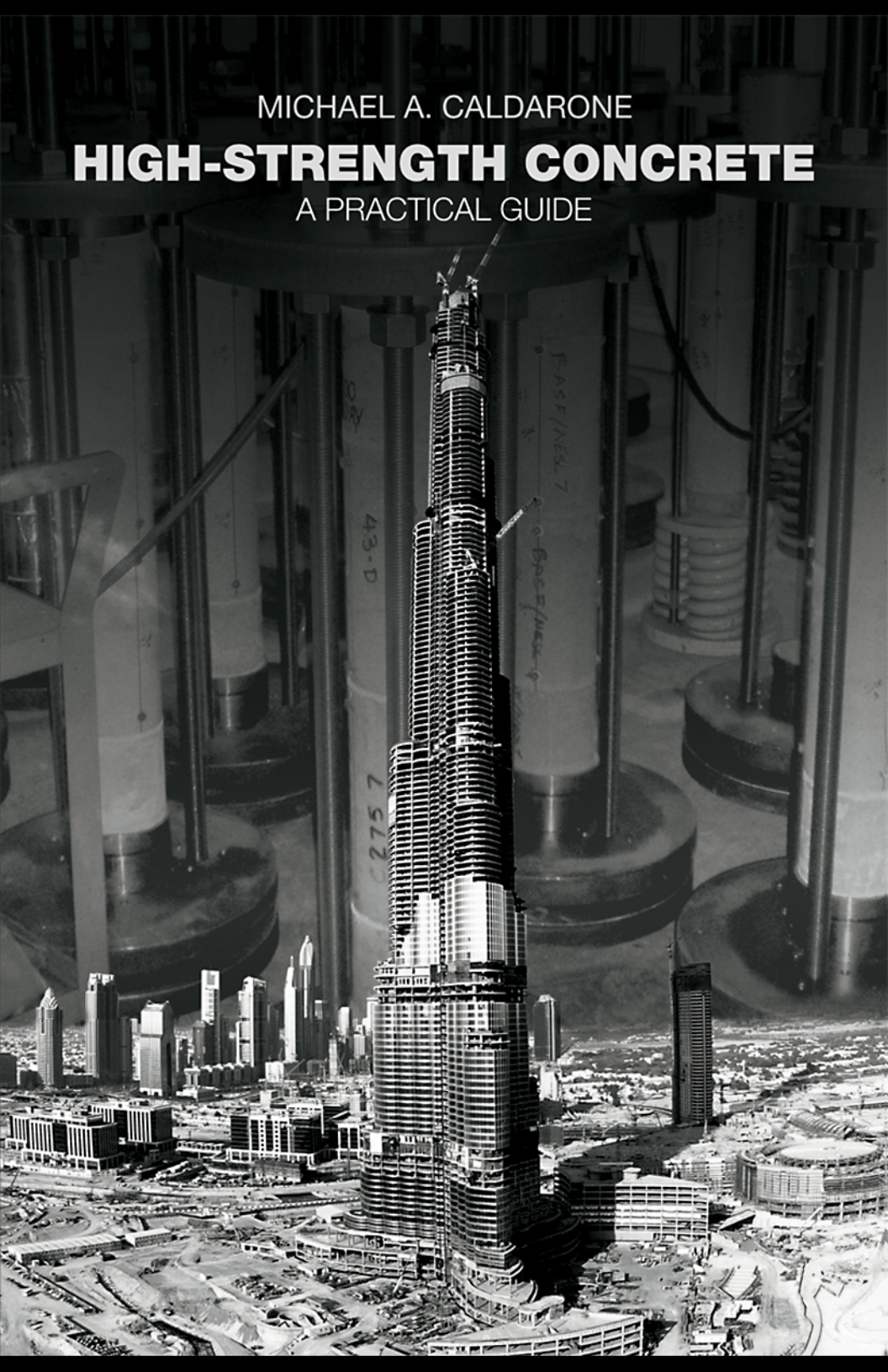
MICHAEL A. CALDARONE

HIGH-STRENGTH CONCRETE

A PRACTICAL GUIDE



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A practical guide

Michael A. Caldarone

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To Ilona and Andrew

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Contents

List of illustrations x
Preface xiii
Acknowledgments xviii
List of abbreviations xix

1 Introduction **1**

Unit conversions 2
Terminology 2
Historical background 12
Applications 15
References 20

2 Constituent materials **21**

Introduction 21
Cementitious materials 21
Aggregates 48
Water 53
Chemical admixtures 54
Air-entraining admixtures 59
References 61

3 Mixture proportioning and evaluation **64**

Introduction 64
Identifying relevant concrete properties 65
Statistical variability 67
Proportioning considerations 68
Designated acceptance age 84
ACI 318 code requirements for strength acceptability 85
Trial evaluation 86
Proportioning high-strength concrete: an example 88
References 96

www.technobetar.ir

4 Properties **99**

Introduction 99
Mechanical properties 99
Durability properties 115
Constructability properties 124
References 126

5 Specifications **131**

Introduction 131
Prescriptive vs. performance-based specifications 131
The pitfalls of arbitrarily established limits 133
The relevancy of the slump test 135
Constituent materials 136
Quality management plans 136
Producer qualifications 137
Submittals and conditions of sale 138
Testing laboratory qualifications 139
Preconstruction conferences 139
Post-28-day designated acceptance ages 140
References 142

6 Production and delivery **143**

Introduction 143
Order taking 144
Dispatching 144
Quality control 145
Plant operations 147
Delivery 150
References 152

7 Placement, consolidation, and finishing **153**

Introduction 153
Preconstruction conferences 154
Preparation 155
Placement 155
Consolidation 159
Finishing 159
Case study: When self-consolidation is not enough 160
References 162

8 Curing **164**

Introduction 164
Moisture requirements 166

www.technobetar.ir

Temperature requirements 171
Curing high-strength precast concrete 171
References 173

9 Quality control and testing **174**

Introduction 174
Testing variables influencing compressive strength 175
Standard cured vs. field cured specimens 186
In-place evaluation 187
Profiling constituent materials in the laboratory 191
Case Study: Jobsite curing in limewater 193
References 197

10 Problem solving **200**

Introduction 200
Incompatibility 200
Early stiffening and erratic setting 211
Poor strength development 212
Aesthetic defects 213
Petrography 218
Case study: When color becomes a concern 221
Case study: An autogenous shrinkage cracking investigation 223
References 225

11 Summary **226**

Glossary 237
Institutes and standard writing organizations 242
Index 245

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Illustrations

Figures

1.1	Office building at 225 West Wacker Drive, Chicago	14
1.2	Two Union Square, Seattle	15
1.3	Cross section and prestressed strand patterns Texas U54 beams	17
1.4	Parking structure at 900 N. Michigan Ave, Chicago	18
1.5	The super skyscraper Burj Dubai	18
1.6	An extraordinary increase in attainable building height	19
2.1	Micrograph of Type I Portland cement	23
2.2	Portland cement clinker	24
2.3	Relative reactivity of cement compounds	25
2.4	28-day compressive strength of two concretes	28
2.5	A micrograph of fly ash showing typical spherical particles	33
2.6	Compressive strength of concretes produced with fly ash	35
2.7	Common setting characteristics comparing low and high calcium fly ashes	36
2.8	Particle size comparison of conventional and ultra-fine fly ash	39
2.9	Micrographs of conventional fly ash and ultra-fine fly ash	40
2.10	Micrograph of ground granulated blast-furnace slag grains	41
2.11	Scanning electron microscope micrograph of silica fume particles	43
2.12	Micrograph of densified silica fume	46
2.13	Scanning electron microscope micrograph of metakaolin particles	47
2.14	Effects of aggregate type and blend on mean 28-day compressive strength	52

www.technobetar.ir

3.1	Illustration of the relationship between W/B ratio and strength	72
3.2	Schematic representation of two fresh cement pastes	73
3.3	Illustration of interfacial transition zone	75
3.4	Effect of cement content on compressive strength at 28 days	77
3.5	Strength efficiency of Portland cement	78
4.1	Typical stress-strain relationship for high-, moderate-, and conventional-strength concrete	101
4.2	As compressive strength increases, failure takes on an increasingly explosive mode	102
4.3a	(SI units) Measured modulus of elasticity at 28, 91, and 426 days	104
4.3b	(inch-pound units) Measured modulus of elasticity at 28, 91, and 426 days	105
4.4	Measured modulus of elasticity at 91 days	105
7.1	Placement of high-strength concrete simultaneously with conventional-strength concrete	157
7.2	Honeycombs in monolithic spandrel beams	161
8.1	Severe plastic shrinkage cracking caused by ineffective interim curing	167
9.1	Effect of high temperature initial curing	180
9.2	Initial jobsite curing by immersion in lime-saturated water	181
9.3	Temperature controlled jobsite-curing box	182
9.4	Transportation boxes	183
9.5	Improper storage of test cylinders	
9.6	Rebound number determination using a “rebound hammer”	184
9.7	Evaluating surface hardness by means of penetration resistance method	189
9.8	Deviation from average strength at 3 days	190
9.9	Deviation from average strength at 7 days	194
9.10	Deviation from average strength at 28 days	194
9.11	Deviation from average strength at 56 days	195
9.12	Initial jobsite curing by immersion in lime-saturated water	196
10.1	Profile of normal paste hydration	201
10.2	Illustration of an ideally balanced paste during the early stages of cement hydration	202
10.3	Illustration of an “under-sulfated” paste	202
10.4	Illustration of an “over-sulfated” paste	203
10.5	1-day strength values for mortars made using cement with various levels of sulfate, Class C fly ash used as a 25 percent cement replacement, and a carbohydrate-based water-reducing admixture	206

www.technobetar.ir

10.6 The "mini-slump" test performed on paste samples 208
10.7 Mini-slumps pats taken at 2, 5, 10, 30, and 45 minutes 209
10.8 Examination of thin sections using a polarized-light microscope 219
10.9 Air voids cluster along the periphery of a coarse aggregate particle 220
10.10 Thin-section photomicrograph showing concrete damaged by expansive alkali-silica reaction (ASR) 221
10.11 Color contrast of darker column concrete with slab concrete 222

Tables

2.1 Bulk specific gravity of cementitious materials 22
2.2 Abbreviated notations used in cement chemistry 24
2.3 Primary compounds in Portland cement clinker 24
2.4 Various forms of calcium sulfate ($CaSO_4$) 26
2.5 American (ASTM) and Canadian Standards Institute (CSA) Portland cement classification 27
2.6 European (EN) "common cements" 27
2.7 ASTM C 595 classification for blended hydraulic cements 29
3.1 Example constituent material combinations for pastes of varying W/B ratios 88
3.2 Constituent materials used in first series of laboratory trials 90
4.1 Coefficient of thermal expansion of various structural concrete aggregates 114
9.1a Example mixtures used for laboratory evaluation of various cement samples (SI units) 192
9.1b Example mixtures used for laboratory evaluation of various cement samples (inch-pound units) 192

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Preface

My purpose for writing this book is principally to provide those who specify, produce, test, and construct with high-strength concrete practical guidance about a material that continues to be viewed as mysterious, exotic, and to some degree, even experimental. My hope is that this book will also aid the reader in understanding the fundamental mechanics of how structural concrete in general, not just high-strength concrete, works. High-strength concrete is not a new material. The availability of commercially produced high-strength concrete can be traced back to the late 1950s. In many markets worldwide, the commercial availability of concretes capable of developing compressive strength three to five times greater than typical conventional concrete is well established. In other markets, high-strength concrete is still considered novel.

As all new technologies are born in the research laboratory, intellectual concentrations naturally shift from primarily academic interest to practical and economic applicability. While the academic community continues to research and publish ways in which to expand the feasible boundaries of new materials and design methodologies, a shift from theoretical feasibility to practical applicability occurs. Once the transition from research laboratory to real world takes place, the amount of information published about the technology, and its practical applications, decreases.

Although the topic of this book is high-strength concrete, it does not define "high strength" by any single numerical value. My preference is to define "high strength" relative to what might be considered as "normal" or "conventional" strength in the geographic location it is being produced. Even though high-strength concrete usually accounts for no more than a very small fraction of all of the concrete used in modern construction, my personal appreciation for this material is not limited to the ability of making it in and of itself, but more so, the practical knowledge gained that is applicable to conventional concrete also. So even though the primary subject of this book is high-strength concrete, it should come as no surprise to the reader that it also contains information dealing with conventional concrete. Discussing the rationale supporting the technology of high-strength concrete in relation to conventional concrete is far more beneficial than simply

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presenting a set of guidelines and empirical relationships merely for purposes of rote memorization. Stated differently, simply knowing that two plus two equals four has far less potential for advancement compared to knowing *why* two plus two equals four.

Several key considerations were addressed prior to writing this book. First, is the presumption that the reader is generally familiar with both the fundamental terminology and basic principles associated with concrete technology.[1] With this in mind, the reader may find that some concepts considered fundamental to concrete technology have, for the most part, been omitted. In the event of an unfamiliar term, material property, or construction practice being encountered, several informative publications are suggested at the beginning of Chapter 1. Second, to be consistent with the book's title, only concrete-making materials, production methods, and construction practices considered "mainstream" to the industry will be covered. Exotic materials and manufacturing processes will not be addressed. This book will address high-strength concrete made using the same type of cements, aggregates, admixtures, and water that can be used to produce conventional-strength concretes.

There is a unique set of challenges for authors whose mission it is to write a book about concrete meant for an international audience. Each country has its own set of standards for concrete and its constituents. The true challenge for authors or academics when attempting to absorb the seemingly countless number of standards is that no international criteria for measuring concrete properties or defining the physical characteristics of concrete and its constituents yet exist. The exercise is like attempting to assimilate a set of books written in multiple languages, presumably about one particular area of knowledge, that are virtually non-translatable. Fortunately, international standardization attempts are being made with organizations such as the European Committee for Standardization (CEN). Unfortunately, it may take many years before such harmonization occurs on a global scale. In fact, with the vast amount of knowledge that has been collected about concrete and its constituents in modern times, the absence of more universally-oriented standards is an unfortunate roadblock to the concrete industry worldwide. Since the standards and test methods that apply to concrete vary so significantly worldwide, unless considered central to the subject at hand, much of the information presented will refer largely to North American guides and standards published by organizations such as the American Concrete Institute (ACI) and American Society for Testing and Materials (ASTM).

Compared to conventional-strength concrete, the use of high-strength concrete offers a multitude of advantages considering both the technical and economical aspects. The prime objective of this book is to provide the reader with an understanding of the principles and methodologies associated with the commercial applicability of high-strength concrete. Doing so will require identifying some popular myths and misconceptions about concrete.

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This book was not written with the implicit intention of identifying common misunderstandings or misconceptions rooted within the concrete industry; however, in order to satisfy the prime objective, it will occasionally be necessary to distinguish the myth from the fact. Without doing so, it would be much more difficult to develop a comprehensive understanding of why the principles governing high-strength concrete can be so different from those governing concrete of a more conventional strength.

The most notable evolutionary period thus far in the development of ready-mixed high-strength concrete unquestionably occurred in Chicago between the years 1960 and 1990. It was in Chicago, during the early 1960s, that a "perfect storm" of opportunity for the development of commercially available high-strength concrete came together. What emerged was nothing less than a golden age in the history of a state-of-the-art construction material. In 1962, high-strength concrete with a design compressive strength of 42 MPa (6000 psi) was supplied to Chicago's 40-story Outer Drive East high-rise condominium project. At that time, the commercial availability of 40 MPa (6000 psi) was considered a breakthrough. By 1989, commercially available concrete with a design strength of 96 MPa (14,000 psi) was supplied for six stories of columns along with one 117 MPa (17,000 psi) experimental column at the 225 W. Wacker project (Moreno, 1990). What came together was a rapidly growing high-rise building market and an engineering community ready to reap the advantages that come with higher-strength materials; high-quality locally available raw materials (including the new "chemical" admixtures that were making their way into mainstream industry), and lastly, at the focal point of it all was a premier ready-mixed concrete producer with a progressively minded technical staff. The company was Material Service Corporation (MSC) and the individuals principally responsible for the birth and continual evolution of high-strength concrete in Chicago were Ron Blick, Ralph Vencil, Mike Winter, Chuck Peterson, John Albinger, Art King, and Mike Pistilli.

Since the early 1960s, the market demands, material supply, and knowledge in the art and science of making high-strength concrete came together in Chicago. In order for all the necessary components to come together and make it possible, there was one more critically essential element needed. In the case of high-strength concrete in Chicago, the essential element prompting the coming together of high-strength material and design was the communication between the material supply and design communities. For those familiar with both materials engineering and structural design, it will come as no surprise that structural engineers and materials engineers appear to speak different languages. If it had not been for the efforts of the technical and engineering staff, and individuals like Jaime Moreno, working side by side with local designers, the author does not believe that the evolution that took place in Chicago could ever have been possible. In three short decades, there was nearly threefold increase in commercially

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available compressive strength. The history of high-strength concrete is covered in more detail in Chapter 1.

My personal interest in materials engineering can be traced back to the inspiring lectures of my first materials engineering instructor, Dr Antoine Naaman, at the University of Illinois, Chicago. My personal interest in high-strength concrete was born during a field trip to Material Service Yard No. 1 near downtown Chicago. Dr Naaman arranged this field trip every semester for his students. In fact, it was on the day of that field trip that I suspected (or at least hoped) where my future interests would lie. Just before the tour bus departed, I recall Art King's final words and the profound effect they would have on my life—"and remember Material Service makes good concrete." Years later, I reminded myself how important it is to maintain a focus on long-term objectives because, in 1989, I was offered a position with MSC. Although the evolutionary years of high-strength concrete in Chicago was in its twilight, the five years I spent in the Quality Control Department at MSC under the guidance of Art King was an excellent personal opportunity in and of itself. Sadly, in 1994, as a purely business decision, MSC sold its ready-mixed concrete operation.

Of course, Chicago was not the only place where great things happened with high-strength concrete. What happened in Chicago became a great source of inspiration for others. Interestingly, in the preface of his book (Aïtcin, 1998), Professor Pierre Claude Aïtcin of University of Sherbrooke wrote:

> My first exposure to high-strength concrete dates back to 1970, when I first heard John Albinger of Material Service make a presentation on the high-strength concrete he was delivering in the Chicago area at that time. He was so convincing and enthusiastic about high-strength concrete that I decided to end my concrete class at the University of Sherbrooke every year with a contest whose objective was for my students to make the strongest concrete with a maximum amount of cement and supplementary cementitious materials of 600 kg/m^3 (1000 lbs/yd^3).

Unlike Dr Aïtcin, when I first met John Albinger in the mid-1980s, my interest in high-strength concrete was already solidly established, thanks to Dr Naaman's thoughtfully planned student field trips. John's passion and enthusiasm about high-strength concrete had not waned. Looking back, there is no question that John Albinger was a major source of inspiration for me. Passion about one's chosen field is both marvelous to have and difficult to hide.

The intention of this book is to pass on as much useful, practical information that the forthcoming pages will allow. This book, in many respects, expands upon the principles contained in American Concrete Institute's State-of the-Art Report on High-Strength Concrete published by Committee 363. Also included is knowledge learned over the past 20 plus years from colleagues, along with some "real world" case studies.

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There is countless advancement still to be made in the field of cement and concrete. Scientists, mathematicians, and engineers, including Sir Isaac Newton (1642–1727), recognized that most advancement in knowledge is built upon the achievements of those who came before them. In a letter to fellow scientist Robert Hooke on February 5, 1676, Newton modestly wrote: *"If I have seen further it is by standing on the shoulders of giants."* To illustrate, Newton's Law of Gravitation, which could be used to mathematically describe, among other things, planetary motion, was developed using the highly precise calculations of Johannes Kepler (1571–1630). Kepler's contributions to scientific knowledge, on the other hand, may never had occurred had he not stood on the shoulders of giants with such names as Galileo (1564–1642), Brahe (1546–1601), and Copernicus (1473–1543).

Although perhaps not as intriguing as planetary physics, the evolution of knowledge in the field of cement and concrete is no different. In industry and academia, there are many great shoulders yet to be climbed.

Michael A. Caldarone
November 2007

Notes

1 The term "concrete" can be used to describe any composite material comprised of filler and binder. In this book, *concrete* will be an abbreviated term meaning *hydraulic cement concrete*.
2 Later renamed *Building Code Requirements for Structural Concrete*.

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I am fortunate to have had the opportunity to work closely with John Albinger, Art King, Ron Burg, David Crocker, Tony Fiorato, Larry Roberts, Gene Daniel, Russell Hill, Joe Lee Holmes, Karthik Obla, Wilma ("Willy") Morrison, Michael Morrison, Bennie Proctor, Dale Bentz, Ken Rear, Jim Shilstone, Peter Taylor, Phil Smith, Mark Chiluski, Eugene Harbour, Gyu Dong Kim, Colin Lobo, and Rom Young. Their friendship and professional collaboration has meant a great deal to me, and their conversations have clarified my thinking on principles discussed in this book and many other matters.

Serving as Chair of American Concrete Institute Committee 363 on High-Strength Concrete for the past five years has been both an honor and a pleasure, and it has given me an opportunity to both learn and contribute to the advancement of the concrete industry. I would like to thank the members of Committee 363 who contributed their time and efforts, particularly Henry Russell, Paul Zia, Jim Cook, Nick Carino, John Myers, Dan Jansen, Mike Pistilli, Bob Sinn, John Bickley, and Mark Luther, and staff at ACI, including Pat Levicki, Miroslav Vejvoda, Todd Watson, and Dan Falconer.

A number of organizations have graciously allowed me to use some of their material as illustrations and examples. In this regard, I am indebted to the Portland Cement Association, CTL Group, American Concrete Institute, Boral Material Technologies, Inc., and Samsung Engineering and Construction.

Lastly, I personally wish to thank you, the reader, for taking the time to learn more about high-strength concrete. The fact that you are reading this book shows that you are dedicated to the advancement and sustainability of hydraulic cement concrete, the world's most versatile used construction material.

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Abbreviations

AAR	Alkali-aggregate reaction
ACI	American Concrete Institute
ACR	Alkali-carbonate reaction
AIJ	Architectural Institute of Japan
ASR	Alkali-silica reaction
ASTM	American Society for Testing and Materials
BSI	British Standards Institute
CEN	European Committee for Standardization
CH	Calcium hydroxide
CSA	Canadian Standards Institute
CSH	Calcium silicate hydrate
DEF	Delayed ettringite formation
DIN	German Institute for Standardization
HPC	High-performance concrete
HRWR	High-range water-reducing chemical admixture
HSC	High-strength concrete
ISO	International Organization for Standardization
l/d	length-to-diameter ratio
MSC	Material Service Corporation
NRMCA	National Ready-Mixed Concrete Association
PCI	Pressed Concrete Institute
QMP	Quality Management Plan
SAC	Standardization Administration of China
SCC	Self-consolidating concrete
SCM	Supplementary cementitious material
TEA	Triethanolamine
UFFA	Ultra fine fly ash
VMA	Viscosity modifying admixture
W/B	Water-binder (ratio)
W/C	Water-cement (ratio)
W/C+P	Water-cement plus pozzolan (ratio)
W/CM	Water-cementious materials (ratio)

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1 Introduction

Perhaps an appropriate way to begin this book is not to discuss what high-strength concrete is, but rather, what it is not. Having the word "strength" in its name undeniably suggests a bias towards one property only; however, high-strength concrete can be an advantageous material with respect to other properties, both mechanical and durability related. Nevertheless, it is crucially important to recognize that the achievement of high strength alone should *never* summarily serve as a surrogate to satisfying other important concrete properties. It would seem logical that strong concrete would be more durable, and in many respects, the lower permeability that comes along with higher strength often does improve concrete's resistance to certain durability-related distress, but unlike strength, the prerequisites for durability are not easily defined. In fact, depending on the manner in which higher-strength is achieved, the durability of high-strength concrete could actually diminish. For example, if cementing materials are not carefully chosen, higher-strength mixes could conceivably contain an objectionably high quantity of soluble alkalis that could promote cracking if aggregates that are potentially susceptible to alkali reactivity are used. Throughout this book, the reader will frequently encounter references stressing the importance of identifying all relevant properties when developing high-strength concrete. However, equally important is identifying properties that are *not* relevant that could *impede* the ability to achieve the truly important properties.

There are extraordinary differences when comparing the properties of a very high-strength concrete having a compressive strength of 140 MPa (20,000 psi) to that of a conventional-strength structural concrete with a compressive strength of 30 MPa (4000 psi). When considering the adjustments to the principles of mix proportioning necessary in order to satisfy mixture performance requirements, it is interesting to note that no abrupt change in material technology occurs at any one particular level of strength, or at a particular water–binder (W/B) ratio. Rather, the changes that occur when progressing up the strength ladder are quite subtle with each advancing step. As the W/B ratio changes, so do the principles governing mix proportioning, which in turn establishes strength and other mechanical properties. In order to develop an intuitive understanding of how it is possible to

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produce concretes four to five times stronger than conventional concrete, any beliefs that the principles governing concrete proportioning change little should be abandoned from this point on.

It is only natural that hydraulic cement concrete would be viewed as a single material, but in reality, concrete is much better understood when viewed as a composite material comprised of two fundamentally different materials—filler (i.e. aggregate) and binder (i.e. paste). Material properties, principally those mechanical in nature are fundamentally derived from the relative similarities (or differences) in the properties of the aggregate and paste. For this reason, the laws governing the selection of materials and proportions of concrete are by no means static. The most influential factor affecting the strength and largely influencing the durability of concrete is the water-binder (water-cement) ratio.

Hydraulic cement concrete is a two-component composite material fundamentally consisting of aggregates and paste. The principles applicable to proportioning structural concrete are primarily driven by the relative mechanical properties of paste and aggregate. For this reason, proportioning guidelines that might be viewed as "best practice" for one strength level might be quite inappropriate for concrete of a different strength class. The requisite properties of constituents and material proportions will subtly vary from one W/B ratio to another. This fundamental principle applies to the entire spectrum of strength achievable with hydraulic cement concrete when using mainstream, non-exotic constituent materials. This book primarily addresses normal-weight high-strength concrete using constituents and construction practices appropriate for producing compressive strengths with an upper limit of approximately 140 to 150 MPa (20,000 to 22,000 psi) using mainstream materials and testing standards. This book does not address high-strength concrete produced with exotic materials or uncommon manufacturing or evaluation methods.

Unit conversions

Both SI and inch-pound units are expressed in this book, with SI being the primary unit of measurement. In most of the information presented, the values stated in each system will be rounded to only reasonable approximations, but more precise conversion values will be made when warranted. For example, when addressing a general principle, 60 MPa would be rounded to 9000 psi, yet when discussing a particular project where 60 MPa was specified, 8700 psi will be expressed.

Terminology

A section addressing terminology has been placed at the beginning of this book in the hope that the reader will be able to navigate through the coming pages with a minimal amount of needless, terminology-induced stress. The

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meanings of most of the terms used in this book are generally accepted among the major standards writing organizations and institutes worldwide: however, in some circumstances, terms used in one part of the world can have a different meaning in other parts. For example, in the US, the term "admixture" refers to a material other than water, aggregates, hydraulic cementitious material, and fiber reinforcement that is used as an ingredient of a cementitious mixture to modify its freshly mixed, setting, or hardened properties and that is added to the batch before or during its mixing. In the UK, "admixture" is used to mean a material added during the mixing process of concrete in small quantities related to the mass of cement to modify the properties of fresh or hardened concrete. When a powdered admixture is added to factory-made cement during its production, it is called an "additive" and not an admixture. Most would probably agree that the implications of misapplying the term "admixture" would be relatively innocuous; however, with other terms, the consequences can be more serious. For example, in the US, "slag cement" is one of several terms used for the material most accurately described as "ground granulated blast-furnace slag." However, in other parts of the world, "slag cement" can refer to blended hydraulic cement containing ground granulated blast-furnace slag as a major constituent (adding to the confusion in the US, until recently, "slag cement" also referred to blended hydraulic cement containing ground granulated blast-furnace slag!).

Please note that the terms discussed in this section and defined in the Glossary are strictly for the purpose of this book and are based largely on the author's personal preferences.

