterioration at the second stage of cracking was much higher in high strength concrete than in lower strength concrete. This confirms the need for better quality control and assurance when high strength concrete is used in pavement and bridge construction.

4.4 Durability

When properly designed and carefully produced with good quality control, concrete is inherently a durable material. However, under adverse conditions, concrete is potentially vulnerable to deleterious attacks such as frost, sulfate attack, alkali-aggregate reaction, and corrosion of steel. Each of these processes involves movement of water or other fluids, transporting aggressive agents through the pore structure of concrete. Therefore, porosity and permeability are important properties which affect the durability of concrete.

4.4.1 Porosity and Permeability

An excellent review of the pore structure and its influence on permeability of cement paste and concrete has been presented by Young [1988]. It is generally agreed that for normal-weight concrete, its porosity resides principally in the cement paste. The pore structure of the paste can be classified into two types: (1) intrinsic pores in the cement gel resulting from hydration; and (2) capillary pores originating from the space initially filled with water. The size and distribution of these pores cover a very large range, from much less than 2.5 nm to 10,000 nm. Typically, the size of the gel pores is less than 10 nm whereas the size of the capillary pores is more than 10 nm. Pores of 10,000 nm or larger are classified as air voids.

In the previous state-of-the-art report [Zia et al. 1991], the basic information on how porosity and permeability are affected by W/C ratio, curing, mineral and chemical admixtures has been presented, and the various methods commonly used to measure porosity and permeability has been discussed. In the past several years, a large number of porosity and permeability studies have been reported in the literature. Some of these recent studies will be summarized in the following sections.

4.4.1.1 Porosity

Winslow and Lui [1990] demonstrated that the cement paste in concrete and mortar has a pore size distribution different from that of plain paste hydrated without aggregate. For mortar and concrete, additional porosity occurs in pore sizes larger than the plain paste's threshold diameter as measured by mercury intrusion. Based on the assumption that these larger pores are essentially present only in the interfacial zones surrounding each aggregate, Winslow et al. [1994] designed an experimental program in which the volume fraction of sand in a mortar was varied in a systematic fashion and the resultant pore system probed using mercury intrusion porosimetry. The intrusion characteristics were observed to change drastically at a critical sand content of 48.6% volume fraction. Similar results were observed for a series of mortar specimens in which the cement paste contained 10% silica fume. To better interpret the experimental results, a hard core/soft shell computer model was developed to simulate percolation of the interfacial zones in mortar and concrete specimens. The interfacial zone thickness (15-20 mm) provided by the mercury intrusion experiment and computer model for mortar was somewhat less than that conventionally measured using the SEM imaging technique but the difference was to be expected given the inherent differences in the two measurement methods. The computer model indicated that for an interfacial zone thickness of about 20 mm, interfacial zone percolation would occur in most typical construction concrete mixes. This observation was supported by the generally large permeability of concrete relative to plain paste. By decreasing the inerfacial zone thickness or the porosity in the interfacial zone or reducing the quantity of aggregates in a concrete, the probability of interfacial zone percolation would be reduced. Thus they suggested that the engineering of interfacial zone microstructure and aggregate content and size distribution might be critical in increasing the service life of concrete.

Based on energy, structural stability and durability criteria, Xu et al. [1993] proposed an ideal structural model for very low porosity cementitious materials. It was suggested that the ideal structure would consist of surface reactive fine fillers and cementitious products combined to form a chemically stable mature network with good interfacial bond. They investigated a new method involving treatment of aggregate surfaces for use in forming ideal structures, and the effects of matrix and filler characteristics on interfacial bond were examined. A method of replacing a large quantity of unhydrated cement in very low porosity cementitious systems was investigated. The test results demonstrated that interfacial bond was an important factor affecting mechanical properties, and that replacing unhydrated cement by inert fine aggregates resulted in a considerable decrease of strength. Surface treatment of aggregates was effective in maintaining the compressive strength of mortar at the same level as pure cement paste. Surface hydraulic aggregates were effective in producing the ideal structure for very low porosity cementitious materials. It seemed that a key factor in approaching ideal structure is to change inert aggregate surfaces into chemically reactive surfaces.

Mercury intrusion porosimetry is a common method for measuring pore size distribution of hydrated cement products. However, an accurate measurement of this is difficult to obtain. Feldman and Beaudoin [1991] used mercury intrusion porosimetry to 414 MPa to measure the pore size distribution of cement pastes prepared at W/C ratio of 0.8, 0.6, and 0.45. Specimens were predried before intrusion measurements by several techniques including solvent replacement with methanol or isopropanol, evacuation and/or heating for various periods and conditioning at 11% RH. Second intrusions were also performed to investigate the effects of first intrusion. They concluded that it was not possible to obtain an actual pore size distribution of cement paste by mercury intrusion because of its sensitivity to stress.

4.4.1.2 Permeability

There is no recognized standard test method to measure the permeability of concrete. Different investigators have used different techniques and procedures. In general, there are three categories of methods: air(or gas) permeability, hydraulic permeability, and chloride ion permeability. An excellent review of different methods for measurement of permeation properties of concrete on site has been presented by Basheer et al. [1993b].

A. Air (or Gas) Permeability Bunte and Rostasy [1989] described three selected methods that are suitable for on-site testing of concrete permeability, among them are the air permeability test methods by Schonlin and Hilsdorf and by Figg. Both methods measure the air permeability of the surface layer of concrete.

In the Figg method [Figg 1992], a small hole (10 mm diameter and 40 mm deep) is drilled into the concrete and the hole is plugged to half its depth with silicone rubber, either cast in-situ or using a specially shaped preformed bung. A 17-gage hypodermic needle is then pressed through the plug to reach the small test cavity which is evacuated to a pressure of 55 kPa below atmospheric and the time (in seconds) for a 5 kPa pressure increase is taken as a measure of the concrete permeability. See Fig. 4.13(a).

In the Schonlin and Hilsdorf method, a small suction chamber is adhered to the concrete surface by creating a partial vacuum inside the chamber. See Fig. 4.14(a). The reduction of vacuum inside the chamber due to flow of air through the concrete into the chamber is monitored with respect to time. The permeability index of the concrete is determined from the change of pressure in the chamber within a give time period. If V is the volume of the vacuum chamber and pa is the atmospheric pressure outside the chamber, Dt is the lapse of time during which the initial pressure in the chamber pi is increased to p, then the permeability index is calculated by the following equation:

$$I_{perm} = \frac{V}{\Delta t} \frac{\Delta p}{(p_a - \Delta p / 2)} \quad \text{and} \quad \Delta p = p - p_i$$

Schonlin and Hilsdorf [1988] had also extended their test procedure to determine the air permeability of a concrete disk of 150 mm diameter and 40 mm thick. See Fig. 14(b). The concrete disk is cast directly into a rubber ring and is subsequently cured for 7 days, unless the duration of curing is the variable, and then stored in a constant environment of 20oC, 65% RH up to the time of testing at an age of 56 days. The vacuum chamber is adhered to one side of the disk specimen with the other side of the disk exposed to the atmospheric pressure. The vacuum chamber is evacuated first to a stabilized pressure and a stop cock between the vacuum pump and the instrument is closed. As the air flows through the concrete disk, the air pressure in the vacuum chamber increases. A time t0 is taken when the pressure reaches a value of p0 = 20 mbar and time t1 is taken when the pressure reaches a value of p1 = 50 mbar. Based on these pressure and time measurements, the permeabilty coefficient K (m2/sec) is obtained from the following expression:

$$K = V_s \frac{(p_1 - p_0)}{(t_1 - t_0)(p_a - \frac{p_1 + p_0}{2})} \frac{L}{A}$$

in which is the volume of the vacuum chamber, is the atmospheric pressure, is the thickness of the specimen, and A is the cross-sectional area of the specimen.

Hilsdorf [1989] investigated the question of whether a single parameter such as the air permeability of concrete is suitable to characterize concrete durability in a general way. He pointed out that a characteristic air permeability coefficient K as defined by Eq. (4.7) for a standard concrete specimen cured and preconditioned in a standardized way can be determined rapidly and reliably with the test procedure described above. The characteristic air permeability coefficient K reflects the effects of W/C ratio and curing, and correlates well with the progress of carbonation under laboratory conditions [Burieke and Hilsdorf 1993]. Lower air permeability coefficient also indicates a higher resistance to freezing and thawing, to chemical attack, and to penetration of chloride ions. However, the relationships between K and depth of carbonation and other factors affecting durability may no longer be unique if concretes contain larger amounts of admixtures (both mineral and chemical). Thus he concludes that air permeability of concrete is not the unique parameter to describe concrete durability, and it is unlikely that such a parameter exists. Nevertheless, he argues that it is advantageous to characterize concrete not only in terms of its standard compressive strength but also in terms of K since together with a knowledge of the type of cement it reveals several technological parameters such as composition and curing on the potential durability of a particular concrete.

Using a test device and procedure virtually identical to Schonlin and Hilsdorf's, Leeming [1993] conducted the surface air permeability test to determine the relative durability of concrete. See Fig. 4.15. The initial results of the test showed that the test was sensitive enough to differentiate between the sides, top and bottom of a concrete cube and that it also related in some measure to the carbonation depth. However, the test was sensitive to the moisture content of the concrete as all other known permeability test procedures. It was suggested that one potential application was the testing of surface coatings for their ability to restrict the carbonation of concrete. Another possible application would be to test whether the surface of the concrete is sufficiently dry so that silanes will achieve adequate penetration.

Following virtually the same concept as that of Schonlin and Hilsdorf, Whiting and Cady [1992] developed a device for measurement of air permeability of concrete as a part of their SHRP project. A series of comparative tests was carried out and it was established that a linear relationship existed between results obtained on the same specimens using the laboratory and field versions of the method. See Fig. 4.16. A series of field trials was also conducted to evaluate the device under actual test conditions. The results from three test sites showed that there was a general relationship between the readings taken in the field and the air permeability values determined from cores taken at the sites. Just as other similar devices, Whiting and Cady's device is portable, can take measurements on overhead and vertical surfaces, has a short testing time, and is fairly simple to operate. However, the device has an effective depth of measurement of only approximately 12.5 mm (0.5 in.) and the air intake is from an undefined surface area. It also has difficulty in achieving a good seal when the test surfaces are rough or contain microcracks, and it has been reported that the consistency of the test results in the field appears to be highly operator-dependent.

In contrast to the devices described above, which utilize vacuum to draw air out of concrete, Basheer et al. [1993a, 1993b] developed "Autoclam" permeability system for measuring the in-situ permeation properties of concrete by applying a pressure over the concrete surface to force air into it. See Fig. 4.17(a). Thus the base ring of Autoclam must be fastened to the concrete surface by strong adhesives or mechanical anchors. Using the equipment, the rate of decay of air pressure in the cell is recorded for the air permeability test. It was found that statistically satisfactory results could be obtained from a mean of three tests, and since the flow lines were largely concentrated within 40 mm from the test surface, reliable data could be collected by dry the surface even if the surface under test was initially wet. Tests were performed with W/C ratio, aggregate/cement ratio, and number of days of wet curing as the variables.

Using a test cell developed at Leeds University [Cabrera and Lynsdale 1988], Cabrera et al. [1989] conducted an extensive laboratory study of oxygen permeability of concrete involving 25 mixes with varying W/C ratios and five different types of superplasticizer. The mix designs were based on the criterion of minimum porosity by selecting the components of concrete to achieve maximum packing. Some mixes were also prepared by substituting 30% of cement with fly ash. The specimens were cured in a fog room kept at 20oC and 100% RH. Prior to testing for porosity and oxygen permeability, the specimens were dried in an oven at 105oC. In addition to air content, slump, flow table reading, and compressive strength measured by the standard methods, the porosity was obtained by helium pycnometry using a Micromeritics Autopycnometer. With the abundance of data from the 25 mixes, the data were analyzed by using a standard statistical package (SAS) achieving a confidence limit of 95%. Based on the statistical analysis, they developed a model relating permeability to several key parameters as follows:

$$LogK = -16.853 + 0.077(A)(W/C) - 0.012(t)^{0.66} + 0.037(P_p)(W/C)$$

in
which

K = oxygen permeability (m^2)

A = air content (%)

t = age of concrete (days)

Pp = porosity of the fraction of cement paste of concrete (%)M

W/C = water/cement ratio

If the data for concrete mixes with and without fly ash were separately grouped, the following two models relating oxygen permeability to porosity and compressive strength were obtained.

For concrete without fly ash,

$$LogK = -15.54 + 1.114 Log(P_p / f_{cu})$$

and for concrete with fly ash,

$$LogK = -15.95 + 1.01 Log(P_p / f_{cu})$$

in which f_{cu} = compressive strength of cube in MPa.

To use a permeability test for quality control would require measurements to be made at early ages. Therefore relations to predict the long-term permeability of concrete from values at early ages would be very useful. From the data collected in this study, a statistical model to predict the permeability of concrete up to the age of 90 days from its one-day value was obtained.

$$LogK = -3.688 + 0.789 LogK_1 + (0.356/t)$$

where K_1 = one-day value of permeability in m^2, and t = concrete age in days.

Using Eq. (4.10), one can establish tentative one-day permeability values as a criterion for designing for durability at the stage of trial mixes for a particular job, from the knowledge of the permeability of "good" and "bad" in-service concrete. The investigators obtained the permeability values of in-service 30 year old concretes, both "good" concrete and "bad" concrete showing signs of distress and cracking. The values were 5 x 10-17 m2 and 190 x 10-17 m2 respectively. By using these values as the target long-term permeabilities, one can determine the one-day permeability values from Eq. (4.10) as follows:

For "good" concrete, **K** =1.02 x 10-16 m^2 = 10.2 x 10-17 m^2

and for "bad" concrete,$\mathbf{K_1}$ =1.02 x 10-14 m^2 = 1,020 x 10-17 m^2

The above illustrates an approach to design for durable concrete with permeability as control. It should be understood that the models can be of more general use if the data base is expanded to cover a wider range of concrete mixes. Based on the above results, it was suggested by the authors that the target design value for oxygen permeability at one day should be 10 x 10-17 m2.

Air permeability test was also used by Dhir et al. [1991] to characterize the intrinsic permeability of concrete. The effect of micro-cracking on air permeability of concrete was studied by Nagataki and Ujike [1991]. The micro-crackings were induced by the differences of thermal expansion coefficients between aggregate and mortar or between aggregate and cement paste under elevated temperature as well as internal crackings formed around deformed tension bar. The air permeability coefficient of concrete under elevated temperature above 100oC was substantially higher than that of concrete at normal temperature. For reinforced concrete specimens subjected to sustained tensile loading, the air permeability coefficient through concrete cover was also much higher than that of the specimens without tensile loading. As one would expect, the air permeability coefficient increased as the tensile stress increased and the diameter of the steel bar became larger.

Torrent and Jornet [1991] conducted a comprehensive research program to evaluate the quality of the outer layer of concrete (the 'covercrete'), in terms of its durability and protective value to the underlying reinforcing bars. The effects of the concrete grade and cement type on the oxygen permeability were investigated. The specimens were moist-cured for 7 days followed by dry curing for 21 days. The results showed that the permeability to oxygen was reduced 40 times when the concrete strength was increased from 25 to 85 MPa. However, the cement type (including portland and blended cements) seemed to play only a secondary role in the quality of the 'covercrete' of concretes of the same grade, especially above 25 MPa.

The permeability of blended cement concrete was compared to that of the alkali activated slag concrete and portland cement concrete by Hakkinen [1992] using carbonation tests and gas penetration. The test results indicated that the high strength slag concrete activated by cement was very dense. The carbonation test results showed that in high strength concrete the dense structure of the paste compensates for the lack of calcium in blended cement concrete. In the alkali activated slag concrete the structure contained microcracks at the microscopic level, and the carbonation was significantly higher in alkali activated slag concrete than in other concretes. The gas permeability results revealed the same trends at the carbonation tests.

B. Hydraulic Permeability Absorption is a liquid transport mechanism due to capillary suction in pores of concrete. Balayssac et al. [1993] used the water absorption test for assessing both cover concrete porosity and largest capillary size, which are significant factors for concrete durability. The criterion used was the amount of water absorbed after one hour. The value is sufficiently representative of the mean radius of the largest capillaries. The results showed that the absorption test could be used to assess the effects of cement content on porosity of cover concrete and to account for the beneficial effects of curing on capillary size. Correlations were also established between carbonation depth and amount of water absorbed after one hour, which confirmed the validity of the tests for assessment of the resistance of concrete to carbonation.

McCarter and Ezirim [1990] also examined the movement of water into concrete using sorptivity test by measuring the depth of water penetration. Cylinders of 150 x 300 mm (6 x 12 in.) were cast with a high strength concrete (W/C = 0.35) and a low strength concrete (W/C = 0.6), both with ordinary portland cement. After 24 hours, the cylinders were demolded and the sides of the cylinders were sealed with a latex based paint, and left to cure at 21oC, +55% of -5% relative humidity. After curing, the cylinders were placed in plastic containers on stainless steel rods and the containers were filled with water up to 10 mm above the exposed surface. The specimens were removed after 1, 4, 9, 25, and 169 hours and split longitudinally. The depth of water penetration was detected by dusting the fractured surface with a moisture sensitive dye. By plotting the depth of penetration of water against square root of time for the complete test period, an equation was obtained for calculating the depth of penetration of water.

The air permeability test method by Figg described in Section A can also be used easily for water permeability test [Figg 1992]. To conduct the water permeability test which must be subsequent to the air test, the drilled cavity is filled with water at a hydrostatic head of 100 ml and the time for 0.01 ml of water to be absorbed by concrete is measured. See Fig. 4.13(b). Thus the permeability is expressed as a time in seconds. The shorter the time the more permeable the concrete, and conversely, the more durable the concrete the longer the permeability time. Figg used this method to evaluate durability of concrete prisms made with three different aggregate/cement ratios (1:3, 1:6, and 1:9), three different aggregates (flint gravel, limestone, and basalt), and W/C ratios in the range of 0.35 to 0.90. After 28 days of curing under water, the prisms were left outdoors in trays 50 mm deep to accelerate damage from weathering. The most permeable concretes (basalt) suffered damage after 12 winters exposure. Even though the specimens showed unexpectedly good performance, the permeability tests correctly predicted relative durability of the different concretes. Similarly, "Autoclam" developed by Basheer et al. [1993a, 1993b] is so equipped that it can also be used for water permeability test. See Fig. 4.17(b).

Another method for measuring water permeability is the Initial Surface Absorption Test (ISAT) which has been recognized by the British Standards. See Fig. 4.18(a). One of the difficulties with the test method was the leakage problem of the watertight cap. To overcome the problem, a modified cap was developed. See Fig. 4.18(b). It is noted that to fasten the test device to the concrete surface anchor bolts must be installed. Using the ISAT, which is sensitive to curing, cement type and grade of concrete, Dhir et al. [1994] found close correlations between permeation properties and the durability of concrete. In a study lasting for 10 years, a wide range of concrete mixes were designed using ordinary portland cement and blended cement with fly ash and ground granulated blastfurnace slag. Moist curing was varied from 0 to 28 days, and the maximum aggregate size from 5 to 40 mm. All mixes were tested for absorptivity and different aspects of durability including freezing-thawing resistance, carbonation, chloride ingress and mechanical wear. The test results showed that the absorptivity of concrete, measured with the ISAT, could be used as an accurate specification for concrete durability, irrespective of curing, grade or mix constituents. Based on the test results, a tentative classification for concrete durability was proposed as shown in Table 4.4.

Another method for measuring transient permeability in the laboratory was developed by Roy [1989], see Fig. 4.19. Her test setup uses cylindrical specimens contained in a flexible sleeve and connected to an up-stream and down-stream fluid reservoir. At the start of the experiment, both reservoirs and the specimen are maintained at the same constant pressure. Fluid flow is initiated through the specimen by rapidly establishing a pressure gradient between the up-stream and down-stream reservoirs. As the pressure begins to decay through the test sample, it is monitored and from this pressure decay, the permeability is calculated. Simultaneous collection of data from both the up-stream and down-stream reservoirs allows a simpler and more accurate data analysis technique which does not depend on knowing the value of the final pressure at the outset of the experiment. Measurements on several samples have resulted in a permeability of 9×10^{-8} darcys which is consistent with previous measurements on permeability using both liquid (water) and gas systems.

An efficient laboratory test set-up for conducting 40 water permeability tests simultaneously has been developed in Florida and used in an extensive study to evaluate the effects of various mix parameters and curing conditions on the modulus of rupture and permeability of structural concretes commonly used in Florida [Tia et al. 1990]. The test program covered three aggregate types (a Florida porous limestone, a river gravel, and an Alabama dense limestone), three cement types (Type I, II, and III), three W/C ratios (0.33, 0.38 and 0.45), three maximum aggregate sizes (3/8, 3/4, and 1 in.), two pozzolans (fly ash and silica fume) and six curing conditions (steam curing, air curing, curing compound and three moist curing conditions). In addition, concrete samples obtained from six concrete projects in Florida were tested to evaluate the differences between the in-service concrete and the laboratory-cured concrete.

Based on the successful laboratory studies, a field permeability test (FPT) method was developed and implemented in the testing and evaluation of thirteen selected marine concrete structures in Florida [Tia et al. 1992; Meletiou and Tia 1992]. Core samples of the site concrete tested by the FPTs were extracted and evaluated in the laboratory using the standard rapid chloride permeability test. The results of the rapid chlorida permeability test were found to be linearly related to the water permeabilities as measured by the FPT method. The latter was found to be able to provide a relative measurement of permeability which can be used as an indicator of the quality and performance characteristics of structural concrete.

The effect of curing conditions on the permeability and durability of concrete exposed to field conditions was investigated by Ewertson and Petersson [1993]. Two classes of concrete were cured under different conditions and then exposed to three different environments: outdoors exposed to rain fall, outdoors protected from rainfall, and indoors. After one or two years of such exposures, the water permeability and the carbonation depth were determined. The test results showed that the differences between different curing conditions were more pronounced the drier the climate. This means that laboratory tests cannot always be used for predicting the concrete behavior of a real structure. Field exposure tests are to be preferred. The results from the carbonation tests indicated that wet curing and covering with plastic sheet were equally efficient.

The effects on oxygen permeability and water absorption at various depths in the "covercrete" were also investigated by Ballim [1993] with concrete containing ordinary portland cement and blended cements with fly ash and GGBS. A range of strength grades was tested for each of the binder types. Concretes were exposed to moist curing conditions for 1, 3, 7, and 28 days before being tested at 28 days after casting. It was observed that moist curing had a marked influence on the potential durability of concrete and that a relatively greater influence on durability could be effected by extending the duration of early-age moist curing rather than decreasing the water/binder ratio.

C. Chloride Ion Permeability A standard procedure for measuring the movement of chloride ion through concrete is the test method of AASHTO T 277 "Rapid Determination of the Chloride Permeability of Concrete." In this procedure, the specimens are conditioned first by one hour of air dry, three hours of vacuum (pressure < 1 mm Hg), one hour of additional vacuum with specimens under de-aired water, and followed by 18 hours of soaking in water. The test consists of monitoring the amount of electrical current passed through 102 mm (4 in.) diameter x 51 mm (2 in.) long vacuum-saturated concrete specimen when one side of the specimen is immersed in a NaCl solution and the other side in a NaOH solution and a potential difference of 60V dc is maintained on the specimen for 6 hours. The total chage passed, in coulombs, is related to chloride permeability of the specimen.. The lower the total charge, the less permeable and more durable is the concrete.

While the AASHTO method has been widely used by many investigators, it is not without some shortcomings. First, since the method measures the total charge flowing through the test specimen in 6 hours, it tends to heat up the specimen. Higher temperature causes more current to flow through the specimen which, in turn, causes more temperature increase in the specimen. Consequently, the test can show very high total charge values as experienced by Zia et al. [1993c, 1993d, 1993e] and Geiker et al. [1991]. The temperature effect can be overcome by measuring the initial current flowing through the specimen or by measuring the AC impedance of the specimen [Hansen et al. 1993]. Chloride permeability (in coulombs) is essentially a conductance test whereas AC impedance (in ohms) is essentially a resistance test. Since the two properties are reciprocals, an inverse relationship could be expected between the two measurements. Their test data indicated that the initial current could be correlated with the inverse impedance (reciprocal of impedance). In an investigation of the AASHTO T 277 test procedures, Feldman et al. [1994] also found that simple measurement of initial current or resistivity gave the same ranking as conventional tests for the four classes of concretes they studied and that the initial current or resistivity measurement can replace the rapid chloride permeability test with a considerable time saving.

Another potential problem with the AASHTO T 277 test is that it may be affected by admixtures used in the concrete. ASTM C 1202 "Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration" which is basically the same document as AASHTO T 277, cautions that the rapid chloride permeability test can produce misleading results when calcium nitrite has been added and mixed into the concrete. The test has shown higher coulomb values on concrete with calcium nitrite than on identical concrete without calcium nitrite. However, ponding tests on concrete with calcium nitrite have shown it to be as resistant to chloride ion penetration as concrete without calcium nitrite. The high coulomb measurement may be due to additional ions being introduced with the calcium nitrite, making the water in the concrete more electrically conductive and making the concrete appear more permeable. Thus any time ions are added from any admixture or other source, the high coulomb measurement may become a problem. The same consideration should also be true for the AC impedance test.

Both AASHTO T 277 and ASTM C 1202 imply a correlation between the total charge (coulombs) measured and the amount of chloride ingress. However, Pfeifer et al. [1994] reviewed the documents referenced in ASTM C 1202 and concluded that reliable and proper correlations do not exist between the rapid test procedure results and 90-day ponding test results. They recommended that the table relating chloride penetration to coulomb values in these test procedures be removed since it is inaccurate and can be misleading.

The influence of drying on the chloride ion permeability of high performance concretes was investigated by Pigeon et al. [1993]. A wide range of concretes ranging from very high performance to normal concretes with 13 basic mixtures were prepared using four different hydraulic binders at three W/CM ratios. The specimens were dried at three different temperatures. It was found that with the use of silica fume and W/CM of 0.25 or less, it was possible to make high performance concretes that are extremely resistant to the internal damage that can result from drying, even at 110oC. They concluded that it is most probable that, under natural exposure conditions, the resistance to the penetration of chloride ions of the concretes will decrease significantly with time, unless they are not adequately protected against the effect of freezing and thawing cycles.

Given the appropriate lightweight aggregate, the concrete industry is now capable of producing high-strength lightweight concrete. With its increased strength to weight ratio, the structural advantage of such concrete is quite obvious. The issue of durability of high-strength lightweight concrete has received increasing attention from the researchers and engineers. Recent investigations have been reported by both Burg et al. [1990] and Zhang and Gjorv [1991a, 1991b]. Three high-strength lightweight concretes intended for severe, marine exposure were developed by Burg et al. for the unique set of environmental and operating conditions encountered by arctic offshore structures. Two of the concretes had a design compressive strength of 49 MPa (7,000 psi), one with fly ash and the other with silica fume. The third concrete had a design strength of 63 MPa (9,000 psi) and contained silica fume. Salient properties of both fresh concrete and hardened concrete were obtained including chloride permeability. Measured data, where possible, were compared with theoretical values based on compressive strength. Where appropriate, modified relationships between measured compressive strength and other mechanical properties were suggested. Zhang and Gjorv's studies were focused on both the mechanical properties and the resistance of high-strength lightweight concrete (50 to 100 MPa) to water penetration and accelerated chloride penetration. Their results indicated that the permeability of high-strength lightweight concrete appeared to be very low but might be higher than that of normal weight concrete at a similar strength level. The permeability appeared to depend on the porosity of the mortar matrix than the porosity of the lightweight aggregate. There also appeared to be an optimum cement content for permeability. No direct relationship between water permeability and electrical conductivity was observed, but a direct relationship between water permeability and accelerated rate of chloride penetration was observed.

D. Effect of Mineral Admixtures It is generally recognized that the use of fly ash, silica fume, and GGBS will improve the pore structures and thus the durability of concrete. High performance concretes are produced often with large quantities of these mineral admixtures. Such applications not only will help improve the strength and durability characteristics of high performance concrete but also will help dispose more of the industrial by-products which are major environmental concerns.

The effect of mineral admixtures on the pore structure and chloride permeability of concrete was examined by both Geiker et al. [1991] and Torii and Kawamura [1991]. Using the AASHTO T 277 method, they found with mineral admixtures the concrete was much less permeable to chloride ions than the concrete without mineral admixtures independent of curing and environmental conditions. Torii and Kawamura also found that at the surface of concretes with a mineral admixture the hydration of portland cement was considerably depressed and coarse pores were developed when concrete with a mineral admixture were stored in a dry condition for a long time. However, at the depth of 5 cm from the surface of concrete specimen there was little change both in the degree of hydration and the pore structure.

Numerous similar studies [Malhotra 1990; Ellis et al. 1991; Dunstan et al. 1993; Dhir and Byars 1993a, 1993b; Dhir et at. 1993; Ozyildirim 1992, 1994; Ozyildirim and Halstead 1994; Bilodeau et al. 1994] have also been published in the past several years. These studies are generally project-specific with respect to constituent materials, curing methods, environmental exposures, and testing techniques. The results usually indicate the beneficial effects of the three commonly used mineral admixtures.

E. Comparative Evaluations The majority of the permeability studies reported in the literature are carried out usually by using more than one permeability test procedures. Parallel tests for air (or oxygen) permeability, water permeability, carbonation depth, chloride permeability, and/or resistivity are often conducted to develop correlations among these characteristics. For a specific investigation, acceptable correlations can usually be established. However, the correlations are dependent on the specific parameters involved in each study and therefore cannot be generalized. Examples of these comparative evaluations can be found in the publications by Galligo and Rodriguez [1991], Torrent et al. [1991], Zhang and Gjorv [1991b], Hakkinen [1992], Dunstan et al. [1993], and Dhir et al. [1993a, 1993b].

A collection of useful data on rapid chloride permeability, electrical resistivity, corrosion rate, and water absorption of commercially available high-strength concretes has been presented by Burg and Ost [1992]. The ranking of the different mixes tested in terms of water absorption is identical to that of rapid chloride permeability. The measured concrete resistivity is also consistent with the indicated corrosion rate for each mix tested.

4.4.2 Freeze – Thaw

Damage of concrete under repeated cycles of freezing and thawing (frost attack) is a major problem of durability. In the previous state-of-the-art report [Zia et al. 1991], the mechanism with which freezing and thawing damages concrete has been discussed and some of the earlier reseaches have been summarized.

For the normal strength concrete, entrained air of 4 to 8% by volume of concrete provides an effective defense against frost damage and the exact amount is dependent on the maximum size of the coarse aggregate, provided that the coarse aggregate itself is frost resistant [Zia et al. 1993a]. The optimum spacing factor of the air voids should be no more than 0.2 mm (0.008 in.) and the air voids should be small with their diameter being in the range of 0.05 to 1.25 mm (0.002 to 0.05 in.) to ensure that the required spacing factor is obtained with low air contents.

By using concretes with different entrained air void systems subjected to long and short cycles of freezing and thawing in 4% sodium chloride solution, Stark [1989] found that the long freeze-thaw cycles were more severe than the short freeze-thaw cycles for same number of cycles, even where air void spacing factors were no greater than 0.2 mm (008 in.). His findings agree with the ice accretion theory of frost damage in concrete. However, Attiogbe et al. [1992] noted that properly air-entrained concretes containing superplasticizers can have adequate freeze-thaw resistance at calculated spacing factors greater than the ACI recommended maximum spacing factor of 0.008 in. (0.2 mm). Their test data for concretes made with or without superplasticizers showed that virtually all the concretes with adequate resistance under freeze-thaw testing had specific surface values less than the ACI recommended minimum.

Siebel [1989] found that when superplasticizers were used in a high workability air-entrained concrete, the number of pores with a diameter up to 300 mm decreased, while the content of pores larger than 500 mm and the bubble spacing factor increased. Small pores coalesced and formed larger pores. Although the air content of the fresh concrete was sufficient, the superplasticized concrete sometimes had a spacing factor above 0.20 mm. For this reason, concrete with superplasticizers did not always have adequate freezing and thawing resistance. The de-airing agents contained in the superplasticizers were found to have a considerable effect.

At the present time, there is no clear direction as to whether all high-strength concrete will require air-entrainment and the necessary air-void parameters. Some argue that with low W/CM ratio and mineral admixtures, the amount of freezable water in high-strength concrete would be low and its pore size would be decreased to the extent that water in the pore cannot freeze. However, in their study of high performance concretes, Zia et al. [1993c, 1993d, 1993e] found that 5% entrained air was required to achieve a higher level of frost resistance than that required by the ASTM C 666, Procedure A (i.e., a durability factor of 80% versus 60%).

Galeota et al. [1991] studied the freeze-thaw resistance of both non-air-entrained and air-entrained cylindrical specimens (38 specimens of each kind). Half of each kind contained 20% silica fume by weight of cement. After every 50 cycles of freezing and thawing, the dynamic modulus of the specimen was measured and compared with that of the control specimen to determine the durability factor until the durability factor was reduced to 40 per cent. Each cycle of freezing and thawing lasted 12 hours during which the specimens were cooled from 5oC ± 2oC to -25oC ± 2oC in air and then warmed to 5oC ± 2oC in water. They found that the specimens with silica fume, both non-air-entrained and air-entrained, performed very poorly with their durability factors being reduced to less than 40% in 50 and 100 cycles of freezing and thawing respectively. After 250 cycles of freezing and thawing, the non-air-entrained specimens without silica fume achieved an average durability factor of 60%, but the air-entrained specimens without silica fume showed no deterioration at all. It was concluded that the very poor performance of the specimens was probably due to the high void spacing factor because of very high silica fume content, even though the air-entrained specimens had 5% air in the fresh state.

The freeze-thaw resistance of both air-entrained and non-air-entrained concretes was also investigated by Kashi and Weyers [1989]. They tested specimens from 27 batches of high-strength concrete containing silica fume with W/CM ratios of 0.25 and 0.32, and with and without entrained air. The tests were conducted in accordance with ASTM C 666, Procedure A. Another set of similar specimens were moist cured for 28 days instead of 14 days to determine the effect of curing time. The results showed that non-air-entrained high-strength concrete with W/CM ratio of less than 0.30 was frost resistant regardless of the length of curing time. Non-air-entrained high strength concrete with W/CM ratio of 0.32 was durable if silica fume was not used.

Li et al. [1994] investigated the freezing and thawing (F/T) durability of non-air-entrained cement pastes, mortars, and concrete. The test specimens (with four different W/C ratios of 0.24, 0.27, 0.30, 0.33) were cured in 95 ± 3% relative humidity and a temperature of 23oC ± 1oC until testing at 14, 28, or 90 days. Freezing and thawing was performed according to ASTM C 666, Procedure A. The F/T durability of non-air-entrained pastes and mortars was evaluated by measuring the decrease in compressive strength, but the F/T durability of non-air-entrained concrete was determined by the method of ASTM C 666, Procedure A. For comparison purposes, the relative dynamic modulus of the mortar was also measured. At the W/C ratio of 0.24, both the paste and mortar showed excellent F/T resistance at 0, 5, and 10% silica fume levels. When the W/C ratio was higher than 0.24, the paste and mortar durability was significantly reduced. Similarly, at the W/C ratio of 0.24, the non-air-entrained concretes were F/T durable regardless of the silica fume and total cementitious content but the durability was decreased for concretes with higher W/C ratios. The results indicated that factors other than the W/C ratio had little influence on the F/T durability and the critical W/C value was 0.24. The damage in the paste was characterized by surface scaling while in the mortar and concrete a few large cracks led to final failure. [Authors' Note: The research described by Li et al. covers only non-air-entrained pastes, mortars, and concretes, in spite of the misleading title of their publication.]

The F/T durability of non-air-entrained high-strength concrete with a constant W/CM ratio (0.35) and a fixed amount of silica fume (10 % by weight of cement) was investigated by Cohen et al. [1992] to evaluate the effects of the duration of curing in saturated lime-water for 7, 14, 21, and 56 days prior to the freezing and thawing cycles. The aggregates used in the investigation were frost resistant. Therefore, the failure of the non-air-entrained concrete specimens could be attributed only to cracking of the paste. It was found that silica fume modified the frost resistance mechanism of the paste in the concrete. All specimens failed when tested according to the ASTM C 666, Procedure A, using 60% relative dynamic modulus as the failure criterion. The test data suggested the possible existence of a critical curing period of 14 to 21-day in saturated lime-water, which was associated with the largest gains in length and mass, the largest drops in bulk density and compressive strength, and the lowest number of cycles to failure (or lowest durability factor). There were some improvements to the frost resistance properties when the duration of curing was decreased to 7 days or increased to 56 days, but the improvements were insufficient for the concrete to meet the standard test requirements. An interesting observation was made in that the induced damage to the concrete after 300 cycles of freezing and thawing was indicated primarily by a significant drop in the dynamic modulus (from 6.5-million psi to 300,000 psi) rather than a major reduction in compressive strength (with highest compressive strength of 11,000 psi at 0 cycle and lowest strength of 8,000 psi after 300 cycles). The researchers concluded that while an explanation for this phenomenon requires further study of the microstructure, it appears clear that the concrete deterioration process during the freezing and thawing cycles had reached a stage that the normal relationship between the compressive strength and the modulus could no long hold true.

Pigeon et al. [1991] also attempted to determine the influence of various parameters on the limiting value of W/CM ratio below which air entrainment is no longer required for good freezing and thawing resistance. Seventeen high-strength concretes were made using portland cement with and without silica fume, and tested for frost resistance according to ASTM C 666, Procedure A. The parameters included the type of cement, the type of aggregate, and the length of the curing period. The test results, along with previously published data, indicated that the limiting value of W/ CM ratio could be higher than 0.30 in certain cases, but equal to or less than 0.25 in others, depending particularly on the characteristics of cement. They suggested that more research is needed before the values can be used as guidelines, since field exposure conditions differ substantially from laboratory testing conditions, and because the air void spacing factor of non-air-entrained field concretes could be significantly higher than that of laboratory made concretes. It is interesting to note that Burg and Ost [1992] tested five commerically available non-air-entrained high-strength concretes for their freeze-thaw resistance. The W/CM ratios of the five concrete mixes ranged from 0.22 to 0.29, their 28-day compressive strengths of moist cured 150 x 300 mm (6 x 12 in.) cylinders ranged from 11,400 psi (80 MPa) to 17,250 psi (121 MPa), and their silica fume contents ranged from 0 to 16%. Out of the five concretes, only the one with the highest strength, the lowest W/CM ratio, and the largest amount of silica fume sustained the freezing-thawing test without deterioration for over 1,400 cycles. The other four concretes all failed to meet the requirements of ASTM C 666, Procedure A.

In lieu of silica fume, high-reactivity metakaolin has been found as an effective mineral admixture to produce high-strength concrete with good freeze-thaw resistance [Caldarone et al. 1994].

In the past several years, there has been an increasing interest in using large quantities of fly ash or GGBS to produce high performance concrete. The durability of such concrete becomes an important issue since these mineral admixtures are often used to replace a major portion of the portland cement in the concrete. A series of recent studies reported in Canada [Langley et al. 1989; Malhotra 1989; Malhotra and Painter 1989; Malhotra 1990; Bilideau et at. 1994; Bilodeau and Malhotra 1994] showed that air-entrained high-volume fly ash concrete exhibited excellent frost resistance based on the ASTM C 666, Procedure A test. The performance of the concrete was also very good against chloride-ion penetration based on the AASHTO T 277 test. The only exception was the deicing salt-scaling test (ASTM C 672) in which the performance of the concretes investigated was less than satisfactory. The concrete mixes contained class F fly ash (about 56% of the total cementitious materials and W/CM of 0.30) or GGBS (up to 50% of the total cementitious materials and W/CM of not more than 0.55) with roughly 4 to 6% entrained air and a large dosage of superplasticizer for good workability. On the other hand, using class C fly ash, Nasser and Lai [1993] found that the use of high percentage of fly ash in concrete (35 to 50%) reduced its resistance to freezing and thawing even though it contained about 6% air and was cured in water for 80 days. However, concrete containing 20% fly ash gave satisfactory performance if its air content and strength were comparable to control concrete which contained no fly ash.

Several recent studies on the freeze-thaw durability of air-entrained concrete for marine and arctic construction have been conducted. Moukwa et al. [1989] tested concrete with W/C = 0.44 and 4% air in both fresh and seawater. Two laboratory procedures were used, one simulating the field freeze-thaw conditions the concrete undergoes in the tidal zone and the other similar to ASTM C 666, Procedure A. The results of the study suggested that surface effects would probably play an important role in the deterioration of concrete under arctic conditions. Whiting and Burg [1991] tested high-strength lightweight concretes produced from two different lightweight aggregate sources subjected to a variety of freezing and thawing test procedures and conditioning methods. The concrete strengths ranged from 54 to 73 MPa (7,700 to 10,400 psi) and their unit weight varied from 1,920 to 2080 kg/m3 (120 to 130 lbs/ft3). Silica fume, fly ash, and GGBS were used in the different mixtures. The high-strength lightweight concretes exhibited excellent performance with virtually no degradation during the standard freeze-thaw testing. Prolonged exposure was needed to cause significant damage under simulated arctic offshore conditions. Durability was found to be a strong function of cumulative freezing and thawing cycles and moisture content, with saturation of aggregates prior to test leading to premature failure.

In a separate study, Tamura and Tazawa [1991] developed a new mixing procedure in which the lightweight coarse aggregate and 50% of the cement (plus fly ash or silica fume) were mixed first for one half of a minute with 50% of superplasticizer plus a portion of the water to make the W/CM = 0.15. The remaining half of the cementitious materials was then added and mixed for another half of a minute. The fine aggregate (sand) was then added to the mixer at the one minute mark and mixing continued for another half of a minute. At the 1-1/2 minute mark, all the remaining water and superplasticizer were added to the mixer and mixing continued untile it was complete at the 3-minute mark. This mixing procedure allowed the lightweight coarse aggregate to be enveloped first with relatively soft cement paste and then the second addition of cement absorbed the water from the first cement paste coating to ensure a stronger enveloping action. The proposed mixing procedure proved to be most effective in enhancing the freeze-thaw resistance of high-strength lightweight concrete studied. In addition, they study also indicated that there was a close relationship between the water content in the lightweight aggregate before mixing and the resistance to freezing and thawing. For the high-strength lightweight concrete containing fly ash with its air content of 7 to 8%, it was desirable to adjust the water content in the lightweight aggregate to no more than 8%. For the concrete with silica fume, adequate freeze-thaw resistance was obtained even with a water content of 14% in the lightweight aggregate.