Water-binder ratio (W/B)

When first presented by Duff Abrams in 1918, the meaning behind the term "water–cement ratio" was indisputable. At the time, Portland cement was essentially the only binder used for making hydraulic cement concrete. In the early twentieth century, fly ash was still drifting up power plant chimneys, and other materials, such as silica fume, did not yet exist. Ground granulated blast-furnace slag and natural pozzolans, although in use, were not yet "mainstream" to the industry. In later years, with the increased use of supplementary binders, terms such as *water-cement plus pozzolan ratio* (W/(C+P)) and *water-cementitious materials ratio* (W/CM) came into use. When the chemical and physical properties and relative proportions of cementing materials vary (including Portland cements), the relationship between strength and water content, or pore space and water content changes. However, for reasons that will be provided in a more detailed discussion of this important subject in Chapter 3, the author has chosen to adopt the single term *water-binder ratio* (W/B) for expressing the mass ratio of mix water to the combined mass sum of all the binding materials used.

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Supplementary cementitious materials

Pozzolanic materials and hydraulic materials other than Portland cement have traditionally been referred to as *mineral admixtures*. Recently, there has been a shift in terminology to refer to materials such as fly ash, silica fume, ground granulated blast-furnace slag, and natural pozzolans as "supplementary cementitious" or "supplementary cementing" materials. The origin of the term mineral admixture probably traces back to the days when most concretes essentially were comprised of aggregates, Portland cement, and water. Any other material introduced into the mix was considered an "additive" or "admixture." The term *mineral admixture* has been extremely useful for classification purposes, since it differentiates admixtures that are mineral in nature from those that are chemical in nature. Unlike chemical admixtures, which alter the minerals present in a binding system via chemical interaction, mineral admixtures contribute additional mineral oxides to the paste.

> For the purpose of this book, the terms *supplementary cementitious materials, supplementary cementing materials*, and *mineral admixtures* will be used interchangeably.

Strength

This following discussion is presented principally as a premise to providing definitions for the three strength-related terms that will be used most frequently in this book. They are:

- target strength;
- specified strength; and
- required average strength.

In the broadest of terms, strength refers to the maximum amount of stress that a material is capable of resisting until some predefined mode of failure occurs. In engineering, stress flow can be resolved into five fundamental categories—uniaxial compression, uniaxial tension, flexure, shear, and torsion. In the case of hydraulic cement concrete, stresses are most efficiently resisted under uniaxial compression; therefore, attention is almost invariably given to characterizing the mechanical properties of concrete in terms of compressive strength. Being an inherently brittle material, failure in compression is reasonably straightforward to define. A consequence of the internal fracturing that occurs when a brittle material is loaded in compression is that failure usually occurs suddenly. Being less brittle, conventional-strength concrete is capable of more inelastic strain than higher-strength

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concrete. As the strength of concrete increases, the static modulus of elasticity generally increases proportionally with compressive strength.

Hydraulic cement concrete is considered to have "failed" in compression when it is no longer capable of resisting stress due to the internal fracturing that has occurred.

Strength is a relative, not absolute material property. The strength of a material depends on more than just the manner in which stresses are distributed. The measured strength of concrete depends on numerous factors, several of which are age at time of testing, curing history, specimen size, shape, and loading rate. To state that the design compressive strength of a concrete is 60 MPa (9000 psi) has no substantive meaning whatsoever. For example, the measured compressive strength of cylindrically shaped specimens having a 2:1 length-to-diameter ratio (l/d)[1] will usually result in a different (usually lower) value of compressive strength compared to the measured strength of cubically shaped specimens having the same cross-sectional area cured and tested under identical conditions.

Target strength

Target strength simply refers to a desired level of measured strength at a given age, usually when evaluated under a standardized method of testing. It is important to recognize that target strength and design strength are unrelated terms. If a concrete mix was only proportioned to achieve a median average level of strength at which the structure has been designed, the statistical probability that the results of a compression test would be below design strength would be 50 percent. It is important for users of concrete, particularly specifying authorities, to understand that even under the most stringent production and testing processes, there will always exist a statistical probability that the result of a material test will fall below a required level. Though it may certainly be possible to establish processes that would result in unnaturally low probabilities for the occurrence of failures, the costs associated with such processes would likely be extremely high. Engineering is not only about applying scientific knowledge in usable ways, but also being able to do it in a practical and cost-efficient manner, and part of this is defining a threshold for tolerable failure.

Specified strength

Specified strength refers to a defined level of concrete compressive strength chosen by a code-recognized authority in the design of structures,[2] when tested at a designated acceptance age, under standard testing conditions, and evaluated in accordance with the acceptance criteria of a legally adopted design code, such as ACI 318–05.[3] For example, the specified compressive strength (f_c') for a series of columns in a tall building might be 70 MPa (10,000 psi) at 56 days.

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The terms "design compressive strength" and "specified compressive strength" will be used interchangeably. When the designated acceptance age is not given, it will be taken to be 28 days.

Required average strength

The required average strength (f_{cr}') is the average compressive strength used as the basis for the selection of concrete proportions necessary to comply with the strength acceptance criteria of a legally adopted design code, such as ACI 318–05. If the measured strength of concrete equals or exceeds f_{cr}', there is a statistical probability of only about 1 in 100 that the concrete fails to comply with the following strength acceptance criteria:

- Every arithmetic average of any three consecutive strength tests equals or exceeds the specified compressive strength (f_c').
- No individual strength test (average of two cylinders) falls below f_c' by more than 0.10 f_c'.

It is common to relate most of the mechanical properties of concrete to its strength when tested in uniaxial compression, therefore, throughout the book, the terms *strength* and *compressive strength* will be used interchangeably.

High-strength concrete

Defining "high strength" in terms of a universally applicable numerical value is not possible, at least not with any sound degree of rationale. "High strength" is a relative term that is dependent on many things, such as the quality of locally available concreting materials and construction practices. The author does not believe that high-strength concrete need be defined in terms of one numerical value; however, at the end of this section, I suggest a range that most authorities might agree is a reasonable threshold for what would be considered "high-strength concrete," at least at the time this book was written.

Strength is not an intrinsic property of concrete. It is a relative property that depends on numerous factors. Primary factors influencing the measured strength of concrete include specimen geometry, size, age, and curing history; testing equipment parameters, such as loading capacity, lateral and longitudinal stiffness, and the loading rate and uniformity of load distribution. There are geographic considerations also. In regions where compressive

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strengths of 60 MPa (9000 psi) is commercially produced on a routine basis, concrete might not be considered "high strength" until it attains a measured strength in the range of 70 or 80 MPa (10,000 or 12,000 psi). Conversely, in regions where the upper limit on commercially available concrete has been 30 MPa (4000 psi), concrete successfully meeting a design requirement for 40 MPa (6000 psi) might be considered high strength, and for good reason. The reason for such diversity is twofold: need and ability; although it should be realized that both are relative, need to the type of construction and the initiative of the designer, and ability to the commitment of the concrete producer and quality of the locally available materials (Albinger, 1988). Defining high-strength concrete by a specific strength value in essence establishes an arbitrarily selected line of demarcation that the author believes is neither practical nor warranted. The author's principal concern with arbitrarily chosen values defining high strength is that concrete routinely produced in one market might be considered a major achievement in another. For example, during a jobsite meeting to discuss several marginally low 28-day tests that had occurred with a 40 MPa (6000 psi) concrete being used in the construction of a new library in a small community, the author mistakenly referred to the mix (the highest-strength concrete ever attempted by the supplier) as "high-strength concrete." The second time the term was used, the project engineer interrupted to explain to the attendees that ACI had recently changed the definition of high-strength concrete from 41 to 55 MPa (6000 to 8000 psi), and, therefore, the mixture being discussed should not be called "high-strength" concrete. Recognizing that the project engineer was quite correct, the author then proceeded to refer to the mixture as "higher-strength concrete." The term "higher strength" was unprotested and the meeting proceeded. This example was presented merely to demonstrate how easily terminology could divert attention from the things that are *truly* important.

Definitions notwithstanding, for building codes, it should be noted that establishing strength limitations for identifying provisional or newly adopted changes to design details is based on measured material properties and not organizational consensus.

The definition of high-strength concrete is by no means static. Where high-strength concrete has been defined in terms of a precise numerical value, its definition has changed over the years. In the 1984 version of ACI Committee Report 363R–92,[4] 41 MPa (6,000 psi) was selected as a lower limit for high-strength concrete. According to that report, although this value was selected as the lower limit, it was not intended to imply that any drastic change in material properties or production techniques occurs at this level of compressive strength. In reality, all of the gradual changes that take place represent a process that starts with very modest strength levels and continues well into the realm of ultra high-strength concrete. In the course of revising the 1992 version of the *State-of-the-Art of High-Strength Concrete* report, Committee 363 defined high-strength concrete as having a specified compressive strength

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for design of 55 MPa (8000) psi, or greater. Committee 363 also recognized that the definition of high-strength concrete varies on a geographical basis. The Committee recognized that material selection, concrete mix proportioning, batching, mixing, transporting, placing, curing, and quality control procedures are applicable across a wide range of concrete strengths. However, Committee 363 also agreed that material properties and structural design considerations addressed in the report should be concerned with concretes having the highest compressive strengths.

In spite of the author's strong belief that high-strength concrete need not be defined by a hard numerical value, in consideration of the title of this book, it would be reasonable to provide the reader with at least a range of compressive strength and designated acceptance ages that would be considered by most authorities to be the threshold of "high strength." So, with that said, in most industrialized countries producers and users generally consider concrete to be "high strength" when the specified compressive strength of the material is in the range of 40 to 55 MPa (6000 to 8000 psi) at acceptance ages at 28 days or later. Of course, if the industry's rate of advancement in materials technology continues, it might not be long before values of this magnitude become obsolete.

Conventional-strength concrete

Most authors writing about high-strength concrete usually spend an inordinate amount of time mulling over the question "what is the best term to describe that which is not high-strength concrete?" This seems to be one of those questions that truly has no "best" answer, and as a result it is perpetually raised with each new work. Usually the first and most logical choice that comes to mind, the antonym of *high* is the one ruled out the quickest. Referring to non-high-strength concrete as *low-strength concrete*, though technically correct is grammatically appalling. *Low-strength* is a term frequently reserved in the industry to denote failure, a deficiency. Following this, in relatively short order come the other choices: *lower strength*, *normal strength*, and *conventional strength*. Other terms briefly considered by the author in past works have included *regular strength* and *traditional strength*, though, needless to say, these were ruled out almost as quickly as *low strength*.

Perhaps the principal reason why so much time is spent deliberating this particular term is in the hope that readers do not come away believing that high-strength concrete is some sort of exotic or obscure material, which terms such as "normal" could tend to convey. Commercially available high-strength concrete is not new, and is neither exotic nor obscure. High-strength concrete technology has been continually evolving for decades, and it has an extensive record of accomplishment with respect to both its mechanical and durability-enhancing properties. Though high-strength concrete may never come close to the volume of conventional-strength concrete produced,

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the author believes that significantly more structures would benefit, both economically and technically, if it were not perceived as something of an exotic or obscure nature.

After careful and lengthy consideration, the term *conventional strength* has been adopted to describe the type of concrete most commonly specified for civil and structural applications.

High-performance concrete

Provided all performance requirements have been identified and satisfactorily addressed, high-strength concrete (HSC) can be categorized under the much broader term *high-performance concrete* (HPC). Whether identified as HSC or HPC, there are two requirements both must satisfy. They both must be constructible and durable. Just because concrete is strong is no guarantee that it will be durable. For this reason, high-strength concrete should not summarily be thought of as being high-performance concrete.

Very often, the terms *high strength* and *high performance* are used interchangeably, which can make differentiating HSC and HPC a bit confusing. So what are the differences? Why are these terms frequently used interchangeably? Perhaps the source of the confusion is that, in principle, high strength is not a prerequisite for high performance; however, in practice, it is common for strength to increase when steps are taken to improve most durability-related properties. When steps are taken to inhibit the ingress of injurious substances through reduced permeability, concrete strength increases; however, reducing permeability alone will not ensure favorable durability. This book will frequently stress the importance of identifying and addressing all necessary properties prior to selecting materials and mixture proportions. *It is critically important that the preceding statements be thoroughly comprehended and put into practice*. Concrete that has high strength, yet is not engineered to satisfy all necessary durability requirements, should be unworthy of the title "high-performance concrete."

There have been numerous definitions developed for HPC throughout the world. Each one has validity, but slightly different meaning (Russell, 1999). European and UK standards for concrete define HPC as concrete that 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, toughness, volume stability, or service life in severe environments. The term *high-performance* could be attached to any type of concrete that exhibits fresh or hardened properties exceeding those of conventional concrete. In addition to high-strength concrete, other examples of high-performance concrete could include:

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- flowing concrete;
- self consolidating concrete (SCC);
- lightweight concrete;
- heavyweight concrete;
- pervious (no-fines concrete);
- low permeability concrete; and
- shrinkage compensating concrete.

ACI provides the following definition and commentary:

> **Definition:**
>
> **High-performance concrete:** Concrete meeting special combinations of performance and uniformity requirements that cannot always be achieved routinely using conventional constituents and normal mixing, placing, and curing practices.
>
> **Commentary:**
> A high-performance concrete is a concrete in which certain characteristics are developed for a particular application and environment.
>
> Examples of characteristics that may be considered critical for an application are
>
> - Ease of placement
> - Compaction without segregation
> - Early age strength
> - Long-term mechanical properties
> - Permeability
> - Density
> - Heat of hydration
> - Toughness
> - Volume stability
> - Long life in severe environments
>
> Because many characteristics of high-performance concrete are interrelated, a change in one usually results in changes in one or more of the other characteristics. Consequently, if several characteristics have to be taken into account in producing a concrete for the intended application, each of these characteristics must be clearly specified in the contract documents.
>
> (Russell, 1999)

Paul Zia, Distinguished University Professor Emeritus and former Chair of ACI Committee 363, made the following distinction during a private conversation:

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> High-strength concrete and high-performance concrete are not interchangeable terms. High-performance concrete embodies many more attributes than high strength. It 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 placement and compaction without segregation, long-term mechanical properties, early-age strength, toughness, volume stability, or service life in severe environments. Thus it is possible that a high-performance concrete could have a relatively low strength while satisfying other requirements.
>
> (Russell, 1999)

An example of an application of HPC where higher strength was neither needed, nor was a consequence of material selection or proportioning, was for a very low-density structural concrete used for the rehabilitation of a historic building in Chicago (Caldarone and Burg, 2004). A low-density structural concrete was specified for the new roof of one of only a handful of structures to survive the great Chicago fire in 1871. Constructed in 1869, the original roof concrete consisted of highly porous, low-density concrete produced using cinder aggregate and natural cement.[5] The replacement concrete was specified to attain an equilibrium density of 1120 kg/m^3 (70 pcf) and satisfy a specified compressive strength requirement of 20.7 MPa (3000 psi) at 28 days.

Perhaps the most important reason why the terms *high strength* and *high performance* are commonly used interchangeably is that permeability, generally considered the most important property influencing durability, goes hand-in-hand with strength. Both the coefficient of permeability and compressive strength are proportionally related to the W/B ratio. Decreases in permeability consequentially result in increases in strength.

There is perhaps another, more important, reason why high strength should not be considered a prerequisite for high performance. The concrete industry has traditionally used strength as a surrogate for durability. Compared to durability, strength is a much easier property to measure. It is true that in some instances durability correlates well with strength, particularly in cases where the durability property under consideration is proportional to the coefficient of permeability. In such cases, measures required for enhancing durability also result in higher strength. However, in other cases, the opposite holds true; measures taken to produce high strength can be detrimental to durability. For example, the durability of concrete subjected to cycles of freezing and thawing while saturated or in the presence of deicing agents is much more dependent upon the quality of an entrained air-void system than it is on strength. In this example, measures taken to improve air-void system quality could result in decreased strength (Detwiler and Taylor, 2005). Rather than continuing on associating strength, a pure mechanical property, with durability, in the author's view,

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it would be far more meaningful and beneficial to the industry to recognize that permeability, not strength, is the true property directly linking concrete's mechanical and durability properties. Strong concrete is *not* necessarily durable concrete.

Principles of proportioning

The term "principles of proportioning" is used frequently in this book. A primary facet of high-strength concrete technology is that the empirical relationships best suited for determining the quantities of each constituent material is quite different than for conventional-strength concrete. The objectives of the proportioning process remain unchanged; however, the paths, or "principles" required to satisfy those objectives are often very different with high-strength concrete. For example, the size and quantity of coarse aggregate necessary to achieve optimum strength performance at a given age depends on the target strength under consideration. Common objectives include satisfying requirements for strength, durability consistency (slump or slump spread), pumpability, workability, or setting time. Less common, but equally important objectives, if necessary, might involve satisfying requirements for modulus of elasticity, creep, heat of hydration, or shrinkage.

Historical background

In the last 40 years, the compressive strength of commercially produced concrete has approximately tripled, from 35 MPa (5000 psi) to 95 MPa (14,000 psi). This unprecedented escalation in strength was largely made possible because of the following factors:

- advancements in chemical admixture technology;
- increased availability of mineral admixtures (supplementary cementing materials); and
- increased knowledge of the principles governing higher-strength concretes.

In the 1950s, ready-mixed concrete with a design strength of 35 MPa (5000 psi) was considered high strength. The history of true, commercially available high-strength concrete in the US can be traced back to the early 1960s. In 1960, the Washington State Highway Department specified 41 MPa (6000 psi) concrete for prestressed girders, allowing the Highway Department's girders to be among the thinnest in the country. In 1961, Seattle's monorail track girder was specified with 48 MPa (7000 psi), while at the same time 55 MPa (8000 psi) concrete was being specified by the Port of Seattle for use in precast concrete piles (Howard and Leatham, 1989).

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The increased use of chemical and mineral admixtures in the decade of the 1960s quickly led to significant increases in attainable compressive strength. Place Victoria in Montreal, constructed in 1964, reached a height of 190 m (624 ft) using 40 MPa (6000 psi) concrete in the columns (Shaeffer, 1992).

Chicago played a significant role in the early development and evolution of commercially available high-strength ready-mixed concrete. From the early 1960s continuing through the late 1980s, Chicago was a place where progressive design concepts and new material technologies successfully came together. The founders of MSC realized early on that the market of the future was in the development of the Chicago inner city. It became obvious that the Chicago market was sophisticated and required a commitment to quality. In the mid to late 1950s, MSC made such a commitment. They formed a quality control department and even went so far as to hire structural engineers who could communicate with the design community. It was in 1961 that William Schmidt, structural engineer and pioneer in the use of high-strength concrete approached MSC to increase the design strength of normal weight concrete from 35 MPa (5000 psi) to 41 MPa (6000 psi) for the new 40-story Outer Drive East Condominium Project. The request was driven by the project developer's interest in increasing the amount of rentable floor space with higher-strength concrete. So in 1962, concrete having a design compressive strength of 41 MPa (6000 psi) was successfully supplied to the Outer Drive East project. In 1972, the first 52 MPa (7500 psi) was produced for the 52-story Mid-Continental Plaza. In 1974, 62 MPa (9000 psi) concrete was supplied to Water Tower Place, at 74-stories, the world's tallest concrete building at the time. Twenty-five years after the completion of Outer Drive East, commercially available 95 MPa (14,000 psi) was being routinely supplied to numerous projects in Chicago, including the 225 West Wacker building project (Figure 1.1).

Three high-strength concrete bridges, representing the first generation use of high-strength concrete bridges were built for Japan National Railway in 1973. The reasons for selecting high-strength concrete included reductions in deadload, deflections, vibration, and noise, along with an anticipated reduction in long-term maintenance costs. After over 20 years of service, the bridges have performed in accordance with all expectations (CEB-FIP, 1994).

By the late 1980s, very high-strength concrete was being successfully produced in other parts of North America. One of the highest-strength concretes used in any large-scale commercial application thus far has been concrete attaining a target compressive strength of 130 MPa (19,000 psi) in the 58-story, 220 m (720 ft) tall Two Union Square in Seattle (Figure 1.2). The compressive strength originally specified for the structure was 97 MPa (14,000 psi) at 28 days; however, the designer also desired a static modulus of elasticity of 50 GPa (7.2×10^6 psi). Testing demonstrated that an elastic modulus of this scale required concrete with a target compressive strength

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Figure 1.1 Office building at 225 West Wacker Drive, Chicago: constructed with 96 and 117 MPa (14,000 and 17,000 psi) concrete. Courtesy of Portland Cement Association.

on an order of magnitude of 131 MPa (19,000 psi). Results of compressive strength and elastic modulus tests conducted at an age of 4 years were 137 MPa (19,900 psi) and 5.6 GPa (8.1×10^6 psi), respectively (Russell, 1993). Today, high-strength concrete is increasingly becoming a key component in large-scale construction projects, from tall commercial and residential buildings to bridges and tunnels.

In many major metropolitan areas worldwide, 95 MPa (14,000 psi) at 56 days is routinely available. Although the potential certainly exists to achieve similar levels of strength performance by 28 days or less, as will be discussed in Chapters 3 and 5, there are distinct benefits that can be realized when specifying acceptance ages at 56 days or later for high-strength concrete, instead of the long ago, arbitrarily selected age of 28 days.

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Figure 1.2 Two Union Square, Seattle. Courtesy of Portland Cement Association.

Applications

Exceptional benefits, both technical and economical, have been derived using high-strength concrete. Because of these benefits, high-strength concrete is now being regularly used in many applications, including buildings, offshore structures, bridge elements, overlays, and pavements.

High-strength concrete is often used in structures not because of its strength, but because of other engineering properties that come with higher strength, such as increased static modulus of elasticity (stiffness), decreased permeability to injurious materials, or high abrasion resistance.

In bridge structures, high-strength concrete is used to achieve one or a combination of the following mechanical attributes:

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- increase span length;
- increase girder spacing; and
- decrease section depth.

The decreased permeability of high-strength concrete presents opportunities for improving durability and increasing service life. Since 1989, most concrete bridges and highway structures in Norway have been constructed with concretes having a water-binder ratio below 0.40 in conjunction with the use of silica fume to produce very low permeability concrete with improved corrosion resistance (CEB-FIP, 1994). Sandhornoya Bridge in Norway was built in 1989 with lightweight high-strength concrete of 55 MPa (8000 psi). The use of lightweight high-strength concrete provided the advantages of reduced weight and increased strength (Zia *et al*., 1997).

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). A middle span, measuring 61 m (200 ft) 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 (8000 psi). However, the mean strength obtained in the field was 69 MPa (10,000 psi) for the normal weight concrete and 73 MPa (10,600 psi) for the lightweight concrete (CEB-FIP, 1990).

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 (10,900 psi) with a W/B ratio of 0.29 and an air content of 5.0 to 7.5 percent. By using high-strength concrete, 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 (Zia *et al*., 1997).

In the US, the Louetta Road Overpass, which includes two adjacent bridges on State Highway 249 in Houston, Texas, is a showcase project demonstrating the use of high-strength concrete in bridge applications. The structures are the first bridges in the US where high-strength concrete was used exclusively throughout the structure. The structures used pre-tensioned concrete U-beams (Figure 1.3) as an economical and aesthetic alternative to the standard I-beams. Specified compressive strengths ranged from 69 to 90 MPa (10,000 to 13,000 psi) at 56 days (Ralls *et al*., 1993; Ralls and Carrasquillo, 1994).

There are many documented cases of high-strength concrete being used for highway pavements in Norway and Sweden, not for its strength properties, but rather for improved abrasion resistance (Gjorv *et al*., 1990; CEB-FIP, 1994).

In buildings, high-strength concrete presents opportunities for reduced column sizes, resulting in lower volumes of concrete and large reductions in dead loads (Perenchio, 1973). In parking structures, high-strength concrete is additionally used to minimize chloride penetration. Although the cost

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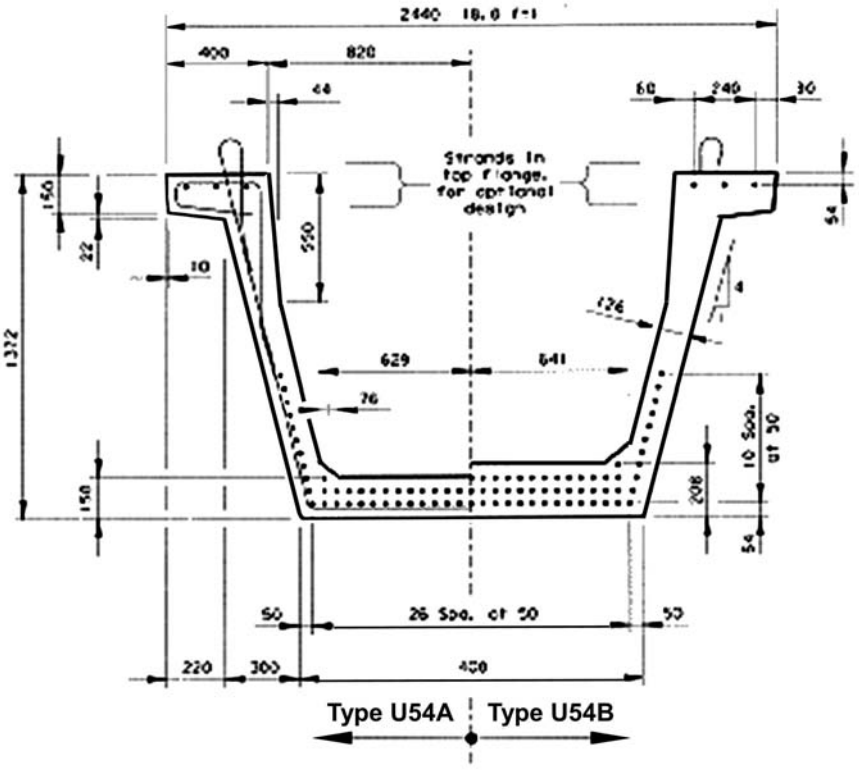

Figure 1.3 Cross section and prestressed strand patterns Texas U54 beams (after Ralls and Carrasquillo, 1994).

per unit volume of high-strength concrete is likely to be greater than that of conventional-strength concrete, given the mechanical advantages of high-strength concrete, the total initial cost of building an engineered structure incorporating high-strength concrete can be less.

In tall buildings, as the elastic modulus of vertical load bearing elements such as columns and shear walls increases, rotational periods decrease. The reduced rotational periods of vibration with stiffer columns and shear walls can be beneficial when considering the occupancy comfort factor of slender buildings. Bridges and parking structures benefit exceptionally well from high-strength, low-permeability concrete. Figure 1.4 shows a 15-story cast-in-place parking structure built with high-strength concrete having a specified compressive strength of 69 MPa (10,000 psi) at 56 days.

At over 150 stories,[6] and utilizing concrete with a specified compressive strength as high as 80 MPa (11,600 psi), the "super skyscraper" Burj Dubai (Figure 1.5) in Dubai will be the world's tallest building. Figure 1.6 illustrates

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Figure 1.4
Parking structure at 900 N. Michigan Ave, Chicago. High-strength concrete helped to reduce column sizes in this 15-story structure. Courtesy of Portland Cement Association.

Figure 1.5
At this point in its construction, the super skyscraper Burj Dubai had already attained the title "world's tallest building." Courtesy of Samsung Engineering and Construction.

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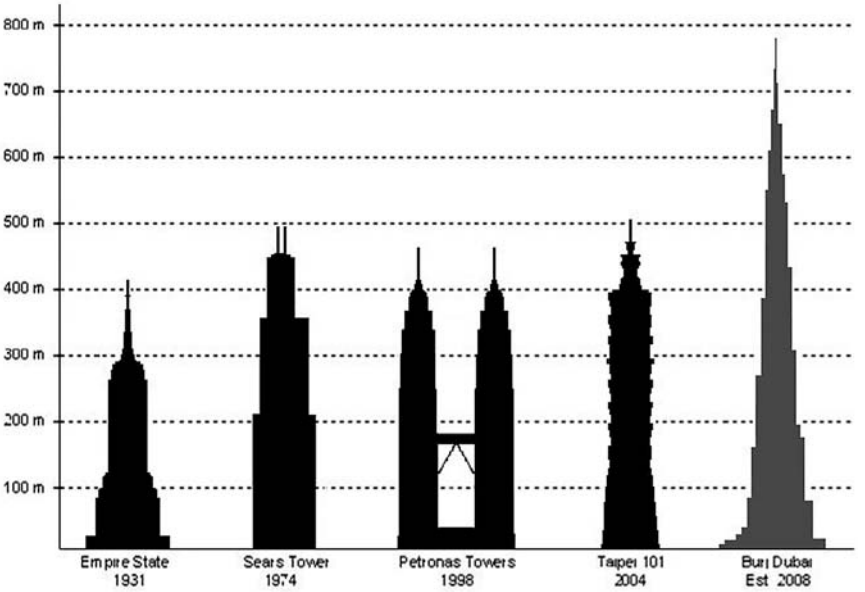

Figure 1.6 With the aid of high-strength concrete, Burj Dubai represents an extraordinary increase in attainable building height. When determining building height, spires are included whereas antennas are not.

the breakthrough height of Burj Dubai in relation to other buildings that have held the title of "world's tallest." Architecturally, it would not have been practical to construct this all-concrete frame building without incorporating high-strength concrete. Concrete building frames, particularly those incorporating high-strength concrete instead of structural steel, a cost-prohibitive material, significantly improves the economic feasibility for constructing tall buildings. Construction of Burj Dubai is scheduled for completion in 2008.

Notes

1 Cylindrical specimens with 2:1 l/d most commonly used for determining compressive strength in the US and continental Europe.
2 For the purpose of this book, "structures" will refer to any application where structural concrete is used, including pavements and plain concrete members.
3 *American Concrete Institute Building Code Requirements for Structural Concrete*.
4 *State-of-the-Art-Report on High-Strength Concrete*.
5 See Glossary for definition of natural cement.
6 During its construction, the completed height of Burj Dubai was undisclosed.

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2 Constituent materials

Introduction

This chapter describes the constituent or "raw" materials used for producing high-strength concrete. It was for the most part written based on the presumption that the reader is already knowledgeable about the basic properties of concrete-making materials. This undertaking has already been successfully accomplished in several comprehensive publications (Neville, 1996; Kosmatka *et al*., 2002; Mindess *et al*., 2003), and addressing it in appropriate depth is beyond the scope of this book. Most of the discussion in this chapter will be devoted to the industry's most commonly used "mainstream" materials. Conceptually, the concreting materials described in this chapter, when appropriately proportioned and combined, have been capable of producing high-strength concrete with long-term compressive strength on the order of approximately 140 MPa (20,000 psi), or even slightly higher.

The selection of suitable cementitious materials for concrete structures depend on the type of structure, the characteristics of the aggregates, material availability, and method of construction. The varieties of high-strength concretes discussed in this book do not require exotic materials or special manufacturing processes, but will require materials with more specific properties than conventional concretes. As the target strength of concrete increases, it becomes increasingly less forgiving to variability, both material and testing-related. Compared to conventional concrete, variations in material characteristics, production, handling, and testing will have a more pronounced effect with high-strength concrete. Therefore, as target strengths increase, the significance of control practices intensifies. It is often possible to produce conventional concretes of suitable quality using marginal quality constituents (provided they are of a generally consistent nature). This is not the case with high-strength concrete. Regardless of how consistent they are, marginal quality materials have no place with high-strength concrete.

Cementitious materials

Concrete performance is largely dependent upon the properties of the cementitious materials, particularly the chemical properties. Understanding

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the complex manner in which cementitious materials interact requires career dedication. Producers of high-strength concrete do not have to become experts, but they should at least appreciate that the cementitious materials chosen are *supremely* important and be knowledgeable with respect to the characteristics to look for. Given the complexity, cement hydration is best thought of as a process that takes place in a "black box." A producer's time would be best spent evaluating what should go into the box in anticipation of what should come out. Trying to understand the mechanics of what actually happens inside the box can lead to confusion or misunderstanding, and is best left in the hands of the cement chemists.

The bulk specific gravities of Portland cement and the supplementary cementitious materials discussed in this chapter are listed in Table 2.1.

Portland and blended-hydraulic cements

Portland cement (Figure 2.1) is indisputably the most widely used binding material in the manufacture of hydraulic-cement concrete. Selecting Portland cements having the chemical and physical properties suitable for use in high-strength concrete is one of the most important, but frequently underestimated considerations in the process of selecting appropriate materials for high-strength concrete. Cements should be selected based on careful consideration of all performance requirements, not just strength. To avoid interaction-related problems, the compatibility of the cement with chemical admixtures and other cementing materials should be confirmed.[1] Concrete producers experienced in making high-strength concrete know firsthand how critically important cement selection can be, and those inexperienced can learn in very hard, expensive ways. In the end, the benefits of the time and resources devoted to material verification testing will considerably outweigh the cost.

The performance of cement can vary widely when attempting to make high-strength concrete. Selecting appropriate cementing materials is the most important first step in the successful manufacture of high-strength concrete. This section will review basic principles about Portland cement—how it is produced, the various ways in which its properties can be altered,

Table 2.1 Bulk specific gravity of cementitious materials

Material	*Bulk specific gravity*
Portland cement	3.15
Fly ash: low calcium (Class F)	2.30 to 2.60
Fly ash: high calcium (Class C)	2.65 to 2.75
Slag cement	2.80 to 2.90
Silica fume	2.20 to 2.25
Metakaolin	2.70 to 2.75

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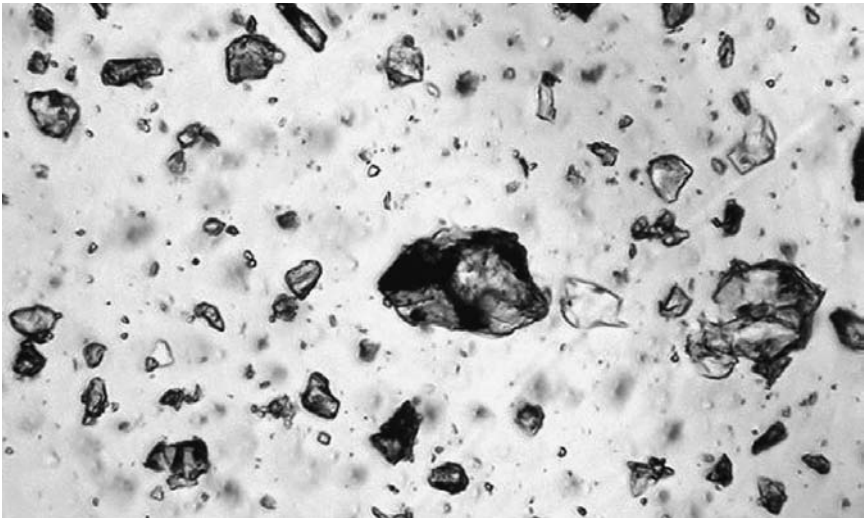

Figure 2.1 Micrograph of Type I Portland cement. Field of view is 400 μm wide. Courtesy of Portland Cement Association.

and the significance of its properties as they relate to concrete performance, particularly strength.

Portland cement is produced by heating sources of lime, iron, silica, and alumina, ground and blended into a raw meal, or "mix design," to a temperature of 1400–1550°C (2500–2800°F) in a rotating kiln, whereupon the raw materials are chemically transformed. The cooled granular product, a complex multiphase clinker (Figure 2.2) consisting primarily of a number of calcium silicate and aluminate compounds, is interground with small amounts of calcium sulfate to a powder of sufficiently large surface area (Hall, 1976).

The chemical composition of Portland cement is traditionally written in an oxide notation used in ceramic chemistry. In this "shorthand" style of notation, each oxide is abbreviated to a single capital letter. A list of the abbreviations used in cement chemistry and the primary compounds formed upon clinkering is shown in Tables 2.2 and 2.3, respectively. Compositionally, Portland cement clinker consists of a mixture of two crystalline calcium-silicate phases, C_3S and C_2S, residing in an interstitial, or "melt" phase composed of C_3A and C_4AF. As Figure 2.3 illustrates, each compound has its own unique hydration reactivity. Major process-related factors that contribute to the characteristics of Portland cement include: burning temperatures, duration of burning, oxygen availability, duration of cooling, and grinding temperatures. These same factors strongly influence the formation of impurities, such as alkali sulfate, periclase, and dead-burnt lime

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Figure 2.2 Portland cement clinker. Courtesy of Portland Cement Association.

Table 2.2 Abbreviated notations used in cement chemistry

	Chemical formula	*Notation*
Lime	CaO	C
Silica	SiO_2	S
Alumina	Al_2O_3	A
Iron	Fe_2O_3	F
Titanium	TiO_2	T
Magnesia	MgO	M
Potassium	K_2O	K
Sodium	Na_2O	N
Sulfur	SO_3	$\bar{S}$
Water	H_2O	H

Table 2.3 Primary compounds in Portland cement clinker

	Chemical composition	*Abbreviated notation*
Tricalcium silicate	$3\ CaO \cdot SiO_2$	C_3S
Dicalcium silicate	$2\ CaO \cdot SiO_2$	C_2S
Tricalcium aluminate	$3\ CaO \cdot Al_2O_3$	C_3A
Tetracalcium alumino ferrite	$4\ CaO \cdot Al_2O_3 \cdot Fe_2O_3$	C_4AF

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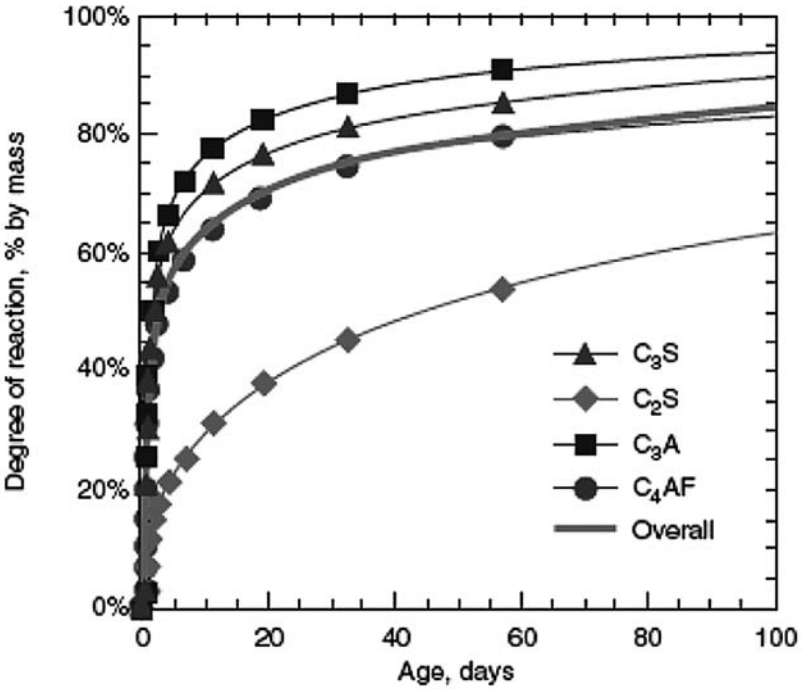

Figure 2.3 Relative reactivity of cement compounds. The curve labeled "overall" has a composition of 55 percent C_3S, 18 percent C_2S, 10 percent C_3A, and 8 percent C_4AF, an average Type I cement composition (after Tennis and Jennings, 2000).

affecting both strength and volume stability. The four primary cement compounds have the following properties:

tricalcium silicate (C_3S): hydrates and hardens rapidly and is largely responsible for initial set and early strength. In general, the early strength of Portland cement concrete is higher with increased percentages of C_3S.

dicalcium silicate (C_2S): hydrates and hardens slowly and contributes largely to strength increase at ages beyond one week.

tricalcium aluminate (C_3A): liberates a large amount of heat during the first few days of hydration and hardening. It also contributes slightly to early strength development. Cements with low percentages of C_3A are more resistant to soils and waters containing sulfates.

tetracalcium aluminoferrite (C_4AF): is the product resulting from the use of iron and aluminum raw materials to reduce the clinkering temperature during cement manufacture. It contributes little to strength.

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> Most color effects that make cement gray are due to C_4AF and its hydrates.
>
> (Kosmatka *et al.*, 2002)

In the finish mill, Portland cement clinker is usually interground with about 2 to 4 percent by mass calcium sulfate. Calcium sulfate is introduced in the form or gypsum, hemihydrate, or anhydrite (Table 2.4). It is primarily introduced in order to control the extremely rapid hydration of C_3A by forming ettringite (calcium trisulfoaluminate). Hemihydrate (plaster) has a much greater solubility rate than gypsum. Excessive amounts of hemihydrate can lead to false set, a recoverable form of severe early stiffening; however, a little hemihydrate is good to have in cement because some SO_3 needs to go into solution quickly in order to control the rapid hydration of C_3A.

In addition to controlling setting and early strength gain, sulfate also helps control drying shrinkage and can influence later age strength (Lerch, 1946). The reactivity of C_3A varies by cement source. The amount and mineral phase of the sulfate can significantly affect the way the cement interacts with supplementary cementitious materials and chemical admixtures, particularly high-strength concrete. As ettringite formation increases, porosity increases. The optimum quantity of SO_3 will occur at *minimum* paste porosity. At later ages, more C_3S hydrates and more space is needed; therefore, the need for SO_3 increases in order to achieve minimum porosity (this is why optimum SO_3 is higher at later ages). Note that the fineness of the aggregate particles has a significant influence on system porosity, which substantially influences optimum SO_3. Therefore, the optimum SO_3 content in a grout made with very fine sand would be less than the optimum SO_3 in concrete. The optimum sulfate content of modern Portland cement is determined at very early ages, usually as early as 24 hours. Unless high-early strength performance is necessary, concretes containing supplementary cementitious materials and chemical admixtures are at a higher risk of becoming under-sulfated. Early stiffening, excessive retardation, and unusual strength development can result with under-sulfated pastes. Calorimeter and mini-slump tests performed on mixture-representative paste samples can be very useful in identifying potential material incompatibilities. Pastes should be prepared representing the sequence in which additives are to be introduced and in the range of concrete temperatures anticipated for the work.

Table 2.4 Various forms of calcium sulfate ($CaSO_4$)

	Chemical composition
Gypsum (calcium sulfate dihydrate)	$CaSO_4 \cdot 2H_2O$
Hemihydrate (calcium sulfate hemihydrate)	$CaSO_4 \cdot \frac{1}{2}H_2O$
Anhydrite (anhydrous calcium sulfate)	$CaSO_4$

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Unfortunately, there is currently no worldwide standardization system for classifying hydraulic cement. Given the different ways in which cement is classified throughout the world, it is not possible to do an "apples to apples" comparison. Portland cement specifications in Canada and the US are structured similarly (Table 2.5). European "common" cements (Table 2.6) include Portland and blended hydraulic cements. Compositionally, European Class CEM I cement would be similar to cements specified under ASTM C 150.[2] Classes CEM II through CEM V would be similar to those cements specified under ASTM C 595.[3]

Identifying "high-strength cement"

The information presented in this section is primarily meant to educate the reader about important principles to consider when selecting cement for use in high-strength concrete. Because cement performance depends on numerous factors, the principles herein discussed should not be considered as absolute, but rather, general in nature.

Almost any modern Portland cement meeting the compositional requirements of ASTM C 150 can be used to obtain concrete with satisfactory

Table 2.5 American (ASTM) and Canadian Standards Institute (CSA) Portland cement classification

ASTM C 150	*CSA A5*	*Description*
I	10	Normal
II	20	Moderate sulfate resistance
III	30	High early strength
IV	40	Low heat of hydration
V	50	High sulfate resistence

Table 2.6 European (EN) "common cements"

Designation	*Description*
CEM I Portland cement	Portland cement. Comprising Portland cement and up to 5% of minor additional constituents
CEM II Portland-composite cement	Portland-composite. cement Portland cement and up to 35% of other single constituents
CEM III Blastfurnace cement	Blastfurnace cement. Portland cement and higher percentages of blastfurnace slag
CEM IV Pozzolanic cement	Pozzolanic cement. Portland cement and up to 55% of pozzolanic constituents
CEM V Composite cement	Composite cement. Portland cement, blastfurnace slag and pozzolana or fly ash

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workability having compressive strength up to about 60 MPa (8500 psi). Cements can vary widely in the manner in which they perform in concrete. As Figure 2.4 demonstrates, cements that perform exceptionally well in conventional-strength concrete may not necessarily perform as favorably in high-strength concrete. Conversely, the strength efficiency of some cement can increase as cement contents increase and W/B ratios decrease.

In order to obtain higher strength while maintaining good workability, it is necessary to carefully study the cement composition, fineness (i.e. particle distribution), and its compatibility with the chemical admixtures used (Mehta, 2005).

Cement manufacturers track compressive strength using 50 mm (2 in) mortar cubes made and cured in a prescribed manner using graded sand according to standardized test methods such as ASTM C 109[4] or BS EN 196 (Part 1).[5] Depending on the type of cement being produced, compressive strength is determined as early as one day, but usually no later than 28 days. Cube testing is performed for several reasons, including tracking strength uniformity, conformance to internal operational quality standards, and compliance with applicable industry standards. Cement strengths based on mortar-cube tests cannot be used to reliably predict how the cement will perform in concrete. In fact, ASTM C109–99 (Section 15) contains a precautionary statement indicating that caution must be exercised in using the results of the test method to predict the strength of concretes.

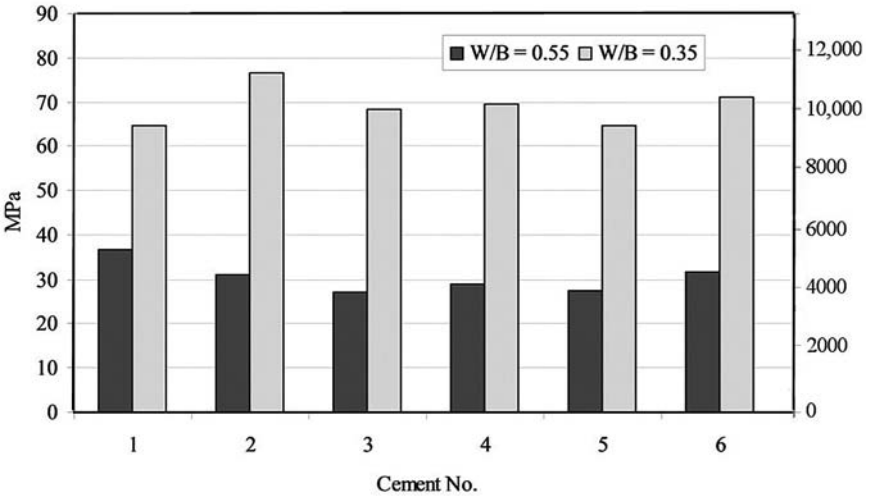

Figure 2.4 28-day compressive strength of two concretes produced at fixed water-binder ratios using six different brands of Type I cement. Note that the relative strength performance of cements that perform well in conventional-strength concrete produced at a 0.55 W/B ratio did not perform as good in high-strength concrete produced at a 0.35 W/B ratio, and vice versa. The mixtures examined contained no supplementary cementing materials or chemical admixtures.

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At best, mortar cube strengths provide a general indication of the relative strength comparing one type or brand of cement to another, relative to standard cement tests, but they should *never* serve as the sole basis for selecting cement for use in high-strength concrete. The compressive strength of such mortars employs fine aggregates and mixing equipment too dissimilar to concrete conditions, and they can be unreliable indicators as to how the cement will perform in concrete. This is especially true for high-strength concretes, which usually contain chemical admixtures and supplementary cementing materials. Certificates of compliance (i.e. mill certificates) usually contain information about the chemical composition and physical characteristics of the cement, and they can be useful documents when tracking compositional consistency; however, mill certificates alone cannot predict the performance of the concrete. In addition, the behavior of cement in concrete can be profoundly influenced by things that are usually not reported on mill certificates, such as the mineral phase or phases of sulfates and the relative reactivity of the Bogue compounds, particularly the highly volatile C_3A component. Therefore, two cements having similar chemical and physical properties based on the mill certificate reports could perform quite differently in high-strength concrete.

Blended hydraulic cements usually consist of blends of Portland cement or Portland cement clinker ground with other ingredients, such as fly ash, silica fume, slag, or natural pozzolans. By replacing Portland cement and using predominantly recycled materials, they are also more environmentally friendly. Under certain conditions, using blended hydraulic cement may be more convenient than introducing SCMs at the concrete production facility. Some of the benefits derived from blended hydraulic cements could include lower rates of heat development, slower strength gain, higher ultimate strength, lower permeability, and enhanced durability characteristics. Classification of blended hydraulic cements in ASTM C 595 is shown in Table 2.7.

Mill certificates can be useful for comparing the chemical and physical properties, but the most reliable way to determine how a cement is going

Table 2.7 ASTM C 595 classification for blended hydraulic cements

Cement type	*Description*
Type IS	Portland-Slag
Type IP	Portland-Pozzolan Cement
Type I(PM)	Pozzolan-Modified Cement
Type S	Slag
Type I(SM)	Slag-Modified Portland Cement

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to perform in concrete requires making *concrete*, especially when high performance concretes are involved. Comparing the relative performance of one type and brand of cement to another in conventional and high-strength concretes can often yield surprising results.

Hester (1989) reported that producers of high-strength concrete have found that the finer grind and higher proportions of tricalcium aluminate (C_3A) and tricalcium silicate (C_3S) phases in general purpose Type I , and even high-early strength Type III cements, make them less susceptible to overdosing of high-range water-reducers in the field and achieve excellent early and long-term strength development. The author's field experience in commercially produced high-strength concretes also supports this observation. Higher C_3A (i.e. greater than 8 percent), low alkali cements have performed very well in high-strength concrete. Note that cements are very complex materials, and basing selections on such broadly stated observations should be avoided. Numerous other factors will influence the manner in which cements will perform in concrete. For example, two cements having similar chemical composition with equal sulfate and C_3A contents can perform very differently if the sulfate is of a different mineral phase and the C_3A has markedly different reactivity. A systematic approach to aid in identifying cements suitable for use in high-strength concrete is presented in Chapter 9.

Consider "plain" concrete as only being comprised of Portland cement, water, and aggregates. As the cement content of a plain concrete of fixed plastic consistency increases, the W/B ratio decreases and strength increases. While strength continues to increase with each incremental increase in cement content, the magnitude of the strength increase gradually decreases. The actual rate of change in strength gain depends on factors related to the properties of the constituents used, particularly the chemical and physical properties of the cement; however, eventually, there reaches a point where adding more cement results in little or no strength increase (and perhaps even some strength loss). For ordinary Portland cements, the point at which no practical benefits come from increasing only the cement content typically occurs at water–cement ratios somewhere in the range of 0.35 to 0.45. The specific threshold at which little or no strength gain (by virtue of cement alone), depends on several factors, including rate of hydration and the specific properties of the cement used. However, it should be noted that prior to reaching such a point, other problems, unrelated to strength might begin to manifest, such as premature stiffening, poor finishability, and cracking at early or later ages.

Supplementary cementitious materials

Supplementary cementitious materials (SCMs) have undeniably played a significant role in the evolution of high-strength concrete. Appreciating just how exceptional these materials can benefit high-strength concrete can be

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challenging given the restricted use resulting from arbitrarily established limits in prescriptive-based specifications. SCMs are important materials that contribute to the properties of concrete when used in conjunction with Portland cement by reacting either hydraulically or pozzolanically. Pozzolans are siliceous or alumino-siliceous materials that, by themselves, possess no hydraulic (cementing) value, but will, in finely divided form and in the presence of water, chemically react with calcium hydroxide to form compounds having cementitious properties. Some pozzolans are highly reactive, whereas others are only nominally reactive. Examples are fly ashes, silica fumes, and raw or calcined natural pozzolans, which include metakaolin, volcanic ashes, calcined shales and clays, and diatomaceous earths.

Depending on the SCM used, benefits derived include higher early strength, higher later age strength, reduced permeability, control of alkali-aggregate reactivity, lower heat of hydration, and reduced costs (Russell, 2002). Fly ash (conventional and ultra-fine), ground granulated blast-furnace slag, silica fume, and metakaolin are discussed.

Fly ash and slag cement are usually the SCMs chosen first for high-strength concrete. When combined with a high-strength Portland cement, these materials have been used for economically producing binary concretes with specified compressive strengths of at least 70 MPa. (10,000 psi). For higher strengths, particularly above about 80 MPa (12,000 psi), ternary mixtures containing very fine, paste densifying pozzolans such as silica fume, metakaolin, or ultra-fine fly ash can be quite advantageous.

Fly ash and slag cement have traditionally been treated as replacements for Portland cement. Silica fume, metakaolin, and ultra-fine fly ash tend more to be treated as performance-enhancing additives that are used in addition to, rather than as replacements for Portland cement. SCMs are not Portland cement equivalents and should not be viewed as only replacement materials. Compared to Portland cement, supplemental materials are different in the ways each interacts and the unique properties they can impart. SCMs essentially contain the same minerals as Portland cement [calcium (CaO), silica (SiO_2), alumina (Al_2O_3), and iron (Fe_2O_3)], only in different proportions and mineral phases. The key to the successful use of SCMs is in understanding their capabilities and limitations in the manner in which they interact with other cementitious materials and chemical admixtures.

When properly understood, the technical and economic benefits SCMs are capable of imparting can be nothing less than remarkable. SCMs have significantly expanded the feasible realm of modern hydraulic cement concrete. Unfortunately, in most studies conducted thus far, fly ash and slag cement have been treated as replacements for Portland cement. As a result, it is only natural that they would predominantly be viewed merely as cement replacement materials. Perhaps future studies of these materials will place more emphasis on their individual merits and not treat them only as cement replacements.

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Supplementary cementitious materials alter both the fresh and hardened properties of concrete. In the fresh state, SCMs can influence rheology, setting characteristics, placeability, and finishability. Depending on the particular one used, SCMs can contribute to the properties of hardened concrete through hydraulic activity, pozzolanic activity, or both. Similar to Portland cement, hydraulic SCMs chemically react directly with water to form binding compounds. In the presence of water, pozzolans react chemically with calcium hydroxide released from hydration to form compounds that have binding properties. Fly ash, slag, and silica fume, have been the most commonly used SCMs in high-strength concrete. Metakaolin and ultra-fine fly ash (UFFA), though newer to the concrete industry, have been successfully used in commercially produced high-strength concretes. For this reason, discussions on metakaolin and UFFA have been included in this chapter.

Fundamentally, there are two systematic ways that cementing materials can be classified. In one system, materials are categorized as either being hydraulic or pozzolanic. In the second, distinctions are made between the primary cementing material and those considered "supplemental" to the system. Several considerations come into play when considering whether to incorporate just one or multiple SCMs in high-strength concrete, including target strength and age, material cost, and any other required properties. With respect to strength, it is not the absolute attainable strength per se, but rather the *efficiency* of the combined cementing materials that should govern in the material selection process. For example, if equivalent compressive strengths were attainable with two mixtures, the first being a binary system and the second being a ternary system, the more appealing of the two would be the mixture that produces the highest strength per unit cost at the designated age under consideration. Of course, the costs of all chemical admixtures incorporated must also be factored into the calculations. Analyses of this nature can be very powerful tools in the mixture selection process. Note that the age of the concrete can significantly influence the results. For example, concrete exhibiting favorable 28-day strength per unit cost efficiency perhaps might not appear as attractive at earlier ages.

When identifying fresh and hardened properties, whether or not the paste constituents are classified as hydraulic or pozzolanic is of little relevance. More emphasis should be placed on what comes out of a system (i.e. performance) rather than what goes in (i.e. prescription). What matters is the rate binder is produced and the binding capacity of the system (performance characteristics) rather than what goes in (prescriptive requirements). Portland cement has traditionally been and still remains at the heart of hydraulic cement concrete, and high-strength concrete is no exception. When making high-strength concrete, significantly better performance is achievable when incorporating SCMs. Supplementary cementitious materials are critically important materials for high-strength concrete, and they should routinely be viewed as necessary mixture constituents.

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An interesting question arises when a "supplementary cementitious material" comprises more than 50 percent of the cementitious material in concrete. For example, if slag cement comprised more than 50 percent of the total cementing material in concrete, would it still be appropriate to refer to it as a "supplemental" material? Understandably, many would view this as a tongue and cheek example; however, it was given for a reason. Terminology systems are useful, but only to a point. In the end, regardless of how things are classified, *performance* is what truly matters.

Fly ash

Fly ash (pulverized fuel ash) is the spherically shaped amorphous, glassy residue that results from the combustion of pulverized coal (Figure 2.5). It is the most commonly used pozzolan in concrete, and it has played a significant role in high-strength concrete since its very birth. Specifications for fly ash include ASTM C 618[6], BS EN 450–1[7], and CAN/CSA A23[8].