To better control water absorption into the lightweight aggregate under high pumping pressure, Asai et al. [1994] developed a new lightweight coarse aggregate with its surface being coated by a high molecular paraffin. The coated aggregate made it possible to produce lightweight concrete with high durability against freezing-thawing, abrasion, and fire.

4.4.3 Scaling

Scaling is another problem of durability. It is caused by repeated application of deicing salts. Concrete surface damaged by salt scaling becomes roughened and pitted as a result of spalling and flaking of small pieces of mortar near the surface. Even high–quality concrete with adequate air entrainment can still suffer scaling by deicing chemicals.

The exact cause of scaling is not well understood but it is recognized that when deicing chemicals are applied to melt ice, the heat consumption causes a rapid drop in the temperature of the concrete just below the surface resulting in damages from the effects of rapid freezing or differential thermal strains. Furthermore, deicing chemicals can accumulate in the surface layer of the concrete, forming relatively concentrated solutions. When water stays on the concrete surface, it flows towards the concentrated chemical solution causing an osmotic action accompanied by hydraulic pressures. These pressures may, in turn, cause salt scaling.

Scaling is most likely to occur where there is a weak layer of paste or mortar at or near the concrete surface. The best prevention of scaling is to eliminate the weak layer of material by proper mix design and good construction practice in placing, finishing, and curing. Over vibration, too much trowelling, and excessive bleeding should all be avoided. Well cured concrete pavements, allowed to dry for a period before deicing salts are applied, generally will have good scaling resistance.

Earlier studies on the scaling resistance of concrete containing silica fume and using Pyrament blended cement have been summarized in the previous state-of-the-art report [Zia et al. 1991]. It was concluded that based on the available research results curing history and moisture condition are important factors to field performance which can not be accounted for by the standard ASTM test procedures. Thus a need exists for new test procedures that will take into account these factors.

More recently, the deicer salt scaling resistance of high-strength concretes made with different cement was studied by Gagne et al. [1991]. Seventeen concrete mixtures were prepared with W/CM = 0.26 and 0.30, which produced high-strength concretes with a 28-day strength in the range of 60 to 90 MPa (8,600 to 12,860 psi). For W/CM = 0.26, Type I cement plus 6% silica fume was used. For W/CM = 0.30, two types of cement and a silica fume were used (Type III, Type III + 6% silica fume, Type I + 6% silica fume). Using sodium chloride as a deicer, all specimens were subjected to 150 daily cycles of freezing and thawing in accordance with ASTM C 672. After 50 cycles, the weight loss for all concretes was lower than 0.75 kg/m2 and after 150 cycles the weight loss was under 2 kg/m2. No clear relationship was found between the scaling resistance and the spacing factor. The test results, along with others, indicated that non-air-entrained high-strength concretes with good deicer salt scaling resistance could be produced with a portland cement plus silica fume and good quality coarse aggregate by using a W/CM of 0.30, even after only 24 hours of curing. With certain Type III cement, it is also possible to produce scaling resistant concrete with air entrainment and silica fume. In their study of non-air-entrained concretes, Li et al. [1994] also found no salt scaling, after 50 cycles of testing, for concretes with W/CM of either 0.24 or 0.27, but some scaling was observed for the specimens with W/CM = 0.30 and the specimens with W/CM = 0.33 suffered severe scaling. After 100 cycles of testing, the specimens with W/CM = 0.27 showed some scaling while those with W/CM = 0.24 still showed no sign of scaling.

In lieu of silica fume, high–reactivity metakaolin has also been used as an effective mineral admixture for high-strength concrete which proved to have satisfactory performance in scaling resistance [Caldarone et al. 1994].

The effect of curing and drying on salt scaling resistance of fly ash concrete was investigated by Bilodeau et al. [1991]. Concretes with 20 and 30% fly ash as cement replacement were produced with two types of aggregate, using W/CM of 0.35, 0.45, and 0.55. The test results showed that, with few exceptions, concrete with up to 30% fly ash performed well under the scaling test. Extended moist-curing or dry periods did not seem to affect significantly the scaling performance of the reference concrete as well as the fly ash concrete. However, when higher volume of fly ash (55 to 60%) was used in air-entrained concrete, the scaling performance of the concrete was less than satisfactory [Bilodeau et al. 1994].

4.4.4 Abrasion

Abrasion is wearing due to repeated rubbing and friction. For pavements, abrasion results from traffic wear. Adequate abrasion resistance is important for pavements and bridge decks from the standpoint of safety. Excessive abrasion leads to an increase in accidents as the pavement becomes polished reducing its skid resistance.

There is no generally accepted criterion for evaluating the abrasion resistance of conventional concrete. The lack of an abrasion resistance criteria is due to the fact that surface wear normally is not a controlling factor in pavement performance. If the pavement surface is provided an adequate texture depth during construction, its design is dictated by other requirements. An exception is in areas where the use of studded tires is permitted.

Abrasion resistance of concrete is a direct function of its strength, and thus its water-cement ratio and constituent materials. High quality paste and strong aggregates are essential to produce an abrasion resistant concrete.

The superior performance in abrasion resistance of concretes made with Pyrament blended cement was summarized in the previous state-of-the-art report [Zia et al. 1991]. Unfortunately, Pyrament cement is no longer being produced and not available to the construction industry.

The use of silica fume high-strength concrete with low W/C ratio for the repairs of abrasion-erosion damage of in the stilling basin at Kinzua Dam and in the concrete lining of the low-flow channel, Los Angeles River was described by McDonald [1989]. It was shown that silica fume offers potential for improving many properties of concrete which are particularly beneficial in repair of hydraulic structures.

The experimental work on the abrasion resistance of concrete pavements subjected to heavy traffic from studded tires was discussed in a paper by Gjorv et at. [1990]. Their test results indicated that if the concrete strength were increased from 50 MPa (7,200 psi) to 100 MPa (14,400 psi), the abrasion of the concrete would be reduced roughly by 50%. At 150 MPa (21,600 psi), the abrasion of the concrete was comparable to that of high quality massive granite. When compared with a high quality asphalt pavement, the abrasion resistance of the very high-strength concrete represents an increase in the service life of the pavement by a factor of nearly ten.

A unique piece of equipment, called TEREDO, for in-situ assessment of abrasion resistance of concrete has been developed by researchers at University of Belfast, Northern Ireland [Montgomery et al. 1989]. It is designed for accelerated abrasion test by using rotating wheels. Three sets of spiked steel dressing wheels are mounted on radial arms around a central spindle rotating at a constant speed of 50 rpm by a small DC motor with a variable speed control. The central shaft is attached to a center-hole jack allowing a force to be applied to the spindle through the jack, thus enabling the rotating wheels to abrade the surface. The machine is so designed that the dressing wheels will follow the contours of the test surface, including any peaks and troughs formed by its abrading action. The depth to which the test surface has been abraded is measured at 8 points along the abraded annular path after 15 minutes of operation. This recorded depth is used as a measure of the abrasion resistance of the surface. The investigation into the sensitivity of the apparatus to the variables of concrete mixes indicated that TEREDO produces test results which relate to the strength of the concrete. Therefore, correlations can be established between the measured abrasion depth and water/cement ratio, curing regime, and the method of surface finish.

Laplante et al. [1991] conducted an experimental program using the ASTM C 779 test method to determine the abrasion resistance of concrete with silica fume addition, W/CM ratio, and coarse aggregate type as variables. They found that coarse aggregate was the most important factor, followed by W/CM ratio in rank, affecting the abrasion resistance of concrete. The abrasion resistance of concrete is strongly influenced by the relative abrasion resistance of its mortar and coarse aggregate. When the coarse aggregate and mortar have nearly the same abrasion resistance, the surface wear of the concrete would be fairly uniform and the concrete can present serious skidding and slipping problems when wet. When the W/CM is very low, it can make the concrete almost as abrasion resistant as high-performance rocks.

Also using the ASTM C 779 test method, Dhir et al. [1993] conducted a study to investigate the abrasion resistance of concrete as affected by the curing method (water vs air), length of curing, variable workability, and variable maximum aggregate size. By measuring also the initial surface absorption (ISAT) at the 10-minute mark, a linear correlation was established between the abrasion of concrete and its initial surface absorption at 10 minutes, see Fig. 4.20.

Abrasion resistance of high-strength concretes containing chemical and mineral admixtures was investigated by de Almeida [1994]. Ten concrete mixtures were evaluated for their abrasion resistance according to a Portuguese Standard, which is similar to the Brazilian Standard and the German Standard

DIN 52108, using the Dorry apparatus, Fig. 4.21. The compressive strength of the concrete varied from 60 to 110 MPa (8,700 to 15,950 psi) at 28 days, and the W/CM varied from 0.24 to 0.42. The concrete mixtures contained silica fume, fly ash or natural pozzolan, with or without a superplasticizer, with workability being kept constant, Fig. 4. 22. From the test results, it was concluded that the abrasion resistance of concrete generally varies inversely with the W/CM ratio (see Fig. 4.23), the porosity, and the cement paste volume in the concrete. Therefore, by using superplasticizer to reduce substantilly the W/CM ratio, the abrasion resistance of concrete would be improved considerably. Introducing mineral admixture without using superplasticizer would reduce the abrasion resistance of concrete since more water would be needed to maintain a constant workability. It is noted that the results of the study should be applied to high-strength concrete mixtures only. However, even the least abrasion resistant concrete produced in the study resulted in surface wear that was only 17% of ordinary concrete.

It is worth noting that Fwa [1989] developed a laboratory test procedure for cyclic wetting and drying to simulate the weathering effects of Singapore's climate on the wearing resistance of concrete pavements. Tests were conducted with a rotating drum on plain cement mortar specimens with and without weathering treatment. While no direct correlations were established between his test data and the field performance of in-service pavements, the test results did suggest that the test procedure could be useful for evaluation of the relative surface wear resistance of concrete pavement.

Abrasion resistance of concrete including rather high volumes of Class C fly ash (50% and 70% replacements of cement) was investigated by Naik et al. [1994]. A reference portland cement concrete was proportioned to produce 28-day strength of 41 MPa (5,950 psi). Abrasion tests were conducted by using the rotating cutter method of ASTM C 944. All the specimens made either with or without fly ash passed the abrasion resistance requirements of ASTM C 779, Procedure B. An accelerated test method, using the rotary cutter device having dressing wheels equipped with smaller size washers, was developed. A measured amount of standard Ottawa sand was added to the surface being abraded at one minute intervals. The results of the accelerated test showed lower abrasion resistance for high–volume fly ash concrete systems relative to no-fly ash concrete.

4.5 Strength Effects

Traditionally, interest in the strength and other properties of concrete has been focused on those at 28 days and beyond. In the past several years, there has been an increasing interest in the strength and other properties of concrete at ages less than 28 days. There are at least three factors which have contributed to this increased interest in early strength: (1) the fast-paced construction schedules that expose concrete to significant structural loads at early ages, (2) the development of specialty cements or admixtures which enable the achievement of higher strength at early ages, and (3) the recognition that long-term performance of concrete is greatly affected by its early-age history.

At the present time, there is no generally accepted definition of early strength. Any strength measured at ages less than the standard 28 days is regarded as early strength. Since the properties of concrete depend closely on the degree of cement hydration, one definition of early strength could be the strength at the age corresponding to 50% hydration of cement. For concrete made with ordinary portland cement (ASTM Type I) and cured at a standard temperature of 20oC, approximately 50% of the cement will hydrate within 3 days. In an Engineering Foundation Conference on Properties of Concrete at Early Ages [Carino et at. 1989], it was recommended that the early age could possibly be defined as the period during which the properties of concrete undergo rapid change.

A total of seventeen papers were presented in the Engineering Foundation Conference. The papers covered various topics including the following:

- Fresh concrete and early hydration
- Development of mechanical properties
- Test methods

- Structural performance at early ages
- Admixtures
- Effects of early-age history on long-term properties

The Conference presented four principal recommendations: (1) the influence of cement paste microstructure on long-term performance of concrete and the effects of early-age history on microstructure need to be understood; (2) codes and standards need to address concrete at early ages; (3) the importance of controlling the temperature rise in structures during early ages needs to be understood and methods for such control need to be implemented; and (4) increased efforts in education and technology transfer are required.

A classical study by Klieger on high early strength concrete with Type I and Type III cements and the later developments of Pyrament blended cement and Regulated Set cement have been summarized by the authors in the previous report [Zia et al. 1991]. In the past few years, a number of valuable researches have been reported in the literature.

Chengju [1989] compared various expressions for maturity of concrete with special regard to the influence of temperature. Various maturity functions and strength-maturity equations were reviewed by comparing with published data, and a parabolic maturity function was proposed. Also reviewed were the principle and performance of a commercial maturity meter, which is based on the evaporation of liquid. It was concluded that the maturity concept may serve as an effective means of predicting the early-age strength of in-situ concrete in spite of its inherent limitations.

Under the SHRP C-205 contract, Zia et al. [1993c, 1993d], developed mixture proportions of two options of very early strength (VES) and high early strength (HES) concretes using four different types of aggregates. Option A of VES concrete, using Type III cement, melamine-based superplasticizer, air entrainment agent, and calcium nitrite as accelerator, achieved a minimum compression strength of 14 MPa (2,000 psi) in 6 hours with W/C of 0.40. Option B of VES concrete, using a proprietary blended cement (Pyrament PBC-XT) without any other admixtures, achieved a minimum compressive strength of 17.5 MPa (2,500 psi) in 4 hours with W/C of 0.23. Both concretes were insulated immediately after casting in order to preserve the heat of hydration so as to speed up the maturity of the concrete. HES concrete, also using Type III cement, naphthalene-based superplasticizer, air entrainment agent, and calcium nitrite as accelerator, achieved a minimum compressive strength of 35 MPa (5,000 psi) in 24 hours with a W/C of less than 0.35 and without insulation. Sections of test pavements, as full-depth full-lane replacement or new construction, were installed in Arkansas, Illinois, Nebraska, New York, and North Carolina to evaluate the constructibility and long-term performance of HES concrete. Some of the pavements were insulated to mimic VES concrete. Satisfactory results were obtained from the field experiments.

Following the work of Zia et al., Whiting and his colleagues conducted field studies of full-depth pavement replacedments and bridge deck overlays for early opening of repaired highways (from 4 to 24 hours) in Georgia, Ohio, and Kentucky under SHRP C-206 contract. In a series of papers [Nagi et al 1994; Nagi et al. 1995; Whiting and Nagi 1994; Whiting et al. 1994], they reported the performance of 11 different concrete mixtures used for pavement repairs and 3 different concrete mixtures for bridge deck overlays. A variety of tests were performed including determination of water content by microwave drying, temperature development in test slabs, early strength gain predictions, long-term strength gain and elastic moduli, as well as durability testing. Usually more than one test methods were applied in each performance test. Based on these extensive field studies, they concluded that (1) concretes used for early highway opening applications showed a high rate of early strength gain and developed considerable heat during the first few hours after placement, (2) a variety of methods can be used to predict in-place strength gain of concretes designed for early highway opening applications, (3) most of the early opening mixtures continued to gain strength after placement up to a period of at least 90 days (the longest test period included in the study), and (4) questions still remained regarding durability of concretes designed for early opening applications. There were indications that microcracking in the concretes, possibly caused by thermal effect, may account for some of the poor performance in freeze-thaw testing. The use of calcium chloride should be avoided, as it contributes to reduced freeze-thaw resistance.

Another interesting and successful field application of high performance concrete was described by Lessard et al. [1994]. To accelerate the traffic reopening in 24 hours, and to minimize the loss of customers and revenue, high performance concrete was used for a routine concrete sidewalk at the entrance of a McDonald's restaurant. The air-entrained concrete was designed to have a compressive strength of 20 MPa (3, 000 psi) in 24 hours and achieved a compressive strength of 69 MPa (10, 000 psi) in 28 days. Economic comparisons indicated that the cost of the sidewalk using the high performance concrete was 16% over the cost of a 30 MPa (4,400 psi) normal concrete sidewalk.

To achieve early compressive strength, an accelerator is usually used in the concrete mix. Rear and Chin [1990] evaluated the performances of three commercially avialable non-chloride accelerating admixtures with five different Type I and Type I/II portland cements, two different Class C fly ashes and two different Class F fly ashes. Their test results showed that the three accelerating admixtures were all effective in increasing early compressive strength with both cement and cement-fly ash combinations. However, the range of performance with different cements, fly ashes, and addition rates points out the importance of testing with local materials to determine the most effective combination.

Early strength development of concrete containing high volume of fly ash has also attracted the attention of several investigators recently. Malhotra and Painter [1989] tested three series of specimens for early-age strength and freezing-thawing resistance. The cement content was kept constant at 145 ± 5 kg/m3 while the amount of Class F fly ash varied from 43% to 65% of the total cementitious materials. The W/CM varied from 0.28 to 0.42 and the air content varied from 4.0 ± 0.5% to 9.5 ± 1.0%. The strength properties of the concrete were determined at ages up to 91 days, and the freezing and thawing tests were conducted according to ASTM C 666, Procedure A. The test results showed that the compressive strength of the concrete increased with decreasing W/CM ratio and with increasing amounts of fly ash. The optimum amount of fly ash was about 58% of the total cementitious materials. The concretes investigated showed adequate early-age strength and satisfactory freezing-thawing resistance, but the test specimens suffered moderate to considerable surface scaling.

Naik and Ramme [1990a, 1990b] described their studies of using Class C fly ash to replace a part of cement in the concrete produced in precast/prestressed concrete plants. The objective was to develop optimum mixture proportions for producing high early strength concrete with high fly ash contents. It was found that fly ash replacement improved workability, decreased water demand, and increased strength while maintaining the high early strength of 34 MPa (5,000 psi) requirements of precast/prestessed concrete operations. Hammons and Smith [1990] examined the development of compressive strength and modulus of elasticity of concrete mixtures containing Class C fly ash in the amount of 25% to 50% of the total cementitious materials from one and one-half hours after final set to 14 days.

Sarkar et al. [1991] observed a sudden upsurge of mechanical strength at 12 hours in a silica fume/fly ash high-strength concrete and the cause for the sudden upsurge of strength was investigated and the microstructural characteristics of the concrete was determined. The concrete was subject to progressive hydration from 1 to 91 days. It was suggested that the early increase in strength in a silica fume/fly ash concrete might be explained by the dissolution of a certain amount of K_2O from the surface of a substantial number of fly ash particles in very early stage of hydration. In contrast, silica fume concrete (without fly ash) with identical cement, aggregates, and W/C did not develop such high early strength, though the trend was reversed from age one day.

Special cements and binder materials are often used to achieve eary concrete strength for special applications. Pyrament cement was used for runway repair at Yeager Airport inCharleston, West Virginia [Anon. 1990]. An extensive study of concretes made with Pyrament cement was conducted by Husbands et al. [1994], and a comparative evaluation of Pyrament cement and two other rapid-setting materials (Set 45 and Silikal R17AF) for rapid runway repair was performed by Wakeley et al. [1991]. In both studies, excellent performance of Pyrament cement as a binder was identified. Unfortunately, the producer of

Pyrament cement suspended its production and the material is no longer available to the transportation industry.

4.6 Interrelationships of Properties

Many of the mechanical properties of concrete discussed in the previous sections are interrelated and their relationships are expressed in quantitative terms. Some of the properties, however, may also be linked indirectly even though their relationships cannot be established explicitly.

In the previous report [Zia et al. 1991], four interrelationships of properties were discussed. They included the relationship between strength and strain capacity, between strength and permeability, between fatigue and tensile strain capacity, and between fatigue and permeability. The authors are not aware of any significant developments in this topical area in the past few years.

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Tables & Figures

Table 4.1

Concrete	Diameter (mm)	Curing	A (MPa)	B (Days)
	200	Air	34.3	0.69
	150	Air	32.6	0.98
35 MPa	100	Air	33.3	0.80
	100	Sealed	35.6	1.11
	100	Water	37.7	1.25
	200	Air	72.3	1.30
	150	Air	80.9	2.22
90 MPa	100	Air	79.9	1.45
	100	Sealed	95.0	2.35
	100	Water	99.8	2.52
	200	Air	100.0	2.10
	150	Air	107.0	2.20
120 MPa	100	Air	101.4	1.91
	100	Sealed	114.5	2.97
	100	Water	125.9	4.33

Table 4.2

Materials	Bond Strength, psi (MPa)	Test Temperature	Type of Failure
Dry Aggregate and Concrete			
Pyrament 505	2,720 (18.8)	73°F (22.8°C)	Bond Failure
Set-45	1,200 (8.3)	73°F (22.8°C)	Bond Failure
Silikal R17 AF	2,760 (19.0)	73°F (22.8°C)	Concrete Failure
Wet Aggregate and Concrete			
Pyrament 505	1,760 (12.1)	73°F (22.8°C)	Bond Failure
Set-45	1,260 (8.7)	73°F (22.8°C)	Bond Failure
Silikal R17 AF	600 (4.2)	73°F (22.8°C)	Bond Failure
Dry Aggregate and Concrete			
Pyrament 505	1,260 (8.7)	15°F (-9.4°C)	Bond Failure
Set-45	590 (4.1)	45°F (7.2°C)	Bond Failure
Silikal R17 AF	2,710 (18.7)	5°F (-15°C)	Concrete Failure

Table 4.3

Concrete Strength	High-Strength Concrete (95 MPa)				Control Concrete (42 MPa)			
Slippage, micro m	10		100		10		100	
Slip Gage	1	2	1	2	1	2	1	2
10 mm Bar	20.4	29.7	53.6	64.9	12.7	1.5	30.7	35.2
16 mm Bar	13.6	17.4	37.4	39.8	10.7	13.1	24.3	28.2
25 mm Bar	10.8	8.5	20.5	24.8	5.7	8.2	15.3	20.1
25 mm Smooth Bar	8.5	11.0	12.1	13.7	6.3	6.7	10.2	12.5

Table 4.4

Durability Classification Ranking	ISAT Measurement taken at 10 minutes ml/m^2/sec x 10^{-2}	Additional Considerations
1	<50	Chloride Diffusion With FA or GGBS, increase ranking by 2 divisions
2	51-70	
3	71-90	Carbonation With FA or GGBS, increase ranking by 1 division
4	91-110	
5	>110	Frost Resistance With FA or GGBS, increase ranking by 1 division

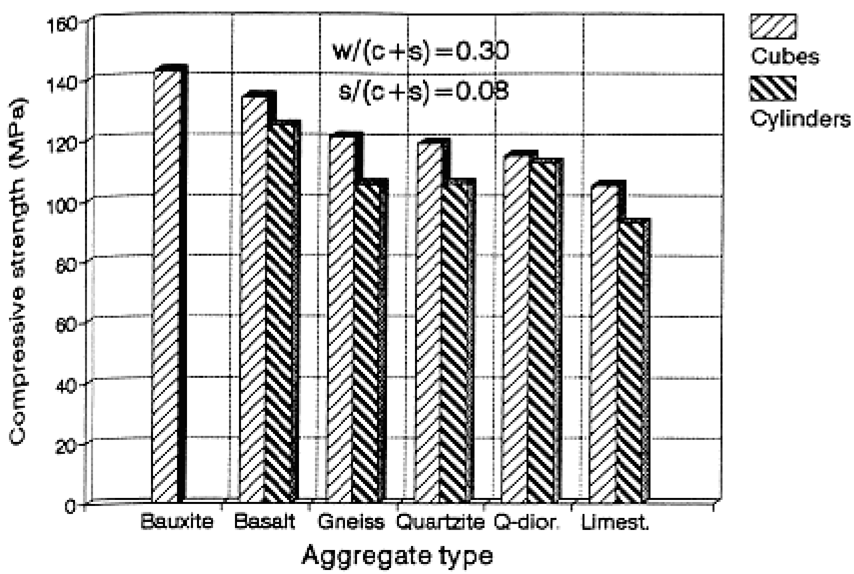


Figure 4.1 The effect of the aggregate type on compressive strength

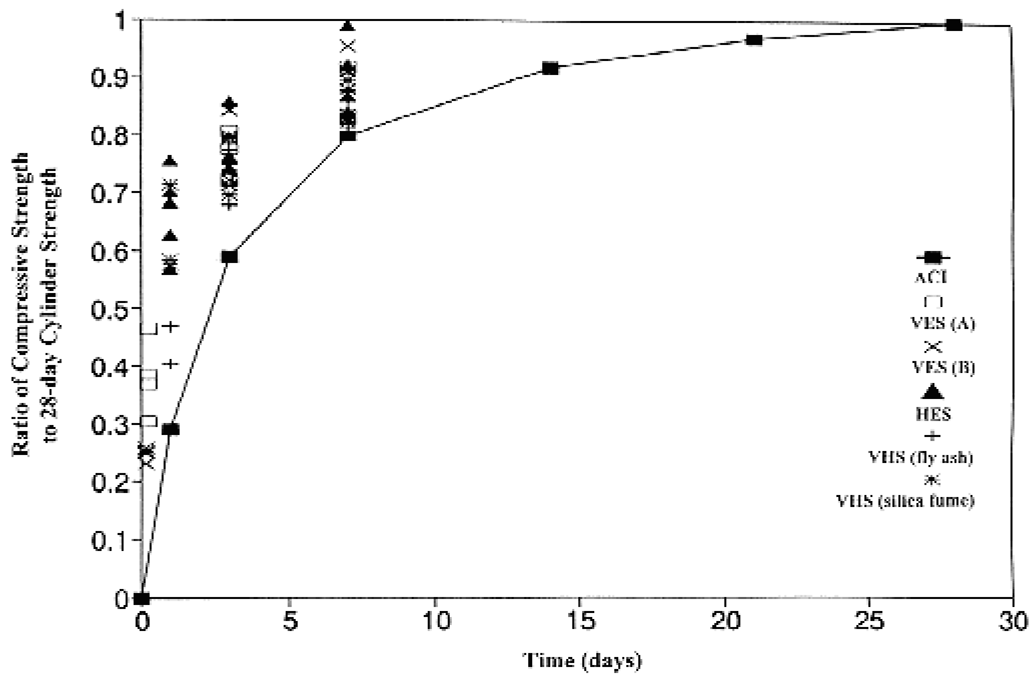


Figure 4.2 Variation of compressive strength of HPC with time

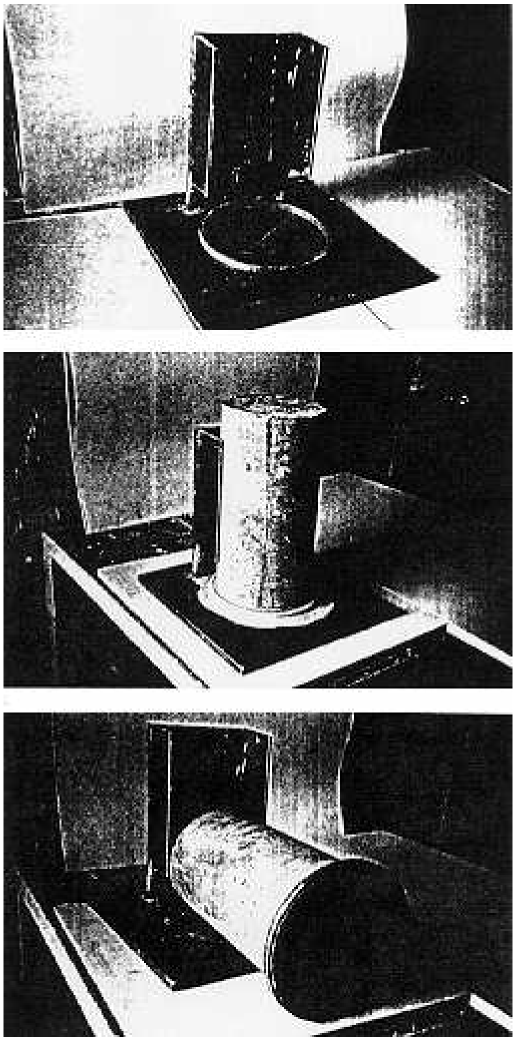


Figure 4.3 Confined capping apparatus for compressive strength testing of HPC cylinders

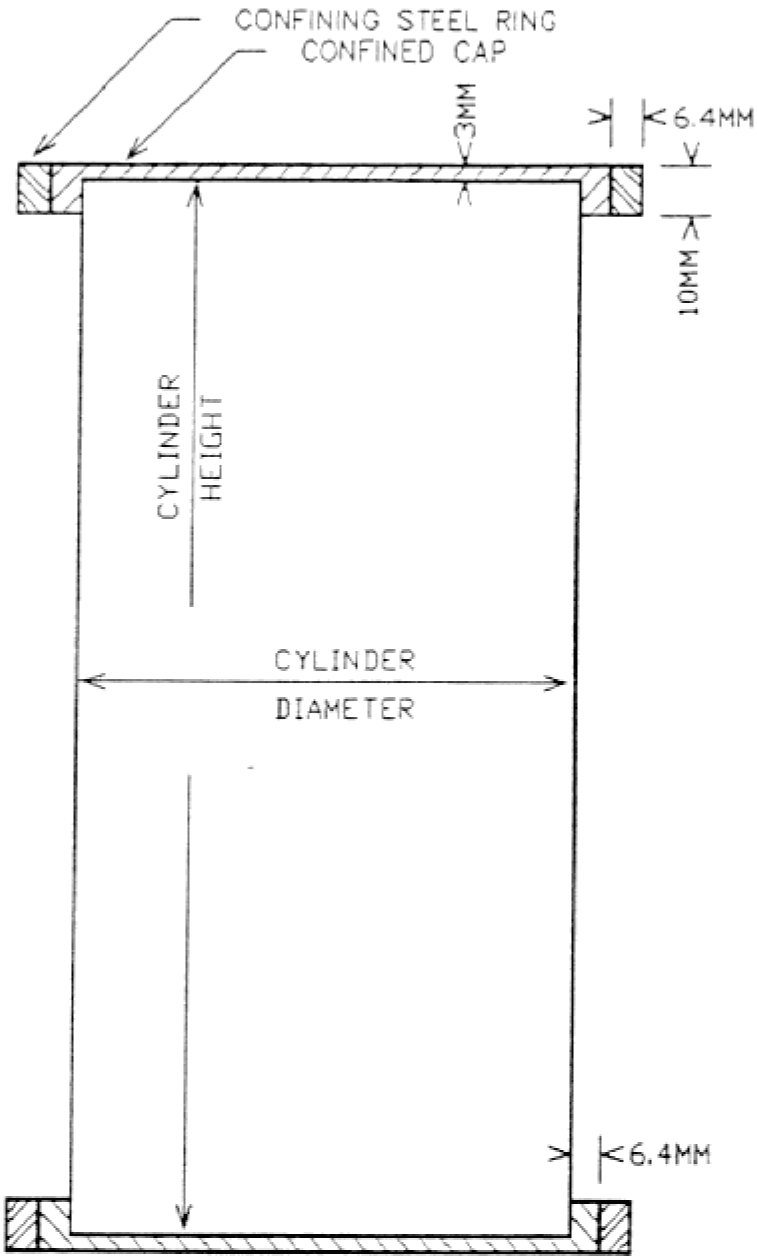


Figure 4.4 Vertical cross-section of a cylinder with prepared confined cap at both ends

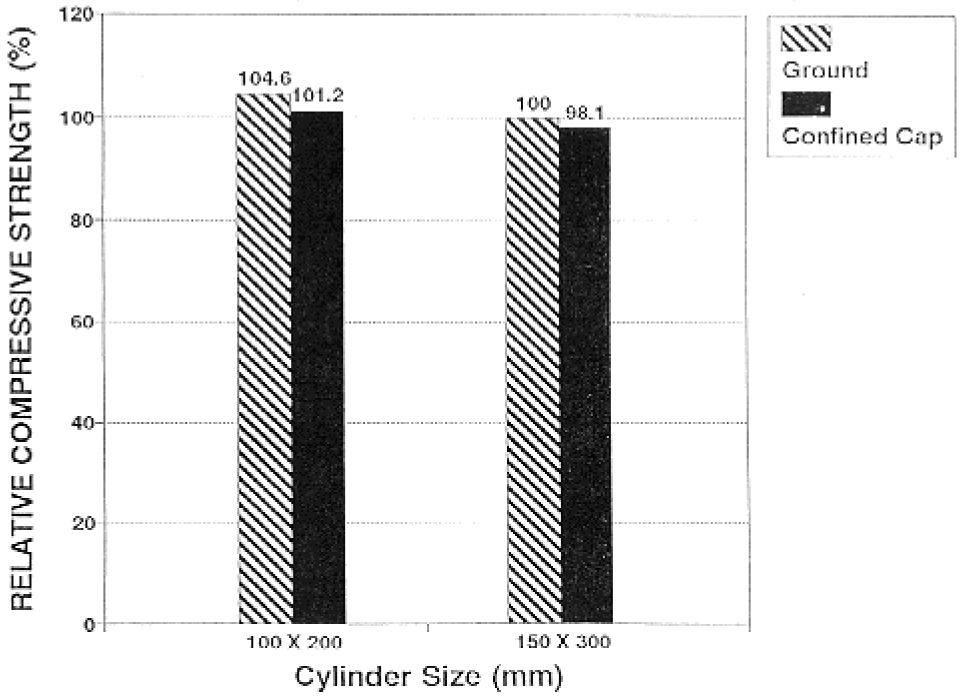


Figure 4.5 Average compressive strength of cylinders relative to strength of standard 150 x 300 mm cylinders with ground ends

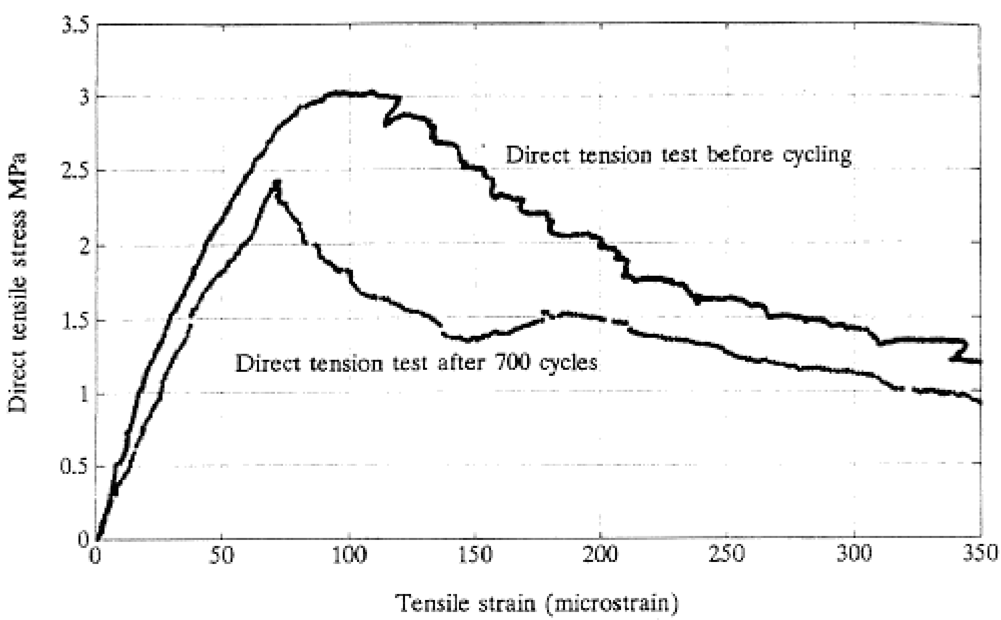


Figure 4.6 Stress-strain curves of high-strength concrete specimens under direct tension before and after 700 cycles of freezing and thawing

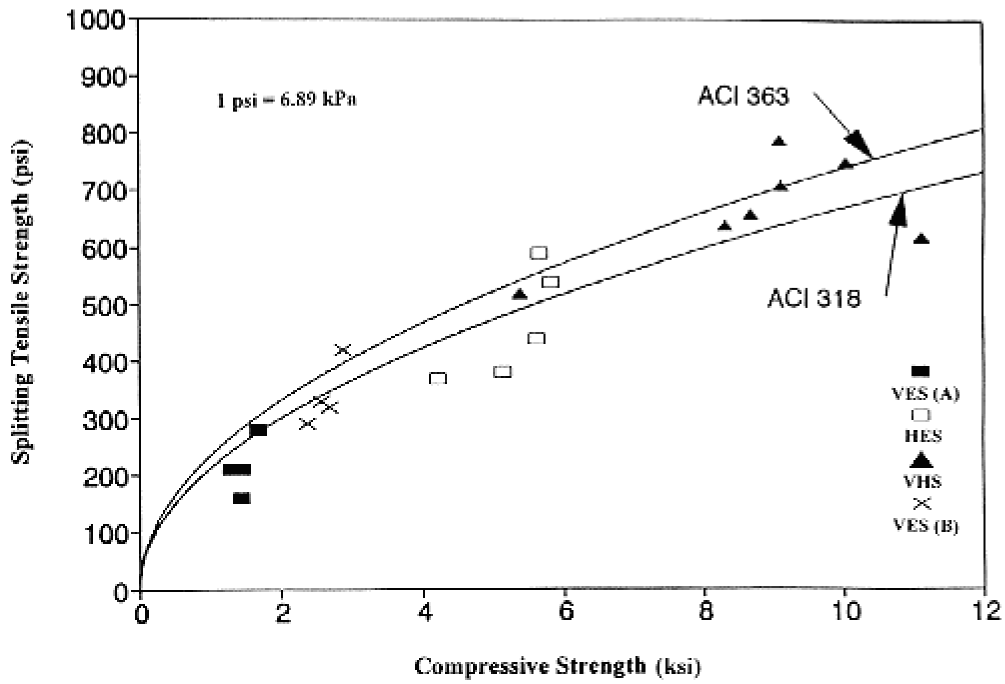


Figure 4.7 Relationship between splitting tensile strength and compressive strength of HPC

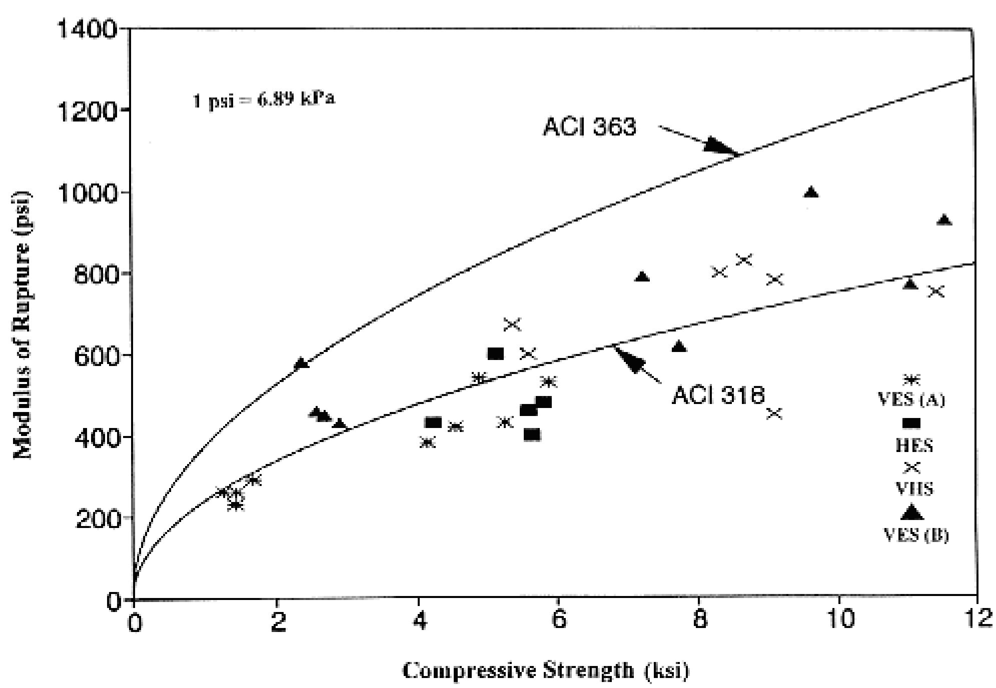


Figure 4.8 Relationship between modulus of rupture and compressive strength of HPC

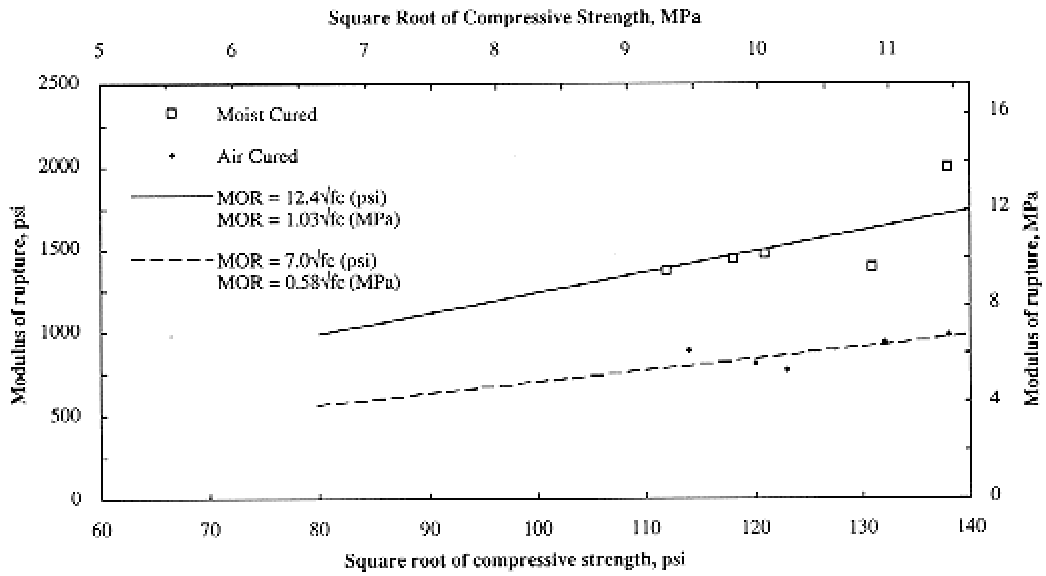


Figure 4.9 Modulus of rupture versus compressive strength of cylinder

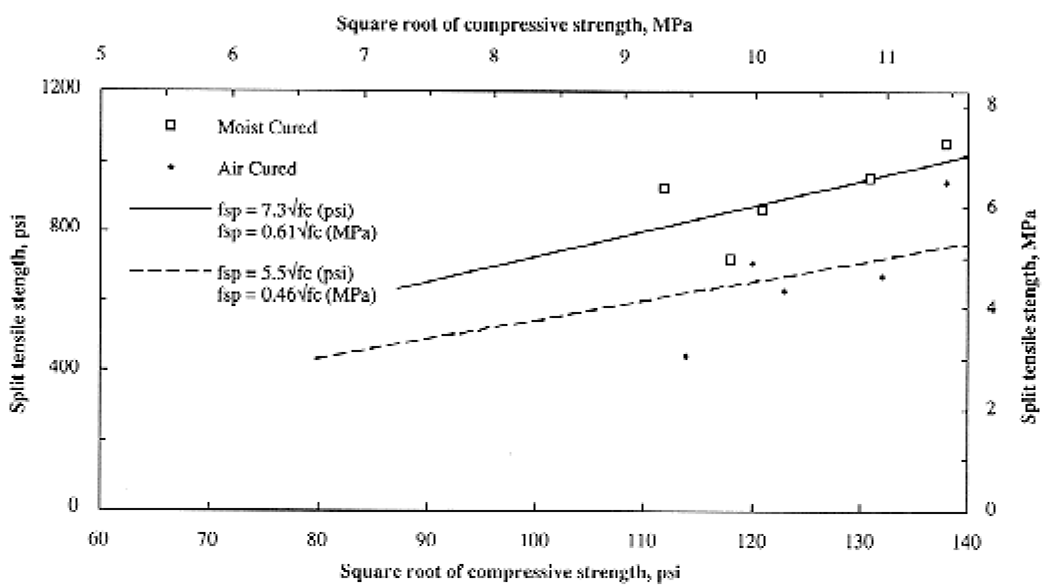


Figure 4.10 Splitting tensile strength versus compressive strength of cylinder

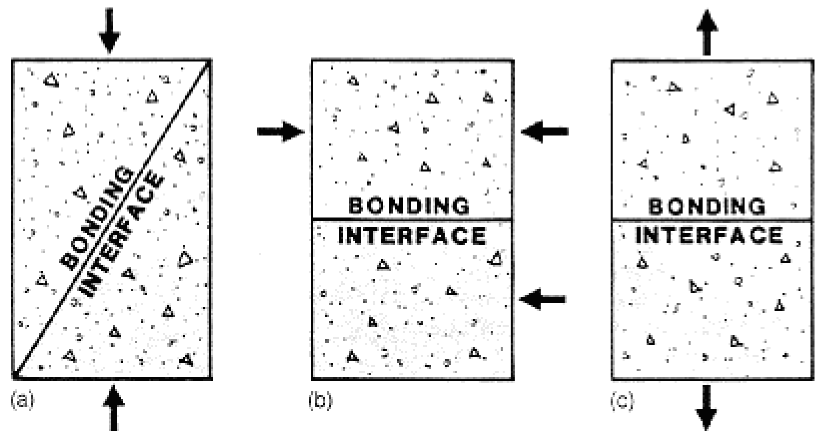


Figure 4.11 Three types of bond strength test: (a) slant shear test; (b) direct shear test; (c) direct tensile test