Coal is formed by the decomposition of plant matter, without free access to air, under the influence of moisture, pressure, and temperature (Vorres, 1979). Coals are ranked based on their degree of coalification. Lignite, the lowest rank of coal, is a high moisture-bearing coal that checks badly upon drying. Sub-bituminous coal is a black and crumbly coal intermediate between lignite and bituminous. Bituminous (soft) coal is the most abundant

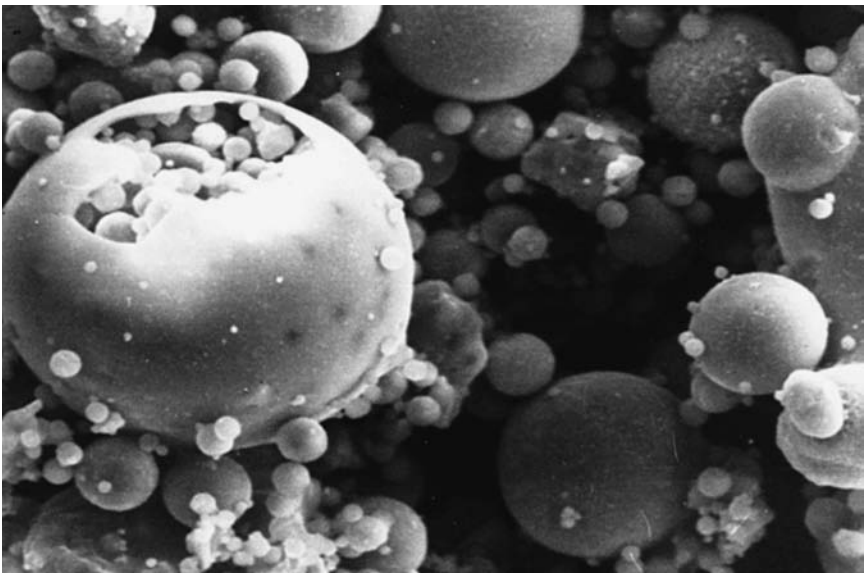

Figure 2.5 A micrograph of fly ash showing typical spherical particles. Field of view is 80 μm wide. Courtesy of Portland Cement Association.

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rank of coal. Anthracite (hard) coal is the highest rank of coal (Helmuth, 1987). Portland cement is rich in lime (CaO) while fly ash is low. Although fly ash contains much lower amounts of lime than Portland cement, the performance of fly ash in concrete is principally driven by its lime content. Low calcium fly ash is normally produced from the combustion of anthracitic or bituminous coal. High calcium fly ash is normally produced from the combustion of lignite or sub-bituminous coal. Fly ashes containing low quantities of calcium oxide (less than about 8 to 10 percent) are considerably different from fly ashes high in calcium oxide (greater than 20 percent). In hopes of minimizing any misunderstanding that could arise when discussing such dissimilar materials that happen to share the same name, the prefixes *low, intermediate,* and *high,* will be used to differentiate fly ashes containing less than 10 percent, between 10 percent and 20 percent, and more than 20 percent calcium oxide, by mass, respectively. Most fly ashes fall into either the low calcium or high calcium groups. Other than the fact that they are by-products of coal combustion, and share similar physical properties, low calcium and high calcium fly ashes have very little in common. Chemically, they are markedly different materials; therefore, it is important that their differences be considered when proportioning concrete. Using low calcium fly ash and high calcium fly ash interchangeably will unquestionably lead to significant variations in performance. Given their dissimilarity, referring to both materials as "fly ash," probably does more harm than good for the industry, particularly among less experienced users.

While there are a number of differences in the chemical composition of each class of fly ash, in general the primary difference is that low calcium fly ash has little or no hydraulic properties of its own, while high calcium fly ash does. When mixed with water, high calcium fly ash will hydrate and form calcium silicate hydrate. In conventional-strength concretes, fly ashes typically comprise 15 to 30 percent by mass of cementitious material. In high-strength concrete, higher percentages are common, particularly when using high calcium fly ash. With respect to strength, for a given set of cementitious materials, the optimum quantity of fly ash in concrete depends largely on the target strength level desired, the age at which the strength is needed, and the chemical and physical properties of the fly ash and other cementitious materials used. For example, the optimum quantity of a given fly ash needed to maximize 28-day compressive strength in a binary mixture containing 300 kg/m^3 (500 lb/yd^3) Portland cement and fly ash might be found to be 25 percent by mass of the total cementitious materials content. On the other hand, in a high-strength concrete containing 500 kg/m^3 (850 lb /yd^3) using the same materials, the optimum quantity of the same fly ash might be determined to be in the range of 40 to 50 percent of the cementitious material. Figure 2.6 illustrates the marked difference the chemical composition fly ash has in the 28-day strength performance of conventional and high-strength concrete. The optimum quantity of fly ash with respect to compressive strength performance depends largely on the properties of the cement and

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fly ash used, the quantity of fly ash used, the total cementitious materials factor, and the age of the concrete. For example, the low calcium fly ash used in the Figure 2.6 study exhibited decreased 28-day strength when comprising more than 25 percent replacement in conventional-strength concrete. On the other hand, no decrease in 28-day strength was observed using the same fly ash in a moderately high-strength concrete. By 56 days, no decrease in strength was observed up to the 40 percent maximum replacement level studied. The optimum proportions for one fly ash may be quite different than for another; therefore, laboratory trial batches should be made to establish optimum performance.

Prescriptive specifications commonly view fly ash as a replacement for Portland cement, with maximum replacements usually in the range of 15 to 20 percent by mass. For special durability needs, such as increased resistance to sulfate attack or alkali reactivity, low calcium fly ashes have comprised 30 to 40 percent of the binder content. In high-strength concrete, optimum post-28-day strengths have been achieved using fly ash contents much higher than the usual 15 to 25 percent maximum allowed by many specifications.

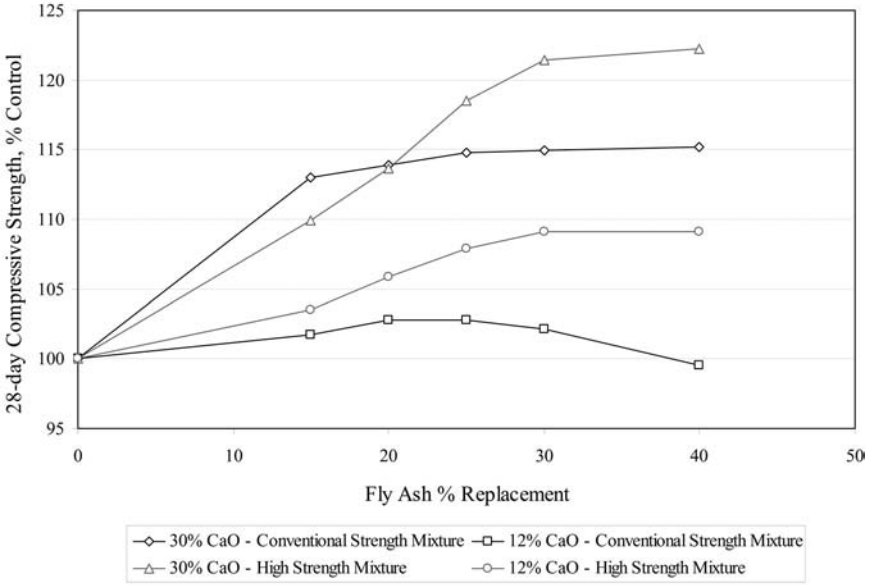

Figure 2.6 Compressive strength of concretes produced with fly ash containing 12 percent and 30 percent calcium oxide in conventional-strength and moderately high-strength concrete. The total binder content of the conventional and high-strength mixtures were 250 kg/m^3 (420 lb/yd^3) and 385 kg/m^3 (650 lb/yd^3), respectively. All batches were produced at a target slump of 125 mm (5 in).

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Figure 2.7 shows the typical time to initial set relationship for low, intermediate, and high calcium fly ash. When comprising less than 50 percent of the cementitious material, both low and high calcium fly ashes retard the setting time of the concrete, but for markedly different reasons. The slower setting time experienced when using increasingly greater percentages of low calcium fly ash is largely physical in nature and due to the dilution of cement, the more early reactive material. In the case of high calcium fly ash, because it contains both aluminates and sulfates, the retardation experienced when used in normal quantities is more chemical in nature rather than merely due to physical dilution of the cement. Interestingly, as the percentage of high calcium fly ash increases, there is a point where the setting characteristic of the paste is controlled by the highly volatile high calcium fly ash. Pastes exclusively produced with high calcium fly ash usually flash set within minutes of being mixed.

The performance of fly ash in concrete is strongly influenced by its chemical composition. In 20 to 35 MPa (3000 to 5000 psi) concrete, fly ash is commonly used at 15 to 25 percent (by mass) of cementing material, and at 30 to 40 percent or more for special applications, such as mass concreting, ASR (alkali-silica reaction) mitigation, or resistance to sulfate attack. High calcium fly ash, which has both hydraulic and pozzolanic characteristics, has been found to be highly suitable for the mechanical properties of high-strength concrete. However, high calcium fly ash may worsen rather than improve sulfate resistance (Tikalsky and Carrasquillo,

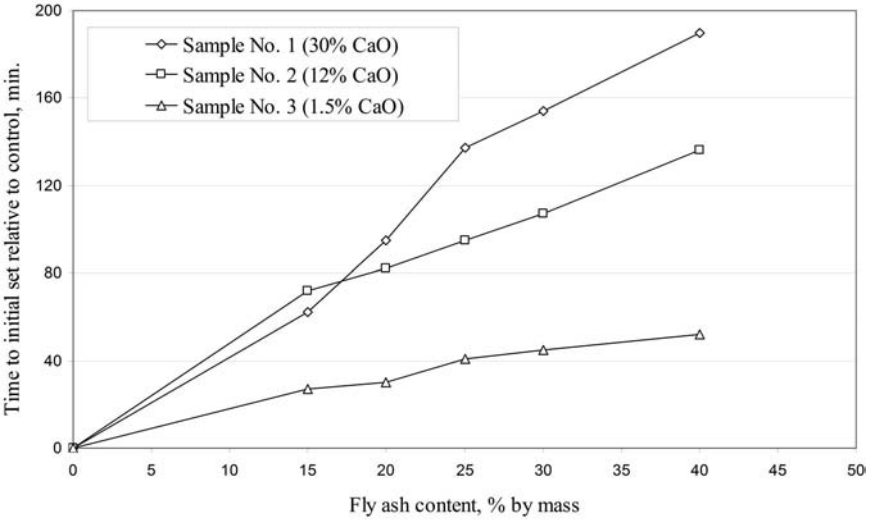

Figure 2.7 Common setting characteristics comparing low and high calcium fly ashes. Actual curves will depend on the specific fly ash used and testing conditions.

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1992) and is notably less effective than low calcium fly ash in resisting alkali-silica reactions (Malvar *et al.*, 2002). High-strength concrete mixtures up to approximately 80 MPa (12,000 psi) have been successfully produced using 30 to 40 percent fly ash by mass of cementitious material without silica fume or high-reactivity metakaolin.

The effect of fly ash content on the rapid chloride permeability of concrete is more significant in cases of wet curing for long period such as 365 days, because wet curing enables the pozzolanic reaction to proceed (Sengul *et al.*, 2005).

There are advantages and disadvantages with each type of fly ash. When used in effective quantities, there are distinct advantages with low calcium fly ash given its ability to mitigate alkali aggregate reactions and reduce concrete's vulnerability to sulfate attack better than high calcium fly ash. Low calcium fly ash, being a slow reacting pozzolan, develops early strength at a slower rate than concrete containing an equal quantity of high calcium fly ash. The slow reactivity of low calcium fly ash present challenges in applications requiring rapid strength development, such as pre-tensioned prestressed concrete, fast track construction, or post-tensioned structures. As a result, low calcium fly ash is rarely used in prestressed concrete or fast track performance applications. Because of its highly pozzolanic nature, long-term strength development is appreciably enhanced. Depending on the chemical and physical characteristics of the entire binding system, it is not unusual for mixtures proportioned with 20 percent or more low calcium fly ash to attain higher strength after at least 21 days. On the other hand, as a direct result of its high calcium oxide content, high calcium fly ash can be effectively used to achieve a higher early strength.

There is a common misconception that fly ash is unsuitable for high-early-strength concrete. This is likely to be true with respect to low-calcium fly ash, but is not necessarily true when using high-calcium fly ash. Under moderate-to-warm temperatures, the author has supplied high-early-strength concretes for post-tensioned elements containing 20 to 40 percent high-calcium fly ash exceeding the compressive strength of concretes containing 100 percent ordinary Portland cement by three days, and, by two days under hot weather conditions.

The spherical shape of fly ash particles impart lubrication to plastic concrete, enhancing workability at the same level of consistency while reducing water demand, and thus reducing the water-binder ratio. The chemical and physical properties of fly ash vary by production source. Even within a given electric generating station, each combustion unit has its own unique burning characteristics. Therefore, unless effective control measures are in place at the production source, greater shipment-to-shipment inconsistencies can occur when receiving fly ash from plants having multiple burning units. Producers of high-strength concrete should review such matters with their fly ash suppliers to ensure acceptable shipment-to-shipment consistency.

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Ultra-fine fly ash

As of yet, UFFA is not considered a mainstream material, but the highly innovative technology associated with its use justifies including it in this book. Rather than including this discussion in the fly ash section of this chapter, the differences between conventional fly ash and UFFA technology are significant enough to warrant a separate section.

The particle distribution of conventional fly ash typically consists of particles ranging from slightly greater than 150 μm to submicron size. Mehta (1985) reported that a majority of the reactive particles in fly ash are actually less than 10 μm in diameter. Typical low calcium fly ash consists of fewer than 25 percent (by volume) of particles with a particle diameter of 10 μm or less (Obla *et al*., 2003). Research by Popovics (1993) and Bouzoubaã, *et al*. (1997) showed that increasing the fineness of fly ash by grinding improves reactivity to a point but eventually leads to increased water demand.

UFFA is a highly reactive processed low to intermediate calcium fly ash designed to increase strength and reduce permeability on an order of magnitude similar to silica fume and metakaolin. By starting with a material having spherically shaped particles with the potential to reduce both water demand and high-range water-reducer demand, the objective in processing conventional fly ash into UFFA is to attain a particle size distribution optimizing workability and pozzolanic reactivity. The mean particle size of commercially produced UFFA has been approximately 3 μm (Figure 2.8), with about 90 percent of the particles smaller than 7 μm (Obla *et al*., 2003). As Figure 2.9 illustrates, UFFA is significantly smaller than conventional fly ash.

Water reduction over other highly reactive pozzolans can range from as much as 10 percent with a corresponding reduction of high range water reducer for similar plastic characteristics. These reductions also yield hardened properties that are equivalent to other high reactive pozzolans (Obla *et al*., 2000). UFFA improves durability through a reduction in permeability, an increase in the resistance to alkali attack, and an increase in sulfate resistance (Obla *et al*., 2003). Through the pozzolanic reaction, the permeability continues to decrease as the concrete continues to cure. A reduction in permeability slows the ingress of ions and other deleterious chemicals through the concrete and towards potentially reactive aggregate and reinforcing steel. Despite any reduction in permeability if the concrete is cracked, it is considered permeable to the bottom of the crack. The potential for cracking with UFFA in concrete versus other highly reactive pozzolans is reduced due to the reduction in autogenous and plastic shrinkage (Hossain *et al*., 2007). The quantity of UFFA used is determined by the desired plastic and hardened concrete characteristics. Typical dosage rates range from 9 percent to 12 percent of the total binder content.

Performance of UFFA in concrete has been demonstrated on several high-strength concrete projects in North America and Africa. Marine applications

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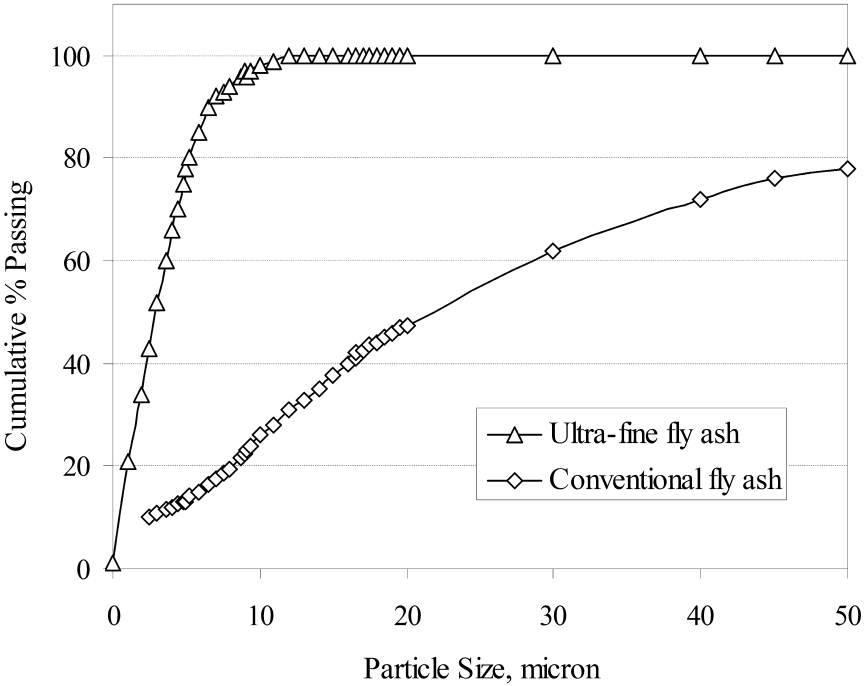

Figure 2.8 Particle size comparison of conventional and ultra-fine fly ash. Courtesy of Boral Material Technologies, Inc.

using UFFA have been used by ready mixed concrete requiring high strength, low permeability, and enhanced placeability.

Slag cement

Granulated blast-furnace is a quenched, glassy granular product that is dried and ground into an off-white powder similar in size to Portland cement. It is a slowly reacting latent hydraulic cement considered to have negligible pozzolanic activity. When ground, slag particles are highly angular (Figure 2.10). A multitude of acronyms and terms are used to describe this material, including GGBFS, *GGBF-Slag, GGBS, GGBFS, slag,* and *slag cement*. In this book, the terms *slag cement* and *GGBFS* will primarily be used. Specifications for GGBFS for use in concrete include ASTM C 989[9] and BS 6699.[10]

In binary concretes, slag cement typically comprises 30 to 50 percent of the cementitious material by mass. Slag cement is a more temperature sensitive material than Portland cement. Temperature reductions have a more pronounced effect with slag cement than with Portland cement. At low temperature, replacement of Portland cement with slag cement results in a

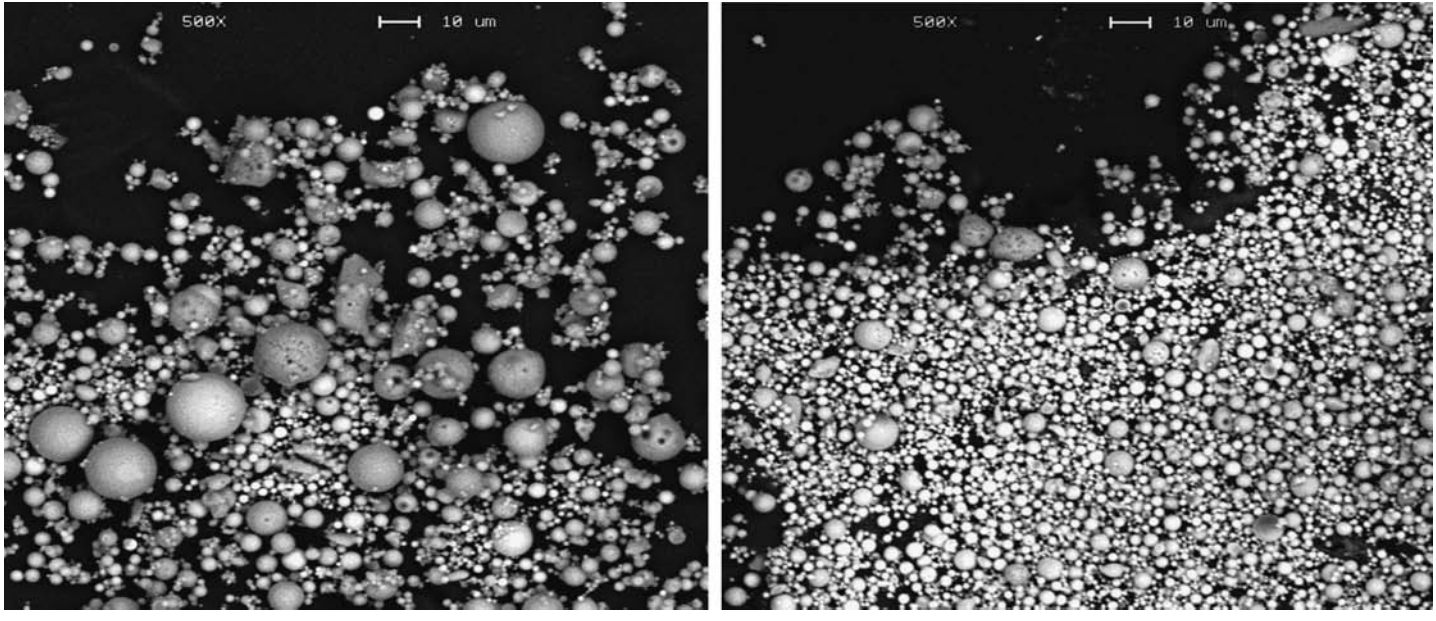

Figure 2.9 Micrographs of conventional fly ash (left) and ultra-fine fly ash (right) at 500 × magnification. Courtesy of Boral Material Technologies, Inc.

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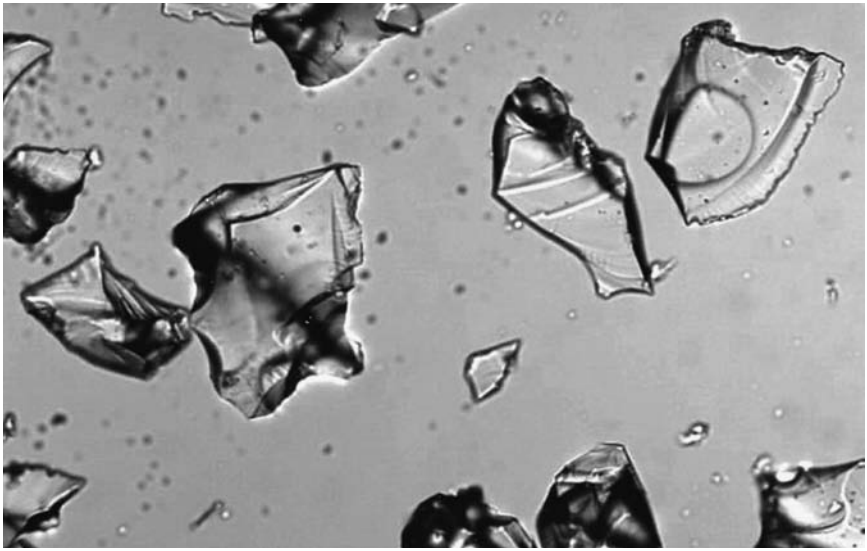

Figure 2.10 Micrograph of ground granulated blast-furnace slag grains. Field of view is 400 μm wide. Courtesy of Portland Cement Association.

substantial loss of early age strength development (Dubovoy *et al.*, 1984). These thermal characteristics can be very beneficial in large-scale elements at risk of cracking caused by high thermal gradients. In large-scale, mass elements, concretes containing slag cement contents exceeding 60 percent are increasingly being used.

Slag cement is exceptionally desirable for use in high-strength concrete. At a given W/B ratio, higher long-term compressive strength can be expected with concretes incorporating slag cement compared to Portland cement-only concretes. When readily available, slag cement warrants strong consideration as a constituent for high-strength concrete. Although generally ground finer than ordinary Portland cement, the water demand with slag cement is generally about the same as or slightly lower than Portland cement. In fresh concrete, slag cement can improve workability and pumpability. When used in effectively high amounts, slag cement can reduce the risk of damage caused by alkali-aggregate reactions (AAR), sulfate attack, and chloride-induced corrosion. In general, strength of Portland cement-slag mixtures increases with an increase in slag cement fineness.

The setting time and amount of heat liberated with pastes comprised of 100 percent slag cement is considerably slower than pastes made with 100 percent Portland cement. The performance of concretes made with combinations of slag cement and Portland cement is strongly influenced by the fineness of the slag cement and the amount of alkalis present in the Portland

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cement. In general, as the soluble alkali content of Portland cement increases, strength, in addition to other mechanical properties increases at a faster rate. Alkali-activated concretes produced only with slag cement binder have been successfully produced in Europe and Russia (Talling and Brandstetr, 1989). Slags that contain high amounts of reactive (glassy) aluminates can affect the optimum SO_3 level. ASTM C 989 limits SO_3 to 4 percent, which generally works if the reactive aluminate content of the slag cement is no greater than about 8 percent; however, some slags have reactive aluminate contents as much as 15 percent.[11] This reality may one day prompt a change to the current 4 percent SO_3 limit in ASTM C 989.

Accelerated curing increases early age strength development for slag cement mixtures. At normal temperatures, early age strength development is retarded when slag cement is used. The period of time at which the strength of a concrete containing both slag cement and Portland cement equalizes to that of a Portland cement-only concrete is a function of the chemical and physical properties of both materials (Dubovoy *et al*., 1984). The magnitude of strength gain from 7 through 28 days can be larger in mixtures incorporating slag cement than reference mixtures made with Portland cement only. For ASR control, replacement levels of 35 percent to 40 percent are generally recommended. For sulfate resistance, replacement levels of 35 percent or greater are usually recommended. For more information, consult ACI 233R-03.[12]

Note that in some parts of the world the term "slag cement" is also used to mean blended (binary) hydraulic cement containing ground granulated blast-furnace slag as a major constituent; therefore, caution should be exercised any time this term is encountered.

Silica fume

No single material has been more responsible for opening the gateway to the achievement of ultra-high strength than silica fume. Silica fume (Figure 2.11) is an ultra-fine mineral residue composed of amorphous glassy spheres of silicon dioxide (SiO_2) generated as a gas in submerged-arc electric furnaces during reduction of very pure quartz in the manufacturing of silicon and ferro-silicon alloys. Silicon alloys are available for numerous specialized applications, and as a result, different types of silica fumes are produced. Depending on the characteristics of the raw materials and process involved, the chemical and physical properties of silica fumes can vary significantly (Malhotra *et al*., 1987). Silica fume is generally dark gray to black in color. Most of the silica fumes used in concrete contain 85 to 95 percent amorphous SiO_2 in glassy spherical particles. The average particle size ranges from 0.1–0.3 μm, approximately 100 times smaller than Portland cement grains. The specific surface of silica fume ranges from 15–30 m^2/g. Silica fume is also referred to as *condensed silica fume* and *microsilica*.

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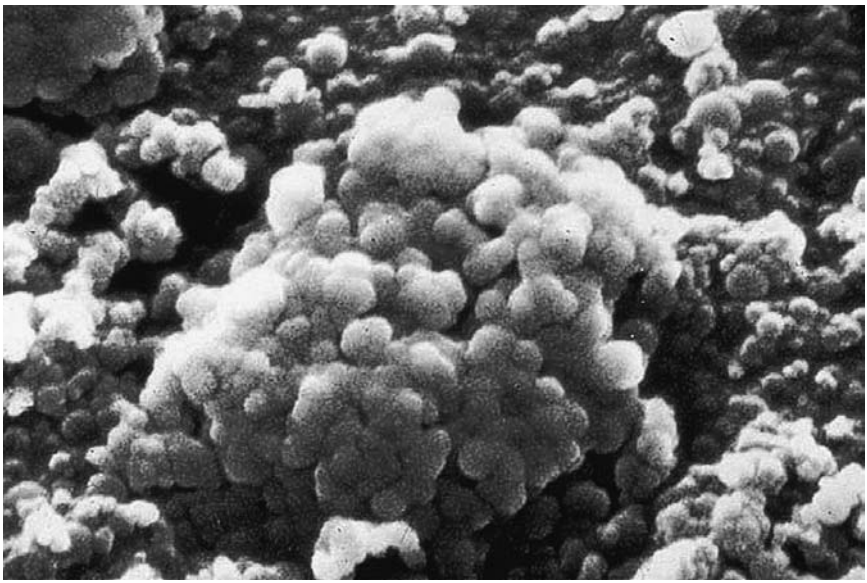

Figure 2.11 Scanning electron microscope micrograph of silica fume particles at 20,000×. Courtesy of Portland Cement Association.

Silica fume is the ultra-fine non-crystalline silica produced in electric arc furnaces as a byproduct of the production of silicon metals and ferrosilicon alloys. It is considered a purely pozzolanic material. Because of its extreme fineness, silica fume dramatically increases the water demand of the mixture. This has made the addition of high-range water reducing admixtures a requirement when silica fume is used. Unless the water demand is offset using a high-range water-reducing admixture, the increase in water necessary to produce needed workability would destroy the properties desired with silica fume. Proprietary products containing silica fume may also include carefully balanced chemical admixtures.

In high-strength concrete mixtures, silica fume is typically used at 5 to 10 percent (by mass) of cementing material. When used correctly, silica fume is an extremely effective material for achieving very high strengths and significant decreases in permeability. Because of its chemical and physical composition, silica fume is highly effective for achieving high strength at both early and later ages (Mazlom *et al.*, 2004). Silica fume is specified under ASTM C 1240[13] and BS EN 13263–2.[14]

Because of its physical nature, silica fume significantly affects the fresh properties and behavior of concrete. The tiny particles increase cohesion; retain free water and prevent segregation and bleeding. From a theoretical perspective, reduced bleeding is highly desirable since it prevents settlement

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around reinforcement bars and precludes the development of bleed channels. Because of its extremely small size, silica fume lubricates the concrete and increase pumpability thereby facilitating easier placements in heavily reinforced elements.

In cementitious compounds, silica fume works on two levels. When silica fume is added to fresh concrete it chemically reacts with the CH to produces additional CSH. The benefit of this reaction is twofold: increased compressive strength and chemical resistance. The bond between the concrete paste and the coarse aggregate, in the crucial interfacial zone, is greatly increased, resulting in compressive strengths that can exceed 105 MPa (15,000 psi). The additional CSH produced by silica fume is more resistant to attack from aggressive chemicals than the weaker CH. The second function silica fume performs in cementitious compounds is a physical one. Because silica fume is so much smaller than a Portland cement particle, it can fill the voids created by free water in the matrix. This function, called particle packing, refines the microstructure of concrete, creating a much denser pore structure. Impermeability is dramatically increased, because silica fume reduces the number and size of capillaries that would normally enable contaminants to infiltrate the concrete. Thus silica fume modified concrete is not only stronger, it lasts longer, because it is more resistant to aggressive environments. As a filler and pozzolan, silica fume's dual actions in cementitious compounds are evident throughout the entire hydration process (Malhotra *et al.*, 1987).

The contribution of silica fume to concrete strength may be expressed in terms of an efficiency factor, K, which relates to the quantity of cement silica fume is capable of replacing while maintaining equivalent strength. For compressive strength, K is in the range of 2 to 5, which means that in a given concrete 1 kg (2.2 lb) of silica fume may replace 2 to 5 kg (4.4 to 11 lb) of cement. The efficiency factor for equivalent compressive strength is valid provided the water content is kept constant and the silica fume dosage is less than 20 percent by mass of cement. In reality, the concept of strength efficiency factors is applicable for all supplementary cementitious materials, but can be particularly useful for very fine SCMs, such as silica fume, metakaolin, and ultra-fine fly ash in their ability to appreciably reduce early heat of hydration by reducing the amount of cementitious material necessary.

The use of silica fume strongly affects the strength development characteristics of the concrete. Regardless of the curing methods used, high-strength concrete with silica fume will gain strength faster during the first 28 days than a similar high-strength concrete mixture without silica fume. Compressive strengths of high-strength concrete with silica fume replacements of 5 to 20 percent of the mass of cement and after 7 days of moist curing were 34 to 57 percent higher than high-strength concrete without silica fume (Hooton, 1993). The higher the silica fume content (up to 20 percent),

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the higher is the compressive strength after 7 days of moist curing. Beyond 28 days, the strength gain of concretes with silica fume is somewhat slower than concretes without silica fume. Beyond 56 days, high-strength concrete with silica fume gains additional strength very slowly, probably due to the effects of self-desiccation. There is general agreement among researchers that the positive influence of silica fume on the strength gain of high-strength concrete occurs mostly during the early age of the concrete (i.e. the first 28 days after placement).

In a study by Detwiler and Mehta (1989), carbon black, a non-pozzolanic material physically similar in size to silica fume was used to evaluate the relative significance of physical and pozzolanic effects. Results show that at an early age, the influence of silica fume on the compressive strength of concrete may be attributed mainly to physical effects. By an age of 28 days, both physical and chemical effects become significant. However, even at an age of 7 days, there is a difference in the resistance to sub-critical crack growth in the cement paste-aggregate transition zone between silica fume and carbon black mixes.

Standard material specifications and test methods do not always characterize the true potential of cementitious materials, and silica fume is a prime example. In order for pozzolans to comply with the requirements of ASTM C 618, the Strength Activity Index of the pozzolan must meet or exceed cube strength values based on the strength of mortars produced at an equivalent flow (i.e. consistency). Comparing the strength performance of mortars prepared on a constant flow rather than constant W/B ratio basis with pozzolans having water-reducing properties, such as fly ash; however, is inappropriate for high-water demand SCMs, such as silica fume and metakaolin. Since the high water demand of these materials are offset using high-range water-reducing admixtures, basing acceptance on strength tests performed without such admixtures is irrelevant. As a result, the strength activity index of silica fume, though at first determined based on water-produced constant flow method (ASTM C 618), is now determined more representatively on a constant W/B basis using high-range water-reducing admixtures to achieve comparative flow values.

In Norway, silica fume is routinely used in concrete. In conventional concretes, addition rates are usually below 5 percent. When used in small quantities, increases in workability with little or no changes to water demand have been reported.

Silica fume has been supplied in the following forms:

- raw powder
- water-based slurry
- densified
- pelletized.

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Silica fume in its natural raw state is difficult to handle. Water-based slurries disperse most efficiently, but they can be quite maintenance intensive since slurries require constant agitation in order to stay suspended. Adding silica fume in the form of a densified powder (Figure 2.12) in bagged or bulk form is the most common and user-friendly way of batching silica fume directly into concrete. As interest in high-strength, high-performance concrete increases, more and more concrete producers are purchasing bulk quantities of densified silica fume. The Pelletized silica fume is not normally used as a batching ingredient in concrete because the pellets will not break up and disperse during the mixing process; however, pelletized silica fume can be interground with portland cement clinker to produce blended hydraulic cement with silica-fume as a constituent.

Due to its extremely high surface area per unit mass, silica fume is a high water demand material. If water alone were added to a concrete mixture with more than 5 percent or more silica fume by mass of total binder, the resulting W/B ratio, for practical purposes, would negate the value of using the silica fume in the first place. Therefore, a high-range water-reducing admixture should always be considered as a necessary ingredient in high-strength silica fume concrete. The use of silica fume improves the early age strength development of concrete and is particularly beneficial

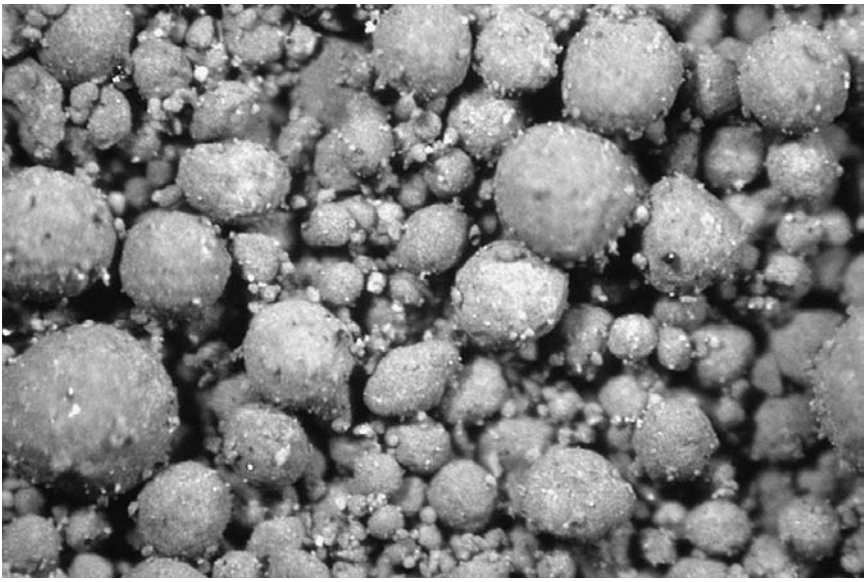

Figure 2.12 Micrograph of densified silica fume. The visible particles are agglomerations of the very small silica fume spheres, some of which can be observed adhering to the larger particles. Field of view is 200 μm wide. Courtesy of Portland Cement Association.

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in achieving high release strengths in precast, prestressed concrete beams. Use of silica fume often allows a reduction in the total amount of cementitious materials. At later ages, concretes made with silica fume can achieve compressive strengths in excess of approximately 115 MPa (17,000 psi).

Metakaolin

Metakaolin (Figure 2.13) is a highly reactive aluminosilicate with the capability of producing mechanical and durability-related properties similar to silica fume. Unlike fly ash, blast-furnace slag, and silica fume, which are byproducts of major industrial processes, metakaolin is a specifically manufactured SCM. In ASTM C 618, metakaolin would be required to meet ASTM C 618 as a Class N (natural) pozzolan.

The raw material necessary for the manufacture of metakaolin is kaolin clay (also known as "china clay"). In its purest form, kaolin clay is a fine, white mineral, comprised primarily of hydrated aluminum di-silicate $(Al_2Si_2O_5(OH)_4$. The temperature at which kaolin transforms into the crystal structure of metakaolin occurs in the range of 600 to 800°C (1100 to 1500°F). If the material is under-fired during pyroprocessing, conversion to an amorphous mineral phase will not occur and the material will not become pozzolanic. If over-fired, sintering and the formation of dead-burned,

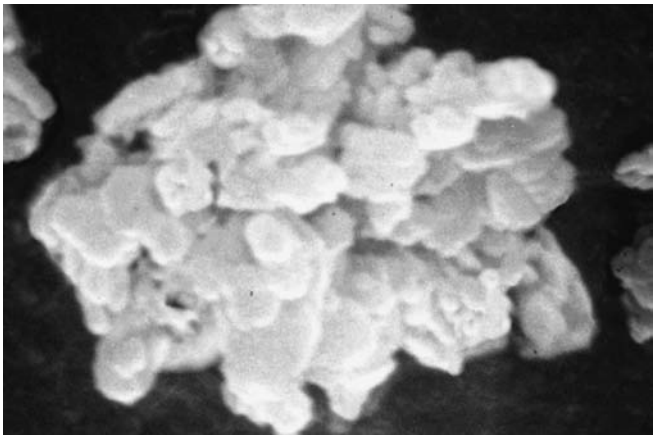

Figure 2.13 Scanning electron microscope micrograph of metakaolin particles at 20,000×. Courtesy of CTLGroup.

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non-reactive mullite ($3Al_2O_3$–$2SiO_2$) will occur. Therefore, conversion of kaolin clay into the highly reactive pozzolan metakaolin is a highly-temperature sensitive process. Metakaolin has been shown to be a quality-enhancing SCM that exhibits high performance properties comparable to silica fume (Caldarone *et al*., 1994).

Aside from the potential to achieve high strength and low permeability on an order of magnitude to that of silica fume,[15] more favorable constructability-related properties can be derived using metakaolin. These advantages are mainly due to its particle size and color. Having an average particle size 20 to 30 times larger than the average particle size of silica fume, the water demand with metakaolin is lower, and the need to offset high water demand with high-range water-reducing admixtures is lower. The result is a high-strength concrete having improved workability, finishability, and a reduced tendency for surface dehydration and plastic cracking. Being much lighter in color than most silica fumes, metakaolin will not darken the color of the paste or mortar, and opens up opportunities to develop high-performance architectural concretes.

In order to offset high water demand, it is a customary industry practice to utilize a high-range water-reducing chemical admixture (HRWR). As previously mentioned, the practice of using HRWRs in conjunction with silica fume is considered a necessity. Rarely would silica fume concrete ever be used without the aid of HRWR. Being a reactive pozzolan with significantly larger sized particles, metakaolin concrete of equal consistency to that of silica fume concrete could be produced using less HRWR, resulting in enhanced workability and lower cost. In addition, concrete containing silica fume does not bleed significantly because of the particle size, leading to a significant risk of plastic shrinkage cracking. The larger particle size of metakaolin is less prone to plastic cracking, and exhibits enhanced finishability.

Metakaolin has a very promising future in the industry as a quality-enhancing additive for high-strength, high-performance concrete. It should be noted that, although the water demand associated with metakaolin is not as high as that of silica fume, when used in the 5 to 12 percent range (by mass of total cementitious material), water demand increases will usually necessitate the use of HRWR, though perhaps not as much. The author is unaware of any extensive field studies conducted using metakaolin in quantities exceeding 12 percent.

Aggregates

Aggregates overwhelmingly occupy the largest volume of any constituent in concrete and profoundly influence concrete performance in both the fresh and hardened states. Selection of appropriate aggregates is important for all structural concretes, regardless of strength. Among the most important

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parameters affecting the performance of concrete are the packing density and corresponding particle size distribution (gradation) of the combined aggregates used. Reasonably efficient aggregate packing improves important engineering properties, including strength, modulus of elasticity, creep, and shrinkage, while generating savings due to reductions in paste volume. Other important parameters influencing packing efficiency include particle shape and surface texture.

Unfortunately, all too often, aggregate purchases are handled by operations or sales managers with little or no technical experience, and cost is usually the primary consideration. Securing an aggregate supply for an upcoming project based on price alone and then addressing the material proportions that will be needed is tantamount to putting the "cart before the horse." Consequently, the concrete will need to be designed around a given set of aggregates, which may or may not be appropriate for its intended usage. When inappropriate aggregates are first selected, it is ironic that once the high-strength mixture has been developed, there is a good chance that the mixture cost will be higher than it would have been had suitable aggregates been selected in the first place. When selecting aggregates for high-strength concrete, the ability to satisfy a strength requirement should never constitute the sole basis of selection. Aggregates that are considered suitable for conventional-strength concrete are not necessarily well suited for high-strength concrete. Aggregates should be selected considering all necessary properties and not just strength. The objective of the aggregate selection process is not to seek out perfect aggregates, but rather, to identify aggregates capable of satisfying all necessary concrete properties in a reasonably cost effective manner. Cost should never supersede quality when selecting concrete aggregates.

Greater considerations are required when selecting coarse and fine aggregates for high-strength concrete. The process of selecting aggregates for high-strength concrete first involves balancing water demand and paste-aggregate bond potential. For equivalent workability, as the maximum size of coarse aggregate increases, the permissible amount of coarse aggregate also increases. Similarly, as the fineness modulus of fine aggregate increases, the permissible amount of coarse aggregate decreases.

High-strength concretes have been produced using lightweight, normal-weight, and heavyweight aggregates. Shideler (1957), Holm (1980), and Hoff (1992) have reported on lightweight high-strength structural concrete using structural lightweight aggregates. Mather (1965) has reported on heavyweight high-strength concrete using high-density aggregates.

If there is a potential for alkali-aggregate reactivity while in service, aggregates should be stringently evaluated. Aggregates proposed for use should be tested to determine its potential for deleterious alkali-aggregate reaction. Tests less than 12 months old for comparable aggregate from the same production facility are usually acceptable for this purpose.

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Fine aggregate

The optimum gradation of fine aggregate for high-strength concrete is determined more by its effect on water demand than on particle packing. High-strength concretes typically contain high volumes of cementitious (i.e. powdery) sized material. As a result, fine sands that would be considered acceptable for use in conventional concretes may be less suited for high-strength concrete due to the sticky consistency that may result. Conversely, coarse sands that may not comply with standard specifications for concrete aggregates may be highly desirable in high-strength concrete. In regards to their impact on workability, the physical grading of fine aggregates is less critical in high-strength concrete mixtures compared to conventional-strength concretes. In order to comply with the requirements of ASTM C 33,[16] the fineness modulus of sands must be between 2.3 and 3.1. Blick (1973) observed that sands with fineness moduli below 2.5 produced high-strength concrete with an overabundance of fine particles. The resulting concrete had a sticky consistency and was difficult to consolidate. Sand with a fineness modulus of 3.0, which would be considered coarse by conventional standards, resulted in the best workability and compressive strength when used in high-strength concrete.

Coarse aggregate

Given the critical role that the interfacial transition zone plays in high-strength concrete, the mechanical properties of coarse aggregate will have a more pronounced effect than they would in conventional-strength concrete (Mokhtarzadeh and French, 2000). Important parameters of coarse aggregate are shape, texture, grading, cleanliness, and nominal maximum size. In conventional-strength structural concretes, it is common for the aggregates to be stronger and stiffer than the paste, aggregate strength is usually not considered a critical factor; however, aggregate strength becomes increasingly important as target strength increases, particularly in the case of high-strength lightweight aggregate concrete. Aggregate properties such as surface texture and mineralogy significantly affect the interfacial paste–aggregate bond and the level of stress at which interfacial cracking commences.

Durability properties notwithstanding, important coarse aggregate properties to consider include strength, stiffness, bonding potential, and absorption (Perenchio, 1973). Caution should be exercised when using extremely stiff coarse aggregates, such as diabase or granite. Depending on the desired concrete properties, stiff aggregates can be either beneficial or detrimental. Several studies (Cetin and Carrasquillo, 1998; Myers, 1999) have found that using coarse aggregate with greater stiffness can increase the elastic modulus while at the same time decrease strength capacity. Designing high-strength concrete to act more like a homogeneous material could enhance ultimate strength potential (Neville, 1996). This can be achieved by

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increasing the similarity between the elastic moduli of coarse aggregate and paste, a subject discussed in more detail in Chapter 4.

As the target strength increases, the properties of aggregates as they relate to water-demand become less relevant and the properties that relate to interfacial bond become more important. Even though the water demand of smaller size coarse aggregates is higher, having greater surface area (and correspondingly greater interfacial bonding potential), smaller aggregates become more desirable as the target strength increases. Rough textured and angular coarse aggregates provide greater mechanical bond and are generally more suitable for use in high-strength concrete than smooth textured aggregates (Neville, 1997). With respect to mechanical properties, even though crushed aggregates usually outperform smooth textured aggregates, smooth textured aggregates should not be summarily dismissed from consideration or restricted based on this characteristic alone. Depending on the required strength and other necessary properties, a clean, well-shaped locally available rounded aggregate might perform satisfactorily.

Aïtcin and Mehta (1990) observed that for high-strength concrete [> 40 MPa (6000 psi)], particularly very high-strength [> 80 MPa (12,000 psi)], it is the mineralogy and strength of the coarse aggregate that ultimately controls the strength of the concrete. It was observed that for concretes produced using identical materials and similar proportions, crushed coarse aggregate from fine-grained diabase and limestone yielded the highest strength results. Concretes made from a river gravel and from a crushed granite containing inclusions of a soft mineral were found to be relatively weaker in both strength and elastic modulus. Note that when considering durability, aggregate mineralogy is critically important.

The crushing process eliminates potential zones of weakness within the parent rock, thereby making smaller sizes more likely to be stronger than larger ones (deLarrard and Belloc, 1997). Smaller aggregate sizes are also considered to produce higher concrete strengths because of less severe concentrations of stress around the particles, which are caused by differences between the elastic moduli of the paste and the aggregate.

For high-strength concrete, aggregate particles should be generally cubical in shape and should not contain excessive amounts of flat and elongated pieces. Note that flatness and elongation are relative terms, and that the definitions vary by location. In the author's view, coarse aggregates containing more than approximately 20 percent of particles having ratios of length to circumscribed thickness greater than three to one, as determined by ASTM D 4791,[17] should be avoided when making high-strength concrete. Aggregate particles should be clean and free of any materials that would degrade, such as organic matter, clay lumps, and soft particles, or adhere to the surface during mixing and impede interfacial transition zone bond. When finely divided materials (i.e. smaller than 75-mm), such as clay, shale, or excessive dust of fracture remain on the surface of aggregates after undergoing batching, mechanical bond at the interfacial transition zone decreases.

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In the case of high-strength concrete, the effect of a weakened paste-to-aggregate bond can be extremely detrimental to strength. For this reason, use of clean, washed aggregate in the production of high-strength concrete is highly suggested. Coatings that impair paste-aggregate bond can be identified through petrographic examination of the suspect aggregate and frequently through petrographic examination of concrete produced with the suspect aggregate.

Aggregate blending is the process of intermixing two or more aggregates to produce an aggregate with a different set of properties. It is not common industry practice to blend crushed and rounded coarse aggregates; however, as the author has seen directly, blending crushed cubically shaped and smooth naturally rounded coarse aggregates can be advantageous for optimizing the properties of high-strength concrete. Luciano *et al.* (1991) incorporated coarse aggregate blending for optimizing concrete with a specified compressive strength of 83 MPa (12,000 psi) at 28 days with additional requirements for modulus of elasticity and pumpability. Included in the optimization program was 9.5 mm (⅜-in.) siliceous gravel composed of rounded quartz particles and a 9.5 mm (⅜-in.) dolomitic limestone composed on angular and sub-angular particles (Figure 2.14).

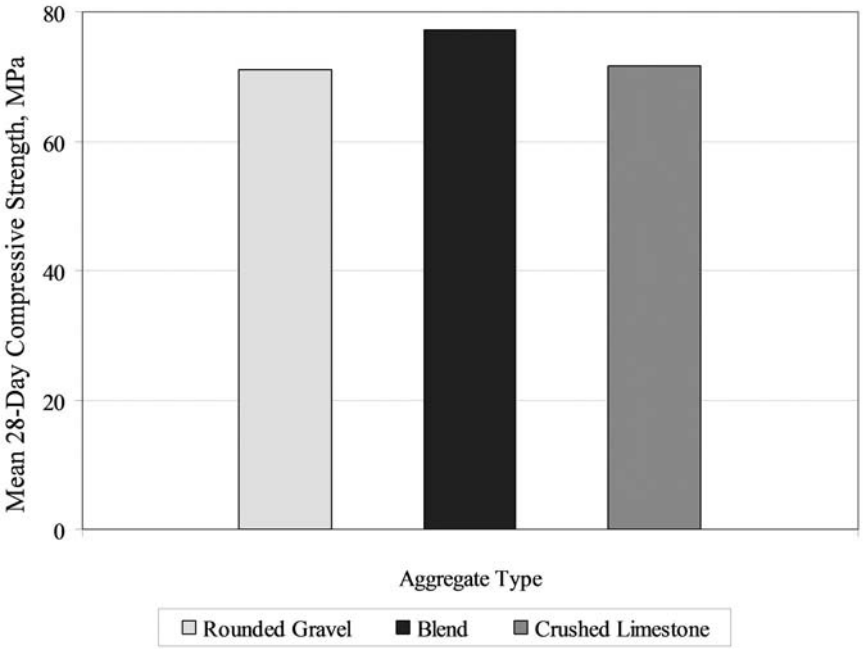

Figure 2.14 Effect of aggregate type and blend on mean 28-day compressive strength (after Luciano *et al.*, 1991).

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Particle packing

Fuller and Thompson's packing theory

It might be reasonable to believe that the best gradation is one that produces the densest packing arrangement. However, some minimum amount of void space is necessary to provide enough paste for satisfactory workability. A commonly used equation to describe maximum particle packing was developed by Fuller and Thompson in 1907. Their basic equation is:

$$P = \left(\frac{d}{D}\right)^n$$

where: P = % finer than the size considered
d = aggregate size considered
D = maximum aggregate size
n = coarseness factor.[18]

Computer simulation

A computer simulation algorithm was developed by Sobolev and Amirjanov (2007) for modeling the packing of large assemblies of particulate materials representing aggregate systems comprising hydraulic cement concrete. The implementation of the developed algorithm allows the generation and visualization of the densest possible and loose-packing arrangements of aggregates. The influence of geometrical parameters and model variables on the degree of packing and the corresponding distribution of particles was analyzed. Based on the simulation results, different particle size distributions of aggregates are correlated to their packing degree.

Water

Because of environmental regulations that prevent the discharge of runoff water from production facilities, use of non-potable water or water from concrete production operations is increasing. Non-potable water includes water containing quantities of substances that discolor it, make it smell, or have objectionable taste. Water from concrete production operations includes wash water from mixers, water that was reclaimed from returned leftover concrete, or storm water runoff collected in a basin at the concrete production facility. Water from these sources should not be used to produce high-strength concrete unless it has been shown that their use will not adversely affect the properties of the concrete. Whether it is used wholly or in combination with potable water, non-potable water should be frequently sampled and stringently tested.

Mixing water includes the free water introduced during and after batching, ice, free moisture on aggregates and water introduced in any significant

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quantity contained in admixtures. Water that is fit for human consumption, has no pronounced odor or taste, and has a history of successful use in producing conventional-strength concrete is usually well suited for producing high-strength concrete. The requirements for mixing water quality for high-strength concrete are no more stringent than for conventional concrete. ASTM C 1602[19] classifies water as follows:

- *Potable water*: that which is fit for human consumption.
- *Non-potable water*: other sources that are not potable, that might have objectionable taste or smell but not related to water generated at concrete plants. This can represent water from wells, streams, or lakes.
- *Water from concrete production operations*: process (wash) water or storm water collected at concrete plants.
- *Combined water*: a combination of one or more of the above-defined sources recognizing that water sources might be blended when producing concrete. All requirements in the standard apply to the combined water as batched into concrete and not to individual sources when water sources are combined.

If used in excessive quantities, water represents concrete's greatest single enemy. Equally true, is that for high-strength concrete to attain its desired fresh and hardened properties, a certain minimum quantity of water is necessary. Producing concrete with an insufficient amount of water can too be an enemy of concrete. A case study presented in Chapter 10 addresses this subject.

Chemical admixtures

Use of chemical admixtures has become an integral part of modern concrete technology. No single group of materials has contributed to expanding the capabilities of hydraulic cement concrete more than chemical admixtures. Prior to the days of chemical admixtures, high-strength concrete usually meant zero-slump concrete. Without materials like high-range water reducing, retarding, and hydration stabilizing admixtures, modern high-strength concrete, as we know it, simply would not be possible.

Unlike supplementary cementitious materials, which contribute minerals, chemical admixtures alter the characteristics of the minerals present in paste; they do so in numerous ways. When properly selected and used, chemical admixtures can enhance both the fresh and hardened properties of concrete, usually doing so in a cost-effective manner. It would be hard to identify even one example where it would not be advantageous to use chemical admixtures in structural concrete. Of course, it is physically possible to produce conventional-strength concrete without the aid of water-reducing or set controlling admixtures; however, slump and setting time would be more difficult to control. Without chemical admixtures, fresh concrete essentially would be at the mercy of time and temperature with respect to

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the ability to transport, place, consolidate, and finish. It would be challenging to produce concrete with design strengths in excess of 35 MPa (5000 psi) consistently without the aid of chemical admixtures. In practical terms, trying to produce, deliver, and place concrete with strengths in excess of 50 MPa (7500 psi) reliably without chemical admixtures would be largely an exercise in futility.

The performance of chemical admixtures in hydraulic cement concrete is principally influenced by the chemical and physical properties, and quantities of cementitious materials used. Due to adverse interactions that can occur between chemical admixtures and cementitious materials, admixtures that have been shown to be effective in some cases may not work well in others; this subject will be described in detail in Chapter 10. Other factors influencing the performance of chemical admixtures include: water content, aggregate shape, gradation, and proportions; mixing time; slump; and temperature of the concrete (Kosmatka *et al*., 2002).

Slump retention, batch-to-batch slump uniformity, and admixture efficiency can be increased when high-strength concrete is initially proportioned with a sufficient quantity of water to produce measurable consistency without the high-range water-reducing admixture. For example, a mixture proportioned with enough water to produce a 25 to 50 mm (1 to 2 in) slump would be expected to exhibit longer slump retention following the addition of high-range water-reducing admixture. This is not always possible when producing very high-strength concretes. Unlike early melamine or naphthalene-based high-range water-reducing admixtures that performed more consistently after pre-wetting the cement, newer-generation high-range water-reducing admixtures, based on polycarboxylate chemistry, can frequently be introduced without pre-wetting the cement. Therefore, once the water content has been established, some newer generation admixtures could conceivably be introduced during the beginning phases of batching rather than at the end.

There is no universal rule of thumb applying to the ways that chemical admixture dosages should be computed. The quantity of most chemical admixtures, such as water reducing and set controlling, is usually determined based either on the amount of cement or total cementitious material.

Conventional water reducing

Water-reducing admixtures (ASTM C 494[20] Type A) are commonly referred to as "conventional" or "normal" water reducers. When used within the manufacturer's suggested dosage rate, conventional water reducer's can be used in one or a combination of the following ways while minimally affecting setting time:

- Reduce the W/B ratio while maintaining constant slump.
- Increase slump while maintaining a constant W/B ratio.

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- Reduce the cementitious materials content and water content while maintaining constant slump and strength.