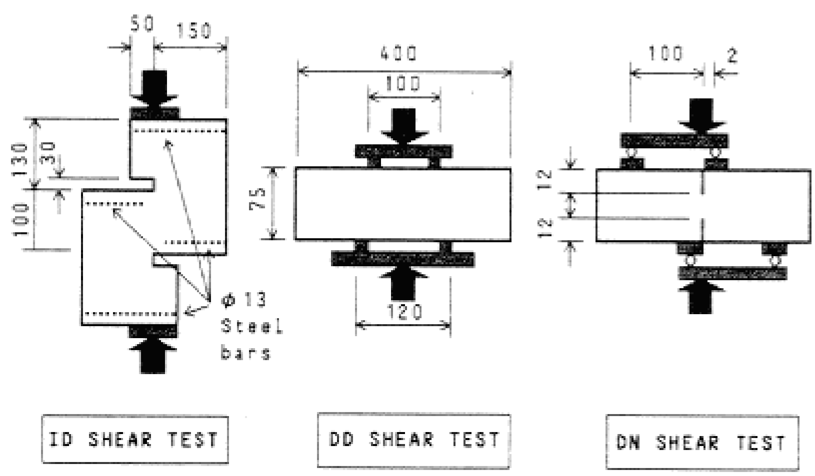
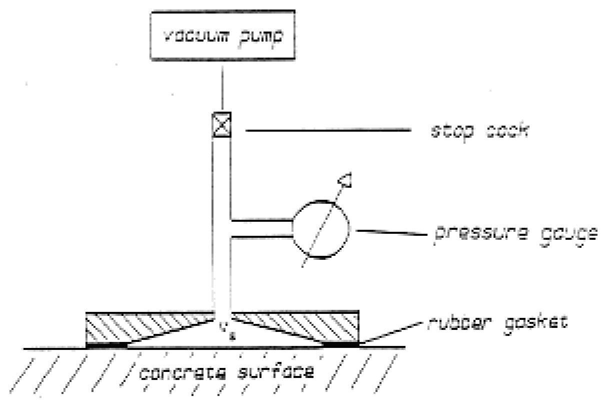
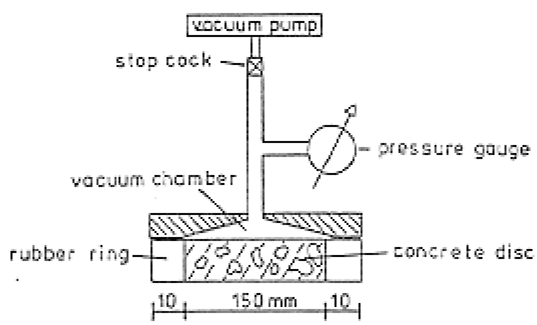


Figure 4.12 Three types of shear test and their loading arrangements (dimensions in mm)



(a) Application on a concrete surface



(b) Application on a precast concrete disk

Figure 4.13 Figg's method for permeability measurement

Figure 4.14 Schonlin & Hillsdorf's air permeability test

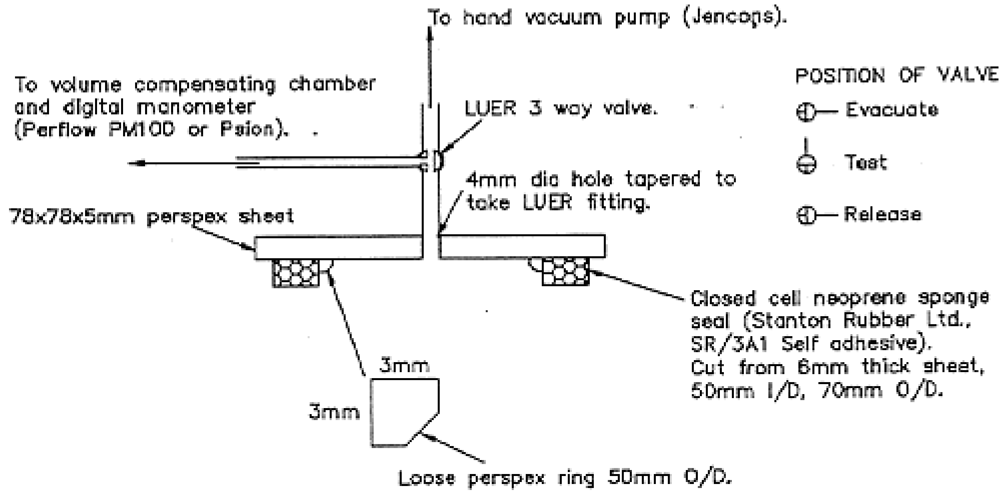
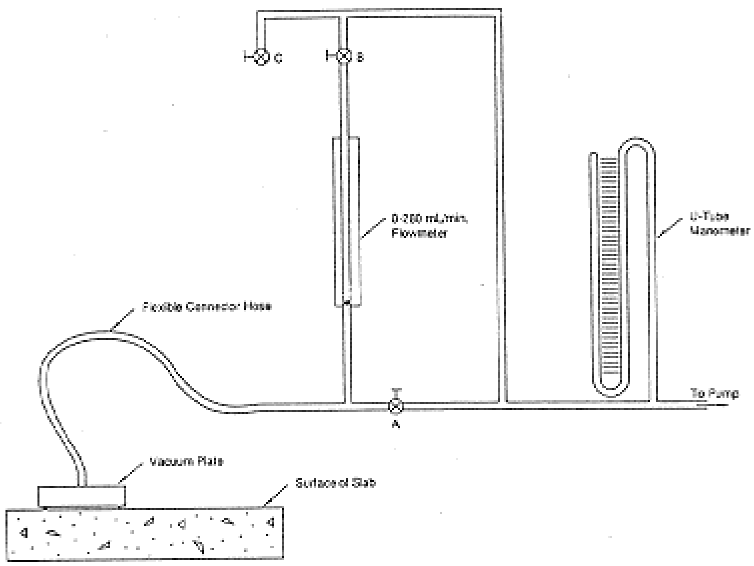
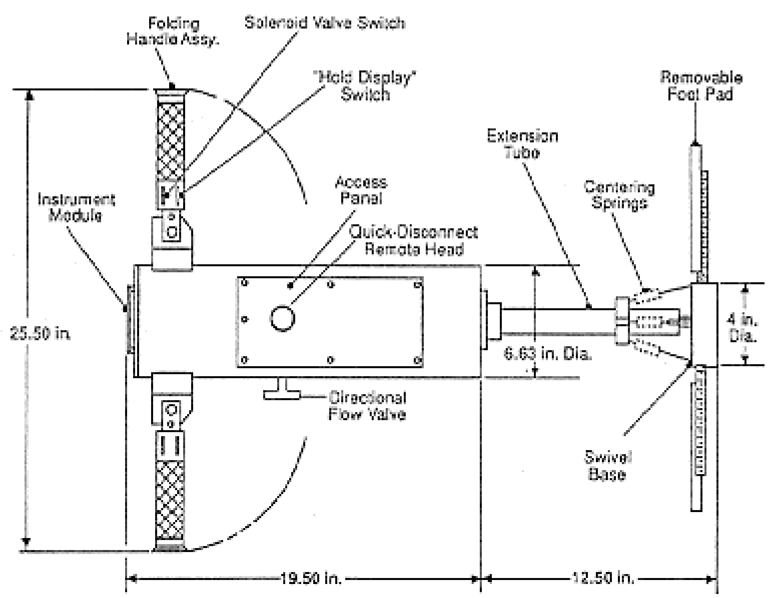


Figure 4.15 Leeming's air permeability test setup

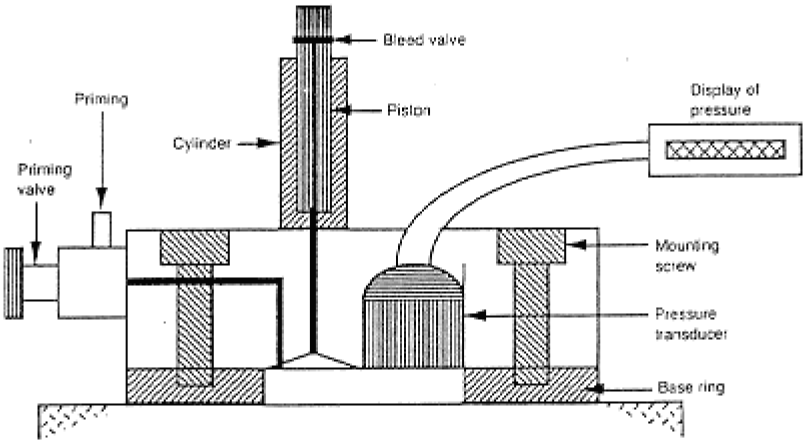


(a) Schematic view of the aboratory test setup

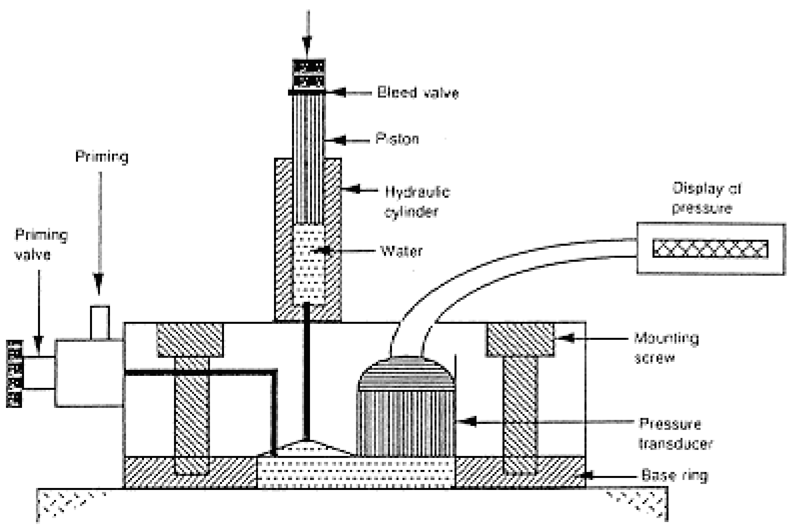


(b) Permeability indicator for field application

Figure 4.16 Whiting & Cady's air permeability test apparatus

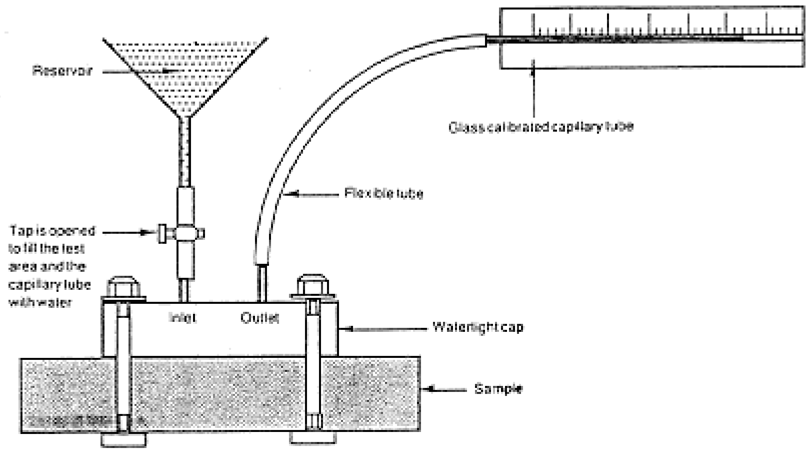


(a) Air permeability test setup

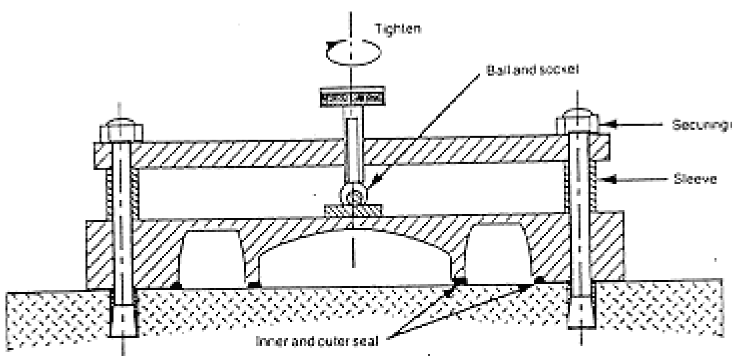


(b) Water permeability test setup

Figure 4.17 Autoclam apparatus for permeability tests

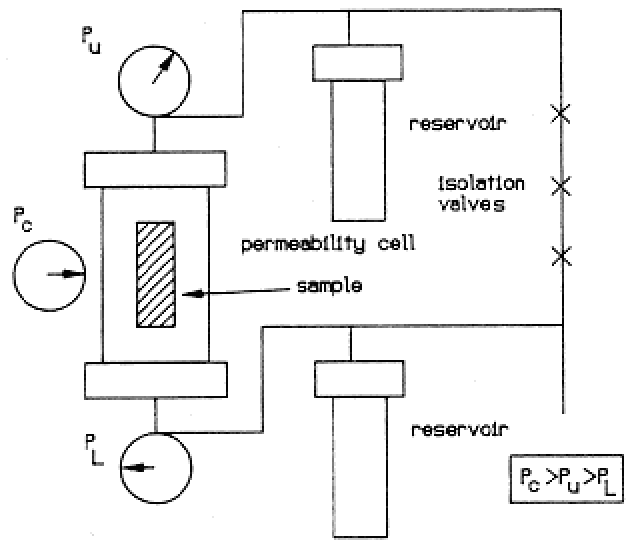


(a) ISAT test setup

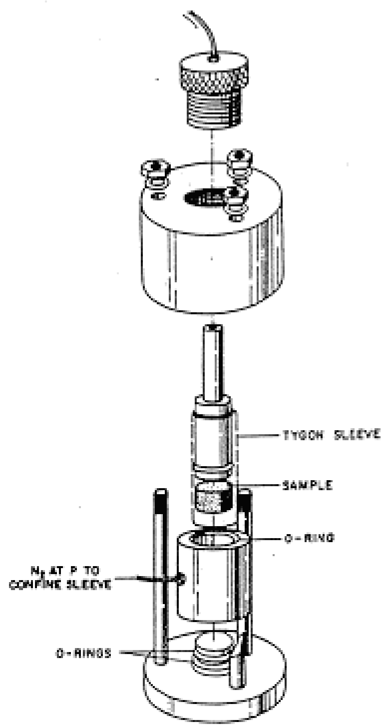


(b) Modified cap for ISAT

Figure 4.18 ISAT test apparatus



(a) Schematic view of pulsed permeability apparatus



(b) Exploded shop drawing of pulsed permeability apparatus

Figure 4.19 Roy's test setup for transient permeability measurement

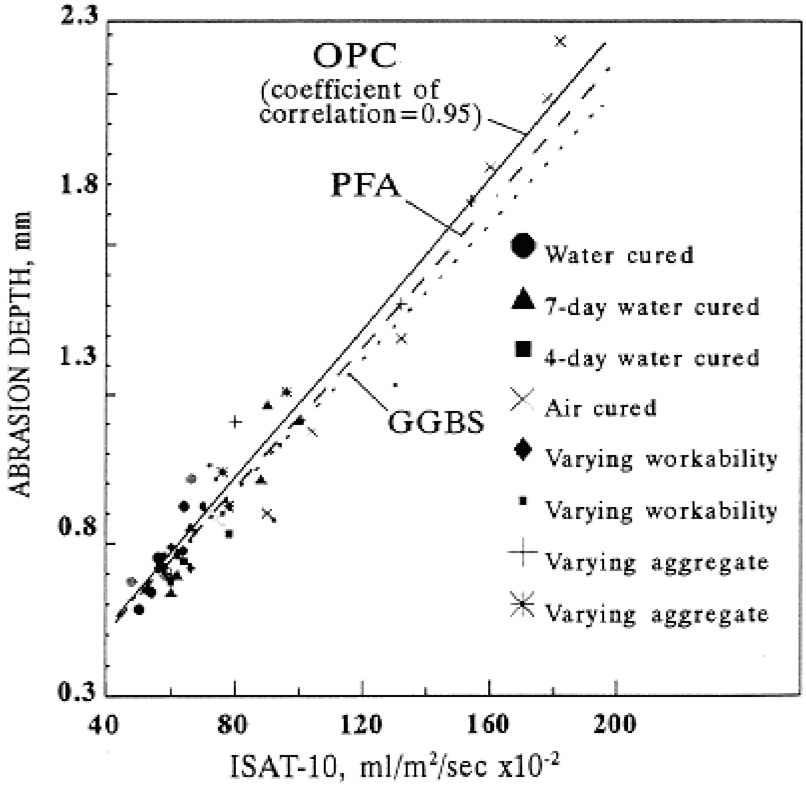


Figure 4.20 Relationship between depth of abrasion and the ISAT-10 reading

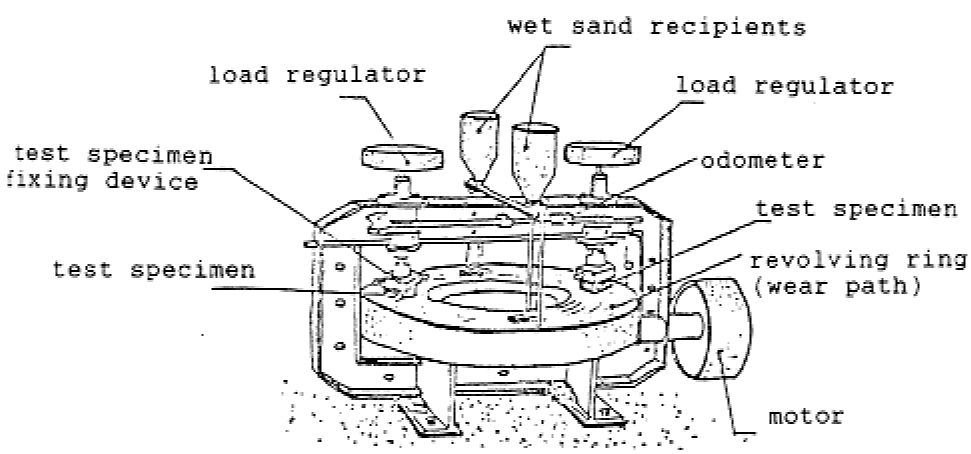


Figure 4.21 Schematic view of Dorry apparatus also known as Amsler-Lafond apparatus

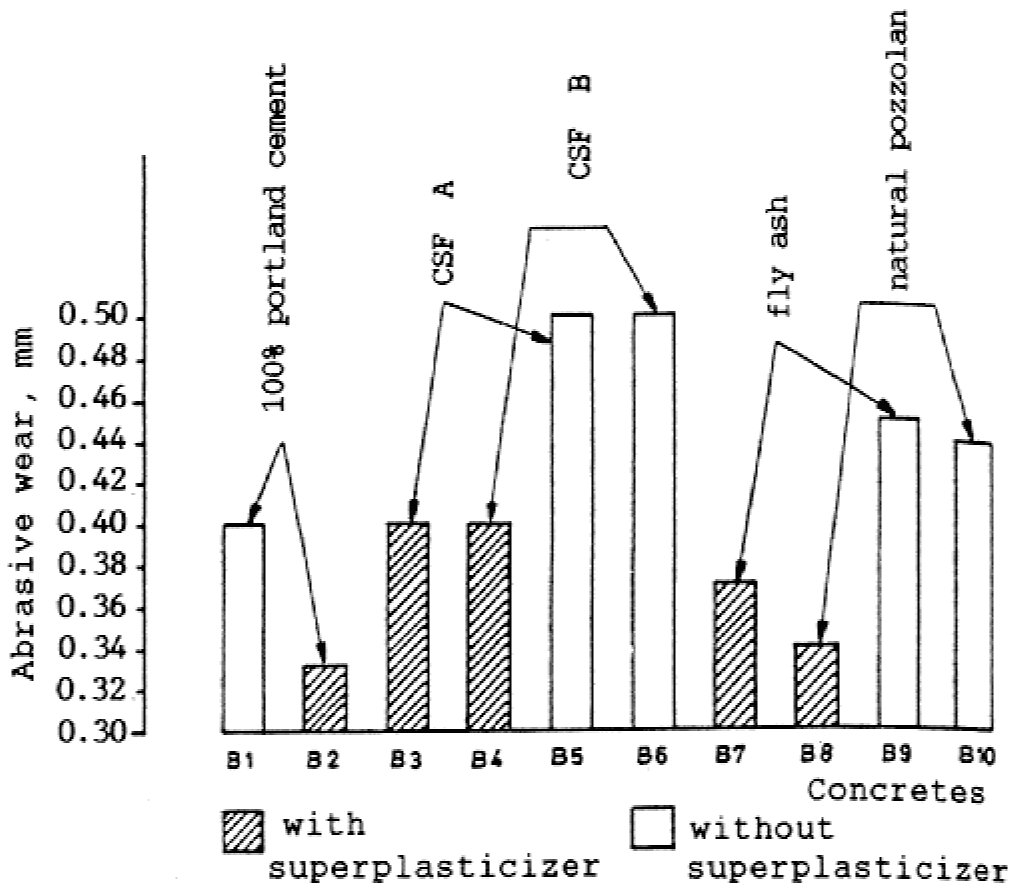


Figure 4.22 Abrassive wear of 10 different concrete mixtures tested by de Almeida

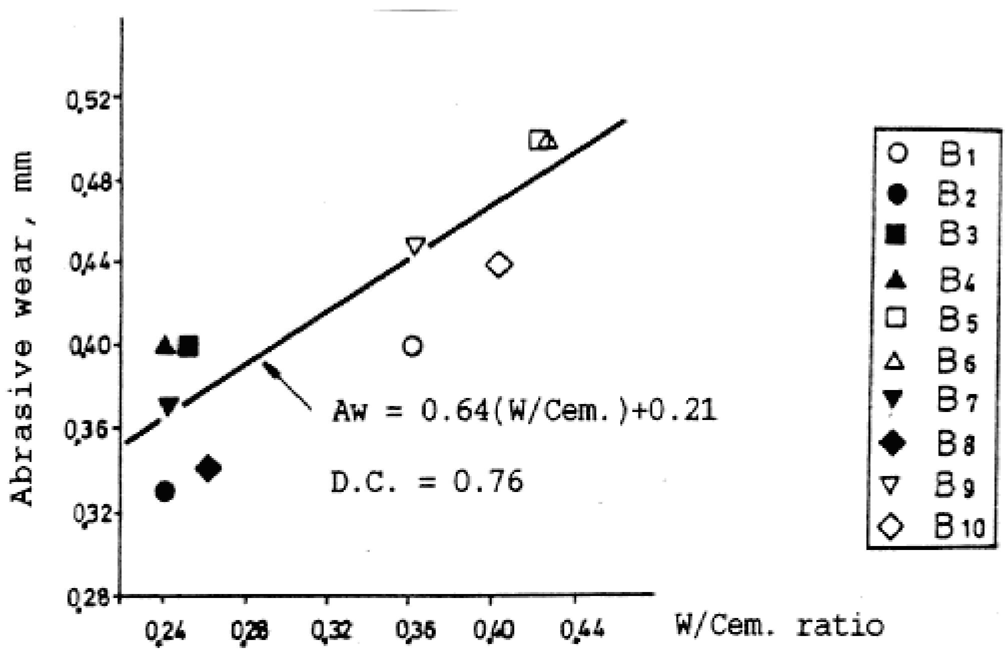


Figure 4.23 Correlation between abrassive wear (Aw) and water-cement ratio

BEHAVIOR OF FIBER-REINFORCED CONCRETE

5.1 Introduction

High-Performance Fiber-Reinforced Concrete (HPFRC) results from the addition of either short discrete fibers or continuous long fibers to the cement based matrix. Due to the superior performance characteristics of this category of HPC, its use by the construction industry has significantly increased in the last 5 years. A very good guide to various portland cement-based composites as well as their constituent materials is available in a recently published book [Balaguru and Shah 1992]. The book provides information on fabrication, mechanical and long-term properties of concretes with short discrete fibers. It also covers special topics such as fiber reinforced cements and slurry-infiltrated fiber concrete. In 1992, the first international workshop on high performance fiber reinforced cement composites (HPFRCC) was held in Mainz, Germany [Reinhardt and Naaman 1992].

For highway pavement applications, concretes with early strength are attractive for potential use in repair and rehabilitation with a view towards early opening of traffic. The work conducted on high early strength fiber reinforced concrete (HESFRC) as a part of the SHRP C-205 project is reported in volume 6 of the SHRP reports [Naaman et al. 1993]. This report provides an extensive database and a summary of comprehensive experimental investigation on the fresh and mechanical properties of HESFRC. The control high early strength (HES) concrete (used with the fiber addition) were defined as achieving a target minimum compressive strength of 5,000 psi (35 MPa) in 24 hours, as measured from 4 x 8 in. (100 x 200 mm) cylinders.

This chapter is divided into two major sections. The first section deals with short discrete fiber reinforced concrete and the second section summarizes the recent developments in the use of continuous fibers as reinforcements to produce high performance fiber-reinforced structural concrete.

5.2 Discrete Fiber-Reinforced Concrete

5.2.1 Strength

The strength of the fiber reinforced concrete can be measured in terms of its maximum resistance when subjected to either compressive, tensile, flexural and shear loads. In field conditions, usually some combination of these loads is imposed; however for evaluation purposes, the behavior is characterized under one type of loading without the interaction of other loads. The strength under each individual type of loading is a useful indicator of the FRC material's performance characteristic for design consideration.

5.2.1.1 Compression

The compressive properties of fiber-reinforced concrete (FRC) are relatively less affected by the presence of fibers as compared to the properties under tension and bending.

The influence of fibers in improving the compressive strength of the matrix depends on whether mortar or concrete (having coarse aggregates) is used and on the magnitude of compressive strength. Studies prior to 1988 including those of Williamson [1974], Naaman et al. [1974] showed that with the addition of fibers there is an almost negligible increase in strength for mortar mixes; however for concrete mixes, strength increases by as much as 23%. Furthermore, Otter and Naaman [1988] showed that use of steel fibers in lower strength concretes increases their compressive strength significantly compared to plain unreinforced matrices and is directly related to volume fraction of steel fiber used. This increase is more for hooked fibers in comparison with straight steel fibers, glass or polypropylene fibers. The typical influence of fibers on the stress- strain curve of concrete composites is illustrated in Figs. 5.1 and 5.2.

Ezeldin and Balaguru [1992] conducted tests to obtain the complete stress-strain curves of steel fiber-reinforced concrete with compressive strengths ranging from 35 MPa to 84 MPa (5,000 to 12,000 psi). The matrix consisted of concrete rather than mortar. Three volume fibers fractions of 50 pcy, 75 pcy and 100 pcy (30 kg/m^3, 45 kg/ m^3 and 60 kg/ m^3) and three aspect ratios of 60,75 and 100 were investigated. It was reported that the addition of hooked-end steel fibers to concrete, with or without silica fume, increased marginally the compressive strength and the strain corresponding to peak stress.

The effect of silica fume on the compressive properties of synthetic fiber-reinforced concrete was studied by Bayasi and Celik [1993] by testing 6 12 in. (150 300 mm) cylinders. Two fiber types were used: fibrillated polypropylene fibers and polyethylene-terphalate polyester fibers. Fiber volume fractions ranged from 0 to 0.6% and fiber length was 12 mm (0.5 in.). Silica fume was used as partial replacement of portland cement on an equal mass basis at 0, 5, 10 and 25%. Fibers had a relatively small favorable effect on the compressive strength. Both types of fibers improved the compressive behavior by enhancing the toughness. Furthermore, both types of fiber appeared to increase the strain at peak compressive stress. At 0.35 and 0.30 percent of polyester and polypropylene fiber content, respectively, use of silica fume enhanced the compressive strength and toughness of fiber concrete with an optimum content of 5 to 10 percent of weight of cement. Bayasi and Zeng [1993] reported that when polypropylene fibers with fiber lengths of 0.5 in. (12.5 mm) and 0.75 in. (19 mm) were used in volume fractions of 0.1, 0.3 and 0.5%, there was no significant effect on the compressive strengths.

Nakagawa et al. [1989] conducted compressive strength tests of concrete with short discrete carbon (both pitch-based type and PAN type), Aramid fibers and high strength Vinylon fibers. The tests were conducted on 100 x 200 mm (4 x 8 in.) cylindrical specimens. The volume percentages used were 1.86 and 3.05% for pitch type carbon fibers, 1.88 and 3.37% for Pan-type carbon fibers, 2.09 and 3.67% for Aramid fibers, and 2.00 and 3.40 for high strength Vinylon fibers. The results indicated that compressive strength tends to decrease when the fiber volume was increased. The effect of large volume of entrained air, due to the increase of fiber volume, had a significant influence on this reduction of strength.

Ezeldin and Lowe [1991], studied the compressive strength properties of rapid-set materials reinforced with steel fibers. The primary variables were (a) rapid-set cementing materials, (b) fiber type, and (c) fiber content. Four fiber types made of low-carbon steel were incorporated in this study. Two were hooked and one was crimped at the ends, and one was crimped throughout the length. Fibers were added in the quantities of 50, 75 and 100 lbs/yd^3 (30, 45, and 60 kg/m^3). The findings indicate that steel fibers can be successfully mixed with rapid-set materials up to a quantity of 75 lbs/yd^3 (45 kg/m^3). An increase in the compressive strength in the range of 5 to 25% within 24 hours was observed. The magnitude of the increase is dependent on the fiber shape and the content.

Zheng and Chung [1989], conducted compressive strength tests on 50 mm (2 in.) mortar cube specimens reinforced with short pitched-based carbon fibers (0.5% by weight of cement or 0.28% by volume of cement mortar) together with water reducing agent and an accelerating admixture. The results indicated tensile strengths of carbon fiber reinforced cement mortar increased by about 18% to 31% compared to the corresponding plain cement mortar values. The ductility was also improved.

5.2.1.2 Flexure

There are a number of factors that influence the behavior and strength of FRC in flexure. These include: type of fiber, fiber length (L), aspect ratio (L/d_f) where df is the diameter of the fiber, the volume fraction of the fiber (V_f), fiber orientation and fiber shape, fiber bond characteristics (fiber deformation). Also, factors that influence the workability of FRC such as W/C ratio, density, air content and the like could also influence its strength. The ultimate strength in flexure could vary considerably depending upon the volume fraction of fibers, length and bond characteristics of the fibers and the ultimate strength of the fibers. Depending upon the contribution of these influencing factors, the ultimate strength of FRC could be either smaller or larger than its first cracking strength.

Generally, there are three stages of the load-deflection response of FRC specimens tested in flexure and schematically they are shown in Fig. 5.3. The three stages are:

1. A more or less linear response up to point A. The strengthening mechanism in this portion of the behavior involves a transfer of stress from the matrix to the fibers by interfacial shear. The imposed stress is shared between the matrix and fibers until the matrix cracks at what is termed as "first cracking strength" or "proportional limit".
2. A transition nonlinear portion between point A and the maximum load capacity at point B (assuming the load at B is larger than the load at A). In this portion, and after cracking, the stress in the matrix is progressively transferred to the fibers. With increasing load, the fibers tend to gradually pull out from the matrix leading to a nonlinear load-deflection response until the ultimate flexural load capacity at point B is reached. This point is termed as "peak" strength.
3. A post peak descending portion following the peak strength until complete failure of the composite. The load-deflection response in this portion of behavior and the degree at which loss in strength is encountered with increasing deformation is an important indication of the ability of the fiber composite to absorb large amounts of energy before failure and is a characteristic that distinguishes fiber-reinforced concrete from plain concrete. This characteristic is referred to as toughness.

The nonlinear portion between A and B exists, only if a sufficient volume fraction of fibers is present. For low volume fraction of fibers ($V_f < 0.5\%$), the ultimate flexural strength coincides with the first cracking strength and the load-deflection curve descends immediately after the cracking load, Fig. 5.4. Typical load-deflection curves of FRC beams observed experimentally for different types of fibers are shown in Figs. 5.5 and 5.6.

Two concepts are proposed in the literature for explaining the factors that affect the magnitude of the "first cracking strength or proportional limit". One concept relates the "first cracking strength" to the spacing of the fibers in the composite [Romualdi and Batson 1963; Romualdi and Mandel 1964]. The other concept is based on the mechanics of the composite materials and relates the "proportional limit" to the volume fraction of the fiber, aspect ratio and fiber orientation.

In the fiber spacing concept, it is stipulated that the volume fraction of fibers and fiber aspect ratio must be such that there is a fiber overlap; however, except for this, the fiber aspect ratio L/d_f which has a significant effect on the flexural strength of FRC is not a parameter in the fiber spacing approach. Experimental results by some investigators [Edington et al. 1974; Swamy and Mangat 1974] tend to show that the fiber spacing concept does not accurately predict the first cracking strength of fiber-reinforced concrete. Additional discussion of the spacing concept can be found in Hannant's book [Hannant 1978].

The law of composite materials is believed to be simple and is proven experimentally [Shah and Rangan 1971] to be more accurate for the prediction of first cracking strength in comparison with the fiber spacing concept. The composite materials approach is based on the assumptions that the fibers are aligned in the direction of the load, the fibers are bonded to the matrix, and the Poisson's ratio of the matrix is zero. In the law of composite materials, the effect of fibers on the cracking behavior of FRC composites can be viewed similarly to conventional reinforcing steel in concrete members. However, because the fibers are randomly distributed, an efficiency factor is commonly multiplied by the volume fraction of fibers to account for their random distribution. The efficiency factor was studied in the literature and was observed to vary between 40% and 80% [Romuldi and Mandel 1964; Nielsen and Chen 1968].

Because of the linear dependence of the ultimate flexural strength of FRC on the volume fraction of fibers and their aspect ratio, it could be stated that the ultimate flexural strength generally increases with the fiber reinforcing index, defined as the product of fiber volume fraction and aspect ratio ($V_f L/d_f$). Based on this observation, Shah and Rangan [1971] proposed the following general equation for predicting the ultimate flexural strength of the fiber composite:

$$f_{cc} = Af_m (1 - V_f) + B (V_f L/d_f) \quad (5.1)$$

where f_{cc} is the ultimate strength of the fiber composite, f_m is the maximum strength of the plain matrix (mortar or concrete), A and B are constants which can be determined experimentally. For plain concrete, A = 1 and B = 0. The constant B accounts for the bond strength of the fibers and randomness of fiber distribution. Swamy et al. [1974a] established values for the constants A and B as 0.97 and 4.94 for the ultimate flexural strength of steel fiber-reinforced concrete and 0.843 and 4.25 for its first cracking strength.

Ezeldin and Lowe [1991] studied the flexural strength properties of rapid-set materials reinforced with steel fibers. The primary variables were (a) rapid-set cementing materials, (b) fiber type, and (c) fiber content. Four fiber types made of low-carbon steel were incorporated in this study. Two were hooked and one was crimped at the ends, and one was crimped throughout the length. Steel fibers were added in the quantities of 50, 75 and 100 lbs/yd^3 (30, 45 and 6 kg/m^3). An increase in the flexural strength was observed. The fiber efficiency in enhancing the flexural strength is controlled by the fiber surface deformation, aspect ratio, and fiber content. The results further indicate that steel fibers are very effective in improving the flexural toughness of rapid-set materials. Toughness indexes as high as 4 for I_5 and 9 for I_{10} can be achieved with fiber contents of 75 lbs/yd^3 (45 kg/m^3).

A comparative evaluation of the static flexural strength for concretes with and without four different types of fibers: hooked-end steel, straight steel, corrugated steel, and polypropylene fibers was conducted by Ramakrishnan et al.[1989a]. The fibers were tested at 0.5, 1.0, 1.5 and 2.0% by volume. It was reported that maximum quantity of hooked-end fibers that could be added without causing balling was limited to 1.0 percent by volume. Compared to plain concrete, the addition of fibers increased the first cracking strength (15 to 90 percent) and static flexural strength (15 to 129 percent). Compared on equal basis of 1.0 percent by volume, the hooked-end steel fiber contributed to the highest increase, and the straight fibers provided the least appreciable increase in the above mentioned properties.

Johnston and Zemp [1991] investigated the flexural performance under static loads for nine mixtures, Table 5.1, using sets of 15 specimens for each mixture. Each set of 102 x 102 x 356 mm (4 x 4 x 14 in.) specimens were prepared from five nominally identical batches and tested under third point loading over a 305 mm (12 in.) span. First crack strengths defined in ASTM C 108 as the point on the load-deflection curve at which the form of the curve first becomes nonlinear, and ultimate strength based on the maximum flexural load (ASTM C 78) were established for the eight fibrous concretes, with only ultimate strength for the plain concrete control. The ultimate strengths based on the maximum load was only slightly greater (2.1% on average) than the first crack strength, with a maximum of 4.2% for mixture 3, containing 1.5% fibers. The deflection at maximum load was likewise little different from the first crack deflection. Trends or relationships involving the first crack strengths were therefore similar to those involving ultimate strengths. Note that this may not be the case for fiber-matrix combinations that produce multiple peaks on the load-deflection curve with the maximum load at deflection much higher than the first crack deflection [Johnston and Carer 1989]. The results of Johnston's work indicated that increasing the fiber content from 0.5 to 1.5% had a significant beneficial effect on the first crack (and ultimate) strengths despite the negative influence of increasing w/c and w/(c+f). The increase in first crack strength of 31%, unadjusted for the differences in w/c and w/(c+f) is quite large, since it is widely believed that increasing fiber content has only a minor effect on first crack strength for many of the types of fiber in current use. With the adjustment in w/c and w/(c+f), for 1.5% of SW(75) fibers in Table 5.1, the increase is 63% over the value for 0.5% of the same fibers.

Although increasing aspect ratio (V_f L/d_f) has long been recognized as a positive influence on performance because of the improved resistance to pull-out of the fibers from the matrix, any benefit for the smooth wire (SW) fiber used by Johnston and Zemp [1991] was obscured by the increase in w/c and w/(c+f) needed to maintain the workability. After adjustment to allow for the differences in w/c and w/(c+f), strengths were seen to increase slightly with increasing aspect ratio as expected from the consideration of the pullout resistance. The effect of the aspect ratio was quite small compared with that of the fiber content.

A change in fiber type often involves a change in aspect ratio. Johnston and Zemp [1991] investigated the effect of the fiber type. The results indicated that for particular fiber types, smooth and deformed wire fibers (SW and SDW) the effect - although small compared to the fiber content - is slightly superior to melt extract (ME) and slit sheet (SS) fibers of the same aspect ratio. This does not means that all wire fibers will necessarily be superior to all melt extract or slit sheet fibers because fiber development is a continuous ongoing process.

Fiber content seems to be the parameter that is of primary importance in determining the first-crack and ultimate strengths under static flexure loading. Fiber aspect ratio and fiber type are of secondary importance in practical concretes where increasing the aspect ratio or changing the type (steel composition, surface area, surface texture, etc.) in a manner that increases water demand may tend to counteract any improvements in strength attributable to changes in these fiber parameters.

Balaguru et al. [1992] conducted flexural tests on deformed steel fibers reinforced concrete beams. The variables investigated were fiber type, length and volume fraction, and matrix composition. The results indicate that fiber content in the range of 30 to 60 kg/m (50 to 100 lbs/yd^3) provide excellent ductility for normal strength concrete. The fiber content has to be increased to about 90 kg/m^3 (150 lbs/yd^3) for high strength concrete. Hooked-end fiber geometry provides better results than corrugated and deformed-end geometry. Fiber length, in the range of 30 to 60 mm (1.18 to 2.36 in.) does not have any significant effect on toughness for hooked-end fibers. Ductile behavior can be obtained using 120 kg/m^3 (200 lbs/yd^3) of fibers, even for concretes containing 20 percent silica fume by weight of cement. The load-deflection curves for normal strength concrete beams and high strength concrete beams with 30 mm long hooked-end fibers are shown in Figs. 5.7a and 5.7b. It has been reported [Ashour 1993] that steel fibers also enhance the strength and ductility of high strength concrete beams.