Set retarding

Set retarding admixtures (ASTM C 494 Types B and D) are critically important in the production of high-strength concrete. These admixtures are most commonly used to control setting time; however, in high-strength concrete their primary role is in controlling hydration as it relates to strength development. High-strength concretes incorporate higher cementitious materials contents than conventional-strength concrete. All else equal, lengthening hydration time will result in increased long-term strength. Set retarding admixtures decrease the rate of C_3S hydration and are primarily used to extend setting time. A retarding admixture can control the rate of hardening in the forms to eliminate cold joints and provide more flexibility in placement schedules. The dosage of a retarding admixture can be adjusted to give the desirable rate of hardening under the anticipated temperature conditions. When the retarding effect of the admixture has diminished, normal or slightly faster rates of heat liberation will usually occur. Depending on the type and dosage of retarding admixture used, early hydration can be effectively controlled while still maintaining favorable 24-hour strengths. Conventional set retarding chemical admixtures can also be beneficial in controlling workability retention, though caution should be exercised because this may not always be the case.

Hydration stabilizing

Hydration stabilizing admixtures (ASTM C 494 Types B and D) may be useful in situations where a controlled extension of set time is desired, such as extended hauls and during large continuous placements. Unlike conventional set-retarding admixtures, hydration-stabilizing admixtures are formulated to provide extended set time control. Depending on the dosage used, set time extensions can range from a few hours to over a day (Caldarone *et al.*, 2005).

High-range water-reducing

Verbeck (1968) described high-range water-reducing admixtures, or "superplasticizers," as linear polymers containing sulfonic acid groups attached to the polymer backbone at regular intervals. Most of the commercial formulations of high-range water-reducers belong to one of four categories:

- sulfonated melamine-formaldehyde condensates;
- sulfonated naphthalene-formaldehyde condensates;
- modified lignosulfonates; or
- polycarboxylate derivatives.

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High-range water-reducing admixtures (ASTM C 494 Types F and G) decrease the W/B ratio and provide high-strength performance, particularly at early ages. Matching the chemical admixture to the cementitious materials, both in type and dosage rate is important. Slump loss characteristics of the concrete will determine whether the HRWR should be introduced at the plant, at the site, or both locations. However, with the advent of newer-generation products, sufficient slump retention can be achieved through plant addition in most cases. High-range water-reducers can be used in one or a combination of the following ways while minimally affecting setting time:

- Reduce the W/B ratio while maintaining constant slump.
- Increase slump while maintaining a constant W/B ratio.
- Reduce the cementitious materials content and water content while maintaining constant slump and strength.

HRWRs may serve the purpose of increasing strength through a reduction in the W/B ratio while maintaining equal slump, increasing slump while maintaining equal W/B ratio, or a combination thereof. The method of addition should distribute the admixture uniformly throughout the concrete. Adequate mixing is critical to uniform performance. Problems resulting from non-uniform admixture distribution or batch-to-batch dosage variations include inconsistent slump, rate of hardening, and strength development.

Accelerating

Accelerating admixtures (ASTM C 494, Types C and E) are not normally used in high-strength concrete unless early form removal or early strength development is essential. High-strength concrete mixtures can usually be proportioned to provide strengths adequate for vertical form removal on walls and columns at an early age. Accelerators used to increase the rate of hardening will normally be counterproductive to long-term strength development. Avoid accelerators when possible. High-early-strength performance is often a routine requirement in many types of construction, such as fast track high-rise construction or at precast plants. A common consequence of speeding up the rate of hydration in a given system is reduced long-term strength. This is not to say that high-strength concrete is unsuitable for certain applications, just that accelerated high-strength concrete requires additional consideration with respect to selection of constituents and proportioning.

Viscosity modifying

Viscosity modifying admixtures (VMAs) are a family of admixtures designed for specific applications. The European Federation for Specialist Construction Chemicals and Concrete Systems cites the following uses:

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- reduce segregation in highly flowable/self compacting concrete;
- reduce washout in underwater concrete;
- reduce friction and pressure in pumped concrete;
- compensating for poor aggregate grading, especially a lack of fines in the sand;
- reducing powder content in self compacting concrete;
- reduce bleeding in concrete; and
- improve green strength in semi-dry concrete.

Self-consolidating high-strength concrete mixtures are frequently produced using high-range water-reducing admixtures in conjunction with viscosity-modifying admixtures, such as cellulose ether, welan, or diutan gum. Some VMAs are based on inorganic materials such as colloidal silica, which is amorphous with small insoluble, non-diffusible particles, larger than molecules but small enough to remain suspended in water without settling. By ionic interaction of the silica and calcium from the cement a three dimensional gel is formed which increases the viscosity and/or yield point of the paste. This three dimensional structure/gel contributes to the control of the rheology of the mix, improving the uniform distribution and suspension of the aggregate particles and so reducing any tendency to bleeding, segregation and settlement.

Most VMAs are supplied as a powder blend or are dispersed in a liquid to make dosing easier and improve dosing accuracy. They have little effect on other concrete properties in either the fresh or hardened state but some, if used at high dosage, can affect setting time and or the content and stability of entrained air (EFNARC, 2006).

Corrosion inhibiting

Corrosion inhibitors are primarily used where chloride salts and the threat to steel corrosion is present, such as parking structures, marine structures, and bridges. Ferrous oxide and ferric oxide form on the surface of reinforcing steel in concrete. Ferrous oxide, though stable in concrete's alkaline environment, reacts with chlorides to form complexes that move away from the steel to form rust. The chloride ions continue to attack the steel until the passivating oxide layer is destroyed. Corrosion-inhibiting admixtures chemically arrest the corrosion reaction. The most widely used corrosion-inhibiting admixture used in concrete thus far has been calcium nitrite. Anodic inhibitors, such as nitrites, block the corrosion reaction of the chloride-ions by chemically reinforcing and stabilizing the passive protective film on the steel; this ferric oxide film is created by the high pH environment in concrete. The nitrite-ions cause the ferric oxide to become more stable (Kosmatka *et al.*, 2002).

Other commercially available corrosion inhibitors include sodium nitrite, dimethyl ethanolamine, amines, phosphates, and ester amines.

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Synergistic effects of combined admixtures

A common practice when producing high-strength concrete is to use a high-range water reducer (superplasticizer) in combination with a conventional retarder or hydration-stabilizing admixture. The high-range water-reducer gives the concrete adequate workability at low water–binder ratios, leading to concrete with greater strength. Retarders slow the hydration of the cement and allow workers more time to place the concrete. Combining high-range water-reducing admixtures with water-reducing or retarding chemical admixtures has become common practice in order to achieve optimum performance at lowest cost. With optimized combinations, improvements in strength development and control of setting times and workability are possible. When using a combination of admixtures, they should be dispensed individually in a manner approved by the manufacturer(s). Air-entraining admixtures, if used, should never come into direct contact with chemical admixtures during the batching process.

Generally, set-neutral water-reducing admixtures or accelerating water-reducing admixtures will not be as beneficial to long-term strength development as admixtures that retard setting. As the specified design strength increases, the ability of set-retarding admixtures to effectively control hydration, which is related to strength, becomes increasingly important.

In high-strength concrete mixtures, high-range water-reducing admixtures are primarily used to enable lowering the water–binder ratio while maintaining workability. Due to the relatively large quantity of liquid that is frequently added in the form of high-range water-reducing admixtures, the water content of these admixtures should be included in the calculation of the water–cementitious materials ratio.

High-range water-reducers enable workable high-strength concrete to be produced at the required water–cementitious materials ratio. They are utilized to control water demand, slump, slump life, placement time, rate of strength gain, and the effects of elevated temperatures and promote favorable consolidation.

Unexpected interactions between otherwise acceptable ingredients in Portland cement concrete are becoming increasingly common as cementitious systems become more complex and demands on the systems get more rigorous. Such incompatibilities are exhibited as early stiffening or excessive retardation, potential for uncontrolled early-age cracking, and unstable or unacceptable air void systems.

Air-entraining admixtures

Air-entraining admixtures are surfactants that entrain small air bubbles that become a part of the cement paste. Air entrainment improves the workability of concrete, reduces bleeding and segregation, and most importantly improves the frost resistance of concrete. Air entrainment is essential to

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ensure the long-term durability of concrete that will become critically saturated with water and then exposed to freezing and thawing conditions. However, air entrainment only protects the paste fraction of the concrete. It does not protect concrete from deterioration caused by non-frost-resistant aggregates.

Air-entraining admixtures are used to purposely introduce and stabilize microscopic air bubbles in concrete. Air entrainment will dramatically improve the durability of concrete exposed to cycles of freezing and thawing. Entrained air greatly improves concrete's resistance to surface scaling caused by chemical deicers. Furthermore, the workability of fresh concrete is improved significantly, and segregation and bleeding are reduced or eliminated. Air-entrained concrete contains minute air bubbles that are distributed uniformly throughout the cement paste that can be produced in concrete by use of air-entraining cement, by introduction of an air-entraining admixture, or by a combination of both methods. Air-entrained cement is a cement with an air-entraining material interground with the clinker during cement manufacture. An air-entraining admixture, on the other hand, is added directly to the concrete materials either before or during mixing (Kosmatka *et al*., 2002). Entrained air can significantly reduce the strength of high-strength concrete, and in addition, increase the potential for strength variability as air contents in the concrete varies; therefore, extreme caution should be exercised with respect to its use. The effects of air entrainment on high-strength concrete are further addressed in Chapter 3.

Notes

1 A discussion on this subject can be found in Chapter 10.
2 *Standard Specification for Portland Cement*.
3 *Standard Specification for Blended Hydraulic Cements*.
4 *Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or (50-mm] Cube Specimens)*.
5 *Determination of Strength*.
6 *Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete*.
7 *Fly Ash for Concrete—Part 1, Definitions, Specifications, and Conformity Criteria*.
8 *Concrete Materials and Methods of Concrete Construction*.
9 *Standard Specification for Ground Granulated Blast-Furnace Slag for Use in Concrete and Mortars*.
10 *Specification for Ground Granulated Blastfurnace Slab for Use with Portland Cement*.
11 Private conversation with Peter Hawkins, retired (formerly with California Portland Cement Co.).
12 *Slag Cement in Mortar and Concrete*.
13 *Standard Specification for Silica Fume Used in Cementitious Mixtures*.
14 *Silica Fume for Concrete. Conformity Evaluation*.
15 When evaluated on an equivalent mass comparative basis.
16 *Standard Specification for Aggregates for Concrete*.

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17 *Standard Test Method for Flat Particles, Elongated Particles, or Flat and Elongated Particles in Coarse Aggregate*.
18 For maximum particle density, Fuller and Thompson used n = 0.5.
19 *Standard Specification for Mixing Water Used in the Production of Hydraulic Cement Concrete*.
20 *Standard Specification for Chemical Admixtures for Concrete*.

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3 Mixture proportioning and evaluation

Introduction

The practice of developing high-strength concrete capable of satisfying needed constructability and serviceability requirements with reasonable economy involves both art and science. Concrete proportioning should not be thought of as a process of selecting tabulated values based on empirical relationships. Concrete proportioning requires cognizant thought processes, particularly when designing mixtures that approach or exceed the limits of the method being used. Empirically-based proportioning methods can be quite useful, but an awareness of their limitations is essential.

Compared to conventional-strength concrete, developing high-strength concrete is a more meticulous process. High-strength concretes incorporate higher quantities of cementing materials used in conjunction with multiple types of chemical admixtures. There are two important points to keep in mind when developing high-strength concrete, and both are related to the W/B ratio. As the target W/B ratio progressively decreases:

- the proportioning principals that were appropriate with conventional-strength concrete progressively become less applicable; and
- some of the constituents that worked well with conventional concrete become less appropriate.

Developing high-strength concrete is still largely done on a trial and error basis. Interestingly, strength is usually not the most important consideration when developing high-strength concrete. The achievement of a mechanical property such as strength is relatively simple and straightforward provided the principles of material selection and mixture proportioning are understood and practiced. Matters related to durability and constructability usually supersede strength during the proportioning process. The true challenge with high-strength concrete is attaining high mechanical properties while still satisfying constructability and durability requirements. For example, slowing down the rate of hydration by using set-retarding or hydration-stabilizing admixtures can be extremely beneficial with respect to long-term

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strength. However, slowing down the rate of hydration too much, though highly favorable for strength, could detrimentally affect construction schedules, therefore trade-offs become necessary. Available options for attaining strength should be identified only after due consideration is given to constructability and durability.

Several articles have been published with suggestions on methods of optimizing the development of particular mixtures by reducing the number of trial mixtures necessary. For example, de Larrard (1990) provided suggestions for high-strength concrete mixtures based on rheological considerations. Domone and Soutsos (1994) have re-examined the maximum density theory for applicability to high-strength concrete. The Absolute Volume Method commonly used to proportion conventional-strength normal-weight concrete forms a solid foundation for proportioning high-strength concrete and much of the information presented in this chapter will be founded on similar empirical principles. Note that this chapter was not written with the intention of merely presenting a set of proportioning guidelines laid out in a systematic flow chart or "cookie cutter" manner. Given the empirical nature of concrete proportioning, doing so would do little good. Learning to proportion high-strength concrete comprehensively should additionally provide the reader with a broader understanding of concrete proportioning in general.

It is important to remember that the process of proportioning concrete is not a means to an end, but rather a means to a *beginning*. It is a process that, when completed, ends up at a starting point. The information in this chapter is presented in order to develop initial estimates of the proportions that can be used for conducting trial batches in the laboratory and subsequently in the field. In all likelihood, adjustments to the initially proportioned mix design are going to be necessary through the course of laboratory and field trial evaluations. Therefore, it is always better to go into the trial evaluation process with an open mind and a willingness to make necessary adjustments. No matter how much a person rationalizes as to how concrete should or could perform when produced, in the end, the only things that will matter are the laws of chemistry and physics.

Whether in a fresh or hardened condition, concrete behavior does not always follow logic, and it is not good practice to make assumptions about how materials will or will not behave in concrete. For example, Kwan (2004) found during the development of high-strength self-consolidating concrete that at W/B ratios below 0.28, the addition of silica fume could substantially increase workability despite the large increase in surface area of the combined cementing materials.

Identifying relevant concrete properties

As is the case with all concrete, before a high-strength concrete mixture can be proportioned, it is essential that all relevant fresh and hardened

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properties have been identified. Careful consideration should be given to the mixture properties needed during both construction and while the concrete is in service. As obvious as identifying relevant properties may seem in theory, this point is emphasized because it does not happen nearly as often as it should in practice. Concrete is often developed based on design criteria only and fails to adequately address the contractor's needs. With respect to concrete properties, the author's preference is to classify them into three principal categories: *mechanical*, *durability*, and *constructability*-related, though it should be recognized that these three property categories are neither necessarily mutually exclusive or inclusive of all concrete properties, such as color or texture. Whether classifying concrete properties into these three categories, or simply by the fresh or hardened state is largely a matter of personal preference. How concrete properties are classified is completely insignificant compared to the importance of identifying and dealing with the properties that are truly relevant. Considering only a few necessary properties, or centering a disproportionate amount of attention on only a few properties could impair performance in both the fresh and hardened state. Concrete mixtures can be developed to meet an array of different properties. Common properties to consider when proportioning high-strength concrete include:

- design-related
  - later-age strength and modulus of elasticity
  - durability
- construction-related
  - consistency (slump or slump spread)
  - workability retention period
  - placeability
  - finishability
  - setting time
  - early strength
  - form stripping
  - post-tensioning.

The process of identifying and disregarding unimportant properties is equally as important as recognizing those that are truly important. Attempting to satisfy irrelevant properties could make it difficult to satisfy the ones that truly are important.

Traditionally, a grossly disproportionate amount of attention has been given to compressive strength. In many cases, strength and durability are indeed mutually exclusive properties. Depending on the service conditions, strong concrete may or may not be more durable. Resolving a strength deficiency by merely increasing the cement content, as so often has been done in the past, may end up worsening durability. For example, if proper consideration was not given to the heat generating characteristics

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of a high-strength concrete used in a massive element, such as a large bridge abutment, high thermal stress gradients could develop. If early-age tensile strength development is insufficient, the concrete will crack.

As the reader's knowledge of high-strength concrete increases, it will become apparent that the achievement of high strength is made possible when steps are taken which do the following:

- reduce paste porosity;
- reduce paste microcracking;
- increase mixture homogeneity; and
- reduce microcracking at the interfacial transition zone.

Statistical variability

It should come as no surprise that the consequence of having greater sensitivity to both material and testing-related variations would be a higher overall variability in test results. As the target strength of high-strength concrete increases, higher coefficient of variations should be expected. ACI 214R-02[1] recognizes this normal characteristic of high-strength concrete and provides a separate ratings table for determining the adequacy of control for concrete having a specified compressive strength (f_c') greater than 34.5 MPa (5000 psi). For example, according to the ACI 214 Tables[2] a varition of 2.8 MPa (400 psi) or less for a concrete with a specified compressive strength at or below 34.5 MPa (5000 psi), tested at the designated acceptance age indicates excellent control. Similarly, a coefficient of variation of 7.0 MPa (1000 psi) or less for a concrete with a specified strength greater than 34.5 MPa (5000 psi) also tested at the designated acceptance age would too indicate excellent control. Note that the same variation, if applied to conventional-strength concrete, would indicate poor control.

This book, along with the ACI 318/318M[3] Building Code, defines a strength test as the average of two or more specimens of the same age taken from a single batch of concrete. From time to time, unusually low or high values (outliers) commonly occur when strength testing concrete. Outliers are more difficult to identify when testing only two specimens at a time. For better statistical confidence, at least three specimens are suggested, especially when testing for acceptance at the designated concrete age.

ACI 363.2R-98 reports on the results of an inter-laboratory test program conducted by Burg *et al.* (1999) that demonstrated that the current requirements for testing platens, capping materials, or specimen end conditions might be inadequate for testing high-strength concrete. For high-strength concrete, greater consideration must be given to testing-related factors, including specimen size and shape, mold type, consolidation method, handling and curing in the field and laboratory, specimen preparation, cap thickness, and testing apparatus (Vichit-Vadakan *et al.*, 1998). These factors are discussed in more detail in Chapter 9.

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When statistically evaluating strength results, it is generally assumed that the data population is normally (symmetrically) distributed about the mean value; however, this is not necessarily the case with high-strength concrete. Cook (1989) pointed out that a skewed[4] distribution may result for high-strength concrete. Data not distributed symmetrically about the mean may be skewed. Distributions that are too peaked or flat may indicate kurtosis.[5] Presuming test data to be normally distributed where in fact it is not can be misleading rather than informative.

Recognizing the increased sensitivity of high-strength concrete to material and testing-related variables, and the resulting higher overall variability, Myers and Carrasquillo (1999) suggested that a trial batching series be conducted by the concrete producer prior to actual production in order to verify that the proposed mix design has a sufficient strength over design factor.

Proportioning considerations

When developing mixture proportions for high-strength concrete, three fundamental factors must be considered in order to produce a mix design satisfying its intended property requirements:

- mechanical properties of the aggregates;
- mechanical properties of the paste; and
- bond strength at the paste-aggregate interfacial transition zone.

Upon satisfactorily addressing relevant mechanical and durability properties, the fresh concrete should be capable of satisfying the following constructability-related requirements:

- be easily produced and delivered;
- exhibit reasonable within-batch and between-batch uniformity;
- maintain the desired consistency throughout the intended placement period;
- resist segregation when placed and consolidated; and
- when necessary, exhibit satisfactory finishing characteristics.

This section presents accepted mixture proportioning principles for high-strength concrete using common materials and production techniques. The most common method used for proportioning normal weight concrete is by calculating the absolute volume occupied by the individual constituents. The fundamental procedures described in ACI 211.1 for proportioning normal weight concrete is generally applicable for proportioning high-strength concrete; however, distinct limitations do exist in the applicability of ACI 211.1 to high-strength concrete. Recognizing this, ACI Committee 211 published ACI 211.4, a revised method for proportioning concrete by

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the absolute volume method that is empirically better suited to low W/B ratio paste-rich mixtures. However, rather than using the modified ACI 211.4 method, the 211.1 method with appropriate modifications will be used in the example problem presented at the end of this chapter (pp. 88–95). In the long term, the reader will be far better served by understanding why such modifications are necessary rather than merely knowing that modifications exist.

Broadly stated, the procedures for proportioning normal weight concrete by absolute volume consists of a series of steps, which when completed provide general estimations for a mixture meeting strength and workability requirements based on the combined properties of the individually selected and proportioned components. The general process being:

1 Identify relevant mechanical, durability, and constructability requirements.
2 Select desired consistency (slump or slump spread).
3 Select the nominal maximum aggregate size (based on dimensional and constructability constraints).
4 Estimate the water content based on the cementitious materials used, aggregate characteristics, admixture characteristics, and air content requirements.
5 Estimate the target W/B ratio considering both mechanical and durability requirements.
6 Estimate amount and proportions of cementitious material based on estimated water content and desired W/B ratio.
7 Estimate the required dosage range of each chemical admixture.
8 Estimate volume of coarse aggregate considering physical properties of coarse and fine aggregates and workability requirements.
9 Calculate required fine aggregate content.
10 Conduct laboratory trials for the purpose of evaluating the ability of the mixture to satisfy required mechanical, durability, and constructability properties, while checking for possible constituent material incompatibility and adjusting the materials or mixture proportions as needed.
11 Conduct field trial tests replicating anticipated job conditions, adjusting the materials or proportions as needed.

Depending on the particular property under consideration, there are two useful ways to think about the composition of concrete. Fundamentally, concrete is a dual-component composite substance comprised of two materials —paste and aggregate (coarse and fine aggregates). However, sometimes it is useful to think about concrete as a material comprised of mortar (paste + fine aggregate) and coarse aggregate. For example, the characteristics of the mortar fraction of concrete can profoundly influence the entrainability, size, and spacing of the air-voids in air-entrained concrete. Although the

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physical properties and volume of coarse aggregate can strongly affect the stability and uniformity of an air-void system itself, the air-voids themselves reside within in the mortar. During the development of self-consolidating concrete, Wong and Kwan (2005) observed that even when the fluidized concrete showed serious signs of segregation, the aggregate particles that were smaller than 1.2 mm (0.05 in) tended to remain bound to the cement paste. This indicated to the authors that better mixture optimization could be derived by considering the coarse and fine aggregates separate from each other, that is, viewing the concrete as a mixture of mortar and coarse aggregates rather than a mixture of paste and combined aggregates.

Concrete composition limits the ultimate strength that can be obtained and significantly affects the levels of strength attained at early ages. In conventional-strength concrete technology, the two dominant factors that are considered to control maximum concrete strength are the aggregate and paste characteristics. However, as target strengths progressively increase, the characteristics of the paste-aggregate interfacial transition zone takes on paramount importance. In fact, the ability to achieve ultra-high compressive strength ultimately becomes governed by the quality of the interfacial transition zone bond.

A common mistake when first attempting to produce high-strength concrete is to apply proportioning principles that would be more appropriate for conventional-strength concrete. Despite the fact that the principles of proportioning high-strength concrete have been identified and validated, nonetheless, it is an all too common occurrence. The objective of this section is to identify principal factors to consider when proportioning high-strength concrete.

Water-binder ratio (W/B)

The distance cementing particles are spaced at the time of hardening establishes the capillary porosity, or "gel-space ratio" of hardened cement paste, and it is the single most important factor influencing the strength, and largely influencing the durability of concrete. Insomuch as this principle is at the heart of the water–cement (W/C) ratio "theory," it is seldom stated in this manner. Of course, most courses in the fundamentals of concrete would not be complete without mention of the inverse relationship between the W/C ratio and strength; however, quite often only the relationship itself is presented without an explanation as to *why* the W/C ratio is so intimately connected to strength and other important properties, such as permeability.

The relationship between the W/C ratio and compressive strength was first described by Duff Abrams in December 1918 at an annual meeting of the Portland Cement Association. After conducting countless tests on various concretes and mortars over a four-year period at the Lewis Institute in Chicago, Abrams first published his findings in 1919 in *Design of Concrete Mixtures*. Provided that concrete is of a workable (plastic) consistency,

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Abrams surmised that for given materials, strength depends only on one factor—the ratio of water to cement.

Mathematically, this relationship was expressed with the following formula (Abrams, 1919):

$$S = \frac{A}{B^x}$$

where: S is the compressive strength of concrete,
x is the ratio of the volume of water to the volume of cement.
A and B are constants whose values depend on the quality of the cement used, the age of the concrete, curing conditions, etc.

The constants A and B correlating the relationship between the W/C ratio and compressive strength depend on the quality of the cement and various other factors as stated above. Abrams recognized that the relationship between the W/C ratio and compressive strength was dependent on the particular cement chosen. Unlike the single curve water–cement ratio vs. compressive strength relationships frequently presented in concrete proportioning guides, in actuality, different cements produce different curves. As obvious as this principle seems, it is emphasized because concrete making materials, cement notwithstanding, are all too often viewed as commodities. Regardless of the strength class of the concrete, constituents should *never* be viewed as mere commodities. When making high-strength concrete, the selection of a conducive cement is initially important.

Since the relationship between the water–cement ratio and compressive strength for Portland cement concrete cannot be described by any single curve, it would seem appropriate that a harmonized term relating to the mass ratio of water to all cementitious materials could be established. In lieu of terms such as water–cement ratio (W/C), water-cement plus pozzolan ratio (W/C+P), and water-cementitious materials ratio (W/CM), this book places more emphasis on the term *water-binder ratio* (W/B). The practice of including pozzolans and other hydraulic materials when calculating the water–cement ratio is a long accepted industry practice. Understanding that a describable relationship exists between water-binder ratio and compressive strength for a given type of binding system is what matters, not the magnitude of strength correlated to any one binding system. As Figure 3.1 demonstrates, given the many different types and feasible combinations of cementitious materials for use in the production of hydraulic cement concrete, it would seem more appropriate to envision the relationship between water-binder ratio and strength in terms of a strength envelope rather than a single curve. A similar relationship was suggested by Aïtcin (1998). Whether a material is classified as hydraulic or pozzolanic when first combined is irrelevant compared to the manner in which the materials interact, what they ultimately become, and the manner in which they become

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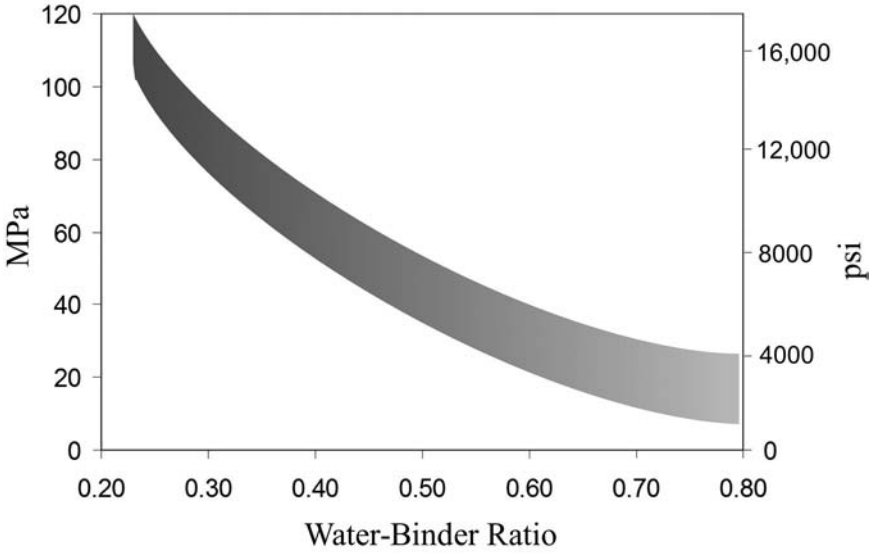

Figure 3.1 Illustration of the relationship between W/B ratio and strength for various combinations of cements and pozzolans.

it. It is for this reason, the expression W/B will be used in place of the traditional expressions W/C, W/CM, and W/(C+P).

Expounding on Abrams theory, Gilkey (1961) theorized that for a given cement and acceptable aggregates, the strength that may be developed by a workable, properly placed mixture of cement, aggregate, and water (under the same mixing, curing, and testing conditions) is influenced by:

a ratio of cement to mixing water;
b ratio of cement to aggregates;
c aggregate grading, surface texture, shape, strength, and stiffness; and
d maximum aggregate size.

Although factors b) through d) are highly important for establishing concrete strength, factor a) plays the most critical role. The W/B ratio is what establishes paste density. The primary factor influencing concrete strength is the density of the hydrated cement paste. The role of paste density as it relates to strength is described in the next section.

Paste density

The preceding section explained how the W/B ratio is the single most important factor influencing the strength of concrete. In actuality, the water-binder ratio is what establishes *paste density* (Smith, 2003). The principal

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factor determining concrete strength is the *density* of the hydrated cement paste. As Figure 3.2 demonstrates, as the W/B ratio decreases, the distance between cementing materials decreases. Optimum density for a simple cement-water paste occurs at the point of maximum particle packing with 100 percent of the inter-particle voids filled with water. Further decreasing the W/B ratio beyond this point will cause paste density (and measured strength) to decrease.

In conventional-strength concrete, hydration and strength go hand-in-hand. In general, as long as the cementing material within concrete continues to hydrate, strength should continue to rise. It would then seem logical that the converse to this principle is equally true, that is, less hydration leads to lower strength. It turns out that the validity of the latter statement is conditional, and depends on the W/B ratio of the concrete. For a given set of paste constituents, as the W/B ratio continues to decrease, there reaches a point where paste density is maximized. Continuing to decrease the W/B ratio further will cause paste density to decrease and with it, strength. *Note that this is a cornerstone principle of high-strength concrete technology.* The specific W/B ratio at which density is maximized will depend on the paste constituents. Optimum density with one combination of constituents might be 0.25. Other combinations might be slightly higher or lower. 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/B ratio and degree of hydration (Powers, 1947). Most high performance concretes are produced with a W/B ratio of 0.40 or less.

In an effort to measure the amount of water consumed by hydration, Powers (1949) categorized the water contained in cement paste into three different types:

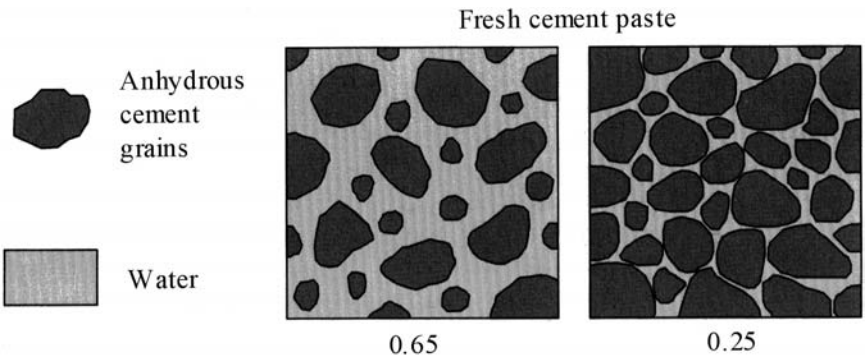

Figure 3.2 Schematic representation of two fresh cement pastes having water-cement mass ratios of 0.65 and 0.25 (after Aïtcin, 1998).

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- chemically bound water;
- physically adsorbed water; and
- free water.

Chemically bound water is the fundamental component of C-S-H gel after hydration occurs. Physically adsorbed water is adsorbed at the external surfaces of the layers of C-S-H, occupying the so-called *gel pores*. Remaining water, including water residing in the capillary pores is considered free water. Physically adsorbed water and free water are usually identified together as evaporable water. Both the evaporable and non-evaporable water content depends on factors such as W/B ratio, age, and the characteristics of the cementing materials used. By measuring the non-evaporable content of the cement paste, Powers (1949) suggested that the degree of hydration of cement can be calculated, and went on to determine that for complete hydration of Portland cement, the water–cement ratio should be greater than about 0.42. With respect to strength, density takes precedent over the amount of hydrated material within the system. Powers theorized that at W/C ratios below 0.42, cement undergoes self-desiccation, leading to autogenous shrinkage. When using supplementary cementitious materials, depending on the particular type being used, the W/B ratio for complete cement hydration to occur is very likely to be different.

W/B ratios that produce the densest pastes do not always make for the most appropriate concretes after all the necessary properties are taken into consideration. Depending on the particular application of the concrete, it might not be feasible to proportion the mixture at the W/B ratio that will result in the densest paste. In practice, higher W/B ratios may be necessary when all of the necessary properties of the concrete are taken into account. It might be determined that a mixture produced at an optimally low W/B ratio exhibits an objectionably cohesive consistency given the significantly larger amount of chemical admixtures that would likely be needed for favorable workability or pumpability. Such a mixture might be perfectly suitable for pumping a short distance into a column or wall, but might be unsuitable for pumping long distances or finishing horizontal surfaces. For example, it might be determined that the optimum W/B ratio for producing densest paste occurs at 0.26. When proportioned at a 0.26 W/B ratio, the concrete might be too sticky to work favorably, yet when proportioned at a W/B ratio of 0.28, the concrete might exhibit much better performance in both the fresh and hardened state. Of course, altering the proportions of the cementitious materials might produce very different results.

As a footnote to this section, there is a somewhat misconceived notion that the presence of abundant amounts of unhydrated cementing material in hardened paste is a bad thing. Unusually higher than "normal" amounts of unhydrated material has often been cited as a contributing factor in low strength investigations. On the contrary, in a properly designed high-strength concrete, higher amounts of unhydrated cementitious material should be anticipated.

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Interfacial transition zone

The interfacial transition zone between cement paste and aggregate particles is one of the most important factors influencing the mechanical and durability properties of high-strength concrete. Improving the density and bonding characteristics of this zone is fundamental to the production of high-strength concrete. Pozzolans, particularly silica fume, are beneficial in this respect (Scrivener *et al*., 1988; Domone and Soutsos, 1994).

In spite of the fact that large, maximum sized coarse aggregates reduce water demand, and thus W/B ratio, it has been found that using larger aggregates impede the ability to attain high values of strength. This is principally due to the inherent incompatibility between the aggregate and the hardened cement paste in terms of their elastic moduli and Poisson's ratios. Consequently, in order to achieve high strength, there requires a reduction of the thickness of the interfacial transition zone in high-strength concrete. The densification of the interfacial transition zone allows for efficient load transfer between the cement mortar and the coarse aggregate, contributing to the strength of the concrete. For high-strength concrete where the matrix is extremely dense and paste-aggregate bond strength is high, a weak aggregate can become the weak link with respect to strength. The transition zone between the aggregate surface and the hardened paste is typically 10 to 50 μm (0.0004 to 0.0020 in) wide, and usually the weakest part of most hardened concrete (Mehta, 1986). Smaller sized coarse aggregates

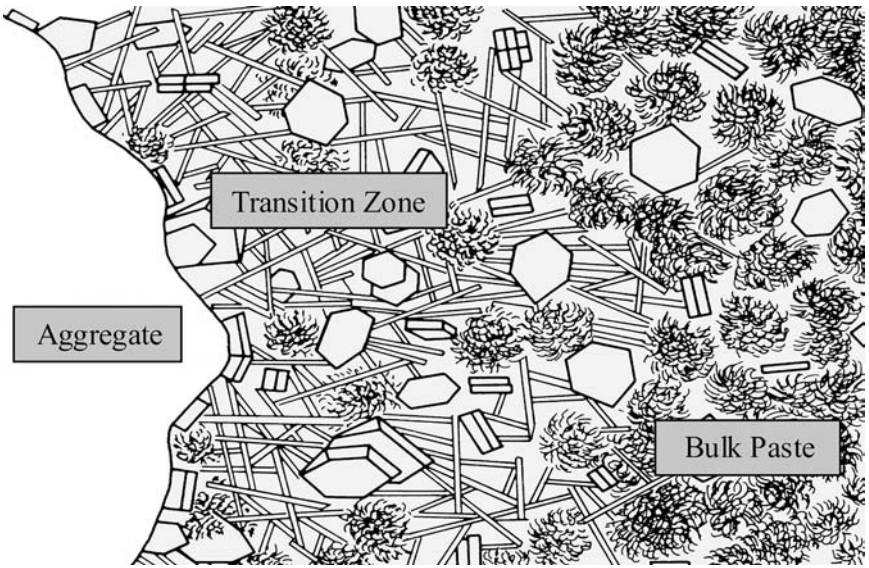

Figure 3.3 Illustration of interfacial transition zone. Courtesy of Portland Cement Association.

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offer a larger total surface area and therefore lower bond stress in the transition zone, thus increasing bond capacity. Fracture surfaces consequentially pass through the coarse aggregate particles as well as through the hardened cement paste, both under compressive and under tensile loading.

Particle distribution

The influence particle grading plays on the fresh and hardened properties of concrete is well recognized for aggregate particles (Fuller and Thompson, 1907). Perhaps less well recognized is that this same principle applies to cement-sized particles. Improvements to the size uniformity of cementing particles have been found to have a notable improvement in strength. Increases of 10 to 20 percent have been reported in the compressive strength of cement cubes produced with particle size-controlled Portland cements (Farny and Panarese, 1994). At a given W/B ratio, as the grading uniformity of cementitious particles increases, paste density also increases, but only to a point. In theory, highest density (i.e. optimum W/B ratio) for a perfectly graded cementitious system is achieved when 100 percent of the remaining space is filled with water. Continuing to decrease the W/B ratio further creates unfilled space, thus causing reductions in density. For pastes comprised of a given set of materials, in theory, maximum strength occurs at maximum achievable density. Berntsson *et al.* (1990) considered the compactibility of pastes to be governed in part by particle geometry and in part by charge at the particle surface, the latter being controllable by the dispersing effects of chemical admixtures. The original size, spatial distribution, and composition of Portland cement particles have a large influence on hydration, microstructure development, and ultimate properties of cement-based materials.

The effect of cement particle-size distribution on concrete properties was investigated by computer simulation along with experimental studies (Bentz *et al.*, 1999). Properties examined include setting time, heat release, capillary porosity percolation, diffusivity, chemical shrinkage, autogenous shrinkage, interfacial transition zone microstructure, and internal relative humidity evolution. The effects of flocculation and dispersion of the cement particles in the starting microstructures on resultant properties were also studied. Using two cement particle-size distributions bounding those commonly used and three different W/B ratios (0.50, 0.30, and 0.25), the results of the study suggested that as the W/B ratio decreases, the use of coarser cements become increasingly more beneficial. It is the author's view that notable improvements to long-term mechanical and durability properties can be achieved using coarser high-strength cements produced at optimal particle distribution.

Aggregate characteristics

The size, shape, texture, and grading characteristics of coarse aggregates significantly affects the fresh and hardened performance of high-strength

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concrete. When used individually, there are advantages and disadvantages associated with the use of crushed and naturally rounded coarse aggregates. The benefits or shortcomings of each depend on the specific concrete properties under consideration and the properties of each aggregate. Important aggregate properties that will determine the optimum blending ratio include gradation, shape, angularity, and hardness.

The strength-attaining limitations of larger-sized coarse aggregates become apparent when attempting to produce high-strength concrete. Figure 3.4 shows the effect of aggregate size when producing plain (Portland cement

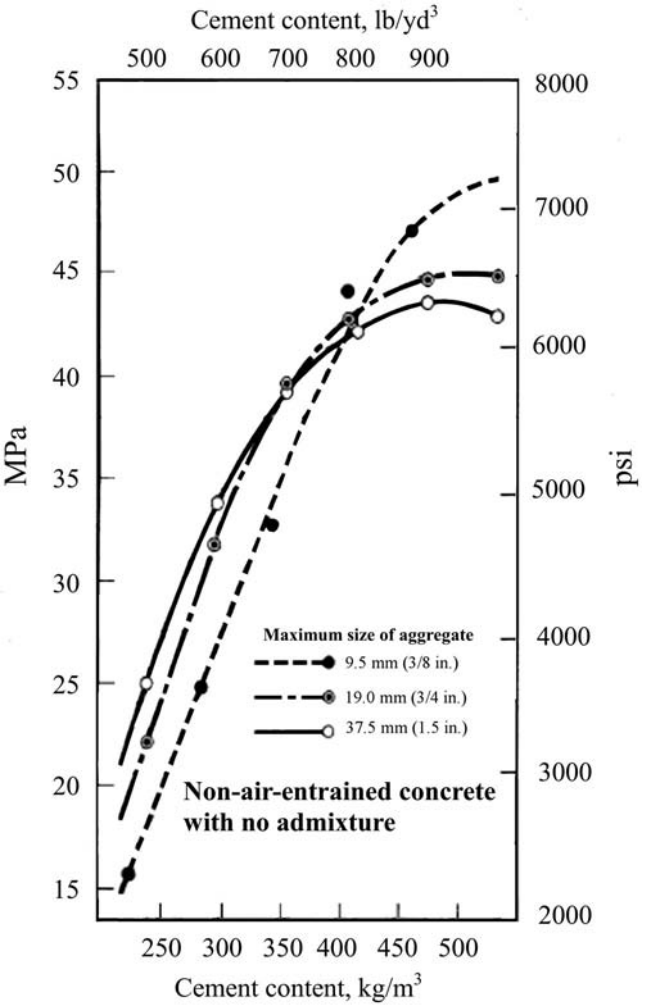

Figure 3.4 Effect of cement content on compressive strength at 28 days for various maximum sizes of aggregate (after Farny and Panarese, 1994).

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only) concrete at a fixed slump of 100-mm (4-in). Note that at cement contents below 350 kg/m^3 (600 lb/yd^3); the largest of the three aggregates yielded the highest compressive strength at 28 days. At cement contents above 400 kg/m^3 (700 lb/yd^3), the smallest of the three aggregates yielded the highest compressive strength at 28 days.

Figure 3.5 shows the relationship between maximum aggregate size and strength per unit mass of cement used (strength efficiency). The strength in MPa (psi) obtained for each kg (lb) of cement used per unit volume of concrete is plotted to form a strength efficiency envelope. As these data

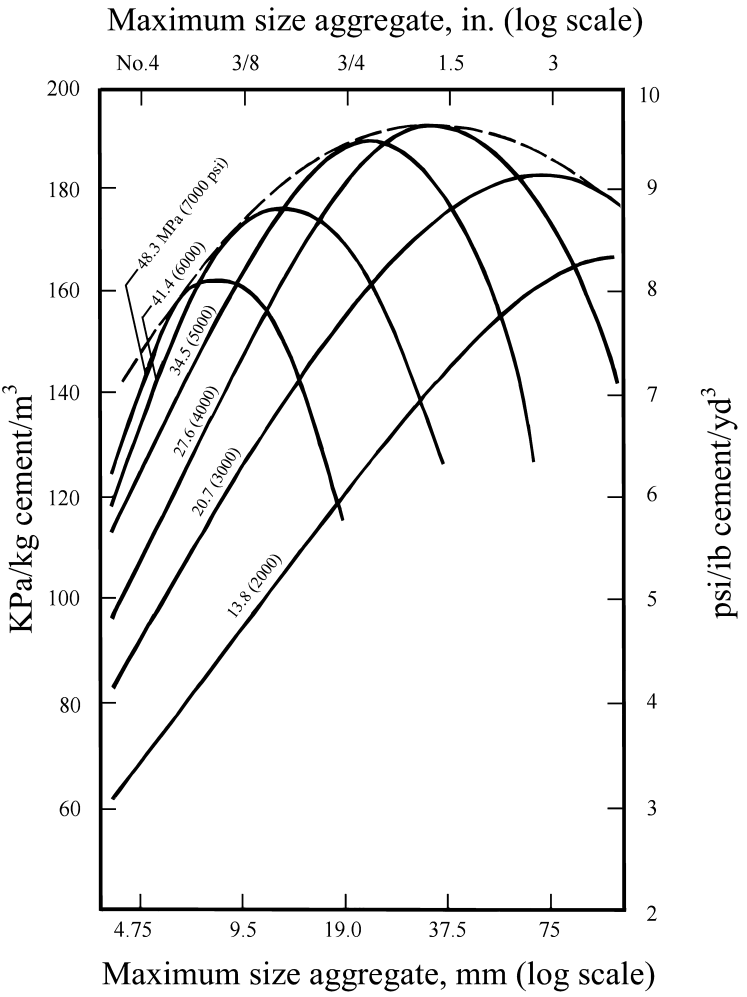

Figure 3.5 Strength efficiency of Portland cement in concretes produced with various sizes of coarse aggregates (after Farny and Panarese, 1994).

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suggest, higher strength efficiencies are obtained at higher strength levels with smaller maximum aggregate sizes (Farny and Panarese, 1994). Note that using the largest practical size coarse aggregate in high-strength concrete is still quite important when modulus of elasticity, creep, and drying shrinkage are principal considerations; therefore, trade-offs between strength and other needed properties often become necessary.

The preceding chapter stressed the importance of identifying cementing materials having quality characteristics suitable for satisfying all mixture performance requirements, not just strength. The objective of aggregate optimization is to produce aggregate blends with high packing densities capable of attaining high performance using lower paste contents. As the paste content decreases, the frequency of paste-related durability problems, such as heat generation, porosity, and drying shrinkage will also decrease.

In various guidelines for proportioning conventional and high-strength concrete, certain assumptions are made with respect to constituent material properties, such as aggregate shape, grading, and angularity, and the relationship between W/B ratio and strength. Whenever possible, the selection of concrete proportions should be based on knowledge of the actual constituents to be used. When using unfamiliar materials, a greater number of iterations should be anticipated during the trial evaluation process.

The use of a larger maximum size of coarse aggregate affects strength in multiple ways. Larger size aggregates have less surface area per unit volume; therefore, as the aggregate size increases, water demand generally decreases. For this reason, a lower W/B ratio can be used, and thus a higher strength is achieved. However, as the target strength of concrete increases, the bond strength at the interfacial transition zone becomes increasingly important. As the size of coarse aggregates decrease, the surface area per unit volume increases, thus causing an increased water demand to produce concrete of equal consistency. Thus, in order to maintain equal strength (i.e. equal W/B ratio), the binder content must be increased. With respect to its influence on strength, the effect of transitioning from a larger to a smaller size coarse aggregate depends on how the increase in water demand is addressed. Merely increasing the water content in order to maintain equal consistency will cause strength to decrease. However, changing from a larger to smaller aggregate while maintaining the W/B ratio fixed will necessitate an increase in the cementitious materials content. Given the increased amount of paste-aggregate bond provided for by the smaller aggregates, the net result of maintaining a fixed W/B ratio would be an increase in measured strength. No matter how addressed, when transitioning from larger to smaller aggregates, note that the coarse aggregate volume will need to be decreased if workability is to be sustained. The water reduction capacity of the particular chemical admixture used will affect the magnitude by which the cementitious materials content will need to increase.

For a given volume of concrete, using larger aggregates results in a smaller volume of paste, thereby providing more restraint to volume changes of

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the paste. This may induce additional stresses in the paste, creating microcracks prior to application of load, which may be a critical factor in high-strength concrete. Therefore, it is generally agreed 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 (⅜ in) and 25 mm (1 in). A high-range water-reducing admixture was used in all of the mixtures studied. In general, for a given W/B ratio, the smallest size of the coarse aggregate produced the highest strength; however, it was feasible to produce compressive strengths in excess of 70 MPa (10,000 psi) using a 25 mm (1 in) maximum size aggregate when the mixture was properly proportioned with a high-range water-reducing admixture. A similar study was conducted by de Larrard and Belloc (1997) using crushed limestone aggregates, Portland cement, silica fume, and high-range water-reducing admixture for eight different mixtures. The results suggested that better performances and economy could be achieved with 20 to 25 mm (¾ to 1 in) maximum size aggregates even though previous researchers had suggested that 10 to 12 mm (0.4 to 0.5 in) is the maximum size of aggregates preferable for making high-strength concrete.

The principle that smaller coarse aggregates produce higher-strength concrete can be a difficult concept to embrace, since it is opposite to the same principle in conventional-strength concrete, where smaller aggregates reduce strength. In order to understand the relationship between strength and aggregate size, three things must be known:

- aggregate size;
- water-binder ratio; and
- consistency (i.e. slump, slump spread, etc.).

Accepting the principle that smaller-sized coarse aggregates are actually more conducive when making high-strength concrete has been found to be one of the more difficult concepts to fully embrace. Once understood and accepted, many of the other principles associated with the technology of high strength should fall right into place.

If sand with a fineness modulus of less than 2.5 is employed, the resulting high-strength concrete could be overly cohesive (sticky), resulting in poor workability and possibly a higher water demand. In general, because of the increased cementitious fines content of a high-strength concrete, the volume of sand is kept to the minimum necessary to achieve workability and consolidation ability.

As target strength increases, bond strength at the paste-aggregate interfacial transition zone progressively takes on greater significance. For concretes having compressive strengths below about 35 MPa (5000 psi), the quality of the interfacial transition zone seldom requires too much consideration. At a compressive strength of 110 MPa (16,000 psi), the

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density and resulting bond characteristics of the interfacial transition zone become *supremely* important. Uncrushed gravels, though favorable for providing water reduction compared to similarly shaped crushed stone, are generally much less suitable for use in high-strength concrete. The increased bond at the paste-aggregate interface that is provided with crushed aggregate is significantly more advantageous than the water reduction afforded using rounded gravel. Aggregates for use in high-strength concrete should be free from any type of coating that would impair paste-aggregate bond.

For mixtures rich in cementitious material, such as high-strength concretes, it is better to use fine aggregates (sands) with higher fineness moduli (> 2.90) than would normally be used for concretes having lower cementitious materials contents. High-strength concrete mixtures already have large amounts of powdery fines; therefore, fine sand particles will not lead to improved workability. Conversely, using finer sands will require more water in order to maintain the same workability.

Estimating coarse aggregate volume

Selecting an appropriate volume of coarse aggregate for high-strength concrete is one of the most challenging aspects for beginners. It is at this point in the proportioning process that the customary empirical relationships between coarse aggregate volume, coarse aggregate size, and fineness modulus of fine aggregates seriously break down. In general, as the fineness modulus of fine aggregates decrease, it is possible to use higher volumes of a given coarse aggregate without sacrificing workability. In fact, this is one of the cornerstone principles making it possible to proportion concrete in a systematic empirically based manner. However, there are certain presumptions behind empirically based selection tables, such as those in ACI 211.1, and there are boundaries at which the applicability of the proportioning method breaks down. Being paste rich, the workability of high-strength concretes can be maintained using coarser sands. In fact, when considering both fresh and hardened properties, it is the author's view that the overall performance of high-strength concrete improves with sands approaching, and in many cases exceeding the 3.1 upper limit fineness modulus stated in ASTM C 33. This being the case, selecting a coarse aggregate volume using a proportioning method designed for lower paste content mixtures will result in an over-sanded high-strength mixture.

Unless the high-strength concrete is being proportioned in accordance with a method specifically designed for paste-rich mixtures, caution should be exercised when estimating the volume of coarse aggregate. Initially, more trial and error may be necessary in order to identify a coarse aggregate volume suitable for satisfying both fresh and hardened properties. If using the ACI 211.1 method for proportioning normal weight high-strength concrete, for initial estimating purposes, the author suggests increasing the coarse aggregate volume (i.e. volume of coarse aggregate per unit of volume

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of concrete) initially by approximately 40 percent. Doing so should reduce the number of laboratory trial iterations needed to identify the most suitable coarse aggregate volume for the materials being used. This principle will be addressed in the proportioning example presented at the end of this chapter. By increasing the coarse aggregate volume by approximately 40 percent, it will become necessary to reduce the estimated water content from the tabulated values in ACI 211.1. For initial estimating purposes, the author suggests reducing the tabulated values by about 10 percent.

Calibrating consistency

Batch-to-batch consistency and performance in both the fresh and hardened state can be improved by having the ability to calibrate the W/B ratio with slump. From time to time, it is good practice to periodically check the slump of superplasticized concrete produced at a fixed W/B ratio *without* the inclusion of the high-range water-reducer. Whenever possible, it is suggested that superplasticized concrete be proportioned in such a way that it has a plastic, verifiable consistency exclusive of the high-range water-reducer. It takes a threshold amount of water (and water reducing admixtures) to produce concrete with a plastic, measurable consistency. If the slump test were performed prior to reaching that threshold, the result would be a zero slump concrete. If the quantity of added water were still below the threshold for achieving measurable consistency, the result would again be zero; therefore, it would not be possible to calibrate the W/B ratio with slump. It should be noted that this is not always possible when producing concretes with exceptionally low water-binder ratios.

Water contained in admixtures

Water from all sources should be identified and compensated for when proportioning high-strength concrete. For practical purposes, the amount of water contained in low dosage chemical admixtures such as conventional water-reducers and set-controlling admixtures is usually negligible. It is good practice to consider the water contained in higher dosage admixtures, such as high-range water-reducers, and corrosion inhibitors.

Air entrainment

Air entrainment is the single most beneficial mechanism for improving the durability of concrete subjected to freezing and thawing while critically saturated or in the presence of deicers; however, air entrainment and high strength are inherently incompatible properties, and satisfying both properties can be quite challenging. For elements that are exposed to freezing and thawing while critically saturated, there is no well-documented field

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experience to prove that air-entrainment is not needed. (Kosmatka *et al*., 2002); however, exterior exposure in and of itself should not justify the use of air entrainment. For horizontal exposed elements, it would be difficult not to justify the need for air entrainment, but this is not the case with vertical members such as columns and walls. Since the strength of high-strength concrete can be dramatically reduced due to the presence of entrained air voids, when proportioning high-strength concrete, it should be noted that concrete's resistance to the distress caused by repeated freeze-thaw cycles while critically saturated or in the presence of deicing chemicals is a function of several key factors. In addition to the presence of a finely distributed system of air voids throughout the mortar fraction, concrete strength, curing, and coarse aggregate durability all contribute to the freeze-thaw durability of the concrete. Recognizing the moderate improvement to freeze-thaw durability that occurs with increasing strength, ACI 318 allows for up to a 1 percent reduction of the permissible air content when the specified compressive strength of the concrete (f_c') exceeds 35 MPa (5000 psi).

When entrained air is genuinely needed, the size and spacing characteristics of the entrained air voids in the mortar fraction is much more critical than the total volume of air in the mixture. Industry recommendations[6] suggest air-void spacing factors should be no more than 0.2 mm (0.008 in) and the air voids should be small with a specific surface of at least 24 mm^2/mm^3 (600 in^2/in^3). It is desirable to achieve these values with a minimum total volume of air because strength commonly decreases as the air content increases (Jana *et al*., 2005). Philleo (1986) discusses durable high-strength concretes, including concretes with air contents below 4 percent and spacing factors greater than 0.20 mm (0.008in). In the case of concretes produced at lower W/B ratios and containing HRWRs, research findings have been mixed. One study suggests that properly air-entrained concretes containing HRWRs can have adequate freeze-thaw resistance at calculated spacing factors greater than the industry recommended maximum spacing factor of 0.20 mm (0.008 in). Test data for concretes made with and without high-range water-reducing admixtures showed that virtually all the concretes with adequate resistance to freezing and thawing had specific surface values less than the industry recommended minimum (Attiogbe *et al*., 1992). However, Siebel (1989) found that when high-range water-reducing admixtures were used in a high workability air-entrained concrete, the number of smaller diameter pores decreased, while the content of larger diameter pores and the spacing factor increased. Small pores coalesced and formed larger pores. Although the total air content of the fresh concrete was within the permissible range, the concrete sometimes had a spacing factor above 0.20 mm (0.008 in) (ACI 363R-92, 2007). For this reason, concrete with superplasticizers did not always have adequate freezing and thawing resistance. Prior to actual use, caution should be exercised and verification freeze-thaw testing using the proposed constituent

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materials should be performed. If a justified need for air entrainment exists, extreme caution should be exercised when proportioning, producing and placing high-strength concrete. Otherwise, air entrainment in high-strength concrete should be avoided at all cost. This subject is further addressed in Chapter 4.

Note that some cement-admixture combinations tend to entrain air, in which case, more than the amount estimated during the initial proportioning calculations might actually be produced. Material combinations that tend to increase air contents should be avoided when producing high-strength concrete.

Workability

The terms *slump* and *slump flow* are frequently used interchangeably with *workability*. Slump and slump flow are terms used to describe the consistency of concrete (i.e. stiff, plastic, normal, flowable, and fluid). It would not be difficult to show that two concretes, produced using different constituents yet having equivalent slump or slump flow (spread) values could respond in considerably differently ways when attempting to work. Adjustments intended to enhance workability may work well with one mixture, yet produce an opposite effect with the other.

The definition of workability has been debated between scientists and engineers for several decades: workability generally refers to the consistency, flowability, pumpability, consolidation ability, and harshness of a mixture. Several tests have been developed to assess workability, including the slump, flow table, compacting factor, Vebe consistometer, and Kelly ball penetration test. Although these methods are useful as quality control tools, they are largely qualitative measures based on arbitrarily defined scales (Saaka *et al*., 2004). The rheological properties of fresh concrete—namely yield stress and plastic viscosity, can be used to predict behavior under different workability conditions. This is particularly important for high-strength concretes, which are typically produced for flowing or self-consolidating consistencies. Chidiac *et al*. (2006) reported good correlation between slump flow measurements and yield stresses predicated using most analytical and empirical rheological models.

Designated acceptance age

Increasing the cementitious materials content merely to achieve an arbitrarily imposed 28-day strength requirement can be counterproductive to both the long-term mechanical and durability properties, including creep and shrinkage, particularly if the structure might not require the strength for several months or years. Historically, there has been reluctance on the part of many specifiers to permit acceptance ages beyond 28 days citing concerns that if a strength problem existed, it could go undiscovered for long periods.