The effect of silica fume on the flexural strength of synthetic fiber-reinforced concrete was studied by Bayasi and Celik [1993]. The specimens were 100 x 100 x 350 mm (4 x 4 x 14 in.) and were subjected to a four point (1/3 point) flexural load-deflection behavior test over a span of 300 mm (12 in.) according to ASTM C 78 and C 108. Two fiber types were used: fibrillated polypropylene fibers and polyethylene-terphalate polyester fibers. Fiber volume fractions ranged from 0 to 0.6% and fiber length was 12 mm (0.5 in.). Silica fume was used as partial replacement of portland cement on an equal mass basis at 0, 5, 10 and 25%. The results indicate that polyester fibers and polypropylene fibers have an inconsistent effect on the flexural strength but significantly increases the flexural toughness and the post-peak resistance of concrete. These improvements continues as fiber volume increases, except in ultimate strength, for which it starts to decrease beyond fiber volume of 0.35 percent. It was also shown that silica fume enhances toughness and post-peak strength of plastic fiber concrete.

Bayasi and Zeng [1993] proposed that flexural behavior of polypropylene fiber be characterized by the post-peak flexural resistance (load or stress). It was found that, for volumes equal or less than 0.3 percent, 19 mm (0.75 in.) long fibers were more favorable for enhancing the post-peak resistance. For 0.5 percent volume, 12.5 mm (0.5 in.) long fibers were more effective. The typical results are shown in Fig. 5.8.

Balaguru [1992] also investigated the flexural strength and the flexural load-deflection behavior of polymer-fiber reinforced rapid-setting concrete. The test variables were the matrix type and fiber type. The first product was a portland cement with a pozzolanic addition termed as PY. The second product was a blend of magnesium oxide and ammonium di-hydrogen phosphate with a small amount of fly ash identified as SE. The fibers were made of fibrillated polypropylene and single-filament nylon 6. Typical load-deflection curves are shown in Fig. 5.9 indicate that post-peak drop is steeper for polymer fibers as compared to steel fibers. This should be expected because of the low modulus of elasticity of polymers fibers.

Nakagawa et al. [1989] conducted flexural tests of concrete with short discrete carbon (both pitch-based type and PAN type), Aramid fibers and high strength Vinylon fibers. The specimens had a crossection of 50 x 100 mm (2 x 4 in.) and the length of the specimens was 1,100 mm (43 in.). The specimens were loaded with two point loads with a shear span of 300 mm (11.8 in.). The constant moment region was 300 mm (11.8 in). The volume percentages used were 1.86 and 3.05 for pitch type carbon fibers, 1.88 and 3.37 for Pan-type carbon fibers, 2.09 and 3.67 for Aramid fibers, and 2.00 and 3.40 for high strength Vinylon fibers. The results indicated that limit of proportionality (LOP) and the modulus of rupture (MOR) increased with fiber volume, and the strength of the fiber used. The typical load-deflection results for various types of fibers are shown in Fig. 5.10.

Zheng and Chung [1989] conducted flexural tests on 4 x 4 x 16 cm (1.57 x 1.57 x 6.30 in.) beams with third point loading. The mortar specimens were reinforced with short pitched-based carbon fibers (0.5% by weight of cement or 0.28 volume % of cement mortar) together with water reducing agent and an accelerating admixture. The results indicated that flexural strengths of carbon fiber reinforced cement mortar increased by about 89% to 112% compared to the corresponding strengths of plain cement mortar.

Cementitious composites with higher volume percentages (in the range of 12 to 16 percent) of fibers have been investigated to exploit the beneficial effects of fibers. These cementitious composites termed as SIFCON (Slurry infiltrated concrete) were investigated in an experimental study dealing with the behavior of reinforced concrete beams containing a SIFCON matrix [Fritz et al. 1992]. The results indicated that the presence of SIFCON in overreinforced concrete beams led to ductility indexes exceeding three times those obtained without it. Crack widths and spacing were more than an order of magnitude smaller than in conventional reinforced concrete. The experimental results also suggested that there is no need for stirrups in flexural members with SIFCON matrix.

One promising new development uses steel fiber mats to reinforce concrete matrix. This new approach, called SIMCON (Slurry infiltrated mat concrete), produces concrete components with extremely high flexural strength [Hackman et al.1992; Krstulovic-Opara et al. 1994]. The advantage of steel fiber mats over a large volume of discrete fibers is that the mat configuration provides inherent strength and can utilize fibers with much higher aspect ratios. The fiber volume is less than half that required for SIFCON (slurry infiltrated fiber concrete), while achieving similar flexural strength and energy absorption capacity. The typical aspect ratios for FRC range from 40 to 100, although special handling procedures may be required as the aspect ratio approaches 100. SIMCON utilizes fibers with aspect ratios exceeding 500. Since the mat is already in a pre-formed shape, handling problems are minimized and balling does not become a factor. Hackman et al.[1992] conducted flexural tests with SIMCON. The fibers were manganese carbon steel, approximately 9.5 in (241 mm) with an equivalent diameter of 0.010 to 0.020 in.(0.25 to 0.50 mm). Stainless steel mats were also produced using a 9.5 in. (241 mm) long fiber with an equivalent diameter of 0.010 to 0.020 in. (0.25 to 0.50 mm). The fiber volume percent for the manganese carbon steel ranged from 1.2 to 3.6, while the stainless steel mats were packed to a density of 5.7 percent. Two specimens of 2 x 4 x 20 in. (50 x 100 x 500 mm) of SIMCON were tested in flexure. For comparison purposes, three SIFCON 2 x 4 x 14 in. (50 x 100 x 350 mm) beam specimens were also prepared with SIFCON slurry containing 14 percent 1.0 in.(25 mm) long 304 stainless steel fibers. The flexural tests were conducted in accordance with ASTM C 1018, using a third-point loading and a 12 in. (300 mm) span. The SIMCON composites with fiber mats of only 3.3 and 3.6 volume percent provided an ultimate flexural strength of roughly 75% of the flexural strength of SIFCON specimens at 25 percent of the fiber volume used in SIFCON. With a 5.7 volume percent of stainless steel fiber mat, the resulting SIMCON composite exhibited an average of 85% of the flexural strength of SIFCON with a 41% of the fiber volume. Figs. 5.11a to 5.11c show the load-deflection behavior of various SIFCON and SIMCON composites. The superior performance of the SIMCON over SIFCON is related to the bonding of the mat fibers in the composite. In the standard SIFCON, the relatively short embedment length of 1 in. (25 mm) results in fiber pullout as the primary failure mode. In the SIMCON composites, the failure mode comprises of multiple cracks and ultimate failure occurs through fiber breakage in the high tensile stress zones of one or more of the crack planes. In the mat reinforced composites, the yield strength of the steel is fully utilized.

5.2.1.3 Tensile and Splitting Tensile

The failure in tension of cement-based matrices is rather brittle and the associated strains are relatively small in magnitude. The addition of fibers to such matrices, whether in continuous or discontinuous form, leads to a substantial improvement in the tensile properties of the FRC in comparison with the properties of the unreinforced matrix. The enhancement of the properties is particularly noticeable.

Most investigations in the field of FRC derive tensile properties of the composite indirectly on the basis of observations from flexural tests or split cylinder tests. This is because there are difficulties associated with the interpretation of results obtained from direct tension tests. The difficulties are due to differences in specimen sizes, specimen shapes, instrumentation and methods of measurement. As yet, no standard test specimen is available for direct tension tests. As such, the observed stress-strain or force-elongation curve in direct tension is expected to vary, depending on the size of the specimen, stiffness of the testing machine, gauge length used to calculate strains and the number of cracks developed within the gauge length. The primary difficulty in characterizing the tensile response of FRC composites is that the post cracking behavior is generally dominated by the widening of a single major crack as observed in several experimental studies [Visalvanich and Naaman 1983; Gopalaratnam and Shah 1987]. The concentration of deformation at the crack location leads to a non-uniform definition of strains in the cracked region which depends on the prescribed gauge length.

The stress-strain or load-elongation response of fiber composites in tension depends mainly on the volume fraction of fibers. In general, the response can be divided into two or three stages, respectively, depending on whether the composite is FRC (fiber volume less than about 3%) or Slurry Infiltrated (SIFCON) where the volume of fibers normally varies between 5% and 25%. Typical stress-strain or load-elongation curve for SIFCON and conventional FRC composites are shown in Figs. 5.12 and 5.13.

Before cracking, the composite (both SIFCON and FRC) can be described as an elastic material with a stress-strain response very similar to that of the un-reinforced matrix. Several approaches can be used to predict the main characteristics of the tensile stress-strain curve of fiber composites in the first linear stage before cracking. These include, the mechanics of composite materials, fracture mechanics, damage mechanics, and empirical approaches. Using the mechanics of composite materials, the tensile stress in the composite at cracking can be predicted from the following equations [Naaman 1987]:

$$\sigma_{cc} = \sigma_{mu}(1 - V_f) + \alpha_1 \alpha_2 \tau (V_f L / d_f) \quad (5.2)$$

where s_{cc} is the tensile strength of the un-reinforced matrix, V_f and L/d_f, are the volume fraction and aspect ratio of fibers respectively, a_1 is a bond coefficient representing the fraction of bond mobilized at matrix cracking strain, and a_2 is the efficiency factor of fiber orientation in the uncracked state of the composite. Equation (5.2) shows that a slight improvement in the first cracking strength is expected at low volume fraction of fibers.

After cracking and in bridging the cracked surface, the fibers tend to pull out under load resulting in a sudden change in the load-elongation or stress-strain curve. If the maximum postcracking stress is larger than the cracking stress, such as in SIFCON (Fig. 5.12), then a second stage of behavior can be identified as the multiple cracking stage, and corresponds to the portion of the load-elongation curve that joins the cracking stress point to the maximum postcracking stress point (peak point on the curve). Beyond the peak point, a third stage of behavior exists characterized by failure and/or pullout of the fibers about a single critical crack. The corresponding descending branch of the load-elongation curve can be steep or of moderate slope depending on the fiber reinforcing parameters and whether a brittle or ductile failure occurs. Along stages I and II (Fig. 5.12), the elongation of the composite (measured along a defined gage length) can be transformed into equivalent strain. However, along stage III, the elongation corresponds primarily to the opening of a single critical crack and cannot be transformed into strain since crack opening is independent of the gage length.

The multiple cracking stage described above occurs only if the maximum postcracking stress is larger than the cracking stress; otherwise, in the case of conventional FRC, with a relatively small volume fraction of fibers, the second portion of the curve vanishes and is replaced by a sudden drop in the load-elongation curve joining the cracking load to the postcracking load. Hence the load-elongation response is reduced to two main parts (stages I and II) as illustrated in Figs. 5.13a and 5.13b. The curve of Fig. 5.13 is for high modulus fibers such as steel fibers, while the curve of Fig. 5.13b is for low modulus fibers such as polypropylene. A comparison of actual stress-elongation curves of steel fiber reinforced mortar and SIFCON is shown in Fig. 5.14.

The postcracking strength increases with increasing bond strength, aspect ratio and volume fraction of fibers. Several empirical equations were derived in the literature to calculate the ultimate strength of fiber composite in tension [Gasparini et al. 1989; Lim et al. 1987; Naaman et al.1974; Naaman 1987]. Almost all equations expressed the ultimate tensile strength s_{pc} in linear function of the fiber reinforcing index $V_f L/d_f$ and fiber bond strength i_u as follows:

$$\sigma_{pc} = k\tau_u V_f L/d_f$$

where k is a constant (k < 1.0) that takes into account the orientation, bond, and distribution characteristics of the fibers.

Experimental tests on splitting tensile strength of FRC are not as numerous as tests conducted in direct tension, flexure and compression. However, the same main factors that affect the behavior of FRC in direct tension, flexure and compression are expected to affect its behavior in splitting tensile mode; namely, the volume fraction, the aspect ratio, and the bond characteristics of fibers. Increasing the volume fraction of fibers and/or increasing their aspect ratio increases the splitting tensile strength of the fiber composite. Also, hooked and deformed fibers are expected to offer better splitting tensile resistance compared to straight or non-deformed fibers.

It is now generally accepted that the type and amount of fibers currently used do not significantly enhance the first cracking stress of the fiber reinforced composite. This is demonstrated in Fig. 5.16, where the tensile stress-strain curves for concrete reinforced with two types of fibers are shown. Note that the first cracking stress was observed to be the same, regardless of the volume and type of fibers.

Fribrillated continuous uniaxial polypropylene fiber specimens were manufactured by means of a pultrusion process by Krenchel and Stang [1988]. Specimens were manufactured with either epoxy or cement-based materials and tested in uniaxial tension at Northwestern University [Mobasher et al. 1990b]. Fig. 5.16 shows composite stress-strain curve for a set of specimens as well as the corresponding calculated matrix contribution by rule of mixtures. The tensile strength of the matrix can be as high as 15 MPa (2,150 psi), and even at an average strain of 2 percent, the matrix contributes about 8 MPa (1,150 psi) of tensile stresses. The development of cracking for these specimens was studied using the fluorescent microscopy and laser holography [Mobasher et al. 1990a, 1990c]. It was observed that when the load-displacement curve reached a distinct change in slope from an elastic to a semi-plastic response, the contribution of the matrix reached its maximum value. At this point distributed microcracks seemed to localize into macrocracks.

Many of the current applications of fiber reinforced concrete involve the use of fibers ranging around 1.0 percent by volume of concrete. Recently, it has been possible to incorporate relatively large volumes (ranging up to 15 percent) of steel, glass, and synthetic fibers in concrete. According to Shah [1991], analysis of results of tensile tests done on concretes with glass, polypropylene and steel fibers, indicate that with such large volume of aligned fibers in concrete, there is substantial enhancement of the tensile load carrying capacity of the matrix. This may be attributed to the fact that fibers suppress the localization of micro-cracks into macro-cracks and consequently the apparent tensile strength of the matrix increases.

Nakagawa et al. [1989] conducted tensile strength tests of concrete with short discrete carbon (both pitch-based type and PAN type), Aramid fibers and high strength Vinylon fibers. The direct tensile tests were conducted on dog-bone shaped specimens. The critical section was 30 mm (1.18 in.) wide, 80 mm (3.15 in.) in length and 12 mm (0.5 in.) in thickness. The volume percentages used were 1.86 and 3.05 for pitch type carbon fibers, 1.88 and 3.37 for Pan-type carbon fibers, 2.09 and 3.67 for Aramid fibers, and 2.00 and 3.40 for high strength Vinylon fibers. The results indicated that ultimate tensile strength increased with the increase in the fiber volume.

Zheng and Chung [1989] conducted tensile tests on briquette mortar specimens reinforced with short pitched-based carbon fibers (0.5% by weight of cement or 0.28 volume % of cement mortar) together with water reducing agent and an accelerating admixture. The results indicated tensile strengths of carbon fiber reinforced cement mortar increased by about 113% to 164% compared to the corresponding strengths of plain cement mortars.

5.2.1.4 Shear Strength

Shear failure can be sudden and catastrophic. This is true for critical sections where, due to construction constraints, little or no reinforcing steel may be placed. For more than 30 years, fiber reinforced concrete (FRC) has been the object of studies dealing with various loading conditions including compressive, flexural and tensile loadings. The use of fibers to improve the shear behavior of concrete is also promising; however, reported research efforts on the shear behavior of FRC are limited [Valle 1991]. Tests performed to study the shear behavior of FRC can be categorized into two general groups: direct shear tests, and tests on beams and corbels. The direct shear tests are required to understand the basic transfer behavior of concrete, while the tests on beams and corbels are necessary to understand the behavior of structural members reinforced with fibers.

The investigations on direct shear behavior include those of Swamy et al.[1987], Barr [1987], and Tan and Mansur [1990]. Shear tests involving corbels have been reported by Fattuhi [1987], and Hara and Kitada [1980]. A number of studies have reported combined shear and flexural tests on beams to investigate the improvements in the behavior due to the addition of steel fibers [Ward and Li 1990].

From these studies, it can be stated that the addition of fibers generally improves the shear strength and ductility of concrete. It has been reported that the stirrups as shear reinforcement in concrete members can be partially or totally replaced by the use of steel fibers [Lim et al. 1987; Mansur et al. 1986]. Most of the work has been limited to concretes of normal strength. The lack of research in this area is even greater for FRC involving high strength concrete.

Valle and Buyukozturk [1993] reported the results of an investigation on the strength and ductility of fiber reinforced high strength concrete under direct shear. Three parameters were investigated: (i) concrete type — high strength versus normal strength concrete, (ii) type of fiber — steel versus polypropylene fibers and (iii) the presence of steel stirrups crossing the shear plane. The average strengths were 4,500 psi (31.5 MPa) for the normal strength concretes and 9,000 psi (63 MPa) for the high strength concretes. The direct shear transfer behavior of fiber reinforced concrete was investigated through testing of initially uncracked pushoff specimens. The specimens had dimensions of 21 x 10 x 3 inches (525 x 250 x 75 mm), with a shear plane of 30 in^2 (18,750 mm^2). In general, fibers proved to be more effective in high strength concrete than in normal strength concrete, increasing both ultimate load and overall ductility (Figs. 5.17a and 5.17b). Greater shear strength increases were found with fiber reinforced high strength concrete specimens (60% with steel and 17% with polypropylene fibers) than with fibers reinforced normal strength concrete specimens (36% with steel fibers and no increase with polypropylene fibers) when compared to the strengths of their respective unreinforced plain concrete specimens. The enhancement performance of fibers in high strength concrete is attributed to the improved bond characteristics associated with the use of fibers in conjunction with high strength concrete. For the specimens with steel fibers, significant increases in ultimate load and ductility were observed. With polypropylene fibers, a lower increase in ultimate load was obtained when compared to the increase due to steel fibers. Ductility of the polypropylene fiber reinforced specimens was greater than that of the steel fiber reinforced specimens. In the tests involving the combination of fibers and conventional stirrups, slight increases in ultimate load with major improvements in ductility were observed in comparison to the corresponding values for plain concrete specimens with conventional stirrups.

More research, both experimental and analytical is needed to develop a better understanding of shear behavior of composites and members with different types of fibers. The research should utilize metallic as well as non-metallic fibers.

5.2.2 Deformations

Information on deformation characteristics is essential for estimating the serviceability limits and the energy absorption capacity of FRC materials.

5.2.2.1 Modulus of Elasticity

The modulus of elasticity of a material, whether in tension, compression, or shear, is a fundamental property that is needed for modeling mechanical behavior in various structural applications. Tests have been devised to measure the moduli of elasticity of a given material. For pure materials, such as steel or glass, observed experimental values are tabulated once and for all, then used in practice. However, for FRC composites made out of at least two different materials, the modulus of elasticity depends on various parameters.

Numerous studies have addressed the modulus of elasticity of composite materials. They lead to numerous models that range from the very simple to the very sophisticated. Among the simplest models for composites made out of two different materials, the upper- and lower-bound solutions or a combination of them (described below) only depend on the volume fraction and the modulus of each material. More advanced models developed for fiber reinforced composites include, in addition, the properties of the interface between the two materials, whether the fibers are discontinuous or not, the distribution and orientation of the fibers, the aspect ratio (length to diameter) of the fiber, and the like.

The most common and the simplest models to predict the modulus of elasticity of FRC as a composite made out of two materials are the upper-and the lower-bound solutions or an arithmetic combination of both. They are described in details in many textbooks on composite materials and only the final solution is given below:

The upper-bound solution assumes that the fibers are continuous and oriented in the direction of loading along which the modulus of elasticity is needed. It leads to the following equation:

$$E_{cL} = E_f V_f + E_m (1 - V_f)$$

in which the subscripts c, L, f, and m stand respectively for composite, longitudinal, fiber, and matrix.

The lower bound solution assumes that the fibers are lumped with their axis normal to the direction along which the modulus is measured. It leads to the following equation:

$$E_{cT} = E_f E_m / [(1 - V_f) E_f + V_f E_m]$$

in which the subscript T stands for transverse.

For a composite with randomly oriented fibers, Halpin and Tsai [1969] suggested an equation based on a combination of Eqs. (5.4) and (5.5). Although their predictions of longitudinal and transverse moduli were different from the above upper- and lower-bound solutions, their equation can be used as a first approximation with the above equations. It is given by:

$$E_c = (3/8)E_{cL} + (5/8)E_{cT}$$

Examination of Eqs. (5.4) to (5.6) shows that for the same volume fraction of fibers, steel fibers should improve the modulus of elasticity of the composite more than glass fibers ($E_{steel} = 3\ E_{glass}$). Also, polypropylene fibers having a modulus of elasticity lower than that of concrete should lead to a decrease in the elastic modulus of the composite. However, for the range of fiber volume V_f normally used in practice, the increase or decrease in E_c is expected to be of the same order as the variability in the experimental data. The same is also true of the flexural stiffness of FRC composites.

Experimental studies [Fanella and Naaman 1985; Shah et al. 1978] have shown that the addition of fibers have only a slight effect on the ascending branch (modulus of elasticity) of the stress-strain curve of the composite. Investigators have also observed that the effect of adding fibers up to 4% by volume is small and linear for composites tested in flexure [Nielsen and Chen 1968; Edington et al. 1974; Ramakrishnan et al. 1989a], direct tension [Nielsen and Chen 1968; Edington et al. 1974; Mangat and Azari 1985] and compression [Nielsen and Chen 1968; McKee 1969]. Results showing the influence of steel fibers on the flexural stiffness of fiber composites is shown in Fig. 5.18.

Ezeldin and Lowe [1991] used three different amounts (30, 45 and 60 kg/m^3 or 50, 75 and 100 lbs/yd^3) of steel fibers in concretes with strengths ranging from 35 to 85 MPa (5,000 to 12,000 psi) to obtain the secant modulus of elasticity under compression. It was found that there was a marginal increase in the modulus of elasticity with the addition of steel fibers.

A comprehensive investigation on the modulus of elasticity of fiber-reinforced cement-based composites was conducted at the University of Michigan [Najm and Naaman 1992]. The results indicate that, although the factors suggested in Eqs. (5.4) to 0(5.6) above do influence the modulus of elasticity of the composite, other factors such as the length or aspect ratio of the fibers, their orientation, and the bond at the fiber matrix interface also have noticeable influence. It should be noted, however, that unless the fiber content is very large (more than 3% by volume), the approximate equation should give adequate results in all cases. Additional precision may not be warranted when compared to the variability usually encountered in the test results.

For the range of fiber volume normally used in practice, the dynamic modulus of fiber-reinforced concrete is little different from that of the plain, unreinforced concrete. Tests conducted by Swamy and Mangat [1974] have shown that the dynamic modulus of FRC reinforced with up to about 2% by volume of steel fibers varied within 5% of the control unreinforced matrix. Hence, the conventional solutions for the static elastic modulus also apply for the dynamic modulus of fiber-reinforced concrete.

Nakagawa et al. [1989] conducted tests for tensile modulus of elasticity of concrete with short discrete carbon (both pitch-based type and PAN type), Aramid fibers and high strength Vinylon fibers. The direct tensile tests were conducted on dog-bone shaped specimens. The critical section was 30 mm. (1.18 in.) wide, 80 mm (3.15 in.) in length and 12 mm (0.5 in.) in thickness. The volume percentages used were 1.86 and 3.05 for pitch type carbon fibers, 1.88 and 3.37 for Pan-type carbon fibers, 2.09 and 3.67 for Aramid fibers, and 2.00 and 3.40 for high strength Vinylon fibers. The results indicated that tensile modulus of elasticity increased with the increase in the fiber volume.

5.2.2.2 Creep and Shrinkage

Based on limited experimental data, the ACI committee 544 report [1982] indicates that wire fiber reinforcement has no significant effect on the creep behavior of portland cement mortar. However, recent results on the creep characteristics of FRC appear to contradict the above statement.

Creep tests conducted by Balaguru and Ramakrishnan [1988] in accordance with ASTM C 512 on steel fiber reinforced concrete (V_f = 0.6%, L/d_f = 100) subjected to a sustained load between 19% and 25% of the compression strength (stress to strength ratio between 0.19 and 0.25) showed that the creep strains were consistently higher than those of plain concrete. Also, creep tests conducted by Houde et al. [1987] on polypropylene and steel fibers showed that the addition of fibers increases the creep strains of the fiber composite by about 20% to 30% in comparison with the unreinforced matrix.

Unlike the observation made by Balaguru and Ramakrishnan [1988] and Houde et al. [1987], Mangat and Azari [1985] reported reduction in the creep strains with increasing content of steel fibers in comparison with plain concrete. For instance, at 3% by volume of fibers and at stress to strength ratio of 0.3, a reduction of about 25 % in creep strain compared to plain concrete is achieved after 90 days. However, it was observed that the steel fibers were less effective in restraining the creep at high stress to strength ratio (equal to 0.55) in comparison with low stress to strength ratio (equal to 0.33). The low effectiveness of steel fibers in decreasing the creep strains at large stress to strength ratio was attributed to the reduced interfacial bond characteristics of the fibers under creep. Large stress to strength ratios increase the lateral strains and hence decrease the interfacial pressure between the fibers and the surrounding concrete. This in effect reduces the restraint to sliding action between the fibers and the concrete matrix and results in larger creep strains.

The same factors that influence the shrinkage strain in plain concrete influence also the shrinkage strain in fiber reinforced concrete; namely, temperature and relative humidity, material properties, the duration of curing and the size of the structure. The addition of fibers, particularly steel, to concrete has been shown to have beneficial effects in counterbalancing the movements arising from volume changes taking place in concrete, and tends to stabilize the movements earlier when compared to plain concrete.

The effect of fibers in restraining the free drying shrinkage strains was found to be insignificant [Lim et al. 1987; Shah and Grzybowski 1989] or to cause a slightly smaller shrinkage than that of plain concrete [Paul et al. 1981; Balaguru and Ramakrishnan 1988]. If the purpose of fibers is only to restrain the free drying shrinkage strain, then the use of short and randomly distributed fibers is more beneficial than long fibers because of the larger number of fibers available in a given volume of concrete [Swamy 1985].

The primary advantage of fibers in relation to shrinkage is their effect in reducing the adverse width of shrinkage cracks [Swamy 1985; Lim et al. 1987; Shah and Grzybowski 1989]. Shrinkage cracks arise when the concrete is restrained from shrinkage movements. The presence of steel fibers delays the formation of first crack, enables the concrete to accommodate more than one crack and reduces the crack width substantially [Swamy 1985]. Tests conducted on restrained shrinkage behavior of FRC have shown that the addition of small amounts of steel fibers (0.25% by volume) reduced the average crack widths by about 20% [Shah and Grzybowski 1989] and the maximum crack width by about 50% [Lim et al. 1987] in comparison with unreinforced plain concrete. Comparing the effect of different types of fibers on the restrained shrinkage characteristics, one finds that polypropylene fibers are much less effective in reducing crack widths than steel fibers [Shah and Grzybowski 1989]. Randomly distributed fibers could enhance the mechanical properties of shrinkage-compensating concrete by restraining the composite uniformly in all directions without adversely affecting the mechanical properties of such composites. Tests conducted by Paul et al. [1981] on the effect of steel fibers on the expansion and drying shrinkage characteristics of expansive cement composites (Type K-conforming to ASTM C 845) have shown (Fig. 5.19) that the 7 day restrained expansion of the shrinkage-compensating concretes reinforced with straight fibers, crimped fibers and conventional steel bars were about 67%, 54% and 47% respectively of the expansion of unrestrained concrete.

High strength concretes with silica fume undergo early cracking when deformation is restrained. This phenomenon, which occurs even when concrete is protected against any evaporation, is attributed to autogenous shrinkage, because of the exceptionally low water-cement ratio (about 0.26). This phenomenon can be corrected by the use of fibers. Paillere et al. [1989] conducted shrinkage tests with two types of hooked fibers with ratios of lengths (in mm to diameter in hundredths of mm) of 30/60 and 50/50. The tests were conducted at fiber volume fraction of 0.8 percent. For each type of fiber, the optimum sand-aggregate ratio which gave the maximum slump was determined so as to magnify the shrinkage characteristics. The fiber concretes exhibited an autogenous shrinkage lower than the reference concrete and undergo cracking at a later age under restrained deformation.

Soroushian et al. [1993] investigated the effects of polypropylene fibers and construction operations on the plastic shrinkage cracking of concrete slabs. Polypropylene fibers at relatively low fiber volume fractions were observed to reduce substantially the total area and maximum crack width of slab surfaces subjected to restrained plastic shrinkage movements. The rate of screeding of fresh concrete was also a critical factor (particularly in plain concrete). Slower screeding rates led to reduced plastic shrinkage cracking.

5.2.2.3 Strain Capacity

The ability to withstand relatively large strains before failure, the superior resistance to crack propagation and the ability to withstand large deformations and ductility are characteristics that distinguish fiber-reinforced concrete from plain concrete. These characteristics are generally described by "toughness" which is the main reason for using fiber-reinforced concrete in most of its applications.

Unlike plain concrete specimens, the presence of fibers imparts considerable energy to stretch and debond the fibers before complete fracture of the material occurs. Toughness is a measure of the ability of the material to mobilize large amounts of post-elastic strains or deformations prior to failure. Typical load-deflection curves of FRC specimens in comparison with plain concrete specimens are shown in Fig. 5.20.

ASTM C 1018, provides Method A for evaluating the toughness of fiber reinforced composites through the use of a toughness index. The toughness index is calculated as the area under the load-deflection curve up to the prescribed service deflection divided by the area under the load-deflection curve up to the first cracking deflection. Three indexes are described in ASTM C 1018: I_5, I_{10} and I_{20} corresponding respectively to deflections of 3, 5.5 and 10.5 times the deflection at first cracking. These indices (computed as shown in Fig. 5.20) provide indications of the shape of the load-deflection response (post cracking) and available ductility. It should be pointed out that the value of I_5, I_{10} and I_{20} is unity for elastic, perfectly brittle material behavior and is equal to 5, 10 and 20 respectively for elastic, perfectly plastic material behavior. Indices higher than those defined above are possible (see Fig. 5.21) depending on the fiber deformation, aspect ratio and volume fraction.

The same variables that affect the ultimate flexural strength of FRC beams also influence the flexural toughness; namely, the type of fiber, volume fraction of fiber, the aspect ratio, the fiber's surface deformation, bond characteristics and orientation.

Ezeldin and Lowe [1991] investigated the flexural toughness for rapid set materials. The primary variables were (a) rapid-set cementing materials, (b) fiber type, and (c) fiber content. Three commercially available rapid-set materials were investigated. Four fiber types made of low-carbon steel were incorporated in the study. Two were hooked at the ends, one was crimped at the ends, and one was crimped throughout the length. Steel fibers were added in quantities of 50, 75 and 100 pcy (30, 45 and 60 kg/m^3). The results indicated that steel fibers are very effective in improving the flexural toughness of rapid-set materials.

Toughness indexes as high as 4 for 5 and 9 for 10 can be achieved with fiber contents of 75 pcy (45 kg/m^3).

Balaguru et al. [1992] studied the flexural behavior of steel fiber reinforced concrete. The focus was on the toughness behavior. The results indicate that fiber content in the range of 50 to 100 lb/yd^3 (30 to 60 kg/m^3) provide excellent ductility for normal strength concrete. The fiber content has to be increased to about 150 lb/yd^3 (90 kg/m^3) for high strength concrete. Hooked end fiber geometry provides better results than corrugated and deformed-end geometry. Fiber length, in the range of 1.18 to 2.36 in. (30 to 60 mm.) does not have a significant effect on toughness for hooked-end fibers. Ductile behavior can be obtained using 200 lb/yd^3 (120 kg/m^3) of fibers, even for concrete containing 20 percent silica fume by weight of cement. It was noted that addition of silica fume improves strength and slightly reduces ductility. The load-deflection behavior of steel fiber reinforced concrete of normal and high strength were earlier shown in Figs. 5.7a and 5.7b. Corresponding to these load-deflection curves, the toughness indexes for hooked end fibers for normal and high strength concretes are shown in Figs. 5.22a and 5.22b. The toughness indexes were calculated based on the ASTM C 1018 procedure. Based on these results the following observations were made: (i) Increase in fiber content results in consistent increase in ductility and energy-absorption capacity. The postpeak load-deflection responses are flatter and the toughness indexes are higher. (ii) Toughness Indexes 5 and 10 computed using the ASTM procedure are not sensitive enough to show the variations that are present in the load-deflection responses. If deflections are measured accurately, values 50 and 100 can be computed for all fiber types and fiber contents greater than or equal to 50 pcy (30 kg/m^3). (iii) Higher fiber contents result in much higher load-retaining capacity at large deflections. In almost all cases, there was a considerable difference in 100 between fiber contents. (iv) The magnitude of toughness indexes are quite different from those reported in the literature [Stevens et al. 1995; Balaguru et al. 1992]. (v) Typically, for a given fiber content, toughness indexes are smaller for high-strength concretes compared to normal strength concretes. The differences are less significant at higher fiber volume fractions.

Liang and Galvez [1990] investigated the peak load capacity and toughness of cementitious composite beam specimens reinforced with polyester fibers. A total of more than 50 beam specimens measuring 2 x 2 x 11 in. (50 x 50 x 275 mm) were tested in flexure. Only one fiber diameter (gauge No. 14) with a nominal diameter of 0.078 in. (2 mm) was used. The variables included effect of fiber length, fiber volume fraction and cement matrix mixture on flexural behavior of composites. The fiber length was varied between 1 to 3 in. (25 to 75 mm). The volume fraction of the fibers was varied between 2.5% to 15%. The results indicated that the peak load capacity and toughness of beam specimens were improved with addition of fibers. While the increase of peak load capacity is dependent upon fiber length and fiber volume fraction, the improvement of toughness index shows less sensitivity to these two parameters. The results also indicated that use of water-reducing admixture (RH 1000) apparently reduced the peak-load and toughness of the beam specimens. It was speculated that the addition of RH 1000 in the cement mix may have deteriorated the chemical bonding between the fiber and the matrix.

Tests conducted on steel fiber-reinforced concrete [Otter and Naaman 1988] showed that the increase in toughness was directly related to the volume fraction of fibers with values of toughness index in compression of up to about 4 for specimens reinforced with 2.1 % volume fraction of hooked steel fibers (see Fig. 5.2). For concrete reinforced with straight, smooth fibers, increasing the aspect ratio of fibers from 45 to 80 resulted only in a minimal increase in compressive toughness (see Fig. 5.2b). The use of glass and polypropylene fibers led to low toughness values compared to steel fibers. Otter and Naaman [1988] attributed in part the low performance of glass and polypropylene fibers to the high aspect ratios of the fibers used which led to difficult mixing and possibly higher porosity.

A comprehensive investigation was carried out to study the compression behavior of normal and high strength fiber-reinforced concrete [Ezeldin and Lowe 1991]. The strengths of the concrete were varied from 5 ksi to 12 ksi (35 MPa to 84 MPa). Silica fume was used for high strength concretes. Three fiber volumes and three aspect ratios were considered. The influence of fiber-reinforcing parameters on the peak stress, corresponding strain, the secant modulus of elasticity and the toughness of concrete were investigated. They also showed that increase in silica fume content renders the fiber-reinforced concrete more brittle as compared to concrete without silica fume (Figs. 5.23a and 5.23b). From these figures, it can be seen that the descending portion of the stress-strain curve with 20% silica fume is steeper than those for 0% and 10%. This indicates reduced toughness for high strength concrete.

Taerwe [1992] reported the test results on normal, medium and high strength concrete cylinders under axial compression. The descending part of the stress-strain curve is very steep for the high strength concrete. Adding fibers is shown to have a beneficial effect on the strain-softening behavior and significantly increases toughness, as measured by the area under the stress-strain curve.

Hsu and Hsu [1994] conducted a series of tests on 75 x 150 mm (3 x 6 in.) cylindrical specimens and obtained the complete compressive stress-strain curves for high strength steel fiber-reinforced concrete with and without tie confinement. The volume fractions of steel fibers in concrete were 0, 0.5, 0.75 and 1 percent respectively. Empirical equations were proposed to simulate the stress-strain behavior of high strength steel fiber-reinforced concrete with compressive strengths exceeding 70 MPa (10,000 psi).

The same factors that affect the load-deformation response of fiber-reinforced concrete composites in the postcracking range in flexure tend to affect their behavior in direct tension. In general, increasing the volume fraction and aspect ratio of fibers and enhancing their bond characteristics (hooked and crimped fibers) tend to increase the slope of the postcracking load-deformation response and hence increases the toughness of the fiber composite. The toughness of the fiber composite in direct tension (defined and measured in accordance with ASTM C 1018) can be one or two orders of magnitude higher than that of plain concrete. This is due to the large frictional and fiber-bending energy developed during fiber pullout on either side of the crack and due to larger deformation encountered as a result of multiple cracks (if they occur) in comparison with the single crack that normally develops in plain concrete.

Little information is available on modeling the descending branch of the stress-elongation curve of fiber-reinforced concrete composites (stage III in Fig. 5.12). However, for steel fiber-reinforced concrete in which fiber pullout occurs through a single crack (stage II of Fig. 5.13), prediction equations were proposed by Visalvanich and Naaman [1983], and Nammur and Naaman [1986]. Fibers are also known to increase significantly the strain capacity of concrete composites under compression loads. It can be seen from Figs. 5.1 and 5.2 that increasing the aspect ratio and/or increasing the volume fraction of fibers produces in general a less steep descending portion, which results in a larger area under the stress-strain curve, higher ductility, and higher toughness of the composite compared to that of plain concrete. The improved toughness in compression due to fiber reinforcement is particularly useful in preventing sudden and explosive type of failure under static loading and in absorbing energy under dynamic loading.

Ezeldin and Balaguru [1992] proposed an analytical expression to generate the complete compressive stress-strain curve for fiber-reinforced concrete without silica fume, using the fiber parameter, , and strain corresponding to the compressive strength, of . The proposed analytical expression is

$$f_c = \frac{\beta \varepsilon_c}{\varepsilon_{of}}$$

$$f_{cf}' = \beta - 1 + (\frac{\varepsilon_c}{\varepsilon_{of}})^{\beta}$$

where f_{cf} is the compressive strength of fiber concrete, of is the strain corresponding to compressive strength, f_c and β ($_c$ are the stress and strain values on the curve and is the material parameter. To use the above equation only two values are needed, namely E_{of} of and β . The authors claimed that the above equation provided a good correlation between predicted and experimental results.

Considerable ductility and toughness can be achieved by using SIFCON and SIMCON. Experimentally observed stress-strain behavior of SIFCON in compression in comparison with plain concrete, fiber-reinforced concrete, and mortar is given in Fig. 5.24.

5.2.2.4 Coefficient of Thermal Expansion

The authors are not aware of any investigation dealing with the thermal expansion of fiber-reinforced concrete. Since the coefficient of thermal expansion of steel is of the same order as that of concrete, it is expected that the coefficient of thermal expansion of steel fiber-reinforced concrete will be similar to that of the plain concrete matrix. When other fibers such as polypropylene or glass fibers are used in small volume fractions, the same conclusions can be drawn. However, for large volume fractions of fibers, it would be reasonable to use the simple rule of mixtures, as a first approximation, to determine the coefficient of thermal expansion, provided that the temperature to which the composite is subjected does not affect significantly the properties of the fibers and their interfacial bond with the matrix.

5.2.2.5 Poisson's Ratio

Little information exists on the Poisson's ratio of fiber-reinforced concrete. In most analytical studies, the Poisson's ratio is generally assumed to be the same as that of concrete. This may be a reasonable assumption provided that the composite remains in the elastic range of behavior. As soon as cracking develops, the confining effects of the fibers bridging the cracks will have a significant effect on the lateral deformation, thus the value of the measured Poisson's ratio. At the time of this writing, no investigation is known to have addressed this issue.

5.2.2.6 Fracture Toughness

Cementitious matrices such as mortar and concrete have low tensile strength relative to their compressive strength, and fail in a brittle manner. One way to improve their fracture properties is to reinforce them with randomly distributed fibers. There have been increasing attempts in recent years to characterize cementitious composites (i.e. concrete and fiber-reinforced concrete) by their fracture properties. Both linear-elastic and elastic-plastic fracture mechanics techniques were applied. The crack growth mechanism in these materials is described in terms of three different zones: a stress-free zone, a pseudo-plastic zone, and a process zone [Visalvanich and Naaman 1983]. The stress-free zone is the zone where the fibers have either completely pulled out or failed; the pseudo-plastic zone is the zone where the matrix has cracked but the fibers bridging the crack provide some resistance to pullout; the process zone is the distributed region in front of an advancing crack due to the stress concentration field. The pseudo-plastic zone provides the main contribution to the fracture energy of fiber-reinforced cement composites. Apparent critical fracture energies observed [Visalvanich and Naaman 1983] at stabilization of crack propagation were of the order of 50 lb-in/in^2 (8,750 J/m2) for fiber-reinforced mortar containing 1% fibers by volume, in comparison to 0.5 lb-in/in^2 (88 J/m2) for plain mortar. However, the fracture toughness or the critical fracture energy cannot solely characterize fracture of fiber-reinforced concrete. The entire crack growth resistance energy is considered essential because it describes crack initiation, the slow stable crack growth process, and the crack extension prior to rapid propagation and fracture.

The postcracking toughness (measured by means of a toughness index) of high strength concretes with steel and polypropylene fibers increases with increase in volume fraction fibers [Benaiche and Baar 1989]. The fracture toughness of cement-based composites reinforced with relatively high fiber volume (up to 15 percent) was studied by Mobasher et al. [1990b]. Crack propagation and damage distribution were examined by laser holographic interferometry Based on fracture mechanisms observed during experimental studies, a R-curve approach was proposed to predict the toughening of matrices due to fiber reinforcement. The theoretical predictions show a good agreement with the experimental results for both the steel fiber composites and glass fiber composites.

The fracture energy of synthetic FRC depends on the fiber type, volume fraction and fiber lengths, and on the particular fiber finish and pretreatment. Fig. 5.25 shows the range of fracture energy obtained for a variety of synthetic FRCs (fibers used as is from the manufacturer), based on uniaxial tensile tests results by Wang et al. [1990]. The data for concrete cement and high strength aluminum alloy was added by Li et al. [1989] for reference. It is clear that the fracture energy of some synthetic FRC may reach that of certain high strength metallic alloy. Further improvements may be expected when fiber, matrix and interface parameters are optimized for the FRC composites. It appears that the influence of fiber length dominates the composite toughness over other parameters, including the fiber type. The frictional pullout is apparently the most important toughening mechanism.

Further improvements in toughness can be achieved by means of fiber pretreatment to effect local crimp or irregularities or to induce controlled damage of fiber during the pullout process. The success in toughening high strength concrete depends critically on overcoming the issues related to workability with higher fiber volume fractions, compressive strength reduction problems and also on using high performance fibers which have high modulus, high tensile strength and a high bond strength with the matrix.

5.2.2.7 Impact Resistance

Impact resistance is essential for applications such as the bridge piers. It is well recognized that the addition of fibers to concrete enhances the impact resistance. A number of studies have been conducted to develop information on the impact behavior of fiber-reinforced concretes. A summary of the work reported since 1989 is presented below.

Ramakrishnan et al. [1989b] used four types of steel fibers to evaluate the impact resistance. The impact specimens were tested at 28 days by the drop weight test method [ACI Committee 544 1990]. This method is simple, inexpensive and can also be conducted in the field. The impact tests were conducted for hooked-end steel (Type A), straight steel (Type B), corrugated steel (Type C), and polypropylene (Type D) fibers. Type A had an apparent aspect ratio of 100, while Type B had an aspect ratio of only 40. Fig. 5.26 shows the number of blows for first crack and full failure. The maximum increase in impact resistance results from the use of Type A fiber, but Type C fiber also contributes a higher impact resistance at higher fiber contents. The impact strength at first crack increased considerably with the increase in fiber content. Compared with plain concrete, the increase in impact strengths at full failure were 640%, 847%, 1,824% and 2,806% respectively for concretes with 0.5, 1.0, 1.5, and 2.0% (volume) Type C fiber content. The results of the study prove that fiber concretes incorporating hooked-end and corrugated steel fibers (Types A and C) have excellent impact resistance.

The impact resistance for concretes with various volume fractions of fibrillated polypropylene fibers [Alwahab and Soroushian 1987] is shown in Fig. 5. 27. The results indicate that significant improvement in impact resistance of concrete can be achieved with relatively low volume fraction of polypropylene fibers. Bentur [1989] studied the impact resistance of normal and high strength concretes with low volume of friblillated polypropylene fibers. The results indicated that low volume content of polypropylene fiber reinforcement (0.1% to 0.5%) had only a small positive influence on the impact resistance of both normal and high strength concretes.

Bayasi and Celik [1993] reported that polypropylene and polyester fibers enhanced the impact resistance of concrete. Furthermore, adding silica fume at 5 and 10% (mass) increases the impact resistance even more. It is postulated that this increase is attributed to the improvement in fiber dispersion and in bond between fibers and concrete caused by silica fume. Their data indicated that a silica fume content of 5% (mass) is optimal for impact resistance. It can also be postulated that the adverse effects on workability, caused by high contents of silica fume or fibers, resulted in the reduction in the impact resistance of the material.

5.2.3 Fatigue

In many applications, particularly in pavements, bridge deck overlays, and offshore structures, the flexural fatigue strength and endurance limit are important design parameters because these structures must be designed for fatigue load cycles.

No standard test (specimen size, type of loading, loading rate, fatigue failure criteria) is currently available to evaluate the flexural fatigue performance of fiber reinforced concrete. However, several earlier experimental fatigue studies were conducted on steel fiber-reinforced concrete and mortar in bending [Batson et al. 1972; Zollo 1972] using a testing procedure, specimen sizes, and loading conditions similar to those employed for static flexural tests of FRC or tests for conventional concrete with reversed and non-reversed type fatigue loading.

Fatigue strength can be described as the maximum flexural fatigue "stress" at which FRC composites can withstand a prescribed number of fatigue cycles before failure. Alternatively, it could be defined as the maximum number of fatigue cycles needed to fail a beam under a given maximum flexural stress level. However, the fatigue strength is often evaluated on the basis of endurance limit. The endurance limit of FRC in flexural bending is defined as the maximum flexural stress at which the beam could withstand a prescribed number of loading cycles (usually 2 million cycles), expressed as a percentage of either: (1) its virgin static flexural strength (first cracking strength or modulus of rupture), or (2) the maximum static flexural strength of similar plain unreinforced matrix (control). The flexural fatigue strength of steel FRC was reported to be about 80% to 90% of its static flexural strength at 2 million cycles when non-reversed loading is applied and about 70% of its static flexural strength when full reversed loading is used [ACI Committee 544 1990].

In evaluating available fatigue data of steel FRC, Anderson [1978] indicated that past investigations could have probably underestimated the fatigue resistance of steel FRC. This is because those investigations used the first cracking strength of the fiber composite as the reference strength. Since fiber addition modifies the cracking strength of plain concrete, Anderson pointed out that proper reference strength for fatigue evaluation of FRC beams should be taken as the unreinforced plain matrix beam strength. Using the proposed method of fatigue evaluation, Anderson showed that the fatigue resistance based on published fatigue data was much higher than reported. Fatigue tests conducted on steel fiber-reinforced concrete by Ramakrishnan et al. [1987b] showed that the addition of collated hooked-end steel fibers results in a considerable increase in the flexural fatigue strength of concrete. The flexural fatigue strength was increased by 200% to 250%, and endurance limit (to achieve two million cycles) was increased 90% to 95 %, when compared to plain concrete.

From tests of similar beam specimens with dimensions 6 x 6 x 21 in. (150 x 150 x 525 mm) in flexural fatigue under 20Hz non-reversed loading with different types of fibers (hooked, straight, corrugated steel fibers and polypropylene fibers) and different volume fractions of fibers (0.5% and 1.0%), Ramakrishnan et al. [1989b] observed that the fatigue strength and endurance limit (to achieve two million cycles) increased with the addition of fibers and increasing volume fraction of fibers. For instance, expressed as a percentage of modulus of rupture of plain concrete, the endurance limit for mixes with corrugated end fibers was 71% for the 0.5% fiber content and 86% for the mix with 1% fiber content. This represents an increase of 9% and 32% respectively over the endurance limit of 65% observed for plain concrete. Also it was observed that the improved bond characteristics of fibers improves the fatigue strength of fiber composites. The highest increase in fatigue strength was with hooked-end steel fibers and the lowest increase was with straight steel fibers and polypropylene fibers. Ramakrishnan et al. also reported that, if expressed as percentage of its modulus of rupture rather than that of the plain unreinforced matrix, the improvement of endurance limit with increasing volume fraction of fibers is either only slight (hooked-end fibers) or unfavorable for the other types of fibers used (polypropylene). Hence the improvement in the endurance limit is only evident when expressed in relation to the unreinforced matrix. This is in support of the observation made by Anderson [1978].

Johnston and Zemp [1991] investigated flexural fatigue behavior of steel fiber reinforced concrete involving nine different mixtures, including a control concrete without fibers. The fiber parameters were varied such that the effects of fiber content, fiber aspect ratio, and fiber type could be studied independently, while other fiber variables were held constant. The fiber content was varied between 0.5 to 1.5 percent (volume). The fiber's aspect ratio was varied from 47 to 100. The fatigue tests were conducted to a maximum of 500,000 cycles. A total of 194 fatigue tests and 135 tests for static flexural strength were conducted. Comparison of the S-N relationships based on stress as a percentage of first crack strength under static loading shows essentially the same trends as S-N relationships based on stress as a percentage of ultimate strength under static loading. In terms of S-N relationships based on stress percentage of first-crack strength, both fiber and aspect ratio are quite important. Fiber contents in excess of 1.0 percent (volume) are associated with better performance, as are fiber aspect ratios above 70 compared with those around 50. Stiffer high strength wire fibers perform slightly better than ductile low strength slit sheet fibers with aspect ratios over 70, but at aspect ratios around 50, differences in performance by type between smooth wire, surface-deformed wire and melt-extract fibers are small. Endurance limits at 100,000 cycles are 84 to 89 percent of the static first crack strength for the better combinations of fiber parameters, i.e., 1.0 percent volume or more of at least 70 aspect ratio fibers. For less effective combinations with 1.0 percent (volume) or less of fibers having aspect ratios around 50, the 100,000 cycle endurance limit is 75 to 80 percent of the static first crack strength. In terms of actual applied stress versus number of loading cycles, fiber content becomes the primary governing factor, with aspect ratio and fiber type of secondary importance. The best performance, a 100,000 cycle endurance limit of 6.9 MPa (1,000 psi) was obtained with 1.5% by volume of 75 aspect ratio cold-drawn wire fibers in a concrete with a w/(c+f) of 0.49. For 0.5% (volume) of same fibers, the 100,000 cycle endurance limit is only 5.2 MPa (750 psi) despite a lower w/(c+f) of 0.39.

Batson [1991] studied the flexural fatigue of concrete reinforced with three types of metallic and one type of synthetic fiber in volume percentages from 0.1 to 2.0. The beams reinforced with metallic fibers exhibited greater fatigue strength as compared to beams with synthetic fibers. The cyclic load for any one test was fixed percentage of the 28 day static flexural strength, ranging from 95 to 40% depending on the fiber type and fiber volume percentage. The tests were run until 2 million cycles or failure. The fatigue strength increased with fiber volume percentage for each type of fibers. The fatigue strength of beams varied with deformed shape of metallic fibers. The tests by Batson [1991] showed that hooked-end fibers at 1% by volume had superior fatigue strength compared to straight, crimped, and synthetic fibers with even higher volume percentages. Fatigue strength increased with fiber volume percentage and the synthetic fiber had the lowest fatigue strength approaching the value for plain concrete. The hooked-end fiber concrete had the greatest toughness indices and synthetic fiber concrete had greater toughness indices than the straight metallic fiber. He concluded that the durability of concrete reinforced with steel fibers subjected to fatigue loads could be enhanced significantly. Fatigue strengths greater than 80 percent of the 28 day static flexure strength were achieved depending on the type and volume percentage of the fiber. The S-N diagram showing the fatigue performance of Type A (steel fibers with hooked ends) with 1% by volume and 0.5% by volume, respectively, is shown in Figs. 5.28a and 5.28b.

Flexural fatigue strength of fibrillated polypropylene fiber-reinforced concretes was investigated by Nagabhushanam et al. [1989]. Three volume fractions of fibers were used. The test results indicated that the flexural fatigue strength and endurance limit (for 2 million cycles) of concretes with fibers were significantly improved when compared to plain concrete. The static flexural strength increased after being subjected to fatigue loading.

The fatigue characteristics of steel fiber-reinforced concrete under uniaxial and biaxial compressive stresses was studied by Yin and Hsu [1990]. The S-N curves and the cyclic deformations of fiber concrete were compared with those of plain concrete. It was reported that the S-N curve of fiber concrete is a straight line from 1 cycle to 1 million cycles, rather than a curve which can be approximated by two straight lines as in plain concrete (Figs. 5.29a and 5.29b). The addition of fibers to concrete increased the fatigue life as well as ductility, while the failure mode remained the same.

5.2.4 Durability

5.2.4.1 Abrasion Resistance

A review of abrasion studies of hydraulic structures undertaken by ACI Committee 544 [1982, 1988, 1990] has shown that if erosion of the concrete surface is due to a gradual wearing as a result of small particles of debris rolling over the surface at low velocities, then the quality of aggregate and the hardness of the surface determine the rate of erosion. Hence fibers have no effect in this regard. On the other hand, when erosion is due to abrasion resulting from high velocity flow and impact of large debris, steel fiber concretes have provided significant erosion resistance.

Abrasion tests [Nanni 1989] in accordance with ASTM C 799, procedure C, on field-cut and laboratory-made specimens showed no significant difference between the abrasion resistance of plain concrete and steel or synthetic fiber-reinforced concrete. However, the results indicated beneficial effects of steel fibers on scaling prevention of existing pavements [Nanni and Johari 1989]. As pointed out by ACI Committee 544 [1982, 1988, 1990], abrasion as it relates to pavements, and slabs, wear under wheeled traffic is similar to the low velocity erosion in hydraulic structures where the presence of fibers is not expected to increase the abrasion resistance of concrete.

An extensive study was recently completed by Sustersic et al. [1991] in which erosion-abrasion resistance of steel fiber reinforced concrete specimens was investigated according to CRD-C-63-80 test method and abrasion resistance according to Bohme test method. Nine mix proportions were used. The w/c ratios were varied from 0.30 to 0.65. The proportions of hooked steel fibers were varied from 0.25 to 2.0% by volume at w/c of 0.30 and the other w/c ratios, the quantity of the fibers were held constant. For comparison purposes, mixes without fibers were made at each w/c. Results showed that adding steel fibers in concrete improves the resistance as measured by both test methods. The erosion-abrasion resistance is improved by an increase in the compressive strength and by an increase in fiber content. It can be correlated to improvements of abrasion resistance from the Bohme test method but only at constant w/c and different content of fibers.

5.2.4.2 Freezing and Thawing

As pointed out by ACI Committee 544 [1982, 1988, 1990], the addition of fibers themselves has no significant effect on the freezing and thawing resistance of concrete. That is, concretes that are not resistant to freezing and thawing will not have their resistance improved by the addition of fibers [Schupack 1985; Hoff 1987]. Hence, the well-known practices for achieving durable concrete and the same air entrainment criteria for plain concrete should be used also for fiber reinforced concrete.

Notable work prior to 1989, including those of Bedard et al. [1986] and Balaguru and Ramakrishnan [1986], developed the understanding that the addition of entrained air improves the freeze/thaw durability of FRC in a manner similar to that of plain concrete. Based on their experimental results, Balaguru and Ramakrishnan [1986] proposed that for a w/c of more than 0.4 (for most field applications) and cement content less than 700 pcy (415 kg/m^3), a minimum of 6% (preferably 8%) entrained air should be used to improve the freeze/thaw resistance of fiber reinforced concrete.

Morgan [1991] summarized the results of several laboratory studies in which both wet and dry-mix fiber reinforced shortcretes were tested according to ASTM procedure A (freezing and thawing in water). It was shown that both steel and high volume polypropylene fiber reinforced wet-mix shortcretes can be made freeze/thaw durable, provided that the shortcrete is properly air entrained. Non-air entrained fiber reinforced wet-mix shortcrete deteriorates very rapidly in the ASTM C666 Procedure A test. It is not currently possible to practically produce high volume polypropylene fiber reinforced shortcrete using the dry-mix process so the inherent freeze/thaw durability of such a system is not known.

Very recently, a study was concluded [Vares 1994], which investigated the influence of Dramix steel fibers on the microstructure of steel fiber-reinforced concrete (SFRC) and the influence of the microstructure on frost resistance. The frost resistance of SFRC with 2% and 4% steel fibers was determined after subjecting the concretes to 100 and 200 freeze/thaw cycles, by comparing changes, toughness and microstructure with those specimens stored in water. No serious damage to SFRC microstructure was observed after 200 freeze/thaw cycles.

5.2.4.3 Wet-Dry Exposure

The effect of addition of polypropylene fiber to concrete mix and adequate curing in enhancing the detoriation resistance of concrete surface skin subjected to cyclic wet/dry seawater exposure was evaluated [Al-Tayyib and Al-Zahrani 1990]. Tests were carried out on 30 concrete slabs specimens of 375 x 750 x 75 mm (15 x 30 x 3 in.), made with and without polypropylene fibers. Some specimens were cured under laboratory-controlled conditions and were subjected to the wet/dry cycles for 85 weeks, while others were cured under field conditions and were subjected to the same cycles for 50 weeks. The results indicate that addition of polypropylene fibers effectively retard the deterioration process of the surface skin of the concrete specimens cured in hot weather environment.

5.2.4.4 Alkaline Environment

The durability of fibers in concrete is a concern for non-metallic fibers. Durability studies were recently conducted for synthetic fibers made of nylon 6, polypropylene and polyester [Khajuria et al. 1991]. Long term durability was estimated using an accelerated aging process. In this process the specimens are stored in lime saturated water maintained at 50oC. The integrity and effectiveness of fibers were studied using flexural toughness of 100 x 100 x 350 mm (4 x 4 x 14 in.) prisms tested under four point loading. Results indicate that, at a fiber content of 4.75 kg/m^3 (7.92 pcy), all three types of fibers provide postcracking resistance. Nylon 6 and polypropylene fibers are durable in alkaline environment present in concrete. Specimens with polyester fibers had some loss of ductility when subjected to accelerated aging.

5.2.5 Applications for Pavements

Fiber-reinforced concrete has been used worldwide with and without conventional reinforcement in many field applications. These include bridge deck overlays, floor slabs, pavements and pavement overlays, refractories, hydraulic structures, thin shells, rock slope stabilization, mine tunnel linings and many precast products. The guide for specifying, proportioning, mixing, placing, and finishing steel fiber-reinforced concrete is available from the ACI Committee 544 [1993]. The addition of steel fibers is known to improve most of the mechanical properties of concrete, namely, its static and dynamic tensile strength, energy absorption and toughness and fatigue resistance. Hence proper utilization of steel fiber-reinforced concrete depends on the skill of the engineer in taking advantage of its improved characteristics under a given loading for a given application and the cost effectiveness of the fiber addition.

The use of fiber-reinforced concrete in pavement applications started in the early 1970s. Many actual applications and experimental field studies of fiber concrete pavements and overlays have been reported [ACI Committee 544 1982, 1988, 1993; Hoff 1985; Lankard and Shrader 1983; Schrader 1985; Vandenberghe and Nemegeer 1985] in several countries around the world. These pavement applications included bridge deck overlays, pavements and pavement overlays, airfields, taxiways, aircraft aprons, and industrial floor slabs. The size of applications (overlays and pavements) varied between those of small streets and bridge segments (in residential and rural areas) of 100 ft to 200 ft (30 m to 60 m) long to as large as 570,000 sq. ft (5,300 sq. m) at McCarren International Airport in Las Vegas, Nevada, 315,000 sq. ft (3,000 sq. m) at Norfolk Naval Air Station in Virginia, and about 600,000 sq.ft (5,600 sq.m) at Taoyuan Air Base in Taiwan. As reported by Lankard and Schrader [1983], 22 airport paving projects were completed in the United States as of 1983. Also, over 11 million square feet (one million square meters) of fiber-reinforced concrete slabs were constructed as industrial floors in Europe since 1984 [Vandenberghe and Nemegeer 1985]. Almost all of the fiber-reinforced concrete pavement applications indicated above used steel fibers. Applications in which other commonly known types of fibers such as polypropylene, carbon, or glass fibers or a combination of fibers are almost nonexistent. The amount of steel fiber used in experimental and actual pavement applications (bridge deck overlays, pavements and pavement overlays, airfields) varied from as low as 60 pcy (36 kg/m^3) to as high as 265 pcy (157 kg/m^3) with an average of about 175 pcy (104 kg/m^3). The cementitious content varied between as low as 550 pcy (330 kg/m^3) to as high as 970 pcy (575 kg/m^3).

Typical material properties of steel fiber-reinforced concrete used for pavements and overlays [ACI Committee 544 1988] are: flexural strength = 900 psi to 1,100 psi (6 MPa to 8 MPa), compressive strength 6,000 psi (42 MPa), Poisson's ratio = 0.2, and modulus of elasticity = 4.0 x 106 psi (28 GPa). Recommended mix proportions for normal weight fiber-reinforced concrete can be found in the report of ACI Committee 544 [1990].

Many of the early experimental and actual fiber-reinforced concrete pavements and overlays developed full-width transverse cracks within 24 to 36 hours after placing [Hoff 1985] and exhibited some degree of curling and corner cracking [Schrader 1984]. The amount of curl is typically 1/8 in. (3 mm) with a range of 0 to 5/8 in. (0 to 16 mm). Corner cracking generally begins to occur after 1 year of service and breaks in an arc of about 1 to 4 ft (0.3 to 1.2 m) radius around the corners where longitudinal and transverse joints meet.

Curling is a common problem of concern in concrete pavements and overlays and does not depend on whether FRC or conventional concrete is used. Although it is difficult to find a common denominator to the degree of curling or cracking, several recommendations have been presented [Lankard 1975; Lankard and Schrader 1983; Schrader 1984] to minimize the level of curling and cracking in FRC pavements and overlays. These include: (1) reducing the cement content and increasing the aggregate content of the concrete, (2) replacing a portion of the cement content with fly ash, (3) using shrinkage compensating concretes, and (4) using water-reducing and set-retarding admixtures.

An experimental rehabilitation project was conducted on the Transcanadian Highway where old concrete pavement was covered with a thin, steel fiber reinforced concrete overlay [Chanvillard et al. 1989]. The surface of the old pavement was either sandblasted or scarified. Three different types of steel fibers were used, and all overlays was bonded with a thin cement grout. In addition, two lanes were repaired with some mechanical bonding provided by a 37.5 mm (1.5 in.) steel nails. In 1989, a thin bonded overlay (TBCO) was used in Japan on a jointed portland cement concrete pavement (PCCP) and continuously reinforced concrete pavement (CRCP) [Ibukiyama et al. 1989].

In Canada, twenty six bridge decks were restored with steel fiber reinforced concrete overlays and structural repairs were made to the beams, piers, or abutments of 19 bridges using steel fiber reinforced concrete, dry-process shortcrete [Johnston and Carter 1989].

Road-work performed along autoroute 40 in the suburbs of Montreal provided an opportunity to monitor the behavior of a thin bonded overlay of fiber-reinforced concrete [Chanvillard and Aitcin 1990]. Examination of the road more than 2 years after it was recommissioned clearly indicated that fibers had a positive effect on the condition of the road.

A demonstration project was built in Rapid City, South Dakota [Ramakrishnan et al. 1991]. The entire project consisted of a pavement that was 740 m (2,428 ft) long and 14.6 m (48 ft) wide. The slab was designed as a 19 cm (7.5 in.) thick plain concrete pavement. Two sections were chosen to study the feasibility of construction and evaluation of fiber reinforced concrete. The first section was 27.5 m (90 ft) long and 15 cm (6 in.) thick. The second section was 23 m (75 ft) long and 12.5 cm (5 in.) thick. Transverse and longitudinal joints were placed at 4.5 m (15 ft) and 3.6 m (12 ft) respectively. The experience indicates that, if proper care is taken, results obtained in the laboratory can be reproduced in the field without any problems.

5.3 Continuous Fiber-Reinforced Concrete

In the last 5 years, there has been significant interest and development in the use of continuous fiber reinforcement for improving the behavior of cementitious composites and/or concrete. Fiber Reinforced Polymers (FRP) or sometime also referred to Fiber Reinforced Plastic are increasingly being accepted as an alternative for uncoated and epoxy-coated steel reinforcement for prestressed and non-prestressed concrete applications.

In 1990, the American Concrete Institute formed the ACI Committee 440 on Non-Metallic Reinforcement. The Committee has just developed a state-of-the-art report on Fiber Reinforced Plastic (FRP) for Concrete Structures [ACI Committee 440 1996].

In 1992, the first international conference was held in Montreal, Canada [Neale and Labossiere 1992]. This was followed by an international conference in Ghent, Belgium [Taerwe 1995]. A specialty conference with emphasis on use of FRP for bridges was held in Canada [Mufti 1991]. Recently, another international conference was held in Montreal, Canada [El-Badry 1996].

This section describes the FRP materials and their engineering properties. The applications in the transportation systems of structural concrete reinforced and/or prestressed with FRP materials are also summarized.

5.3.1 Constituent Materials

Concrete reinforced with continuous fibers rather than short discrete fibers is also termed as Fiber-Reinforced Composites. In general terms, Fiber-Reinforced Composites are a materials system. The term "composite" can be applied to any combination of two or more separate materials having an identifiable interface between them, most often with an interphase region such as a surface treatment used on selected constituents to improve adhesion of that component to the polymer matrix. Composites are defined as a matrix of polymeric material reinforced by fibers or other reinforcement with a discernible aspect ratio of length to thickness.

5.3.1.1 Reinforcing Fibers

The principal fibers in common commercial use for civil engineering applications include glass, carbon and aramid. The most common form of fiber-reinforced composites used in structural applications is called a laminate. Laminates are made by stacking a number of thin layers (laminae) or fibers and matrix and consolidating them into the desired thickness. Fiber orientation in each layer as well as the stacking sequence of the various layers can be controlled to generate a wide range of physical and mechanical properties for the composite laminate.

A composite can be any combination of two or more materials so long as the material properties of different and there is a recognizable region for each material. The materials are intermingled. There is an interface between the materials, and often an interphase region such as the surface treatment used on fibers to improve matrix adhesion and other performance parameters via the coupling agent.

The performance of the composite depends upon the materials of which the composite is manufactured, the arrangement of the primary load-bearing reinforcing fiber portion of the composite and the interaction between these materials. The major factors affecting the performance of the fiber matrix composite are: fiber orientation, length, shape and composition of the fibers, the mechanical properties of the resin matrix and the adhesion or bond between the fibers and the matrix.

Glass has been the predominant fiber for many civil engineering applications because of an economical balance of cost and specific strength properties. Glass fibers are commercially available in "E-Glass" formulation (for "Electrical" grade), the most widely used general-purpose form of composite reinforcement, high strength S-2 glass and ECR Glass, a modified E-Glass which offers greater alkali resistance. Although considerably more expensive than glass, other fibers including carbon and aramid are used for their strength or modulus properties or in special situations as hybrids with glass.

There are three sources for commercial carbon fibers: pitch (a by-product of petroleum distillation), PAN (polyacrylonitrile) and rayon. The properties of carbon fiber are controlled by its molecular structure, and degree of freedom from defects. The formulation of carbon fibers requires processing temperatures above 1000C. At this temperature, most synthetic fibers will melt and vaporize; however acrylic does not and its molecular structure is retained during high temperature carbonization.

There are two types of carbon fiber — the high modulus type, Type I, and the high strength type, Type II. The difference in properties between Types I and II is a result of the differences in the fiber microstructure. These properties are derived from the arrangement of the graphene (hexagonal) layer networks present in graphite. If these layers are present in three-dimensional stacks, the material is defined as graphite. If the bonding between layers is weak and two dimensional layers occur, the resulting material is termed as carbon.

5.3.1.2 Polymer Resins

Unsaturated polyester (UP) is the polymer resin most universally used to produce large composites for structural applications. The Composites Institute estimates that approximately 85% of U.S. composites production is based on unsaturated polyester resins. As mentioned earlier, these resins are typically in the form of low viscosity liquids during processing or until cured. However, partially processed materials containing fibers can also be used under specific conditions of temperature, and pressure. This class of materials has its own terminology, with the most common pre-production forms of partially reacted or chemically-thickened materials being prepreg (pre-impregnation) and sheet molding compounds (SMC).

Unsaturated polyesters are produced by reacting a di-basic acid containing maleic anhydride, or fumaric acid. The resulting polymer is then dissolved in a reactive vinyl monomer such as styrene. The viscosity of the solutions will depend on the ingredients, but typically range between 200 to 2,000 centipoises. Addition of heat and/or free-radical initiator such as an organic peroxide causes a chemical reaction which results in non-reversible cross-linking between the unsaturated polyester polymer and the monomer. Room temperature cross-linking may also be accomplished by using peroxides and suitable additives (typically promoters).

5.3.2 Physical and Mechanical Properties

The physical and mechanical properties are obtained by tests on either Fiber Reinforced Polymers (FRP) bars or tendons. It should be recognized that FRP bar is anisotropic, with the longitudinal axis being the strong axis. Second, unlike steel, the mechanical properties of FRPs vary significantly from one product to another. Factors such as volume and type of fiber and resin, fiber orientation, and quality control during the manufacturing, play a major role in establishing the characteristics of the product. Furthermore, the mechanical properties of FRPs, like all composites, are affected by such factors as the loading history and duration, temperature, and moisture.

5.3.2.1 Specific Gravity

FRP bars and tendons have a specific gravity ranging from 1.5 to 2.0; i.e. they are nearly 1/4 of the weight of steel. The reduced weight of the materials leads to lower transportation and storage costs and less handling on the job site and reduced installation time as compared to steel rebars. This is a major advantage which must be included in any cost analysis for product selection.

5.3.2.2 Thermal Expansion

Reinforced concrete itself is a composite material, where the reinforcement acts as the strengthening fiber and the concrete as the matrix. It is therefore imperative that the behavior under thermal stresses for the two materials be similar so that the differential deformations of concrete and the reinforcement are minimized. Depending on the mix proportions, the linear coefficient of thermal expansion for concrete varies from 6 to 12 x 10-6 per oC [Mindess and Young 1981].

5.3.2.3 Tensile Strength

FRP bars and tendons reach their ultimate tensile strength without exhibiting any yielding of the material. The mechanical properties of FRP reported in Table 5.2 were measured in the longitudinal (i.e. strong) direction. It is noted, however, that the values reported for FRP materials cover the range for some of the more commonly available materials. The exact product properties must be obtained from the manufacturer.

5.3.2.4 Tensile Modulus of Elasticity

The longitudinal modulus of elasticity of GFRP bars is approximately 25 percent of that of steel. The modulus for CFRP tendons, which usually employ stiffer fibers, is slightly higher than that for GFRP rebars.

5.3.2.5 Compressive Strength

FRP bars are weaker in compression than in tension. However, the compressive strength of GFRP is not a primary concern for most applications. The compressive strength also depends on whether the rebar is smooth or ribbed. Compressive strength in the range of 317 to 470 MPa (46 to 68 ksi) has been reported for rebars having a tensile strength in the range of 552 to 896 MPa (80 to 130 ksi) [Wu et al. 1991]. Higher compressive strengths are expected for bars with higher tensile strength.

5.3.2.6 Compressive Modulus of Elasticity

Unlike the tensile stiffness, the compressive stiffness of GFRP bars varies with rebar size, type, quality control in manufacturing and the length to diameter ratio of the specimens. The compressive stiffness for GFRP rebars is smaller than the tensile modulus of elasticity. Based on tests of samples containing 55% to 60% volume fraction of continuous E-glass fibers in a matrix of vinylester or isophthalic resin, a modulus of 34 GPa (5,000 to 7,000 ksi) has been reported [Wu et al. 1991]. Another manufacturer reports the compressive modulus at 34 GPa (5,000 ksi) which is approximately 77% of the tensile modulus for the same product [Bedard 1992].

5.3.2.7 Shear Strength

The shear strength of composites is, in general, very low. GFRP bars, for example, can be cut very easily in the direction perpendicular to the longitudinal axis with ordinary saws. This shortcoming can be overcome in most cases by orienting the rebars such that they will resist the applied loads through axial tension.

5.3.2.8 Creep and Creep Rupture

Fibers such as graphite and glass have excellent resistance to creep, while the same is not true for most resins. Therefore, the orientation and volume of fibers have a significant influence on the creep performance of rebars/tendons. One study reports that for a high quality GFRP rebar, the additional strains caused by creep were estimated to be only 3% of the initial elastic strains [Iyer and Anigol 1991].

Under adverse loading and environmental conditions, FRP rebars subjected to the action of a constant load may suddenly fail after a time, referred to as the endurance time. This phenomenon, known as creep rupture, exists for all structural materials including steel. However, for steel prestressing strands, this is not of concern. Steel can endure the typical tensile loads, which are about 75% of the ultimate strength, indefinitely without any loss of strength or fracture. As the ratio of the sustained tensile force to the short-term strength of the FRP increases, the endurance time decreases. Creep tests were conducted in Germany on GFRP composites with various cross sections. These studies indicate that stress rupture diminishes if the sustained loads are limited to 60% of the short-term strength of the sample [Budelmann and Rostasy 1993].

5.3.2.9 Fatigue

CFRP and GFRP bars exhibit good fatigue resistance. Most of the research in this regard has been on high-performance fibers, such as graphite, which are subjected to large cycles of loading in aerospace applications. In tests where the loading was repeated for 10 million cycles, it was concluded that graphite-epoxy composites have better fatigue strength than steel, while the fatigue strength of glass composites is lower than steel [Scwartz, 1992]. In another investigation, GFRP rods constructed for prestressing applications were subjected to repeated cyclic loading with a maximum stress of 72 ksi and a stress range of 50 ksi. The rods could stand more than 4 million cycles of loading before the failure initiated at their anchorage zone [Franke 1981].

5.3.2.10 Durability

In general, FRP materials exhibit excellent durability. Some studies have shown, however, that certain glass and aramid fibers deteriorate when subjected to an alkaline environments, such as that found in concrete. Sen et al. [1993] found that S-2 glass fibers used in prestressed beams showed a rapid and substantial loss in load carrying capacity when exposed to a 15% salt solution. Research by GangaRao et al. [1995]indicated that depending on the resin, strength losses by as much as 64% and stiffness losses up to 9% were found for glass fiber composites that were placed in an alkaline environment. Common aramid fibers can be degraded by strong acids and bases [ACI 440 1996]. Relaxation of Arapree composites were found to be about 40% higher in an alkaline environment at room temperature than in air [Gerritse and Den Uijl 1995]. These fibers are also subject to degradation when exposed to UV rays present in sunlight. This problem can be overcome by embedding the aramid in concrete or by using specially formulated resins. Though the aramid fibers have shown some deterioration, aramid composites have not shown any strength loss. Carbon fibers have been found to be resistant to most chemicals and exhibit excellent durability characteristics.

Matrix materials that resist moisture infiltration or diffusion of hydroxyl ions can help protect fibers against harsh environments. Research by GangaRao et al. [1995] found that the composite resins play a significant role in the durability of glass composite rods. Frequently, durability problems of composite are the result of the properties of the matrix. Moisture absorption in the matrix can result in swelling of the matrix and loss of strength and stiffness of the composite. Fire and high temperature are also a problem more severe for the matrix than the fibers.

5.3.3 Field Applications

Composite materials have been used in a variety of civil engineering applications with both reinforced and prestressed concrete. They are manufactured as reinforcing elements, as prestressing and post-tensioning tendons and rods, and as strengthening materials for rehabilitation of existing structures. Several new structures utilizing FRP reinforcement are currently underway by the West Virginia

Department of Transportation and by the Florida Department of Transportation. One such application is a 52 m (170 ft) three-span continuous bridge deck reinforced with FRP reinforcing bars. This chapter describes FRP applications in concrete reinforcement. The projects are grouped under the method of application, either as reinforced concrete, prestressed concrete, or external reinforcement.

5.3.3.1 Applications in North America

- **A. Bridge in Calgary, Canada**
  A concrete highway bridge was constructed using carbon fiber composite cables (CFCC) and LeadlineR tendons. The bridge is a two-span continuous skew (33.3o) bridge with spans of 22.83 and 19.23 m (75 and 63 ft). Thirteen precast prestressed concrete girders of bulb-tee section were used for each span. The girders are 1.1 m (43 in.) deep and have a 160 mm (6 in.) web thickness. Of the 26 girders, four (two in each span) were prestressed using CFCC cables of 15.2 mm (0.6 in.) in diameter. Two additional girders (one in each span) were prestressed by 8 mm (0.3 in.) diameter LeadlineR rods. These six girders were located at the center of the bridge. The remaining girders were prestressed with steel strands. This bridge is monitored by an optical fiber system consisting of intra-core Bragg grating optic fiber sensors and electric strain gauges attached to CFCC, LeadlineR rods, and steel strands [Rizkalla and Tadros 1994].
- **B. Bridge in Rapid City, South Dakota**
  A precast post-tensioned bridge of 9 m (30 ft) span and 5.2 m (17 ft) width was erected at a cement plant in 1992 [Iyer and Anigol 1991]. It has a 180 mm (7 in) thick deck slab supported by three longitudinal girders. Cables of three different materials were used to prestress the slab. Thirty GFRP cables were used to prestress one-third of the length of the slab, thirty CFRP cables were used to prestress the next third of the length, and steel cable prestressing was adopted for the remaining length. Each GFRP and CFRP cable consists of seven 4-mm (0.156 in) diameter rods. The initial prestress and final prestress after all losses were set at 0.6Pu and 0.5Pu respectively. Plastic ducts housing the cables were grouted with high-strength epoxy and mortar for bonding purposes, and the temporary anchorages were removed. Monitoring of bridge deflections and stresses in cables and concrete was carried out to assess losses in the cables and the actual deflections under moving loads.
- **C. Waterfront Structure in Port Hueneme, California**
  This waterfront structure, constructed in 1994, consists of two full-scale bays; one is a prestressed deck with graphite cables, and the other is a fiberglass composite deck. The bays are supported by twelve 356 x 356 mm (14 x 14 in.) piles, 13.7 m (45 ft) long, and reinforced with six prestressed CFRP cables and CFRP spirals. The pile caps were post-tensioned with GFRP (E-glass) cables. The deck was designed for 1,000 kN (225 kips) applied on a 762 mm (30 in.) square area and tested at service load conditions. A total of 180 CFRP cables were used to prestress the 6 m (20 ft) long deck, which has a 5.5 m (18 ft) width and a 457 mm (18 in) thickness.
- **D. Column Wrapping Projects in California**
  As a part of its general seismic upgrading program, the California Department of Transportation (Caltrans) placed confining jackets around bridge columns using fiberglass mat. Epoxy was used to provide the lap bond splices for the fiberglass mats. Expansive grout was injected beneath the mat to ensure contact with the original concrete. Eleven 1.8 m (6 ft) diameter and four 1.2 m (4 ft) diameter columns were wrapped. These columns, which are located 32 km (20 mi) from Northridge, suffered no damage in the January 17, 1994 earthquake. This wrapping technique has been used on other projects in California. For example, the cities of Los Angeles and Santa Monica used composite wrapping materials on approximately 200 columns in 1993 and 1994.
- **E. Column Wrapping Projects in Reno, Nevada**
  In 1993, the Nevada Department of Transportation wrapped ninety-six 0.3m (1 ft) diameter columns with a proprietary FRP wrapping system. The columns were part of an interstate highway bridge constructed over a casino, with 48 of those columns actually located within the casino. No odor was detected from the TYFOR S epoxy.
- **F. Foulk Road Bridge in Wilmington, Delaware**
  Carbon fiber Forca^{R} tow sheets were used on this 16.5 m (54ft) long, simple-span, prestressed,

precast box beam structure that exhibited cracking indicative of the lack of transverse reinforcement. The bridge's superstructure was composed of 24 prestressed box beams placed adjacent to each other. For demonstration purposes, six of the beams were retrofitted. The design of the rehabilitation replicated the strength that 12.5 mm (0.5 in) diameter steel bars would have provided had they been installed in the original casting of the beams. One ply of unidirectional CFRP sheet, with the fibers running transverse to the beam, was used on four beams. Two other beams used a higher modulus, higher weight fabric, with one of those beams fitted with two plies rather than a single ply.

5.3.3.2 Applications in Europe

- **A. Marienfelde Bridge in Germany**
  A pedestrian bridge in a Berlin park was constructed in 1988 using external prestressing. The superstructure is a two-span double-T beam slab partially prestressed by seven FRP tendons. Spans are 27.6 and 22.9 m (90 and 75 ft). The double-T beam is 5 m (16.5 ft) wide and 1.1 m (3.6 ft) deep.
- **B. Ludwigshafen Bridge in Germany**
  CFCC strands developed in Japan were used as part of the tensioning materials for a prestressed concrete bridge near a chemical plant. The bridge, which crosses a number of railway tracks, carries heavy truck traffic and was designed to a German road bridge specification requiring a 600/300 kN (135/67.5 kip) load rating. The two-lane bridge has a total length of 85 m (280 ft), and consists of four equal spans: two straight spans and two curved spans with a radius of 62.8 m (206 ft). The superstructure is modeled as a slab-and-beam bridge with a total width of 11.2 m (36.8 ft), and a height of 1.12 m (3.7 ft).
- **C. Lunen'sche-Gasse Bridge in Germany**
  Polystal[R] GFRP tendons were used in 1980 in a 6.55 m (21.5 ft) single-span slab bridge. One hundred GFRP rods 7.5 mm (0.3 in) in diameter were grouped in unbonded prestressing tendons with a length of 7 m (23 ft). Four different anchorage systems were tested. Monitoring of the tensile forces, carried out over a period of 5 years on the grouted anchorage, demonstrated satisfactory performance.
- **D. Ulenberg-Strasse Bridge in Germany**
  This post-tensioned bridge was erected in 1986 with a total of 59 multi-cables, each consisting of nineteen 7.5 mm (0.3 in.) diameter PolystalR GFRP rods. The allowable tensile force of the cables under service loads was set at 0.47Pu. Cables were anchored with a mortar made of silica sand and polyester resin. Synthetic resin mortar was injected as a grout to overcome the weakness of glass fiber to alkali attack. Similar to the "Lunen'sche-Gasse" bridge, E-glass fibers in a polyester resin matrix with external coating of polyamide were used. The slab bridge was designed for the German 600/300 kN (135/67.5 kip) load class, and has two spans which are 21.3 m (70 ft) and 25.6 m (84 ft) long.
- **E. Schiessbergstrasse Bridge in Germany**
  This post-tensioned road bridge, designed for the German 600/300 kN (135/67.5 kip) load class, was built in 1990. This is a two-lane triple-span bridge with spans of 16.3, 20.4 and 16.3 m, (53.5, 67 and 53.5 ft). The slab width is 9.70 m (32 ft) and its thickness is 1.12 m (3.7 ft). A total of 27 FRP tendons were used. The bridge has a sophisticated permanent monitoring system with the possibility of on-line diagnostics.
- **F. Notsch Bridge in Austria**
  This post-tensioned bridge, which has PolystalR GFRP prestressing tendons, was built in 1991. It is a two-lane three-span bridge, with spans of 13, 18, and 13 m (42.7, 59, and 42.7 ft). The slab width is 12 m (39.4 ft), and its thickness is 0.65 m (2.1 ft). It has a permanent monitoring system similar to that on the Schiessbergstrasse bridge.
- **G. Ibach Bridge in Switzerland**
  Accidental damage to a prestressing tendon during maintenance work necessitated repair of this bridge in 1991. Three 5 m (16.5 ft) long CFRP laminates, two with 150 x 1.75 mm (6 x 0.07 in.)

cross sections and one with a 150 x 2.0 mm (6 x 0.08 in.) cross section were applied to the bottom surface of the bridge.

5.3.3.3 Applications in the Japan

- **A. Hakui Cycling Road Bridge**
  Erected in 1991, this pretensioned hollow slab system utilizes CFCC strands as tendons [Minosaku 1991]. In addition to the prestressing tendons, straight CFRP wires are used as substitutes for diagonal tension reinforcing bars in one-half of the outside girders.
- **B. Tabras Golf Club Bridge**
  Braided AFRP (FiBRA^R) rods of 14 mm (0.55 in) diameter were used in three of the 21 girders for this slab bridge constructed in 1990 [Tamura and Tezuka 1990]. The bridge is 2.40 m (7.9 ft) wide, with three 11.98 m (39.3 ft) spans, and conforms to the Japan Institute of Standards (JIS) specifications. The allowable tensile force in the AFRP rods was set at $0.5P_u$
- **C. Takahiko Floating Bridge**
  Completed in 1992, this bridge was partially prestressed using 13 and 15 mm (1.0 and 1.2 in) diameter FiBRA^R rods.
- **D. South Yard Country Club Suspension Bridge**
  Constructed in 1990, this bridge was post-tensioned with 4.86 x 19.5 mm (0.2 x 0.75 in.) flat AFRP (Arapree^R) strips. The anchorage for the post-tensioning was provided by using mortar-filled sleeves.
- **E. Iwafune Golf Club Cable-Stayed Bridge**
  Constructed in 1992, this bridge was partially prestressed using GFRP and CFRP rods. Anchorage was provided by resin-filled sleeves. The initial prestressing was $0.3P_u$
- **F. Tsukude Golf Country Club Bridge**
  Constructed in 1993, this cantilevered-type pedestrian bridge has a single span of 99.0 m (325 ft) and a width of 3.6 m (12 ft). The main girders were post-tensioned with 12.5 mm (0.5 in.) diameter CFCC rods. The bridge was designed based on the Japan Road Association (JRA) specification for highway bridges.
- **G. Nakatsugawa Pedestrian Overbridge**
  CFCC strands were used in this pre-tensioned simple slab bridge built in 1989. The bridge was prefabricated in a single piece, and has a width of 2.5 m (8.2 ft) and a length of 8.0 m (26.2 ft). The anchorages used in fabrication were made of threaded steel pipes, with CFCC tendon ends inserted into the pipes and epoxy-injected. The allowable tensile forces were $0.60P_u$ during prestressing, 0.55Pu immediately after prestressing, and $0.50P_u$ under service loads. This bridge was fabricated as a nonmetallic structure, and CFRP reinforcing bars with surface ribs and lugs to improve the bond with concrete were used as stirrups and temperature reinforcement.
- **H. Birdie Bridge**
  This was a post-tensioned suspended slab bridge 2.1 m (6.9 ft) wide and 54.5 m (179 ft) long. Eight AFRP tapes, each with a 4.86 x 19.5 mm (0.2 x 0.75 in.) cross section, were bundled to make a single cable. A total of 16 cables were used, with allowable tensile forces for the cables set at $0.5P_u$ under initial force, and 0.33Pu under service load. The cables were anchored by a sleeve filled with an expansive mortar. AFRP tapes were also used as non-prestressed reinforcement in the slab. For ground anchors, cables consisting of nine 8 mm (0.3 in.) diameter CFRP rods were used.
- **I. Shinmiya Bridge**
  This was the first application where carbon fiber composite cable (CFCC) strands were used as tensioning materials for a prestressed concrete bridge, constructed in October 1988 on a national highway. The span and width of the bridge are 5.76 and 7.0 m (19 and 23 ft), respectively. Seven-wire 12.5 mm (0.5 in.) diameter CFCC strands were arranged into six tendons at the bottom flange and two tendons at the top flange. Allowable tensile forces were $0.60P_u$ during

prestressing, $0.53P_u$ immediately after prestressing, and $0.45P_u$ under service loads. Epoxy-coated steel reinforcing bars were used for stirrups.

- **J. Bachigawa Minamibashi Bridge**
  This 18.6 m (61 ft) single-span bridge was constructed in 1989. The design conformed to the JRA specification for highway bridges. The post-tensioning for the precast hollow girder used Leadline[R] CFRP cable.
- **K. Sumitomo Bridge**
  A road bridge, erected at the freight entrance to a concrete products plant, was reinforced with pultruded AFRP rods [Mizutani et al. 1991]. This was a prestressed concrete road bridge consisting of a 12.5 m (41 ft) span pre-tensioned composite slab, and a 25.0 m (82 ft) span post-tensioned box girder. AFRP rods were used for all tendons in both spans. The pretensioned composite slab used AFRP as stirrups and reinforcing bars. AFRP cables consisted of three twisted strands for the pretensioned composite slab, and 19 and seven twisted strands, respectively, for the internal and external cables of the post-tensioned box girder. The allowable tensile forces were $0.8P_u$ during prestressing, $0.7P_u$ immediately after prestressing, and $0.6P_u$ under service loads.
- **L. Kitakyushu Bridge**
  The width of this bridge is 12.3 m (40 ft) and the total length is 35.8 m (117.5 ft), with one span a 18.25 m (60 ft) pre-tensioned girder and the other span a 17.55 m (57.5 ft) post-tensioned girder. The tendons used were multi-cable bundles of eight 8 mm (0.3 in.) diameter CFRP rods. Eight multi-cables in all were used. The allowable tensile force under service loads was set at $0.55P_u$. Anchoring of the cables was achieved with a steel wedge-type anchor, with field monitoring in progress. The tensioning operation was conducted in three stages to reduce the difference in tensioning force between cables.
- **M. Maglev Guideway**
  The precast side wall beams of this guideway are simply supported girders partially prestressed with 12.5 mm (0.5 in.) diameter CFCC strands. The non-magnetic property of FRP materials was well utilized here. The FRP strand is twisted seven-wire impregnated in epoxy resin. Anchoring was obtained by direct bonding to concrete.
- **N. Kuzuha Quay Landing Pier**
  This structure was built in 1993 based on Japan's harbor structures design standards. Both pre-tensioning and post-tensioning tendons were CFCC strands of 12.5 mm (0.5 in.) diameter. End anchorage was obtained by direct bonding to concrete and by metal die-cast wedges.
- **O. Airport Pavement**
  The non-magnetic test pavement at Haneda International Airport has concrete pavement post-tensioned with CFRP and AFRP bonded tendons. The anchorage for post-tensioning was obtained using resin casting and wedges, and was not required to hold the permanent load.
- **P. Wrapping Projects**
  Forca[RM] tow sheet has been used extensively in Japan in over 200 projects including tunnels, chimneys, side walls, and slabs.

5.4 References

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Table 5.1 Fiber parameters and concrete matrix characteristics (Johnston and Zemp 1991)

Mixture No.	Fiber parameters				Product VL/D	Mixture parameter		Flexural strengths	
	Volume V, %	Size, mm	Type*	Aspect ratio, L/D		w/c	w/(c+f)	First crack, MPa	Ultimate MPa
1	None	-	-	-	-	0.51#	-	-	4.45
2	0.5	19 x 0.25	SW	75	38	0.54t	0.39	6.19	6.31
3	1.0	19 x 0.25	SW	75	75	0.64t	0.46	7.53	7.76
4	1.5	19 x 0.25	SW	75	113	0.68t	0.49	8.11	8.45
5	1.0	13 x 0.25	SW	50	50	0.60t	0.43	7.88	7.93
6	1.0	25 x 0.25	SW	100	100	0.68t	0.49	7.12	7.34
7	1.0	19 x 0.41	SDW	47	47	0.64t	0.46	7.53	7.59
8	1.0	32 x 0.58	ME	54	54	0.64t	0.46	6.92	7.00
9	1.0	25 x 0.5 x 0.25	SS	71	71	0.68t	0.49	6.48	6.62

* SW is smooth uniform wire, SDW is surface-deformed wire, ME is melt extract, and SS is slit sheet. All are straight and uniform in cross section without hooked or enlarged ends.

No fly ash and 312 kg/m^3 (525 lb/yd^3) of cement.

t 119 kg/m^3 (200 lb/yd^3) of fly ash with 297 kg/m^3 (500 lb/yd^3) of cement.

1 in. = 25.4 mm; 1 MPa = 145 psi.

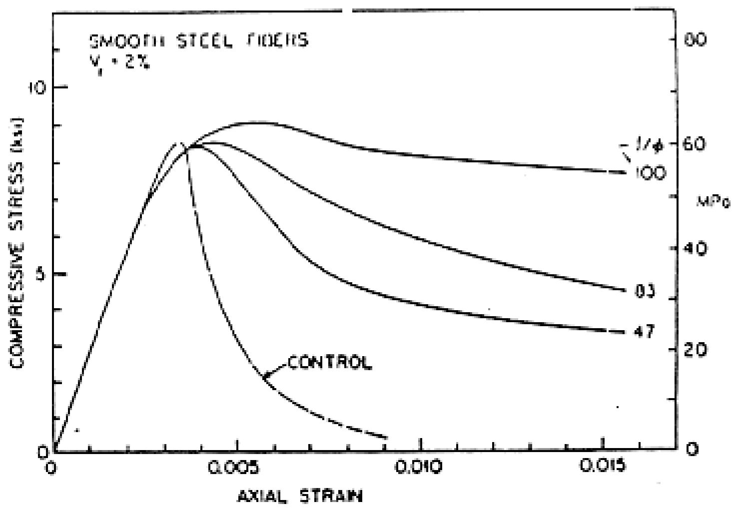
Table 5.2 Comparison of mechanical properties (Longitudinal directions) (ACI 440 1996)

	Steel Reinforcing Bar	Steel Tendon	GFRP Bar	GFRP Tendon	CFRP Tendon	AFRP Tendon
Tensile Strength, MPa (ksi)	483-690 70-100	1379-1862 200-270	517-1207 75-175	1379-1724 200-250	165-2410 240-350	1200-2068 170-300
Yield Strength, MPa (ksi)	276-414 40-60	1034-1396 150-203	Not Applicable			
Tensile Elastic Modulus, GPa (ksi)	200 29000	186-200 27000-29000	41-55 6000-8000	48-62 7000-9000	152-165 22,000-24,000	50-74 7000-11,000
Ultimate Elongination, mm/mm	>0.10	>0.04	0.035-0.05	0.03-0.045	0.01-0.015	0.02-0.026
Compressive Strength, MPa (ksi)	276-414 40-60	N/A	310-482 45-70	N/A	N/A	N/A
Coeff. of Thermal Exp. (10^{-6}/°C) (10^{-6}/°F)	11.7 6.5	11.7 6.5	9.9 5.5	9.9 5.5	0.0 0.0	-1.0 -0.5
Specific Gravity	7.9	7.9	1.5-2.0	2.4	1.5-1.6	1.25

Note:

- All properties refer to unidirectional reinforced coupons
  Properties vary with the fiber volume (45-70%), coupon diameter, and grip system
  N/A = Not Available

Figures



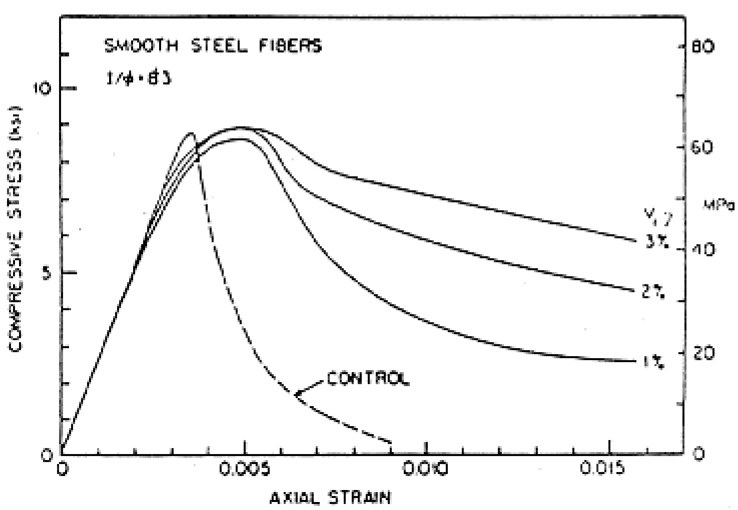


Figure 5.1 Typical effects of volume fraction and aspect ratio of fibers on the stress-strain curve of mortar [Gopalaratnam et al. 1984]

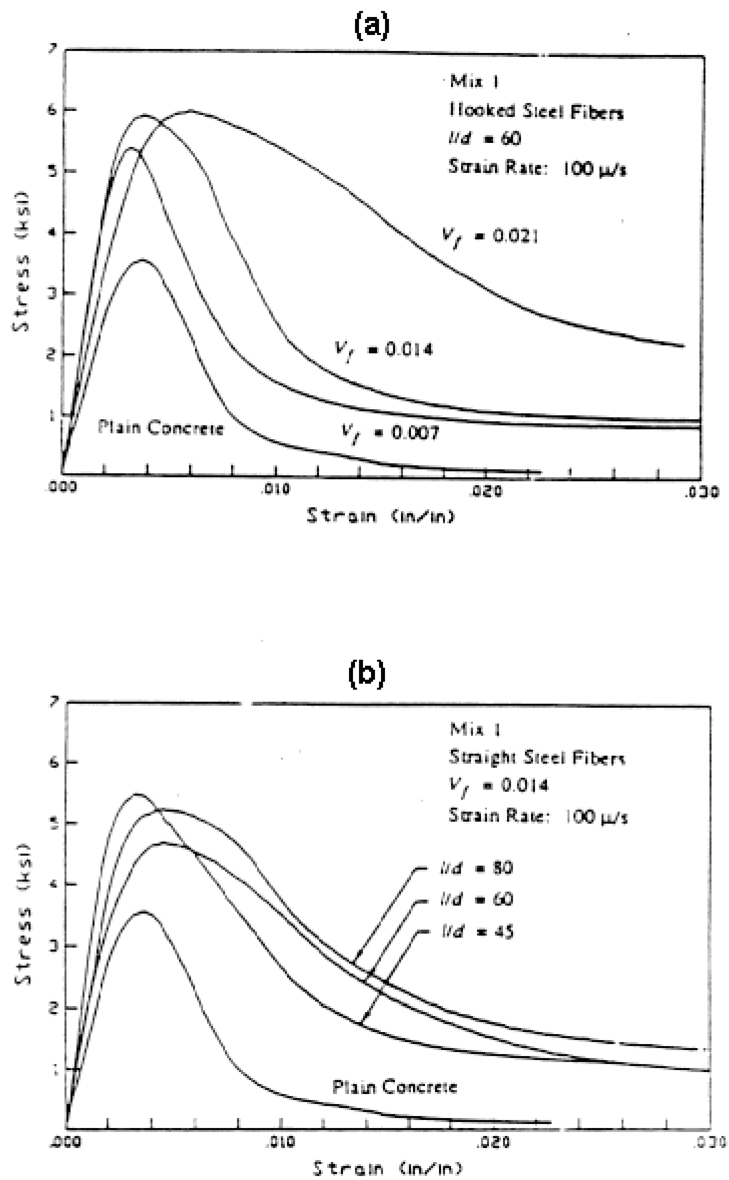


Figure 5.2 Stress-strain response of FRC under compression (a) hooked steel fibers; (b) straight steel fibers [Otter and Naaman 1988]

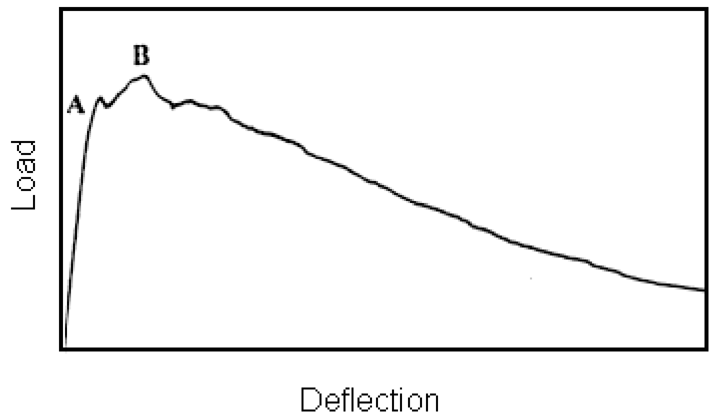


Figure 5.3 Schematic load-deflection diagrams of FRC

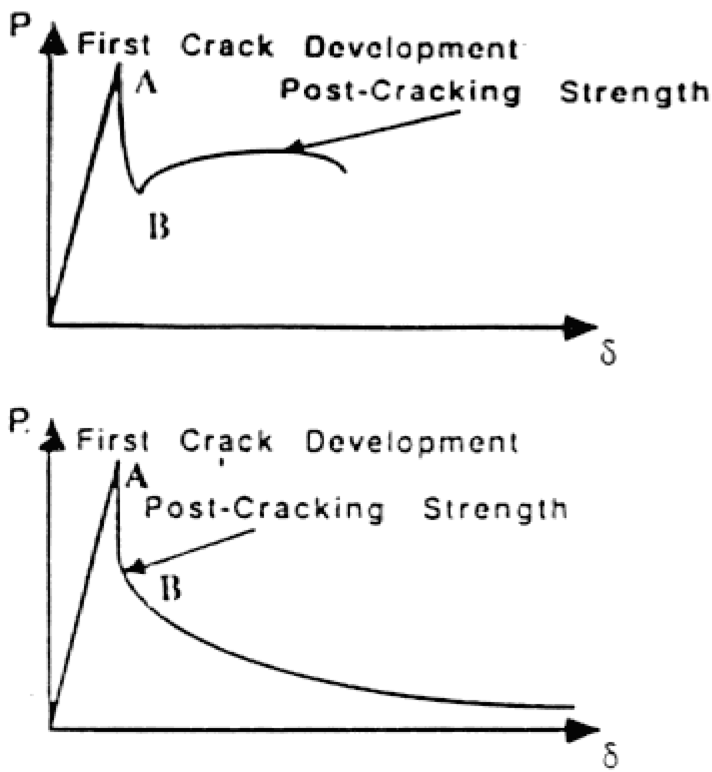


Figure 5.4 Typical load-deflection curves of FRC beams with low volume fraction of fibers

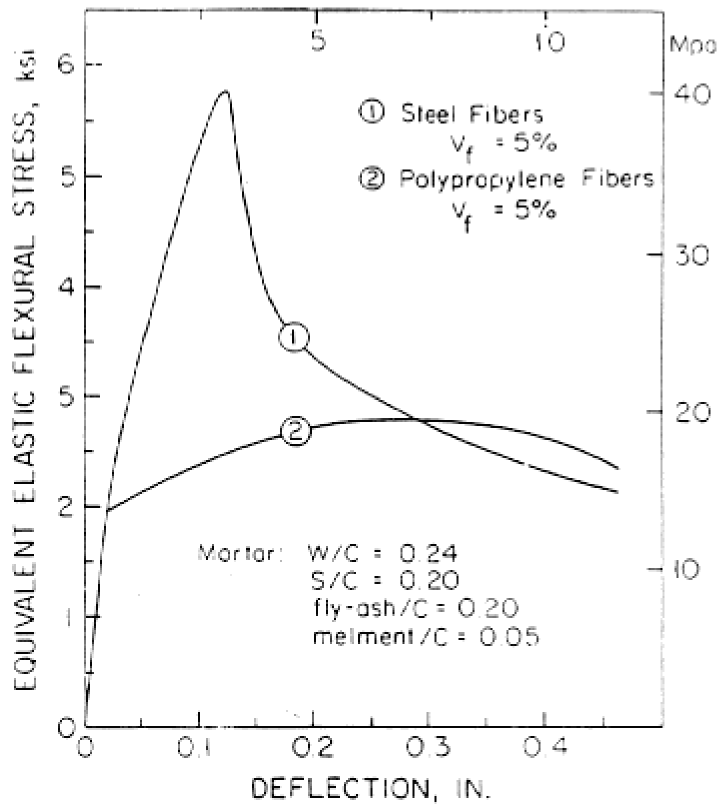


Figure 5.5 Experimentally observed load-deflection curves of steel and polypropylene fiber reinforced concrete beams [Naaman 1985]

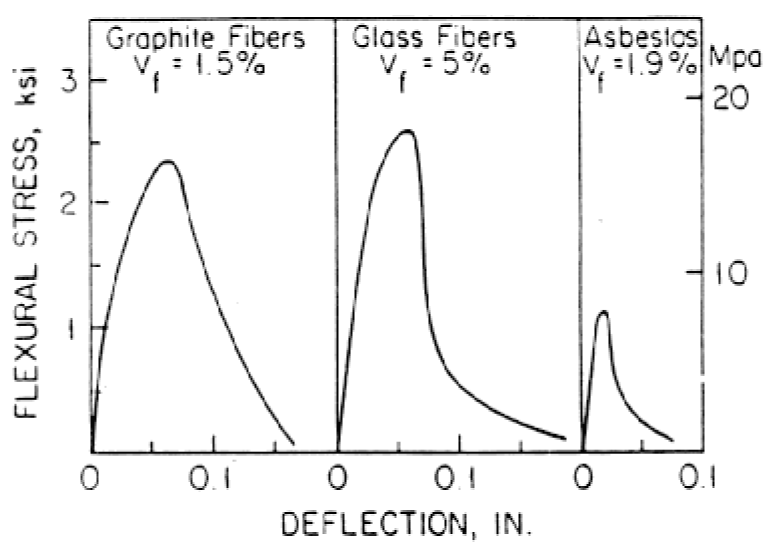
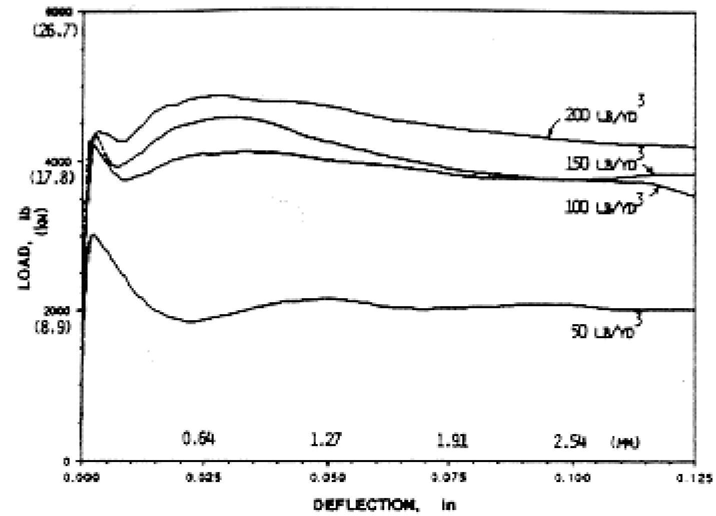
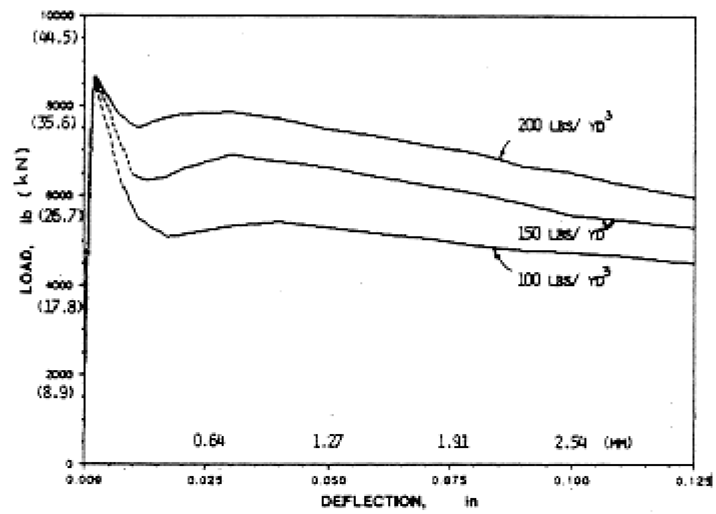


Figure 5.6 Experimentally observed load-deflection curves for different types of fibers [Naaman 1985]

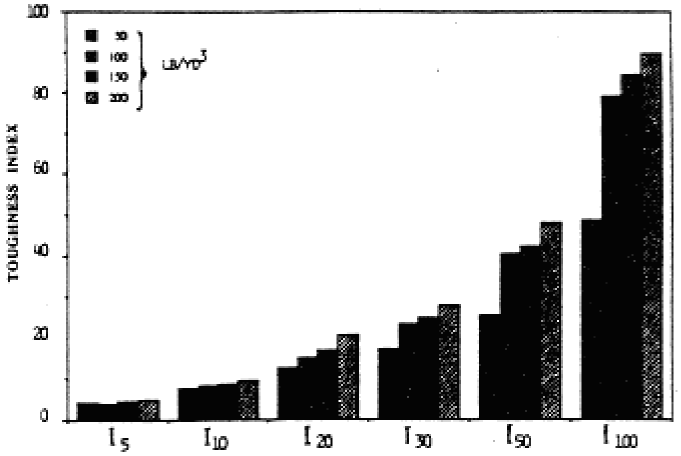


Load-deflection curves for 1.18-in. (30-mm) long hooked-end fibers, normal strength concrete

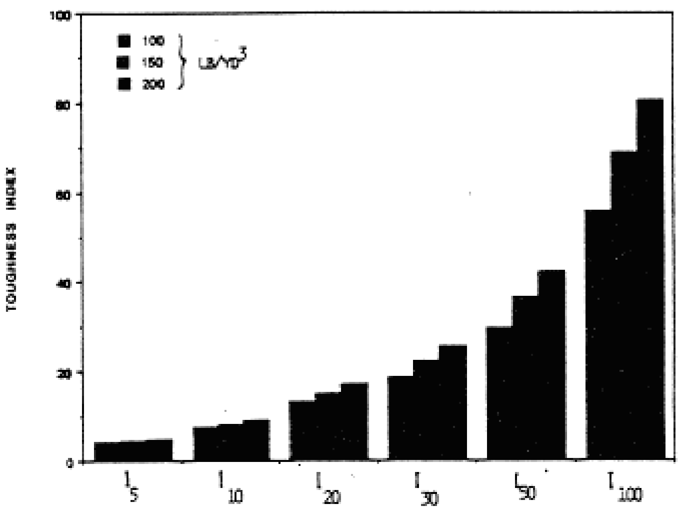


Load-deflection curves for 1.18-in. (30-mm) long hooked-end fibers, high-strength concrete

Figure 5.7a Load-deflection curves for 1.18 in. (30 mm) long hooked end fibers, normal and high strength concretes [Balaguru et al. 1992]

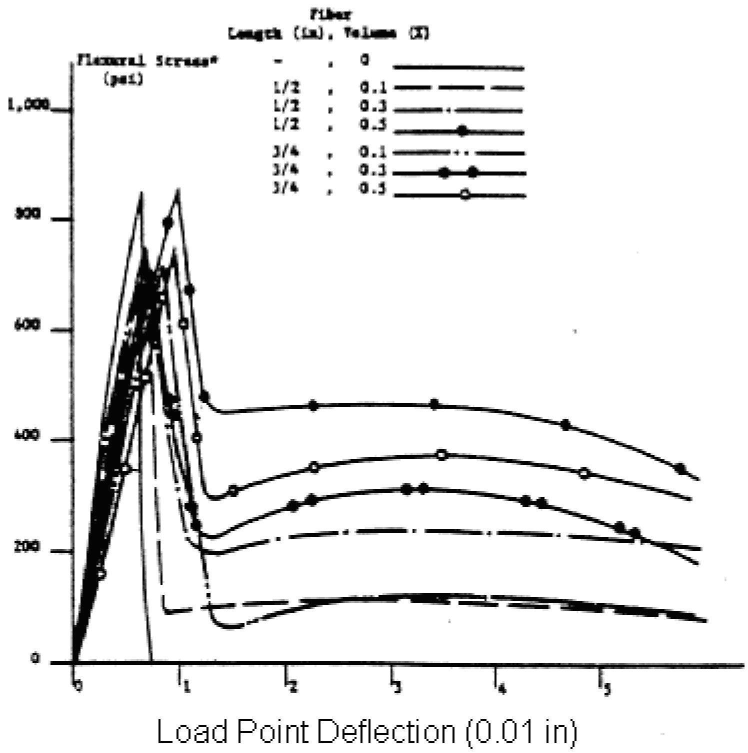


Toughness indexes: Influence of fiber volume fraction on 1.18-in. (30-mm) long hooked-end fibers, normal strength concrete



Toughness indexes: Influence of fiber volume fraction on 1.18-in. (30-mm) long hooked-end fibers, high-strength concrete

Figure 5.7b Toughness indexes for 1.18 in. (30 mm) long hooked end fibers, normal and high strength concretes [Balaguru et al. 1992]



*Obtained based on elastic section analysis.

Figure 5.8 Flexural load-deflection relationships of fibrillated polypropylene fiber reinforced concrete [Bayasi and Zeng 1993]

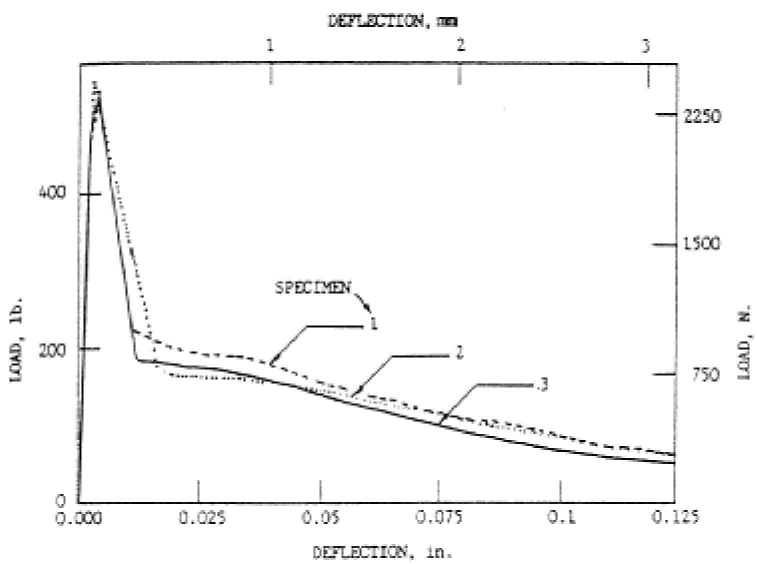


Figure 5.9 Typical load-deflection curves; polymer fibers, 2 x 2 x 13 in. (50 x 50 x 325 mm) beams [Balaguru 1992]

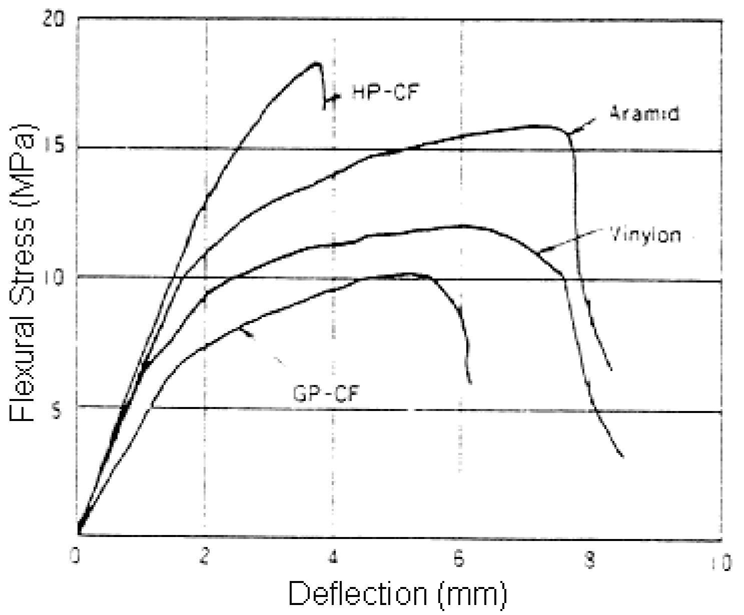


Figure 5.10 Load-deflection curves for fiber reinforced concrete with short discrete fibers [Nakagawa et al. 1989]

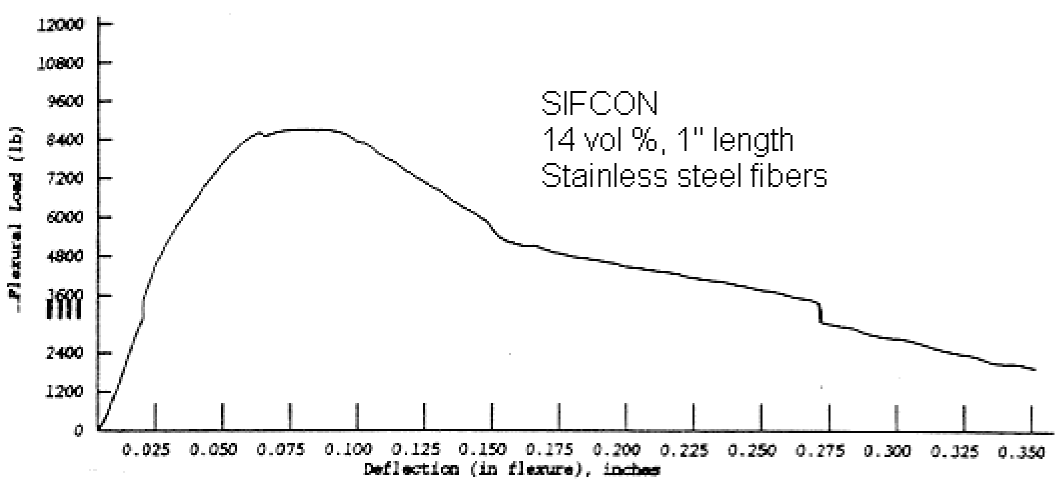


Figure 5.11 Load-deflection behavior of stainless steel SIFCON composites under third point loading with 12 in. span. Crossections are approximately 4 x 2 in. [Hackman et al. 1992]

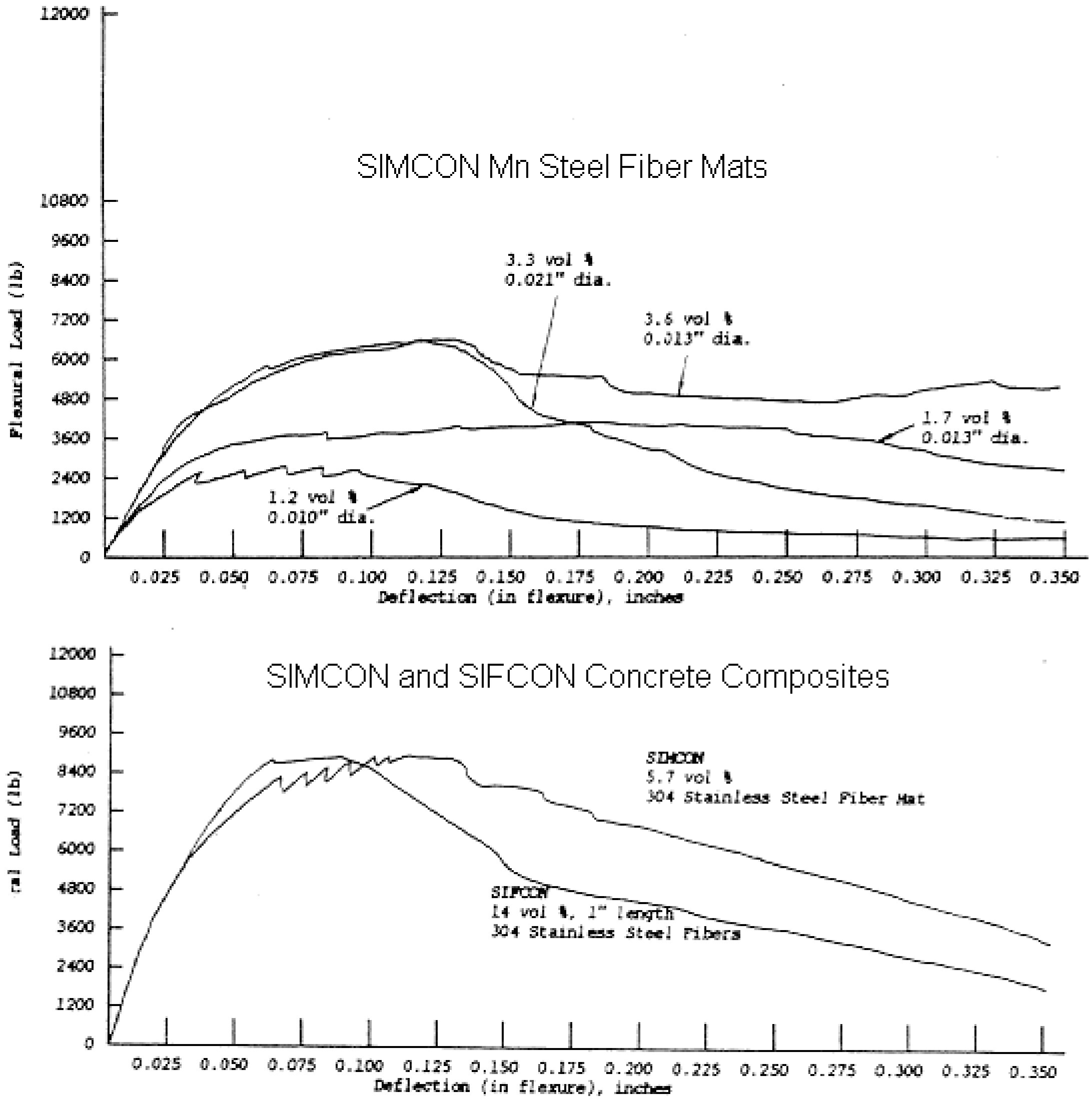


Figure 5.11c Load-deflection behavior of stainless steel SIFCON compared to SIMCON composite under third point loading with 12 in. span. Crossections are approximately 4 x 2 in. [Hackman et al. 1992]

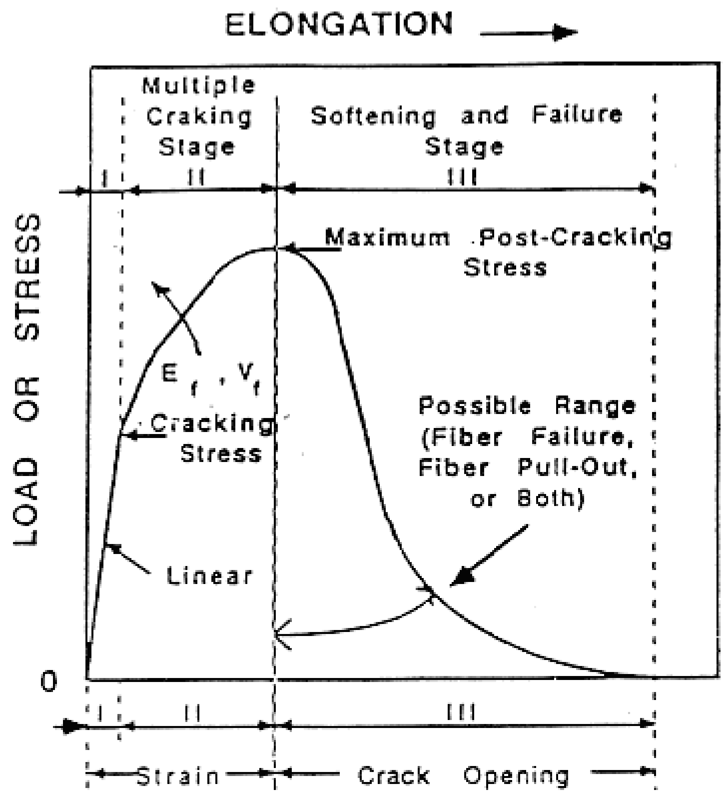


Figure 5.12 Typical load-elongation response in tension of high performance FRC composites as SIFCON [Naaman 1985]

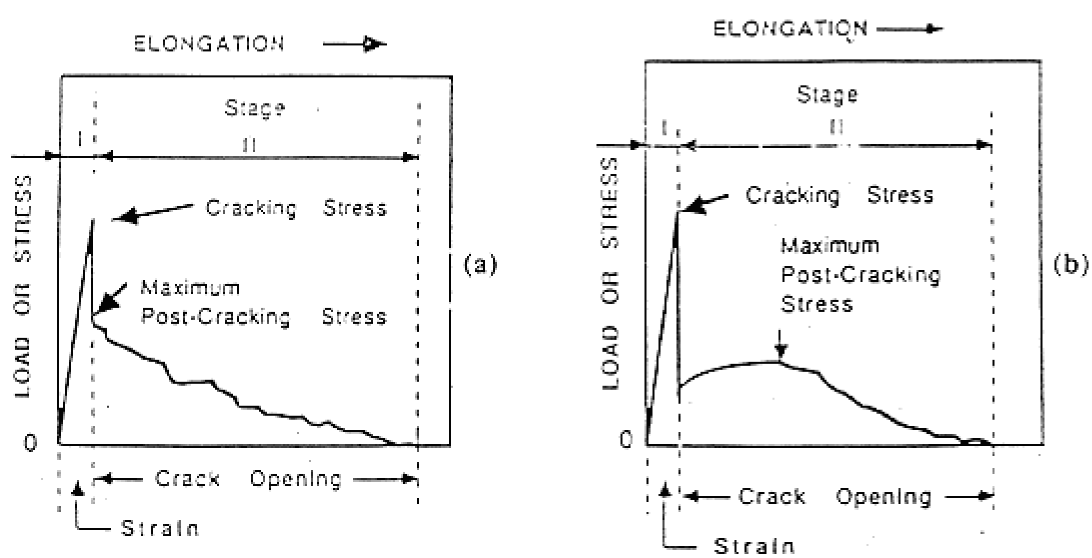


Figure 5.13 Typical load-elongation response in tension of fiber reinforced concrete; (a) using premixed steel fibers, and (b) using premixed polypropylene fibers [Naaman 1985]

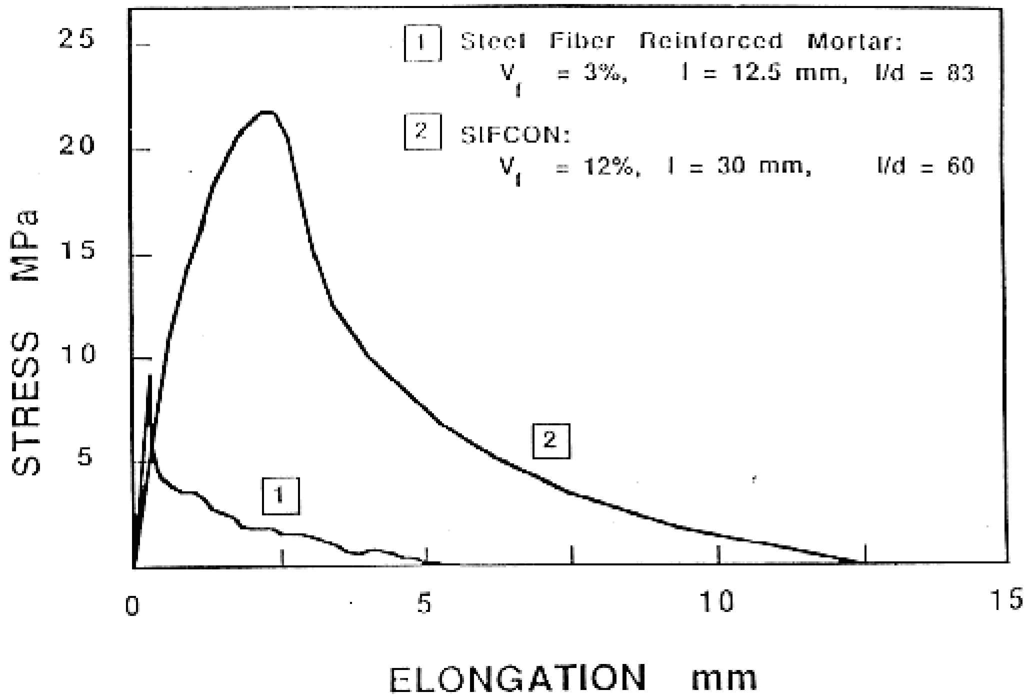


Figure 5.14 Comparison of experimentally observed stress-elongation curves of steel reinforced mortar and SIFCON [Naaman 1985]

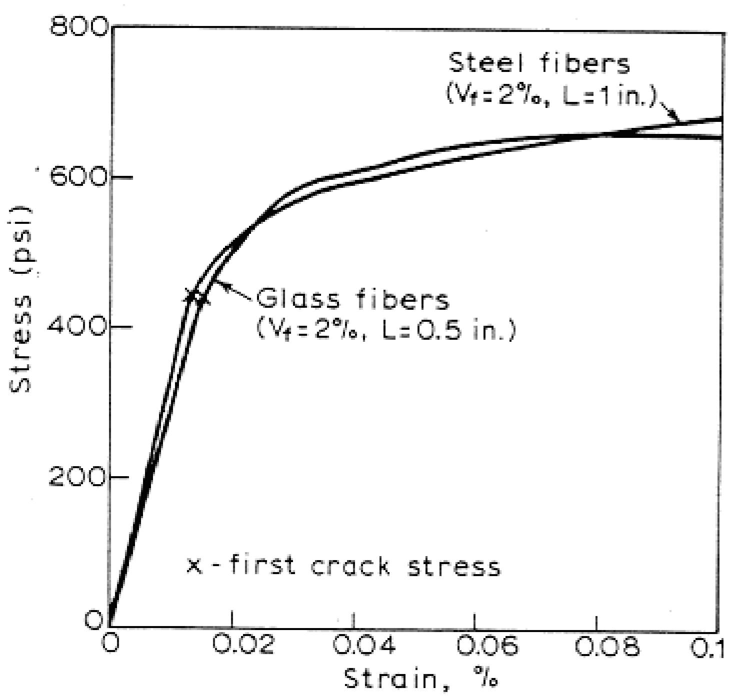


Figure 5.15 Tensile stress-strain curve for fiber reinforced concrete [Shah and Naaman 1976]

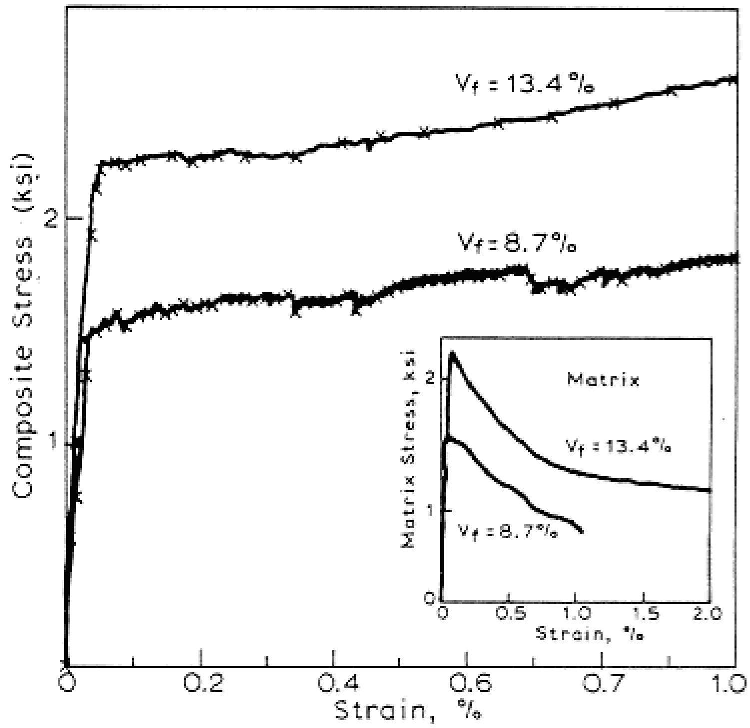


Figure 5.16 Stress-strain response of polypropylene fiber composites and corresponding matrix response computed from the analysis of rule of mixtures [Mobasher et al. 1990b]

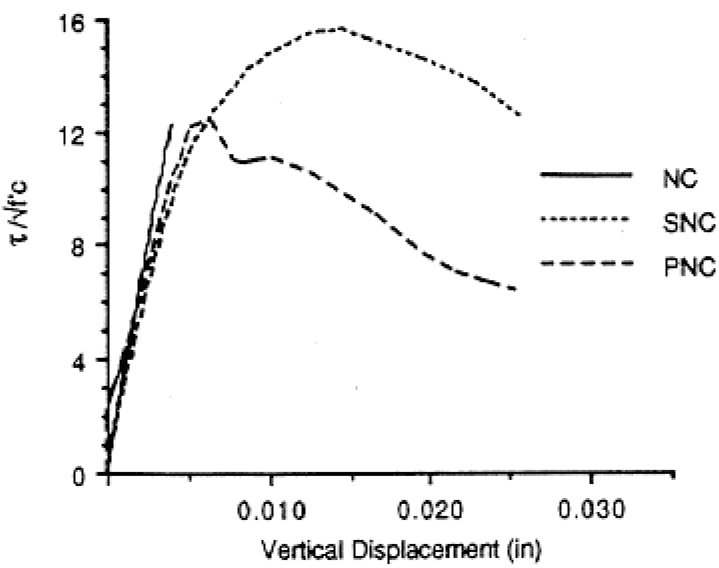


Figure 5.17a Normalized shear-stress versus vertical displacement for high strength concrete (HC), steel fiber reinforced high strength concrete (SHC) and polypropylene reinforced high strength concrete (PHC) specimens [Valle and Buyukozturk 1993]

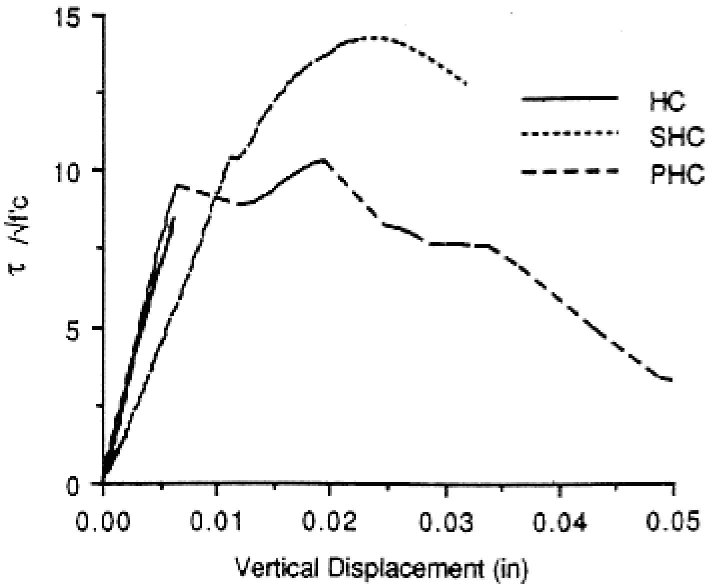


Figure 5.17b Normalized shear-stress versus vertical displacement for high strength concrete (HC), steel fiber reinforced high strength concrete (SHC) and polypropylene reinforced high strength concrete (PHC) specimens [Valle and Buyukozturk 1993]

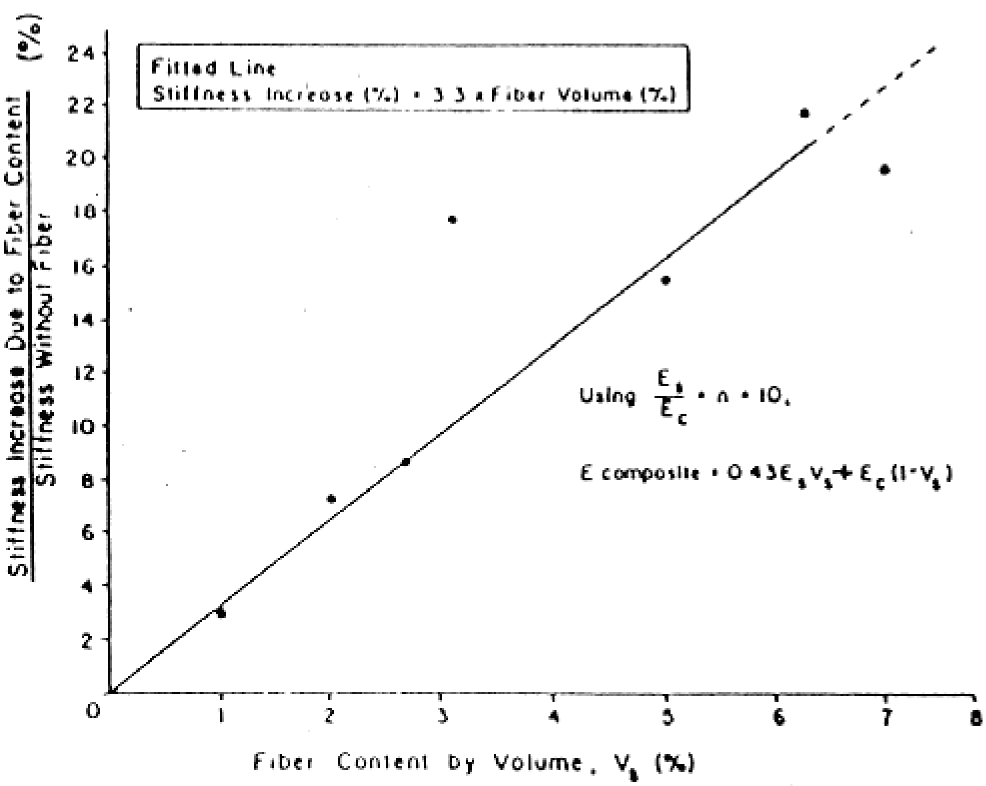


Figure 5.18 Increase of undamaged stiffness versus steel fiber content [Patton and Whittaker 1983]

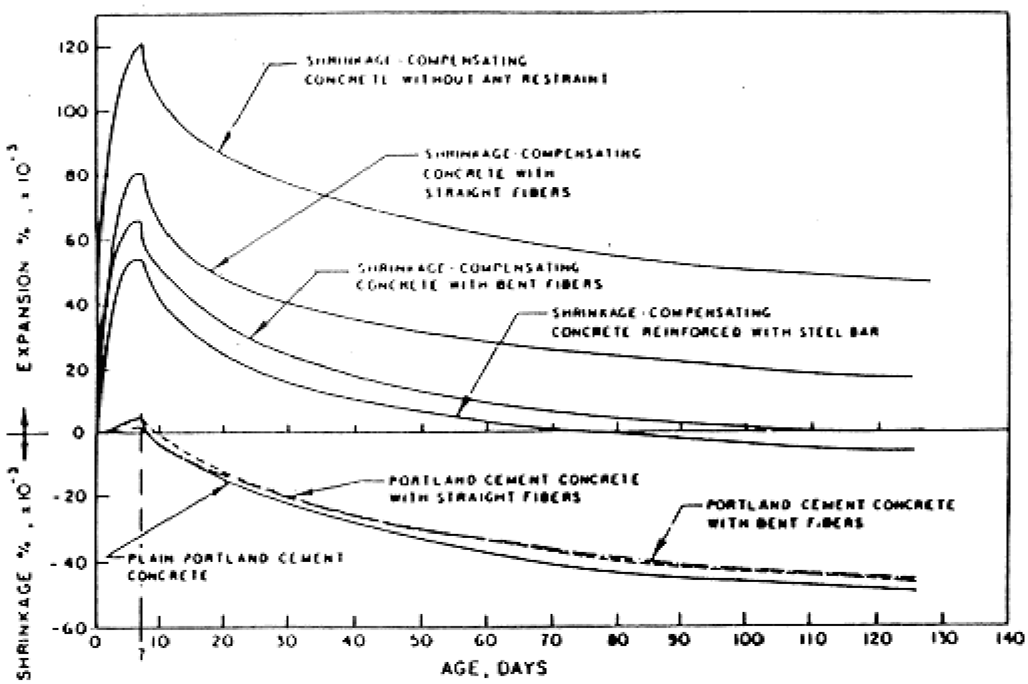


Figure 5.19 Expansion and shrinkage behavior of shrinkage-compensating and portland cement concrete [Paul and et al. 1981]

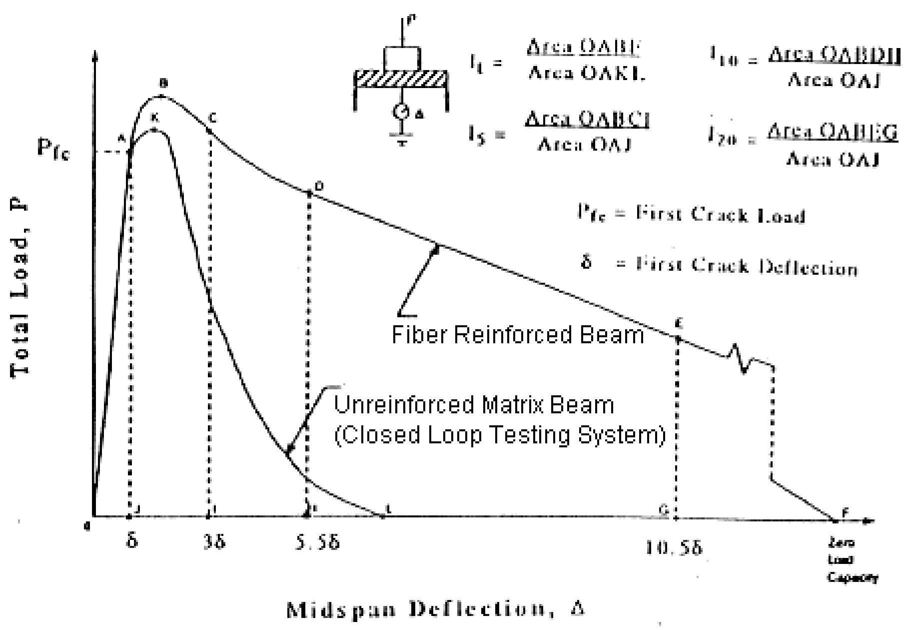


Figure 5.20 Toughness indexes from flexure load-deflection diagram

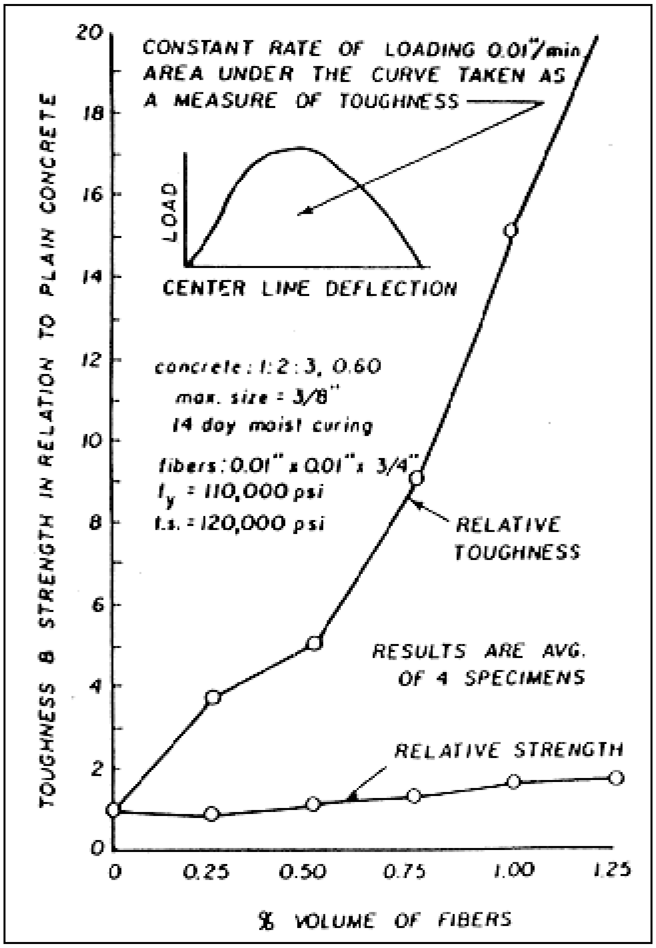


Figure 5.21 Effect of volume of fiber on toughness in flexure [Shah and Rangan 1970]

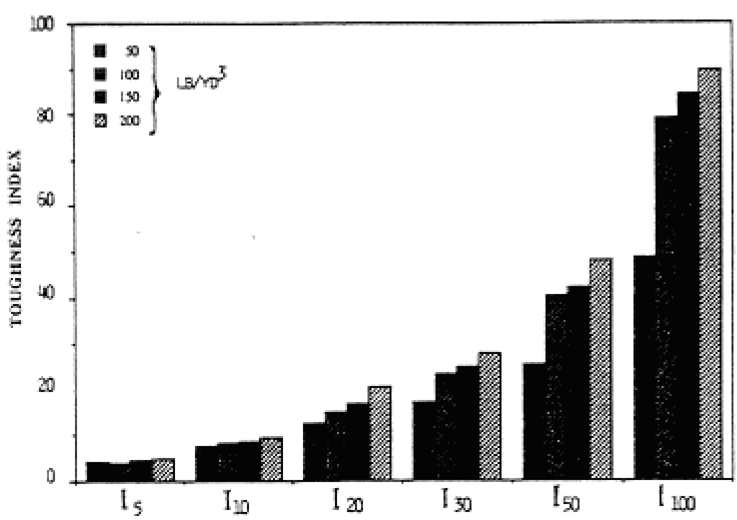


Figure 5.22a Toughness Indexes: influence if fiber volume fraction on 1.18 in (30 mm) long hooked-end fibers, normal strength concrete [Balaguru et al. 1992]

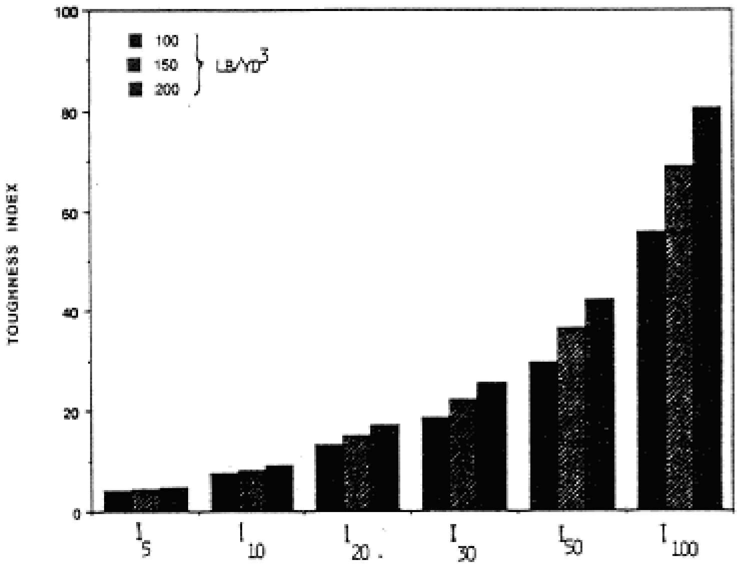


Figure 5.22b Toughness Indexes: influence if fiber volume fraction on 1.18 in (30 mm) long hooked-end fibers, high strength concrete [Balaguru et al. 1992]

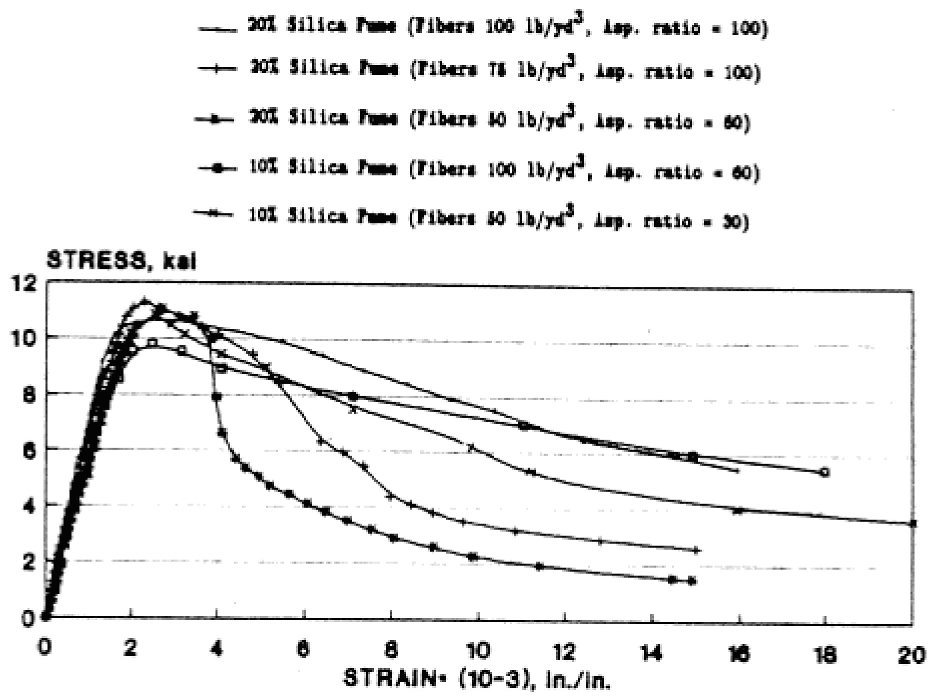


Figure 5.23a Stress-strain compression curves for silica fume fiber reinforced concrete [Ezeldin and Lowe 1991]

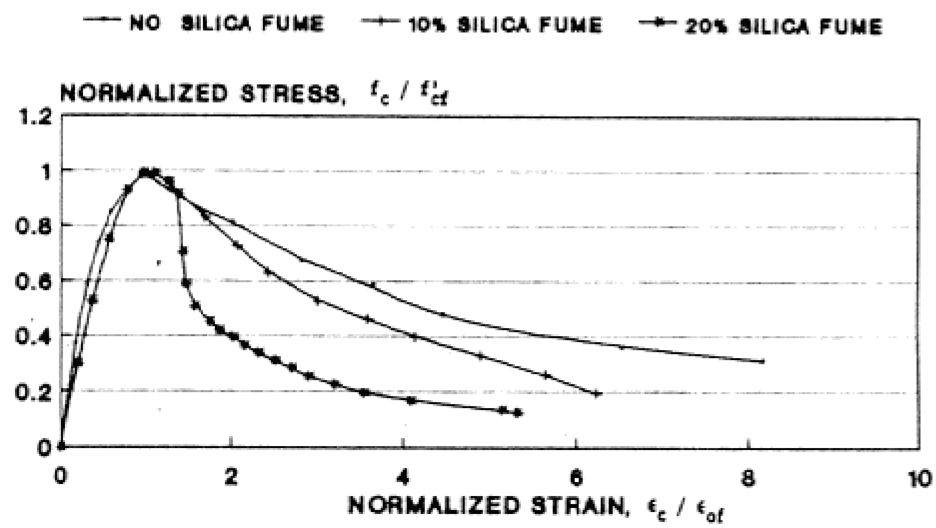


Figure 5.23b Normalized stress-strain compression curves for different silica fume fiber reinforced concrete, reinforcing index = 0.75 [Ezeldin and Lowe 1991]

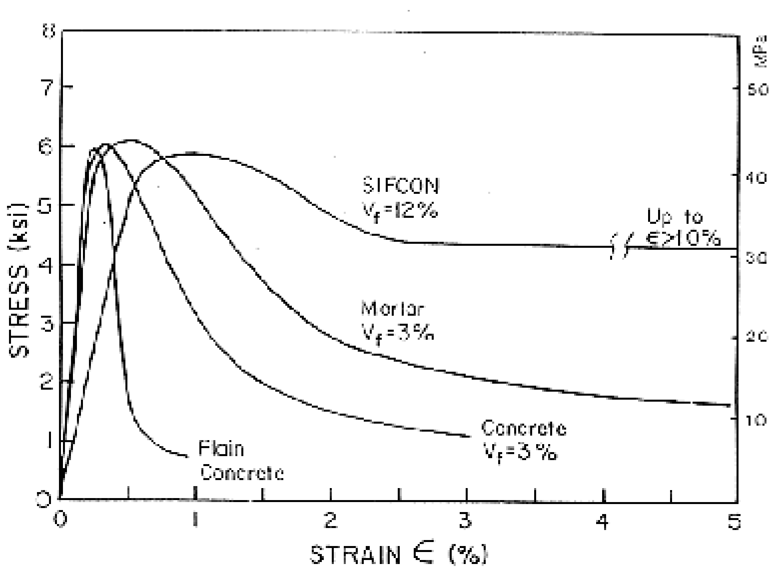


Figure 5.24 Comparison of experimentally observed stress-strain curves of plain concrete, fiber reinforced concrete and mortar, and SIFCON

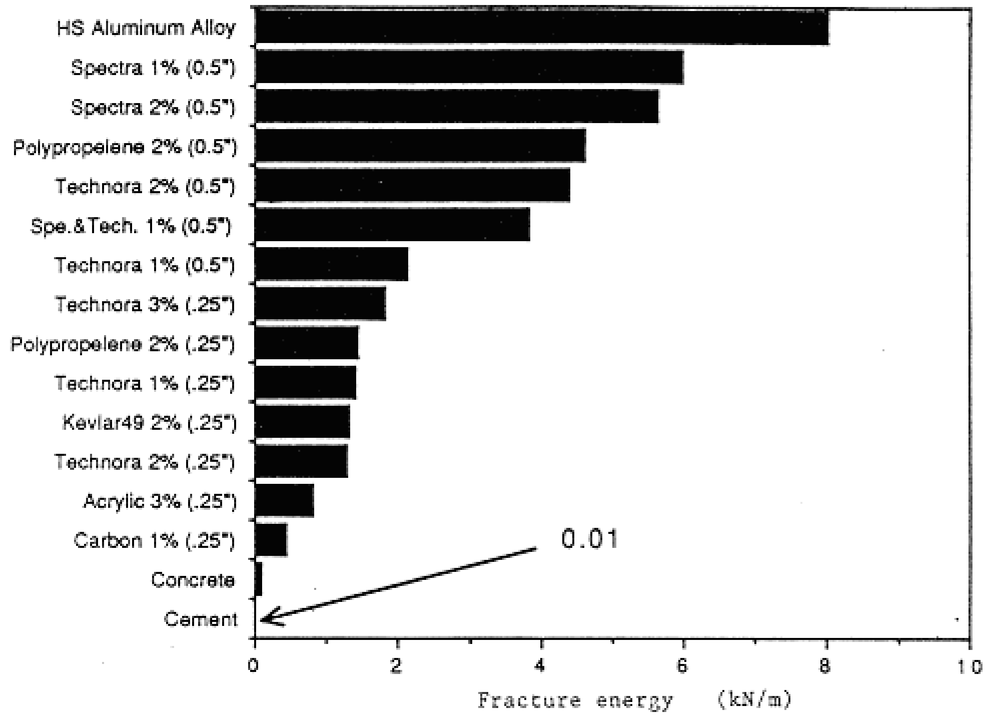


Figure 5.25 Fracture energy of synthetic FRC's [Li et al. 1989]

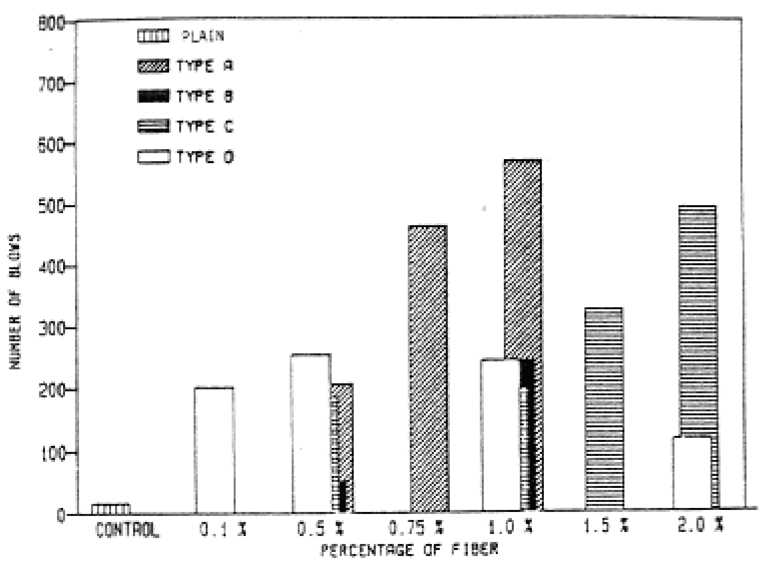


Figure 5.26 Impact results for FRC and control concrete [Ramakrisnan et al. 1989b]

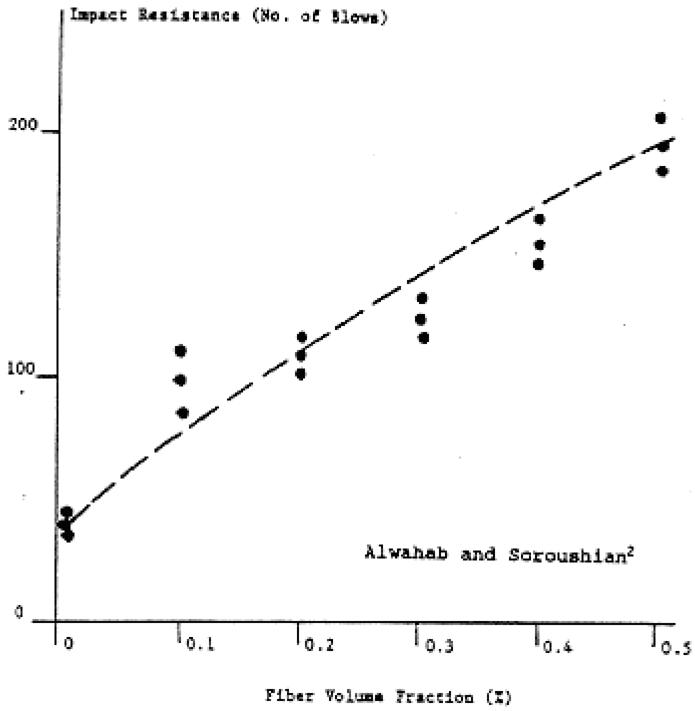


Figure 5.27 Effect of polypropylene fibers on impact resistance of concrete [Alwahab and Soroushian 1987]

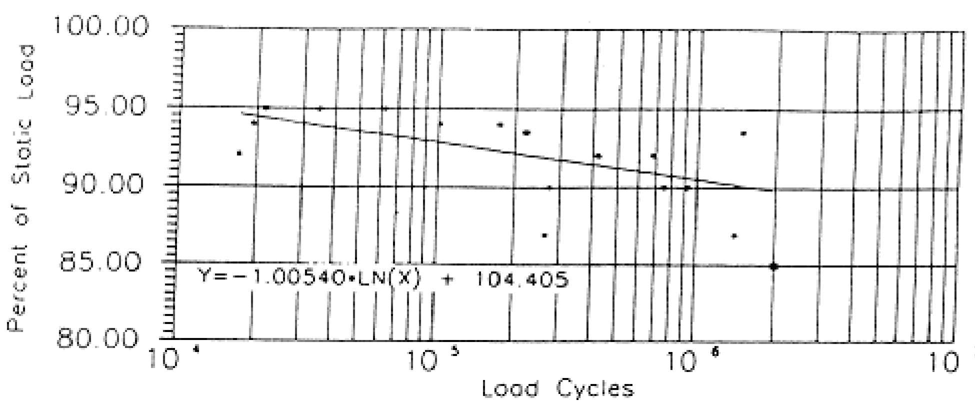


Figure 5.28a Flexure fatigue of 1.00% of type A fibers (steel with hooked end) [Batson 1991]

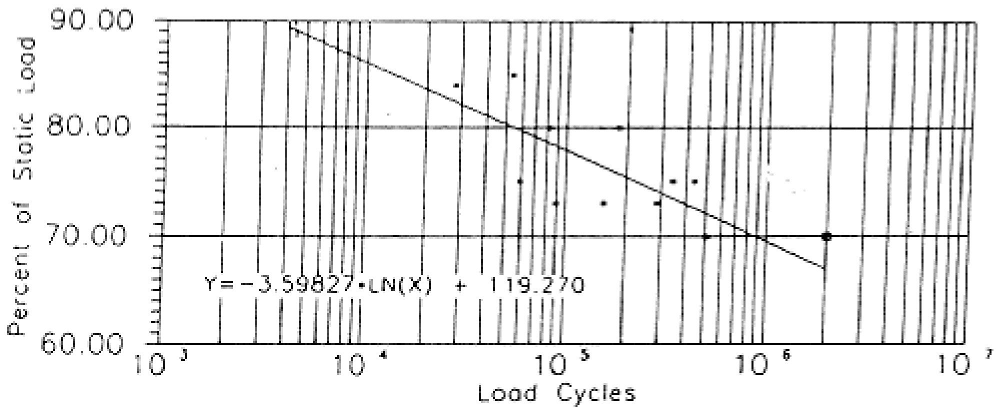


Figure 5.28b Flexure fatigue of 0.50% of type A fibers (steel with hooked end) [Batson 1991]

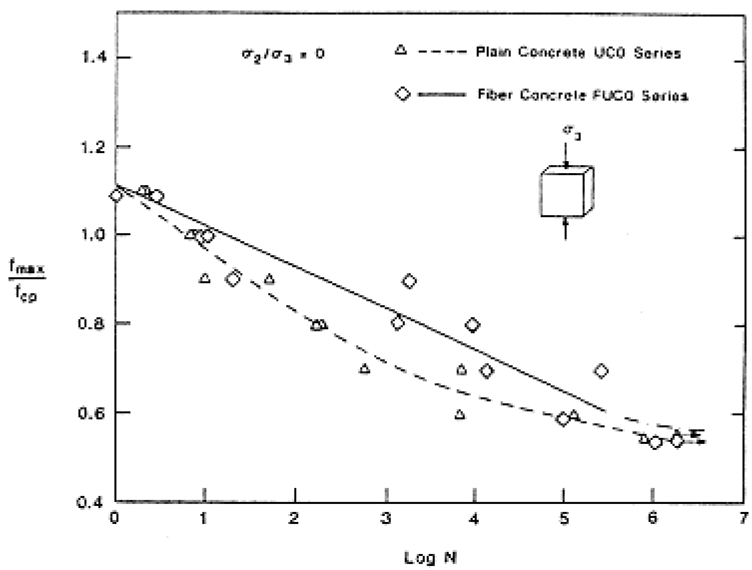


Figure 5.29a S-N curves for plain concrete and fiber concrete under uniaxial compression [Yin and Hsu 1990]

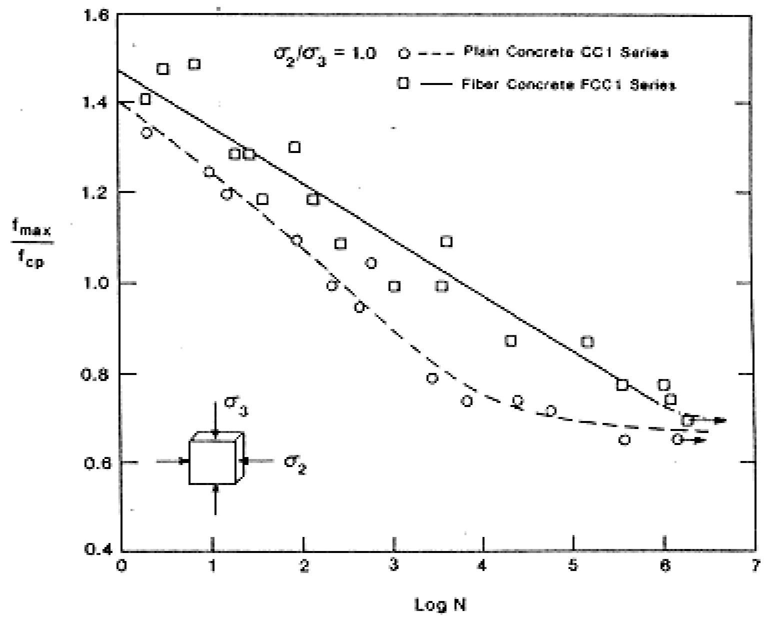


Figure 5.29b S-N curves for plain concrete and fiber concrete under biaxial compression [Yin and Hsu 1990]

APPLICATIONS OF HIGH PERFORMANCE CONCRETE

6.1 Pavements

During the past decade, there has been an increasing interest in using high performance concrete for highway pavements. The main reason for this heightened interest is the potential economic benefit that can be derived from the early strength gain of high performance concrete, its improved freeze-thaw durability, reduced permeability, and increased wear resistance. While the conventional normal strength concrete continue to be used in most cases of pavement construction, different types of high performance concrete are being considered for repair and rehabilitation projects on experimental basis. These projects can be classified into three categories: (1) pavement repairs for early opening to traffice, (2) bridge deck overlays, and (3) special applications and other developments.

6.1.1 Pavement Repairs for Early Opening to Traffic

6.1.1.1 "Fast Track" Concrete

"Fast track" concrete is designed to give high strength at a very early age without using special materials or techniques, and it is durable. The early strength is controlled by the water-cement ratio, cement content and characteristics. Typically, a rich, low-water-content mix containing 1 to 2 percent calcium chloride will produce adequate strength and abrasion resistance for opening to traffic in 4-5 hours at temperatures above 10 C (50 F). Fast track concrete paving (FTCP) was developed originally by the concrete paving industry in Iowa [Grove et al. 1990]. This technology is now well known and widely used in other parts of the country. According to Walker [1990], it has also been introduced to the UK in late July 1990 after a British study team visited Iowa in 1989, which identified five areas of application of the technology: (1) complete pavement reconstruction, (2) partial replacement by an inlay of at least one lane, (3) strengthening of existing bituminous or concrete pavements by a concrete overlay, (4) rapid maintenance and reconstruction processes, and (5) airfield pavements. It was pointed out that the benefits of applying FTCP technology in such applications are (1) a reduced contract period, thus reducing the contract overhead cost, (2) early opening of the pavement to traffic, (3) minimizing the use of expensive concrete paving plant and traffic management systems, and (4) reduced traffic delay costs.

6.1.1.2 SHRP C-205

In their study of high performance concrete under SHRP Contract C-205, the investigators, in cooperation of the various state transportation departments, constructed in June 1991 to July 1992 test pavements with very early strength and high early strength concretes in five states including Arkansas, Illinois, Nebraska, New York, and North Carolina. Except for the North Carolina installation, all sites involved the construction of full-depth and full-lane replacement patches. The patch layout, materials and construction techniques used, and tests performed were similar at four locations. In contrast to the other locations, the North Carolina site involved new construction, made use of a variety of paste and aggregate combinations, and employed both conventional concrete and standard construction techniques (as control) and high performance concrete with a rapid construction schedule.

The five installations provided an opportunity to study the effects of a fairly wide range of exposure conditions. The New York and Illinois installations are in regions where a hard winter freeze is likely. The North Carolina site is in a mild marine environment. The potential for freezing-thawing cycles is high in Arkansas and very high in Nebraska. Traffic varies from light volume with occasional heavy loads in Nebraska to heary volume with a high percentage of trucks in Arkansas. Details of the construction of the five test pavements and the early findings (as of spring 1993) have been described by Zia et al. [1993] and Schemmel and Leming [1993].

6.1.1.3 SHRP C-206

Following the SHRP C-205 project, additional tests of full-depth repair (slab replacement) of concrete pavements using different types of rapid strength cements and admixtures were conducted under the SHRP Contract C-206. Two sites were selected for field evaluations, one a test section of I-20 near Augusta, Georgia and the other a test section of State Route 2 near Vermilion, Ohio. Three different projected traffic opening times were selected: 2 to 4 hours, 4 to 6 hours, and 12 to 24 hours. Three different concrete mixes were used at the Georgia site and eight at the Ohio site, including the Very Early Strength (VES) and the High Early Strength (HES) mixes developed by the reseachers of SHRP C-205. The details of this investigation can be found in a series of publications by the investigators [Nagi et al. 1994; Nagi and Whiting 1994; Nagi and Whiting 1995; Whiting et al. 1994]. Based on the results of their evaluation, it was concluded that it is possible to perform full-depth pavement repairs using a variety of concrete mixtures. Opening of repaired areas to traffic is possible in as short a time as 2 to 4 hours after placement of the repair if special rapid-strength-gain cements are used. Opening can be accomplished in 4 to 6 hours using mixes containing large amounts of Type III cement plus various chemical accelerators and high-range water reducers (VES and HES mixes). More conventional mixes, such as 'Fast Track' designs, may also be used, however, these may take up to 12 hours to develop enough strengths for traffic opening. Regarding durability of the repair mixes, the following conclusions were reached:

1. At the I-20 site in Georgia, VES and Fast Track mixes showed very good freezing and thawing resistance, in comparison with Georgia DOT mix which uses calcium chloride as an accelerating admixture and has relatively low air content.
2. Freezing and thawing resistance of cores taken from the I-20 site, when measured using ASTM C 666, Procedure A, was comparable to that conducted on prisms using the modified ASTM C 666, Procedure B.
3. At the SR2 site in Ohio, durability factor of all mixes except HES and RSC 2 were very low. Expansion exceeded 0.1% at 300 cycles of freezing and thawing.
4. Linear traverse analyses showed that HES and RSC 2 mixes have more than adequate air content and low spacing factor.
5. Performance of core specimens for all mixes, when tested using ASTM C 666, Procedure A, was better than corresponding beams tested using the modified ASTM C 666, Procedure B.

Currently the Federal Highway Administration (FHWA) has an on-going project to monitor the long-term performance of the experimental pavements constructed under SHRP C-205 and SHRP C-206.

6.1.1.4 Other Studies

Field studies of special rapid-strength-gain cements such as MPC (Magnesium Phosphate Cement) used for patching [Seehra et al. 1993], and PBC (Pyrament blended cement) used for full-depth pavement replacement [Ozyildirim 1994] have also been carried out and very satisfactory results have been obtained.

6.1.2 Bridge Deck Overlays

6.1.2.1 Washington Overlays

Twelve concrete bridge decks were rehabilitated and/or protected with latex-modified concrete (LMC) and low-slump dense concrete overlays in the State of Washington. These decks were evaluated by Babaei and Hawkins [1990] to identify the factors that have affected the serviceability of the overlaid bridge decks. The evaluation included overlay freeze-thaw scaling, surface wear and skid resistance, surface cracking, bond with the underlying deck, chloride and water intrusion, and the overlay's ability to retard continued reinforcing steel corrosion. The results of the evaluation indicate that, regardless of concrete deterioration caused by reinforcing steel corrosion, concrete overlaid bridge decks will require resurfacing after about 25 years of service, as a result of traffic action and weathering. Typical forms of distress are freeze-thaw scaling, extensive wear in wheel lines, lack of skid resistance, and the loss of overlay bond. Concrete overlays are resistant but not impermeable to chloride infiltration. If the overlay surface is without cracking, there is indication that corrosion of steel reinforcement in the salt-contaminated underlying deck is less extensive.

6.1.2.2 Virginia Overlays

An alternate to LMC often used by the departments of transportation is dense concrete containing silica fume. In Virginia, a two-lane, four-span bridge deck was overlaid with such concrete with addition of silica fume at 7% or 10% by weight of cement. Test results [Ozyildirim 1993] indicated that the concrete bonds well with the base concrete and has very low permeability, high strength, and satisfactory freeze-thaw resistance. Over a 5-year evaluation period in the field, there was evidence of cracking and increase in half-cell potentials and chloride content, indicating a tendency to corrosion. However, the same evidence was observed with LMC ovelays. Thus silica fume concrete can be used effectively as an alternate to LMC. Just like LMC, plastic shrinkage is recognized as a potential problem with silica fume concrete. Therefore, immediate and proper curing after placement is an essential step to take.

6.1.2.3 Oregon Overlays

Similar study was conducted in Oregon using microsilica modified concrete [Miller 1991]. Seven concrete bridge decks were covered with microsilica concrete in 1989. After one year in service, cracking and delamination were observed in the overlays. However, the cracks and delaminations were not extensive (the worst deck had only 2.5% of its surface delaminated) and comparable to what had also been observed in LMC overlays. More serious crackings and delaminations were observed near construction and expansion joints. The only maintenance performed was the sealing of cracks on one deck with methacrylate and sand at a cost of $4,000. The sealant was effective. The overlay met two of their three design objectives after one year in service. They were adding strength to the deck and providing a smooth and durable wearing surface. However, because of crackings, they could no longer seal the underlying deck from the intrusion of chlorides.

6.1.2.4 Polymer Concrete Overlays

Sprinkel [1993] reviewed the status of polymer concrete overlays for concrete bridge decks, and provided information on the properties of the concretes used, the proper application methods, and the performance record of the overlays. He pointed out that polymer overlays constructed with epoxy, methacrylate, and polyester styrene binders and graded silica and basalt aggregates can provide skid resistance and protection against chloride intrusion for 1 to 20 years. They are an economical technique for extending service life of reinforced concrete decks, especially when the overlays must be constructed during off-peak traffic periods to minimize inconvenience to the travelling public.

6.1.3 Special Applications and Other Developments

6.1.3.1 High Strength Concrete Pavement

High strength concrete has been used for highway pavements in Norway because of the need to provide increased wear resistance to steel studed tires [CEB-FIP 1994; Gjorv et al. 1990]. During the summer of 1989, 110,000 m2 (131,560 yd2) of Highways E-6 and E-18 were paved with high strength concrete. The pavement thickness was 18 cm (7.1 in.) for E-6 and 22 cm (8.7 in.) for E-18. The total volume of concrete used was 22,000 m3 (28,770 yd3). The 28-day cube strength was 90 MPa (13,050 psi) for E-6 and 85 MPa (12,330 psi) for E-18. The slump of the concrete was in the order of 2 to 6 cm (0.8 to 2.4 in.). After four years of service, the wearing resistance of the HSC pavements seems to meet the original expectation. However, at E-6, some longitudinal cracks have appeared close to some of the joints. The problem is now under investigation, but one hypothesis is that the pavement thickness is insufficient such that the damage has been caused by fatigue.

There are many other cases of using high strength concrete for highway pavement in Norway and Sweden to improve the abrasion resistance. The pavements were constructed within the past five year. A fairly extensive listing of these projects can be found in the CEB-FIP report [1994].

6.1.3.2 Long-Term Performance of Fly Ash Concrete

Fly ash has had a long history in its use as a partial replacement for cement in concrete. A study [Vruno et al. 1991] was conducted in North Dakota to determine the effect on properties and performance of paving concrete containing lignite fly ash as replacement for various percentages of portland cement. The fly ashes used in the study do not conform fully to the chemical and physical requirements of the current version of ASTM C 618. However, the fly ashes were used in the pavement construction only after laboratory testing indicated their potentials for providing satisfactory performance. The study included both laboratory and field evaluations of compressive and flexural strengths and freeze-thaw durability. After 15 years of field exposure, the concrete provided very good performance which supports the original laboratory findings.

A similar study has been reported in United Kingdom [Dunstan et al. 1993]. Concrete cores containing high fly ash content were extracted from a number of structures constructed since 1979. The structures investigated were a road pavement, a major road viaduct, water-retaining and industrial structures, and a spillway subjected to marine exposure. After 10 years of service, the concrete properties measured included compressive strength, depth of carbonation, permeability, and chloride and sulphate penetration profiles. Petrographic analysis of thin sections was also made. The test results indicate that the concrete is durable with continued increases in compressive strength beyond 28 days. There is little evidence of cabonation, low to average permeability, and good resistance to chloride penetration.

6.1.3.3 High Performance Base Concrete

A continuously reinforced concrete pavement was constructed in Sydney, Australia [Leshchinsky and Pattison 1994]. The pavement consisted of a 150 mm (6 in.) thick, 5 MPa (725 psi) lean-mix sub-base and a 200 mm (8 in.) thick , 32 MPa (4,600 psi) reinforced concrete pavement. In November and December of 1992, 10,000 m3 (13,000 yd3) of the pavement concrete were placed. A quality assurance program was set up by the concrete supplier to ensure that the concrete strength and its uniformity were achieved. No information on the reinforcement detail is available.

6.1.3.4 Robot Revibrator

A robot revibrator called Rollit Robot Method (RRM) has been developed recently in Sweden for the surface treatments of high strength concrete pavements [Molina and Alvarsson 1994]. The robot travels on the fresh surface layer achieving a higher flatness, and securing a high density of the outer concrete layer. Densification of concrete results from mechanical consolidation and increased hydration. The size of the robot can be produced as a small remotely controlled machine or as a large drivable vehicle that covers the full width of a road or bridge deck. A prototype of the motor vibrator mounted on the robot has been used successfully in Sweden and actual trials at construction sites are planned in Scandinavia.

6.2 Bridges

The benefits of using high strength concrete for bridges are well known to bridge engineers. Over the past several years, there have been a series of design studies published in the literature, all leading to the same conclusion that the use of high strength concrete would enable the standard prestressed concrete girders to span longer distances or to carry heavier loads [Zia et al. 1989; Adelman and Cousins 1990; Schemmel and Zia 1990; Taerwe 1991; Russell 1994].

As discussed in the previous state-of-the-art report [Zia et al. 1991], the use of high strength concrete for bridges has received much wider and earlier acceptance in Europe and Japan even though the concrete strength levels were somewhat lower than what is being used in the U. S. today [FIP-CEB 1990]. However, the trend is towards higher concrete strength levels [CEB-FIP 1994]. Table 6.1 shows a listing of some of the earlier and recent bridges built with high strength concrete. Another fairly extensive listing of high strength concrete bridges, mostly in Japan, Canada, France, and Norway can be found in the report of CEB-FIP [1994]. In the following, brief summaries of selected bridges using high strength concrete, both here and abroad, will be given.

6.2.1 Japan

Three high strength concrete (HSC) bridges built for Japan National Railway in 1973 are of historical importance. The reasons for utilizing HSC were to lower the deadload, to reduce deflection as well as to reduce the vibration and the noise. An additional reason was to reduce the maintenance cost. After over 20 years of service, the bridges representing the first generation of HSC bridges worldwide have performed according to all the expectations [CEB-FIP 1994].

The 2nd Ayaragigawa bridge was the first HSC bridge built and consisted of post-tensioned bulb T-beams with 60 degrees skew. Concrete design strength of 60 MPa (8,600 psi) was chosen to reduce the weight of individual beams to less than 150 tons for lifting. If ordinary (lower) strength concrete was used, the weight would be 170 tons.

Iwahana bridge was the first medium span prestressed concrete truss bridge in Japan made with HSC of over 80 MPa (11,500 psi). The bridge is a 45 m (148 ft) single span Warren truss which was selected to satisfy the clearance under the bridge and to reduce deflection. The members of the truss including the jointing parts were prefabricated in the factory and were transported to the site. The prefabricated members were joined by using concrete and/ or polymer adhesive. The concrete design strength was 89 MPa (12,750 psi) and the average strength obtained by the standard specimens was 84 MPa (12,000 psi) with a coefficient of variation of less than 4%. For the particular project, a steel truss bridge might be more economical but could not be used because of the problem of noise with the trains running on the bridge. Concrete structures are preferable for railway bridges to eliminate noise and vibration problems.

Otanabe railway bridge is a 24 m (79 ft) single span Howe truss built with HSC of 80 MPa (11,5000 psi). HSC was used again to reduce noise and minimize the maintenance cost.

The Akkagawa railway bridge is another 305 m (1,000 ft) truss bridge built in 1975 with main spans of 45 m (148 ft). The required concrete strength for the prefabricated members was 80 MPa (11,500 psi), and the average strength obtained was 96 MPa (13,750 psi) with a standard deviation of 4.4 MPa (630 psi). After cast, the members were steam cured at 65oC (149oF)for 12 hours, then the concrete was autoclave cured at 180oC (356oF) and 10 atmospheres for additional 20 hours. The different parts were finally assembled into 45 m (148 ft) sections and lifted into position. The joints were cast in-situ with 60 MPa (8,600 psi) concrete [FIP-CEB 1990].

CNT Super bridge was built in 1993 as a pedestrian bridge between two laboratories. It is a 40 m (132 ft) single span post-tensioned box beam with outstanding flanges. For aesthetic reason, the beam span/depth ratio was kept at 40. To reduce the beam depth, a very flowable HSC of 102 MPa (14,600 psi) was chosen. The mix proportion of the concrete had water/binder ratio of 0.20 with a 25 ± 2 cm (10 ± 0.8 in.) slump and a slump flow of 60 ± 5 cm (24 ± 2 in.). The shallowness of the beam created a vibration problem which was overcome by the use of a vibration controller attached under the beam deck [CEB-FIP 1994].

6.2.2 France

The Pertuiset bridge is a cable-stayed bridge built over the Loire river in 1987-1988. To make the construction cost effective, a flowable HSC was chosen for the towers and the 18 cm (7 in.) thick deck. Design concrete strength was specified as 60 MPa (8,600 psi), while the maximum stress under sustained load is 23 MPa (3,300 psi) and under extreme overload 38 MPa (5,400 psi). The mean strength of concrete obtained at 16 hours was 33 MPa (4,700 psi) and at 28 days was 80 MPa (11,500 psi). The W/CM was 0.33 and the slump was more than 200 mm (8 in.) [FIP-CEB 1990].

Joigny bridge is an experimental structure built in 1988-1989 to demonstrate the production of HSC in commercial ready-mixed concrete plant without using silica fume. It is a three-span bridge with the center span of 46 m (152 ft). Its double I-sections were prestressed externally. The average concrete strength achieved was 78 MPa (11,200 psi). Design studies indicated that by using a specified strength of 60 MPa (8,600 psi) rather than an ordinary 35 MPa (5,000 psi) concrete, there was a 30% savings of concrete and a 24% load reduction on the pier, abutments and foundations. The reduction in weight also resulted in some savings by reducing the number of prestressing strands. The bridge has been instrumented to obtain its long-term performance record. Temperature and deformation have been monitored since construction [Malier et al. 1989; Malier and Pliskin 1990; Malier et al. 1991; Malier 1992]

Elorn bridge is a 400 m (1,320 ft) span cable-stayed bridge built in 1991-1994. High strength concrete of 97 MPa (13,900 psi) with silica fume was used for structural efficient and durability. For the same reason, high strength concrete of 60 MPa (8,600 psi) with silica fume was chosen for Nomandie bridge, a 850 m (2,800 ft) cable-stayed bridge constructed in 1990-1995 [CEP-FIP 1994]

6.2.3 Norway

Since 1989, the majority of all concrete bridges and highway structures built in Norway have followed a general requirement of using a water-binder ratio of less than 0.40 combined with the use of silica fume so as to improve the chloride resistance due to deicing agents and marine environment. Annual consumption of such concrete ranged from 150,000 to 200,000 m3 (196,000 to 262,000 yd3) [CEB-FIP 1994].

Sandhornoya bridge was built in 1989 with lightweight high strength concrete (LWHSC) of 56 MPa (8,000 psi). It is a 3-span cantilever bridge with a center span of 154 m (500 ft). The use of LWHSC provided the advantages of reduced weight and increased strength.

Stongsundet bridge involving four precast girders was built in 1990. The 65 m (215 ft) long girder was post-tensioned. By using HSC of 75 MPa (10,700 psi) with W/CM of 0.35, the weight of the girder was reduced during transportation and handling.

Stovset bridge built in 1992-1993 is a prestressed cantilever bridge. LWHSC of 74 MPa (10,600 psi) was used for its 220 m (725 ft) center span to obtain the advantages of reduced weight and increased strength.

6.2.4 Denmark

To cross the Great Belt, a major tunnel and bridge connection is now under construction. This US$7 billion project (including financial cost) consists of two single track railway tunnels, each of 8,000 m in length between the islands of Sprogoe and Zealand, and a parallet road bridge (East Bridge) of 6,800 m (22,440 ft). The central part of this bridge will be a suspension bridge with a main span of 1,624 m (5,360 ft) and pylons of 254 m (835 ft) in height. The islands of Sporogoe and Funen will be connected by the West Bridge, a combined road and railway bridge with a total length of 6,600 m (21,780 ft). This structure consists mainly of 110 m (363 ft) long precast concrete girders. The construction work started in 1988 and the connection is expected to be opened in 1998 for the road and in 1996 for the railway. The service life requirement for the project was established as 100 years. Therefore durability is a major consideration. The limits established for the concrete proportions selected for the project are given in Table 6.2 [CEB-FIP 1994].

6.2.5 Germany

Deutzer bridge crossing the Rhine river close to Cologne was built in 1978. The bridge is a free cantilever construction with three spans of 132 m, 185 m, and 121 m (435 ft, 610 ft, and 399 ft). Sixty-one meters (200 ft) of the middle span was cast with a lightweight concrete and the rest of the bridge with a normal weight concrete. The specified strength for both concretes was 55 MPa (7,860 psi). However, the mean strength obtained in the field was 69 MPa (9,890 psi) for the normal weight concrete and 73 MPa (10,500 psi) for the lightweight concrete [FIP-CEB 1990].

6.2.6 Canada

Portneuf bridge in Quebec was constructed in 1992. It uses precast post-tensioned beams of 24.8 m (81.5 ft) span. The average strength of concrete was 75 MPa (10,750 psi) with a W/C of 0.29 and 5 to 7.5% air content. By using HSC, smaller loss of prestress and consequently larger permissible stress and smaller cross-section were achieved. In addition, enhanced durability allowed extended service life of the structure.

A pedestrian bridge of 35 m (115 ft) span with Z-shaped girders was built in Laval, Quebec, in 1992. For the same reasons as the Portneuf bridge, HSC of 70 MPa (10,000 psi) was used. The concrete had a W/C of 0.30 and 5% air content.

St. Eustache bridge in Quebec is a replacement for a 17 m (56 ft) short span bridge superstructure. It uses precast pretensioned channel-shaped girders made with 60 MPa (8,600 psi) concrete. W/C was 0.26 and the air content was 4.5%. HSC was chosen not for strength but for durability. The initial cost of the design proved to be more economical than steel-concrete composite girder.

HSC was also used for a Highway 50 overpass in Mirabel, Canada. It was selected for economy and prolonged service life. The HSC design resulted in a 5% saving. The specified concrete strength was 60 MPa (8,600 psi), but the actual average strength of cylinders was 80.7 MPa (11,560 psi) with 6.2% air content [CEB-FIP 1994].

6.2.7 United States

While the published record indicates that the use of high strength concrete in the United States has had a history of more than 20 years and much of that activity has occurred in the area of high-rise buildings, it is well known that prestressed concrete girder bridges with a design compressive strength of 62 MPa (9,000 psi) have been used in the state of Washington for some time; and in recent years there has been an ever increasing interest in the use of high strength concrete for bridge applications. The trend is clear that more bridges will be built with higher and higher concrete strength in the foreseeable future as the industry becomes more familiar with the technology. However, published record documenting the accomplishments achieved so far is still quite sparse.

An overview of the Federal outlook for high strength concrete bridges has been presented by Lane and Podolny [1993]. Several examples of recent high strength concrete bridges were described and the results of a number of research and design studies were discussed. The issues of codes and specifications and future research needs were addressed.

Brake Lane bridge over I-35 in Austin, Texas is perhaps one of the first bridges constructed in the U. S. utilizing standard pretensioned girders. The bridge is a straight, simply supported, two-span structure with 22 Texas Type C girders, each 26 m (85 ft) long. The girders are spaced at 2.6 m (8.4 ft) apart, and they were designed with a specified concrete strength of 66 MPa (9,600 psi). If the normal concrete strength of 41 MPa (6,000 psi) were used, the maximum span for Type C girder would be only 22 m (71 ft) at 2.6 m (8.4 ft) spacing. Thus the use of the higher strength concrete resulted in a 20% increase in the maximum span length. An extensive materials devlopment program to produce the high strength concrete for the bridge, utilizing the locally available materials, was described by Durning and Rear [1993]. The concrete actually produced for the bridge achieved an average 28-day strength of 92 MPa (13,400 psi) and it also developed a compressive strength of 51 MPa (7,400 psi) in 17 hours, necessary for release of prestressing force.

Three full-sized Texas Type C pretensioned girders of 14.6 m (48 ft) span, made with 69 MPa (10,000 psi) concrete, were tested to evaluate their static and fatigue behavior. Each girder was tested in flexure by a combination of static overloads and repeated service loads.. The number of repeated loads varied from a minimum of 225,000 cycles to a maximum of 700,000 cycles. Two of the girders contained debonded strands and the third girder contained draped strands. The test results confirmed that the behavior of pretensioned girders made with HSC can be adequately predicted by the current design procedures, and that debonded strands is a viable alternative to fully bonded draped strands [Russell and Burns 1993].

An extensive experimental program has been conducted to evaluate the feasiblity of using high-strength concrete in the design and construction of highway bridge structures. Four full-size 137 cm (54 in.) deep pretensioned bulb-tee girders were cast with 68 MPa (9,800 psi) concrete. Each girder is 21.3 m (70 ft) long and contained the same number and configuration of longitudinal prestressing strands and same amount of web reinforcement. A deck slab was cast on two of the four girders. One girder with a deck slab and one girder without a deck slab were tested for flexural and shear strength. The second girder with a deck slab was used to determine long-term behavior under full design dead load over an 18-month period, and the remaining girder without a deck slab was subjected to fatigue test under 5 million cycles of loading. The two girders tested in flexure and shear performed well with respect to both design and specification requirements. The long-term sustained load test indicated that prestess losses were significantly less than expected. Using the measured prestress loss value, measured camber and deflection correlated well with predicted values by conventional analysis. The girder under a fatigue loading of 5 million cycles satisfied all strength and serviceability requirements. Thus this investigation confirmed that high strength concrete girders can be expected to perform satisfactorily over the long term when designed and fabricated in accordance with the current AASHTO provisions [Bruce et al. 1992; Roller et al. 1993; Roller et al. 1995]

Louetta Road Overpass including two adjacent bridges on State Highway 249 in Houston, Texas, currently under construction, is a showcase project to demonstration the use of HSC. The structures are the first bridges in the United States to fully use HSC in all aspects of design and construction. They are also the first on the U. S. to use 15.24 mm (0.6 in.) diameter strands in pretensioned bridge girders and on a 50 mm (1.97 in.) grid spacing. The structures used pretensioned concrete U-beams (maximum span of 41.5 m or 136 ft) with concrete strength in the range of 69 to 89.6 MPa (10,000 to 13,000 psi) at 56 days. The U-beams were specially designed as an economical and aesthetic alternative to the standard I-beams. The U-beams supporting pretensioned concrete panels as stay-in-place forms were made composite with cast-in-situ reinforced concrete deck to provide finished roadways. For purposes of comparison, the southbound mainlanes used 55 MPa (8,000 psi) concrete and the northbound mainlanes used Texas standard 28 MPa (4,000 psi) concrete [Ralls et al. 1993; Ralls and Carrasquillo 1994]

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Table 6.1

Bridge	Location	Year	Max. Span (m)	Max. Design Conc. Strength, (MPa)
Nitta Highway Bridge	Japan	1968	30	59
Kaminoshima Highway Bridge	Japan	1970	86	59
2nd Ayaragigawa Bridge	Japan	1973	50	60
Iwahana Bridge	Japan	1973	45	89
Ootanable Railway Bridge	Japan	1973	24	79
Fukamitsu Highway Bridge	Japan	1974	26	69
Akkagawa Railway Bridge	Japan	1976	46	79
Kylesku Bridge	Scotland		79	53
Deutzer Bridge*	Germany	1978	185	69
Tower Road Bridge	Washington	1981	49	62
East Huntington Bridge	W. Virginia	1984	274	55
Annacis Bridge	Vancouver			55
Sylans Viaduct	France	1986		60
Re Island Bridge	France	1987		60
Braker Lane Bridge	Texas	1987	26	66
Pont du Joigny	France	1988	46	60
Pont du Pertuiset	France	1988	110	65
Arc sur la Rance	France	1989		60
Giske	Norway	1989	52	55
Sandhornoya*	Norway	1989	154	55
Boknasundet*	Norway	1990	190	60
Helgelandsbrua	Norway	1990	425	65
Kwung Tong By Pass	Hong Kong	1990		65

*Lightweight concrete

Table 6.2

	Type A	Type B
Water/Binder ratio*	<0.35	<0.40
Fly ash content of binder	>10%	>10%
Silica fume content of binder	5% ~ 8%	5% ~ 8%
Fly ash plus silica fume	<25%	<25%
Total water content	<135 l/m^3	<140 l/m^3
Entrained air required where exposed to freeze-thaw		

* based on an efficiency factor of 0.5 for fly ash and 2.0 for silica fume. Type A is for precast tunnel segments and in part of the two bridges exposed to sea water, while Type B will be used for the pylons on the East Bridge and the precase girders on the West Bridge.

RECENT ACTIVITIES OF ORGANIZED PROGRAMS ON HPC

7.1 Introduction

Although the concept of high performance concrete as a technology emerged out of laboratory no more than 15 years ago, it has since experienced a phenomenal growth in research and application worldwide during the past decade. While published reports on the subject were rare in the beginning of the 1980's, no one is able to cope with the massive stream of information today. This extraordinary expansion of activities is due in large measure to the stimulus provided by a number of national organized programs to advance the technology in response to need for the massive civil infrastructure renewal and construction around the world. Notable among these national efforts are those of Canada, France, Japan, Norway, and the United States. This chapter will provide an overview of the recent activities of a number of nationally organized programs on high performance concrete.

7.2 Canada

In 1990, the Canadian government created a unique Network of Centers of Excellence on High-Performance Concrete with a 4-year funding of $6.4 million. The Network included seven universities and two industrial partners with its administrative headquarters being located at the University of Sherbrooke. Roughly two-thirds of this organized research effort were focused on materials studies and one-third on structural investigations. Research topics included selection of constituent materials for HPC, durability, evaluation of standard test procedures, nondestructive tests, fiber reinforced shotcrete, bond and anchorage, shear strength, and the use of high strength reinforcement [MacGregor 1993]. The Network made significant advances in the areas of materials improvement, structural design, placing and construction techniques. Methods for producing ready-mixed HPC with compressive strength up to 120 MPa (17,400 psi) were developed in several regions of Canada.

Based on the success of the first 4-year effort, the Network – now named Concrete Canada – received from the Canadian government a renewal grant of $5.5 million in 1994. The second 4-year research program of Concrete Canada focuses on four general areas: infrastructure rehabilitation, durability, cost effectiveness of HPC structures, and the development of new materials and products. In the area of new materials, Concrete Canada participated in the development of what is known as Reactive Powder Concrete (RPC), in which coarse aggregates and sand are replaced by powders with carefully selected grain-size distribution. Using this technology, concretes with more ductility can be produced for compressive strengths ranging from 200 to 800 MPa (29,000 to 116,000 psi), flexural strengths from 30 to 65 MPa (4,350 to 9,400 psi), and toughness varying from 20,000 to 60,000 J/m2 (0.44 to 1.33 kCal/ft^2). These concretes are virtually impermeable and show excellent freez-thaw as well as abrasion resistance.

A vital component of Concrete Canada is its primary commitment to transfer the new technology to the construction industry to enhance its competitiveness. In this effort, Concrete Canada engages 17 Principal Researchers and 29 Collaborating Investigators, working in conjunction with over 60 partner organizations and affiliate member. The following are some examples of technology transfer for HPC in Canada [Concrete Canada 1995]:

- Hibernia offshore oil platform – HPC mix design and structural design.
- Prince Edward Island Bridge – Stabilization of air voids and thermal monitoring.
- Five HPC bridges in Quebec – Development of air-entrained HPC.
- Jacques Cartier and Yamaska Bridges – Mix design, field supervision and testing
- HPC deck replacement.
- Highway 407 Bridges in Toronto – Integrating HPC into this major project.
- Ontario Highway 20 Bridge Replacement – Implementation of HPC.
- Portneuf and Mirabel HPC Bridges – Long-term monitoring for cracks and corrosion.
- Tsable River Bridge in B.C. – HPC mix design and creep and shrinkage studies.

7.3 China

The first application of high-strength concrete in China occurred in the late 1970's when a large arch-type blast door for a naval vessel storage was constructed using concrete with compressive strength of 70 to 75 MPa (10,000 to 10,700 psi). At the same time, a cable-stayed railway bridge across the Red Water River in Guangxi Province was constructed with 60 MPa (8,700 psi) pumped concrete for its three-span (48 + 96 + 48m or 157.5 + 315 + 157.5 ft) prestressed box-section girders. Since then, the use of HSC has expanded rapidly. No less than 20 high-rise buildings in major cities and 13 major highway or railway bridges at strategic locations across the country have been constructed using concrete with compressive strength varying between 50 MPa (7,250 psi) and 80 MPa (11,600 psi), mostly at 60 MPa (8,700 psi). A major 5-year (1987-1991) research program on HSC was supported by the National Natural Science Foundation of China (NNSFC) and the State Ministry of Construction, and was completed by close collaboration among the universities, research institutes, and construction corporations. Based on the research, a "Design Guide for HSC Structures" was compiled in 1993.

Beginning in 1994, a new organized research program on C50 – C100 (7,250 – 14,500 psi) HPC is in progress with joint support of the NNSFC, the State Ministry of Railway, the State Ministry of Construction, and the State Bureau of Building Materials. The emphasis of the program is on long-term behavior of HSC materials and members [Chen and Wang 1996]

7.4 Denmark

The Danish research program "High Performance Concretes in the 90's" is financed by the Danish government and carried out in cooperation between the Technical University of Denmark, several private companies, and Aalborg University. The research has focused on the mix design, uni- and triaxial strength, creep, shrinkage and chloride diffusion of HPC. Applications of HPC have included several relatively short span bridges to obtain knowledge on the full scale performance with respect of the structural behavior and the durability characteristics. This knowledge has been utilized in the design of two major infrastructure projects — the Great Belt Link and the Oresund Link — which are under construction [Nielsen et al. 1996].

7.5 France

Based on the recommendations of a 1985 white paper presented by the National Steering Committee for research in civil engineering, the six-year French National Project on "New Ways for Concrete" was created in 1986 by thirty partners from industry and research institutes with the support of the French Ministry of Public Works and the French Electricity Board. The objective of the Project was to explore and develop new concepts and methods for utilizing high performance concrete by following four basic principles:

- To ensure the best possible interface between fundamental research, applied research, experimental structures, codes, and development
- To promote the idea of closely associating the concepts "new materials", "new designs", and "new construction methods".
- To highlight the notion of "high performance", showing clearly that for various "new developments", improved properties are to be achieved such as high or very high compressive strength, durability, water and air tightness, density, abrasion and impact resistance, resistance to frost, and ease of placement, etc.

The first achievement of this Project was the construction of the Joigny Bridge using 60 MPa (8,600 psi) concrete as described previously in Section 6.2.2. Instrumentation installed in the bridge was monitored to evaluate creep and shrinkage of the bridge. Additional applications of HPC included bridges, buildings, offshore oil platform, barge and tunnel, and nuclear containment vessels, etc. These applications have been described recently by Cheyrezy [1996], Monachon and Gaumy [1996], and Dugat et al. [1996].

Another significant contribution of the Project is the publication of an authoritative treatise on high performance concrete edited by Yves Malier [1992].

7.6 Japan

A five-year research program entitled "Development of Advanced Reinforced Concrete Buildings Using High-Strength Concrete and Reinforcement" was organized by the Japanese Ministry of Construction in 1988. Concrete strengths ranging from 30 to 120 MPa (4,350 to 17,400 psi) were considered along with the strength of reinforcing bars ranging from 400 to 1,200 MPa (58,000 to 174,000 psi). Research studies covered the development of constituent materials for high strength concretes and the determination of their physical properties. In addition, the program also included the development of high strength steel bars, the determination of the mechanical properties of the steel bars and their bond with high strength concrete, structural behavior of beams and columns made with such high-strength materials, as well as structural design methodology including seismic design [Aoyama et al. 1990; Kaku et al. 1992].

Almost concurrent with this major research and development program is the development of "self-compactable" or "flowable" high performance concrete with close cooperation among universities and industry. The primary motivation for the development is to minimize the manual handling of concrete on the job site (to overcome shortage of skilled labor force) with the ultimate goal of achieving automation of concrete construction. An excellent report describing this development has been presented by Okamura and Ozawa [1994].

7.7 Norway

The Norwegain national research program on high performance concrete has been supported by the Royal Norwegian Council for Scientific and Industrial Research (NTNF) and the concrete industry. The program includes research on concrete mix design, durability, mechanical properties, flexural and shear capacities, and fatigue behavior. Some results from this research have been introduced into the revised Norwegian Standard NS 3473 [Jensen 1990]. More recent research activities have focused on early-age cracking, pore structure, and durability of high performance concrete [Sellevold 1996]. There have been four international symposia on the utilization of high strength/high performance concrete, and Norway has hosted the first and the third symposia.

7.8 Sweden

In Sweden, a six-year organized research program on high performance concrete was initiated in 1991. The total budget for the program is at the level of $6.6 million or $1.1 million per year. Funds are provided jointly by a consortium of six companies (Cementa, Elkem Materials, Euroc Beton, NCC, SKANSKA, and Strangbetong) and two government agencies (NUTEK and BFR)on a 50/50 basis. 75% of the budget is allocated to research performed at six universities, one research institute, and one private research unit, the remaining 25% of the budget is allocated to work done by R & D personnel of the industrial partners. The objectives of the program are two-fold: (1) to increase the basic knowledge in selected areas of HPC and therby to create a competent staff of R & D personnel, and (2) to adapt and translate research findings in Sweden and elsewhere into recommendations and guidelines for use by the industry in their own product and process development. The program focuses on three general areas: materials, production technique, and structures. In each of these areas, five to seven specific research topics are pursued. The major concerns are mix design, workability, curing, durability, cracking, bond, toughness, and fatigue. A comprehensive summary of the research program has been presented by Elfgren et al. [1996]

7.9 United States

In the United States, research and development of HPC has been conducted by several nationally organized programs and government laboratories. In addition, several universities are also involved substantially in HPC research, generally with joint support of the Federal, State and private sectors. Presented below is an overview of the major activities. It is not intended to be a comprehensive survey.

7.9.1 ACBM

A unique Center of Excellence for Science and Technology of Advanced Cement-Based Materials (ACBM) was established by the National Science Foundation at Northwestern University in 1989. The Center is a research consortium made up of four universities (Northwestern, Illinois, Michigan, and Purdue) and the National Institute of Standards and Technology (NIST). The primary mission of the Center is to engaged in basic research to develop and transfer knowledge needed to understand the family of complex cement-based materials. ACBM's activities fall in three program areas: research, education and training, and technology transfer.

Now in its 7th year of operation, ACBM's research projects are organized into five key theme areas: Processing, Interfaces, Microstructural Characterization, Transport Phenomena, and Toughening Mechanism. Five or six projects are pursued in each of the theme areas. The issues addressed in these projects are to identify the chemical and physical phenomena that influence the microstructure development, to develop new techniques to characterize microstructure, to define relationships between microstructure and bulk material properties, and to explore methods to modify the matrix to yield tougher, more durable cement-based materials. The Center's objectives are "not only to improve the performance and predictability of high performance concrete through the application of science and technology, but also to develop novel cement-based materials with targeted properties" [ACBM 1995]. A summary of ACBM's activities has been presented by Shah [1996]

Although ACBM conducts basic research for the most part, it is also engaged in applied research through its Industrial Affiliate program. The recent development of a patented extrusion technology to produce cement-based matrix with large volumes (2 to 8%) of discontinuous fibers [Shah 1996] and the announcement of a new Shrinkage Reducing Admixture [Northwestern News 1966] are but two examples of this activity.

7.9.2 SHRP

The Strategic Highway Research Program (SHRP) was a 5-year , nationally coordinated research effort initiated in 1987 at a cost of $150 million. One of the four program areas of SHRP was Concrete and Structures for which funding was budgeted at approximately $18 million. Within the program area of Concrete and Structures, SHRP C-205 on Mechanical Behavior of High Performance Concrete was a 4-year (1989-1993) project conducted by a consortium among researchers at North Carolina State University, the University of Arkansas, and the University of Michigan with a budget of $2 million. Three types of high performance concrete — very early strength (VES), high early strength (HES), and very high strength (VHS) — were developed using four different types of coarse aggregate. A variety of properties of the fresh and hardened concretes related to workability, strength, and durability were determined, and field installations of VES and HES pavements were implemented in five states (Arkansas, Illinois, Nebraska, New York, and North Carolina). The research results were presented in a series of six reports. [Zia et al. 1993a, 1993b, 1993c, 1993d, 1993e; Naaman et al. 1993]. In addition, an annotated bibliography [Leming et al. 1990] and a state-of-the-art report [Zia et al. 1991] were also prepared.

Other SHRP projects related to high performance concrete are the field studies conducted under SHRP C-206 [Nagi and Whiting 1994], the development of HYCON expert system [Clifton and Kaetzel 1994], and the study of resistance of concrete to freezing and thawing [Janssen and Snyder 1994]

7.9.3 NIST

As a member of the research consortium of ACBM, the National Institute of Standards and Technology (NIST) established the Cementitious Materials Modelling Laboratory (CMML) in 1989. The Laboratory is staffed by members of the Inorganic Building Materials Group of NIST's Building Materials Division and visiting personnel from other organizations. The goal of the Laboratory is to develop computer models which will simulate the physical, chemical, and mechanical behavior of concrete and other cementitious materials, thereby gaining insight into factors that affect their engineering performance. Combining such models with other computer-based representations of knowledge will ultimately provide the technical basis and tools for optimum design of high-performance concretes and other new cementitious materials. To model the behavior of portland cement paste, two- and three-dimensional models at the nanometer to micrometer levels have been developed to simulate the hydration reactions occurring between anhydrous cement particles and water and the models have been used to study the phenomena of setting and diffusivity [Bentz et al. 1996].

As mentioned previously, the CMML has also developed under the sponsorship of SHRP an expert system "HYCON" as a diagnostic tool for identifying likely causes of distress in highway pavements and bridge structures [Clifton and Kaetzel 1994]. In addition, NIST in a joint effort with the National Aggregates Association/National Ready Mixed Concrete Association investigated the effects of testing variables (end preparation, cylinder size, type of test machine and testing speed) on the strength of high strength concrete cylinders [Carino et al. 1994].

7.9.4 FHWA

Federal Highway Administration (FHWA) has been engaged in research and technology transfer of high performance concrete. In a multi-year study of development length of uncoated and epoxy-coated prestressing strands at its Turner-Fairbank Highway Research Center, tranfer and development lengths of the types of strands would be determined with full-sized prestressed concrete girders using 10,000 psi (69 MPa) high strength concrete [Lane and Podolny 1993].

To transfer the HPC technology generated from the SHRP projects, FHWA has initiated a major effort in cooperation with the state highway agencies to construct demonstration bridges and to present showcase workshop to promulgate the experience gained from the demonstration projects [Vanikar and Goodspeed 1996]. The details of the demonstration bridges in the states of Texas, Virginia, Nebraska, and New Hampshire are described by Duwadi, Lane, and Berley [1996]. Several other states including Georgia, Oregon, Washington, and North Carolina are expected soon to participate in the demonstration program.

7.9.5 WES

The mission of the U. S. Army Corps of Engineers Waterways Experiment Station (WES) is to serve the needs of the U. S. armed forces, the Army civil works program, and other agencies of the U. S. government. In its role in research and development in concrete technology, WES has had a long history of developing high-performance concrete to serve some specific purpose for which ordinary concrete was not adequate. These developments include tremie concrete for underwater construction, high-performance mass concrete for dams, high strength concrete (up to 223 MPa or 32,300 psi) for hardened military facilities, high-strength and high-density concrete for biological shielding from ionizing radiation, high-performance concretes for nuclear weapons testing and nuclear waste isolation, and high-abrasion-resistant and washout-resisting concretes for repair of abrasion-erosion damage under water to stilling basins below dams. Detailed descriptions of these developments have been provided by Mather [1996].

More recently, through its Construction Productivity Advancement Program (CPAR), WES has been engaged with private industries in several research and development projects in high-performance cementitious materials. In April 1994, WES completed a study on the performance of concretes proportioned with Pyrament Blended Cement [Husbands et al. 1994]. Then, at the request of FHWA, WES conducted a preliminary investigation of a newly developed cementitious material from Ash Bonding Chemicals (ABC) Corporation. The ABC cement is a high-early strength, blended hydraulic cement containing 77 to 95 percent Class C fly ash by weight. The remaining material can be slag and/or portland cement. Four readily available chemical admixtures are used in small quantities to control the workability and setting time of the material. They are a set-suspending agent, an activator, a modifying retarder, and an accelerator. The performance of the concrete produced with ABC cement was compared with the performance of concrete using Type III cement. The results indicated the ABC cement as a possible alternative to Type III portland cement for producing high-early strength concrete. The concrete produced with the ABC cement has an advantage of being able to adjust the setting time of the concrete to whatever is appropriate for a given application, low to very low chloride permeability, good frost resistance even with low air contents, minimal shrinkage at early ages. However, there was more variation in the slump and air contents for the mixtures produced with the ABC cement. The performance of the concrete also varied with the type of fly ash used and with the application of the four chemical admixtures. Therefore a more comprehensive research program has been recommended in order to fully exploit the potentials of ABC cement [Neeley 1995].

WES also collaborated with Master Builders, Inc. to test an admixture system that will enable a concrete producer to tailor the working time of fresh concrete for particular applications and ambient conditions. The research evaluated DELVO stabilizer and activator for standard ready-mixed concrete applications, including long-haul, same-day, and overnight stabilization. In addition, WES is working with HDR Engineering, Inc. to develop very high-performance concretes using reactive-powder concretes (RPC). Concretes with strengths 10 to 35 times that of conventional concrete can be produced with vastly improved material properties. Use of RPC for precast concrete products is being investigated. Furthermore, WES and 3M Corporation are jointly developing and testing a new fiber-delivery system that will allow greater volumes of inexpensive polymer fibers to be introduced into concrete mixtures. The increased volume of fibers will allow the polymer-fiber concrete to match properties achieved with steel fibers. The delivery system is efficient and simple [U. S. Army Corps of Engineers Waterways Experiment Station 1995]

7.9.6 Other Organizations

Though not within any formally organized research programs, there are a number of other public and private institutions where subtantial research in high performance concrete is in progress. These institutions include public agencies such as Virginia Transportation Research Council, Florida Department of Transportation, North Carolina State University, University of Minnesota, University of Texas, University of Nebraska, University of Kansas, University of Southern California, and the private firm Construction Technology Laboratory. The research activities of these organizations have been covered in numerous publications described and discussed in this report.

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CONCLUSIONS

This report is a sequel to a previous state-of-the-art report published by SHRP in 1991 and is based on review of selected literature on high performance concrete with particular reference to highway applications. It covers primarily the six-year period from 1989 to 1994 for which the authors have recently compiled an updated annotated bibliography, containing 776 references, which has been published by the Federal Highway Administration (Publication No. FHWA-RD-96-112). More recent information on the subject is obtained directly from a variety of publications including books, technical journals, conference proceedings, and research reports.

It should be emphasized that, unless otherwise noted, much of the information contained in the previous state-of-the-art report is still current and will not be repeated in the present document. Therefore this document should be viewed as a supplement to rather than a replacement for the previous report. For ease of cross-referencing, the organization and format of the present report have been kept nearly the same as the previous state-of-the-art report. Based on the results of the literature review, the following general conclusions can be drawn:

1. The growth of the amount of research and applications of high performance concrete has been phenomenal in the past seven or eight years. High performance concrete has become widely accepted practically on all continents.
2. A generalized definition of high performance concrete seems to have been accepted by the engineering community. Such a definition is based on achievement of certain performance requirements or characteristics of concrete for a given application that otherwise can not be obtained from normal concrete as a commodity product.
3. Based on the results of the SHRP projects, the Federal Highway Administration has developed a sensible classification of high performance concrete according to different levels of performance requirements. Such a classification would enable design engineers to select appropriate performance criteria of HPC for different highway applications in different environmental conditions.
4. Much of the application of HPC remains in the areas of long-span bridges and high-rise buildings. HPC is used more for bridges than buildings in Europe and Japan, while more buildings than bridges used HPC in the U. S. However, the situation is changing. Use of HPC in buildings is increasing abroad and there is a growing interest in using HPC in bridges in the U. S.
5. There is much understanding of selection of materials, proportioning methods and curing control for the production of high performance concrete. However, much less control is exercised on the concrete placement. In this regard, the development of "self-compactable" concrete in Japan is a significant step toward achieving high performance concrete through automation.
6. Increasing emphasis is being placed on concrete durability than its strength. In many applications, high strength concrete is used only because of its high durability quality rather than the need for its strength.
7. There has been an enormous amount of research performed on durabilty of concrete, but without much correlation largely because the property is "material specific" and dependent on test methods. There is an urgent need for new and improved test methods that would provide more consistent correlation between the laboratory and field results so that the data on durability can be better quantified. More research is needed to develop a rational design methodology for durability.
8. Much research continues to be focused on the mechanical properties of high- and very-high-strength concretes and their structural applications. The results of this research are being incorporated into various national codes of practice. However, more information is needed on the behavior of the concrete at its early age and its relationship to the long-term performance.
9. Comparatively speaking, there is a dearth of information on the mechanical behavior of controlled lower strength concrete. Similarly, the need exists for more research on high performance lightweight concrete so that its use can be more widely accepted in practice.

10. The Slurry Infiltrated Mat Concrete (SIMCON) and the delivery system for non-metallic fibers developed by 3M Corporation are two significant recent developments in the area of high performance fiber reinforced concrete.
11. There has been significant interest and development in the use of continuous fiber reinforcement for improving the behavior of cementitious composites and/or concrete. Fiber Reinforced Polymers (FRP) or sometime also referred to as Fiber Reinforced Plastic are increasingly being accepted as an alternative for uncoated and epoxy-coated steel reinforcement for prestressed and non-prestressed concrete applications.