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A discussion about this topic presented in Chapter 5 addresses this very legitimate concern and suggests the establishment of indicator or target "flags" at earlier ages, such as 7 and 28 days. The designated acceptance age for concrete requires compliance with not only the specified compressive strength, but also the necessary overdesign factor in order to satisfy the required average strength necessary for compliance with building codes such as ACI 318. The example given in Chapter 5 was if the specified compressive strength of a high-strength concrete was 85 MPa (12,000 psi) at 56 days, the specifier might require no less than 75 percent and 85 percent of specified strength be attained no later than 7 and 28 days, respectively. In the event that the target strength is not attained at these ages, remediation procedures would be required.

ACI 318 code requirements for strength acceptability

Concrete structures cannot be designed based on average strength. If so, about half of the concrete tested would have measured strengths that fall *below* the specified value, which, of course, would be unacceptable. Conversely, since strength results tend towards a generally normal distribution, it would be unrealistic and unduly burdensome to require that *all* concrete strength results be above the specified value. Therefore, it becomes necessary to identify what would constitute an acceptable percentage of specimens that fall below the specified value. Once this percentage is identified, and knowing (or assuming) the standard deviation in strength that can be expected, it would then be possible to calculate the required average strength which can be used as a basis for designing mixtures (Mindess and Young, 1981).

In order for an established mixture to be considered acceptable according to ACI 318 Building Code for Structural Concrete,[7] two statistical requirements must be satisfied based on the last 30 test results:

- there is a 1-in-100 (or higher) probability that three consecutive compressive strength tests is below specified strength (f_c'); and
- there is a 1-in-100 (or higher) probability that a single compressive strength test (average of two cylinders) is more than 10 percent below the specified compressive strength (f_c').

When data are available to establish a sample standard deviation (s_s) for concrete having a specified compressive strength with a magnitude greater than 35 MPa (5000 psi), the required average compressive strength (f_{cr}') shall be the larger value computed from the following two equations(ACI 318 Tables 5.3.2.1):

$$f_{cr}' = f_c' + 1.34s_s$$
$$f_{cr}' = 0.90\, f_c' + 2.33s_s$$

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Note that when less than 30 strength tests are available, the required average strength is determined using sample standard deviation modification factors (ACI 318 Tables 5.3.1.2).

When no data are available to establish a sample standard deviation (s_s) for concrete having a specified compressive strength with a magnitude *greater* than 35 MPa (5000 psi), the required average compressive strength (f_{cr}') shall be computed from the following equation (from ACI 318 Table 5.3.2.2):

$$f_{cr}' = 1.10\, f_c' + 5 \qquad \text{(MPa)}$$
$$f_{cr}' = 1.10\, f_c' + 700 \qquad \text{(psi)}$$

During the course of the work, the strength level of an individual class of concrete having a specified compressive strength with a magnitude greater than 35 MPa (5000 psi) shall be considered satisfactory if both of the following requirements are met:

- every arithmetic average of any three consecutive strength tests equals or exceeds f_c'; and
- no individual strength test (average of two cylinders) falls below f_c' by more than 0.10 f_c'.

Trial evaluation

Experience has shown that, where historical data are not available, development of an optimum high-strength concrete mixture requires a much larger number of trial batches than with conventional concrete (Blick *et al.*, 1974; Cook, 1982; Russell, 1999). A laboratory trial-batch program is a highly effective method for determining concrete properties and establishing mixture proportions. Careful attention is required during the trial-batch program to assure that materials and proportions selected will perform satisfactorily under field conditions. Cook (1989) described the laboratory program that was used for developing 70 MPa (10,000 psi) at 56 days for the exterior columns of the 72-story InterFirst Plaza building in Dallas. Because of limited experience at the time with the use of the high calcium (Class C) fly ash for high-strength concrete, comprehensive studies and tests were made to determine material properties and economical mixture proportions.

Trial batches can be tested according to standardized conditions, such as ASTM C 192,[8] or in a manner representative of the anticipated job conditions. For purposes of constituent material evaluation, standardized testing may be preferred; however, prior to use in the work, trial batches representative of actual job conditions should ultimately be performed. Trial batches should be conducted at the anticipated temperatures. This is particularly important for mixtures containing combinations of cementing

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materials and chemical admixtures to identify the presence of incompatible materials. Trial conditions should reproduce the mixing, agitating, and delivery time conditions anticipated during the work. Consistency (slump or slump flow), setting time, and batch temperature should be monitored for the duration of the testing period.

When obtaining material samples for laboratory testing, it is suggested that at least 50 percent more material than theoretically required be obtained in case any batches need to be discarded. All samples initially evaluated should represent the "average" characteristics of the material. Samples believed to represent "best" or "worst" cases should not be initially examined. The detrimental effects of materials known to vary in quality will only be amplified if used to produce high-strength concrete. It would not be difficult for a laboratory study to evolve into something that resembles more of a research project, so when planning a study, focus should always be maintained on the primary and most important objectives of the program. Since this book is a guide for practitioners, given the array of products available for making modern concrete, it would be worth remembering that it does not take too much effort to devise a laboratory study that can start to resemble a large research project. Gutiérrez and Cánovas (1996) carried out an experimental program to identify relevant properties and establish specifications for constituent materials for high-performance concrete mixtures.

Laboratory trial batches do not perfectly replicate field conditions. Fresh and hardened properties achieved in the laboratory are sometimes different from those achieved in full-scale production. Therefore, after the work has been completed in the laboratory, production-sized batches are recommended.

As beneficial as a slower rate of hydration can be for high-strength concrete, it obviously would not be beneficial to a project schedule to retard the setting or strength gaining properties to an unnecessarily high degree. The suggestion would be to use set retarding or hydration-controlling admixtures in order to resist early stiffening and lower ultimate strength that would be expected to occur had hydration not been effectively controlled. Controlling hydration using retarding and hydration-controlling admixtures is critically important during hot weather periods. Compared to producing prescriptive concrete day in and day out, having the flexibility to switch between neutral set and set-controlling admixtures can maintain more consistent performance. Conversely, by not having the ability to control hydration when environmental conditions warrant, greater variations in mixture performance should be anticipated.

As an alternative to evaluating concrete simply on a trial and error basis, several, more efficient practical methods exist for evaluating the compatibility of material combinations at various temperatures, including hydration profiling of paste samples in a conduction calorimeter, and early stiffening of lab prepared mortars (ASTM C 359).[9] Often, the most effective admixture

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type and dosage is determined through trial and error; therefore it is suggested that the proposed combinations of cementitious materials and chemical admixtures be evaluated prior to their actual use.

There is a wide range of cementitious materials and chemical admixture types that have been successfully used to produce high-strength concrete. To demonstrate, Table 3.1 presents various paste compositions that conceptually might comprise high-strength concrete. It cannot be over-stressed that the constituents for making high-strength concrete should never be viewed as commodities. The quality of cements, pozzolans, and chemical admixtures will vary; therefore a systematic trial evaluation program is integral in the mixture development process. A material or material combination found to be suitable for conventional concrete is no guarantee that it will perform favorably in high-strength concrete.

Proportioning high-strength concrete: an example

To work through this example will require a copy of ACI 211.1–91.[10] This example involves proportioning a non-air entrained high-strength concrete for a series of interior, non-exposed building columns. The structure has been designed according to ACI 318–05. The specified compressive strength (f_c') is 70 MPa (10,000 psi) at 56 days. The concrete producer has previous experience successfully making concrete with specified strengths up to 65 MPa (9500 psi) at 56 days. The contractor has indicated that they would like to place this concrete at a 400 to 500 mm (16 to 20 in) slump spread.

Table 3.1 Example constituent material combinations for pastes of varying W/B ratios

Specified compressive strength, MPa [psi]	*< 35 [5000]*	*35–55 [5000–8000]*	*55–80 [8000–11,500]*	*80–120 [11,500–17,500]*
W/B ratio	> 0.45	0.45–0.35	0.35–0.29	0.29–0.25
Chemical admixture # 1	Optional	WRA or HCA	HCA	HCA
Chemical admixture # 2	Optional	WRA or HRWR*	HRWR*	HRWR*
SCM # 1	Optional	Fly ash or GGBS	Fly ash or GGBS	Fly ash or GGBS
SCM # 2	Not necessary	Not necessary	Optional	SF or MK**

* WRA = Water reducing admixture; HCA = hydration-controlling admixture; HRWR = high-range water reducer

** SK = Silica fume; MK = Metakolin

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General considerations

Given the exposure conditions and specified concrete properties, the governing property that will be used to establish this mix design will be the specified compressive strength. Had the producer not been experienced in making concrete of similar strength, a series of laboratory tests profiling each proposed constituent material would be suggested. Given the marginal increase in specified strength and previous success making 65 MPa (9500 psi), they have decided to run their laboratory trials using the same constituent materials. The combination of Portland cement and fly ash proposed has excellent strength development potential. During the development of the 65 MPa (9500 psi) concrete, it was determined that the 56-day compressive strength could be optimized at very low W/B ratios (< 0.35) when the fly ash comprises about 50 percent of the total cementitious material (percent by mass). However, at most working temperatures, strength development during the first 36 to 48 hours is objectionably low when 50 percent fly ash is used in combination with a high-range water reducer. Based on the anticipated time of year that the construction is going to take place, it has been decided that fly ash will comprise 30 percent by mass of the cementitious material. When used at 30 percent, the average water reduction from the fly ash is about 8 percent. The initial chemical admixture dosages will be the standard rates recommended by the admixture manufacturer. At the dosages planned, water reductions from the retarding and high-range water-reducing admixtures are anticipated to be about 5 percent and 20 percent, respectively.

The desired slump prior to the introduction of high-range water-reducer will be 25 to 50 mm (1 to 2 in). Therefore, excluding the HRWR, the estimated water reduction from the use of 30 percent fly ash and standard dosage of retarding admixture is anticipated to be about 13 percent. The quantity of water contained in the high-range water-reducer will be included when calculating the required water content during trial batching. The candidate materials selected for initial laboratory trials are listed in Table 3.2.

Material properties

Portland cement	Bulk specific gravity: 3.15
Fly ash	Bulk specific gravity: 2.72
Coarse aggregate	Clean, well-graded, well-shaped crushed limestone Dry rodded density (unit weight): 1630 kg/m^3 (102 lb/ft^3) Bulk specific gravity (saturated surface dry): 2.68 Absorption 0.4% Total moisture 0.9%
Fine aggregate	Clean, uniformly graded blend of natural sand and crushed manufactured limestone

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Table 3.2 Constituent materials used in first series of laboratory trials

Material	*ASTM specification*	*Description*
Portland cement	C 150	Type I
Fly ash	C 618	Class C (high calcium)
Coarse aggregate	C 33	No. 8: 9.5 to 2.36 mm (⅜ to No.8)
Fine aggregate	C 33	Concrete sand
Water	C 1602	Potable
Retarding water reducer	C 494	Type D
High-range water-reducer	C 494	Type F

	Fineness modulus 2.80 Bulk specific gravity (saturated surface dry): 2.62 Absorption 0.7% Total moisture 5.5%
Water	Potable
Retarding water reducer	Initial dosage 225 L/100 kg (3.5 oz/cwt)
High-range water reducer	Initial dosage 650 L/100kg (10 oz/cwt)

Solution (SI units)

Calculation of required average strength

The sample standard deviation at 56 days for the 65 MPa concrete has been about 8 MPa. Since no data are available to establish a sample standard deviation (s_s) for the 70 MPa, using ACI 318 Table 5.3.2.2, the required average strength f_{cr}' shall be:

$$f_{cr}' = 1.10\, f_c' + 5$$
$$= 1.10\,(70) + 5$$
$$= 82 \text{ MPa}$$

Estimated water and air content

The quantity of water is estimated from Table A1.5.3.3 of ACI 211.1 and then reduced by 10 percent:

For 9.5 mm nominal maximum sized coarse aggregate, 25 to 50 mm slump, non-air-entrained concrete:

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Estimated water (from ACI table):	207 kg
Less 10%	– 21 kg
Revised water	186 kg

Adjusting for a combined 13 percent water reduction, the revised estimated water content per cubic yard will be:

$186 \times (100 - 13)/100 = 162$ kg

The air content estimated from Table A1.5.3.3 of ACI 211.1 is 3 percent. Since this will be flowing concrete produced with HRWR, the air content will be reduced by 0.5 percent. Therefore, in this example, 2.5 percent will be used. Ultimately, the specific materials used and the manner in which they interact will determine the air content and further adjustments may be necessary.

Target W/B ratio

Using Figure 3.1, the feasible W/B ratio range for achieving an f_{cr}' of 82 MPa is approximately 0.29 to 0.35. Given the cementing efficiency of the proposed combination of Portland cement and fly ash based on previous experience, a target W/B ratio of 0.32 has been chosen for the initial laboratory trials.

Estimated binder content

B = 186/0.32 = 581 →	Try B = 580 kg
Portland cement content =	$0.7 \times 580 = 406$ kg
Fly ash content =	$0.3 \times 580 = 174$ kg

Coarse aggregate content

The quantity of coarse aggregate is estimated from Table A1.5.3.6 of ACI 211.1. For a fine aggregate having a fineness modulus of 2.80 and a coarse aggregate having a 9.5 mm nominal maximum size, the indicated volume of coarse aggregate per unit of volume of concrete is 0.46. As previously stated in this chapter, increasing this value by approximately 40 percent is suggested for high-strength concrete. Therefore, the estimated initial quantity of coarse aggregate per cubic meter will be:

Volume = $0.46 \times 1.4 = 0.64$ m^3
Mass = $0.64 \times 1630 = 1043$ kg

Estimated water contributed by the HRWR

The HRWR to be used will be initially dosed at a rate of 650 ml/100 kg of cementitious material. The admixture weighs 1.05 kg/L., 60 percent of which is water.

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per m^3

HRWR volume	$650 \times 5.8 = 3770$ ml
HRWR mass	$3770/(1.05 \times 1000) = 3.6$ kg
Contributed water	$3.6 \times 0.60 = 2.2$ kg → Round to 2 kg
Revised water content	$186 - 2 = 184$

Note: For practical purposes, the amount of water contributed by the retarding admixture is negligible and has been disregarded.

Absolute volume calculations

Material	*Mass (kg)*	*Bulk sp. gr.*	*Absolute volume (m^3)*
Cement	406	3.15	$406/(3.15 \times 1000) = 0.13$
Fly ash	174	2.72	$174/(2.72 \times 1000) = 0.06$
Water	186	1.00	$186/(1.0 \times 1000) = 0.19$
Coarse agg.	1043	2.68	$1043/(2.68 \times 1000) = 0.39$
Air	2.5	n/a	$2.5/100 = 0.03$

Total volume without fine aggregate = 0.80 m^3
Fine aggregate volume = $1 - 0.8 = 0.2$ m^3
Fine aggregate mass = $0.2 \times 2.62 \times 1000 = 524$ kg

Material	*Quantity per m^3*
Cement	406
Fly ash	174
Water	186
Coarse aggregate	1043
Fine aggregate	524
HRWR	3770 ml
Retarder	1305 ml

Theoretical mass per cubic meter[11] = 406 + 174 +186 + 1043 + 524
= 2333 kg

Proportioning a 0.1 m^3 laboratory trial batch

Coarse aggregate free moisture	=	$0.9 - 0.4 = 0.5\%$
Coarse aggregate mass per m^3	=	$1043 + [1043 \times (0.5/100)]$
	=	1048 kg
Coarse aggregate required	=	$1048 \times 0.1 = 105$ kg
Moisture in coarse aggregate	=	$105 - (1043 \times 0.1) = 0.7$ kg
Portland cement required	=	$406 \times 0.1 = 40.6$ kg
Fly ash required	=	$174 \times 0.01 = 17.4$ kg

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Fine aggregate free moisture	=	5.5 – 0.7 = 4.8%
Fine aggregate mass per m^3	=	534 + [534 × (4.8/100)]
	=	560 kg
Fine aggregate required	=	560 × 0.1 = 56.0 kg
Moisture in fine aggregate	=	56.0 – (534 × 0.1) = 2.6 kg
Batch Water	=	(186 × 0.1) – 0.7 – 2.6
	=	15.3 kg
HRWR required	=	3770 × 0.1 = 377.0 ml
Retarder required	=	1305 × 0.1 = 130.5 ml

Solution (inch-pound units)

Calculation of required average strength

The sample standard deviation at 56 days for the 9500 psi concrete has been about 1150 psi. Since no data are available to establish a sample standard deviation (s_s) for the 10,000 psi, using ACI 318 Table 5.3.2.2, and the required average strength f_{cr}' shall be:

$$f_{cr}' = 1.10\, f_c' + 700$$
$$= 1.10\ (10{,}000) + 700$$
$$= 11{,}700 \text{ psi}$$

Estimated water and air content

The quantity of water is estimated from Table 6.3.3 of ACI 211.1 and then reduced by 10 percent:

For ⅜ in coarse aggregate, 1 to 2 in. slump, non-air-entrained concrete:

Estimated water (from ACI table):	350 lb
Less 10%	– 35 lb
Revised water	315 lb

Adjusting for a combined 13 percent water reduction, the revised estimated water content per cubic yard will be:

$$315 \times (100 - 13)/100 = 274 \text{ lb}$$

The air content estimated from Table 6.3.3 of ACI 211.1 is 3 percent. Since this will be flowing concrete produced with HRWR, the air content will be reduced by 0.5 percent. Therefore, in this example, 2.5 percent will be used. Ultimately, the specific materials used and the manner in which they interact will determine the air content and further adjustments may be necessary.

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Target W/B ratio

Using Figure 3.1, the feasible W/B ratio range for achieving an f_{cr}' of 11,700 psi is approximately 0.29 to 0.35. Given the cementing efficiency of the proposed combination of Portland cement and fly ash based on previous experience, a target W/B ratio of 0.32 has been chosen for the initial laboratory trials.

Estimated binder content

B = 274/0.32	=	856 lb	→	Try B = 860 lb
Portland cement content	=	0.7 × 860	=	602 lb
Fly ash content	=	0.3 × 860	=	258 lb

Coarse aggregate content

The quantity of coarse aggregate is estimated from Table 6.3.6 of ACI 211.1. For a fine aggregate having a fineness modulus of 2.90 and a coarse aggregate having a ⅜ in nominal maximum size, the indicated volume of coarse aggregate per unit of volume of concrete is 0.46. As previously stated in this chapter, increasing this value by approximately 40 percent is suggested for high-strength concrete. Therefore, the estimated initial quantity of coarse aggregate will be:

per yd^3

Volume	=	27 × [0.46 × 1.4]	=	17.4 ft^3
Mass	=	17.4 × 102	=	1775 lb

Estimated water contributed by the HRWR

The HRWR to be used will be initially dosed at a rate of 10 oz/cwt[12] of cementitious material. The admixture weighs 8.8 lb/gal, 60 percent of which being water:

per yd^3

HRWR volume	10 × 8.6	=	86.0 oz
HRWR mass	86.0/128 × 8.8	=	5.9 lb
Contributed water	5.9 × 0.60	=	3.5 → Round to 4 lb
Revised water content	274 – 4	=	270 lb

Note: For practical purposes, the amount of water contributed by the retarding admixture is negligible and therefore, has been disregarded.

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Absolute volume calculations

Material	*Mass (lb)*	*Bulk sp. gr.*	*Absolute volume (ft³)*
Cement	602	3.15	602/(3.15 × 62.4) = 3.06
Fly ash	258	2.72	258/(2.72 × 62.4) = 1.52
Water	270	1.00	270/(1.00 × 62.4) = 4.33
Coarse agg.	1775	2.68	1775/(2.68 × 62.4) = 10.61
Air	2.5	n/a	27 × 2.5/100 = 0.68

Total volume without fine aggregate = 20.20 ft^3
Fine aggregate volume = 27.00 – 20.20 = 6.80 ft^3
Fine aggregate mass = 6.80 × 2.62 × 62.4 = 1112 lb

Material	*Quantity per yd³*
Cement	602 lb
Fly ash	258 lb
Water	270 lb
Coarse aggregate	1775 lb
Fine aggregate	1112 lb
HRWR	86.0 oz
Retarder	25.8 oz

Theoretical weight per cubic yard[13] = 602 + 258 + 270 + 1775 + 1112 = 4017 lb
Theoretical fresh unit weight = 4017/27.0 = 148.8 lb/ft^3

Proportioning a 2.0 ft^3 laboratory trial batch

Trial batch factor for 2.0 ft^3 batch = 2/27 = 0.074
Coarse aggregate free moisture = 0.9 – 0.4 = 0.5%
Coarse aggregate mass per yd^3 = 1775 + [1775 × (0.5/100)]
= 1784 lb
Coarse aggregate required = 1784 × 0.074 = 132.0 lb
Moisture in coarse aggregate = 132.0 – (1775 × 0.074)
= 0.65 lb
Portland cement required = 602 × 0.074 = 44.5 lb
Fly ash required = 258 × 0.074 = 19.09 lb
Fine aggregate free moisture = 5.5 – 0.7 = 4.8%
Fine aggregate mass per yd^3 = 1112 + [1112 × (4.8/100)]
= 1165 lb
Fine aggregate required = 1165 × 0.074 = 86.2 lb
Moisture in fine aggregate = 86.2 – (1112 × 0.074)
= 3.91 lb
Batch water = (270 × 0.074) – 0.65 – 3.91
= 15.4 lb
HRWR required = 86.0 × 0.074 = 6.36 oz
Retarder required = 25.5 × 0.074 = 1.89 oz

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Notes

1 *Evaluation of Strength Test Results of Concrete*.
2 ACI 214R-02, Tables 3.2 and 3.3.
3 *Building Code Requirements for Structural Concrete*.
4 Skew (or "skewness") refer to the degree of asymmetry of a distribution.
5 Kurtosis implies greater variance is due to infrequent extreme deviations.
6 Described in ASTM C 457 *Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete*.
7 2005 version [ACI 318–05 (psi), ACI 318M–05 (MPa)].
8 *Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory*.
9 *Standard Test Method for Early Stiffening of Hydraulic Cement (Mortar Method)*.
10 *Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete*.
11 Mass of chemical admixtures was negligible.
12 Ounces per 100 lb of cementitious material.
13 Mass of chemical admixtures was negligible.

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4 Properties

Introduction

There are two fundamental distinctions between conventional-strength and high-strength concrete technology. First is the exchange in the relative strength and stiffness properties between paste and aggregate. On the low end of the strength spectrum, aggregate particles are bound by a weaker, more porous material. On the high end, aggregate particles are bound by a stronger, dense material. Going from conventional-strength to high-strength concrete technology is tantamount to turning a composite material inside out. The second distinction centers on the properties of the interfacial transition zone. Bond strength and degree of stiffness compatibility between binder and aggregate is critically important with high-strength concrete.

Important mechanical properties of normal-weight, high-strength concrete usually include compressive strength, modulus of elasticity, creep, and shrinkage. Depending on the type of concrete or structure, the modulus of rupture, splitting tensile strength and Poisson's ratio may also be essential design parameters. In applications where volume changes and cracking can impair service life, durability-related properties must also be scrutinized. Important durability-related properties often include resistance to alkali-aggregate reactions, sulfate attack, corrosion of embedded metals, and freeze-thaw durability.

Mechanical properties

Being a two-component composite material consisting of paste and aggregate, it is understandable that the mechanical properties of concrete are highly dependent on the relative properties of these two materials. Overall, this and the manner in which bond at the interfacial transition zone is affected is probably the most important, but still underestimated characteristics influencing the service life of most concrete structures. Neville (1997) discusses how bond at the interfacial transition zone and modulus of elasticity are related, but nonetheless, treated separately.

Compressive strength is the common basis for the design of nearly all concrete structures other than pavements, but even then, compressive

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strength is often the common method of routine quality testing (Zia *et al.*, 1997). Mechanical concrete properties such as tensile strength, shear strength, modulus of rupture, bond strength, and stress–strain relationships are normally expressed in terms of compressive strength. Since the laws governing the different mechanical properties of concrete vary, extreme caution should be exercised when attempting to extrapolate relationships that work well for conventional-strength concrete to high-strength concretes. The availability of data for higher-strength concretes requires a reassessment of design equations to determine their applicability with higher-strength concretes (ACI 363R-92, 2007).

Axial stress vs. strain

The stress–strain behavior of concrete is primarily influenced by the relative stiffness of the paste and aggregates, and the bond strength at the interfacial transition zone. All else equal, higher interfacial bond strength is achieved using rough as opposed to smooth textured aggregate. Therefore, for two coarse aggregates of the same size, shape, mineralogy, and stiffness, higher strength (and corresponding strain capacity) would be achieved using crushed stone compared to smooth gravel. Various investigators (Shah *et al.*, 1981, Jansen *et al.*, 1995) have reported higher strain capacities at maximum stress for high-strength compared to conventional-strength concretes. Curves representing typical stress–strain relationships for high, moderate, and conventional-strength concretes are shown in Figure 4.1. As strength increases, the slope of both the ascending and descending portions of the stress–strain curve becomes steeper and ultimate failure in compression becomes increasingly more explosive (Figure 4.2). Therefore, for high-strength concretes, accurate determination of the descending portion of the curve can be difficult to obtain (Wang *et al.*, 1978, Holm, 1980, Shah *et al.*, 1981) and there are yet no established standards for obtaining the complete stress–strain curves for concrete. Since the descending branch is dependent on the test method employed, the stress–strain curve is best used strictly for comparative purposes only.

Modulus of elasticity

Static modulus of elasticity

Few topics are capable of instigating more debate among high-strength concrete authorities than modulus of elasticity. Although it is common to think about the elastic modulus of concrete as a single concrete property, in actuality, concrete has *two* elastic moduli—the elastic modulus of paste and the elastic modulus of aggregate. At the interface between the two materials is the paste-aggregate interfacial transition zone, perhaps the most important factor influencing the mechanical properties of high-strength concrete.

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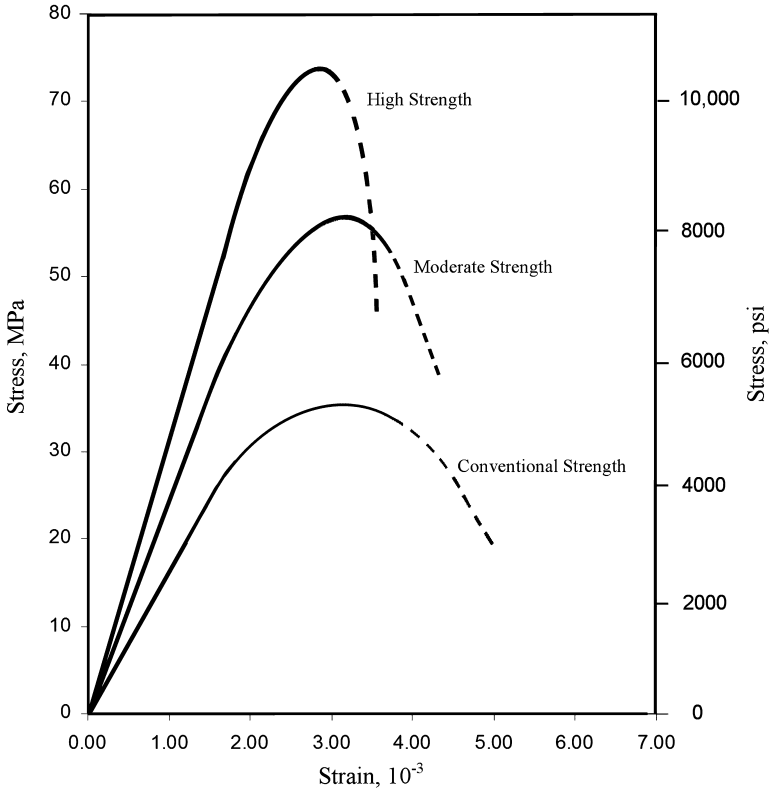

Figure 4.1 Typical stress–strain relationship for high-, moderate-, and conventional-strength concrete.

Although concrete is not considered a perfectly linear-elastic material, Hooke's law of elasticity is applicable to structural concretes for the range of strains commonly used in design calculations. Modulus of elasticity (Young's Modulus) is one of the most important mechanical properties of concrete. Modulus of elasticity is defined as the ratio of normal stress to corresponding strain for tensile or compressive stresses below the proportional limit of a material. It is a key factor influencing the structural performance of reinforced concrete structures and is particularly important as a design parameter in predicting the deformation of tall buildings.

The modulus of elasticity of concrete is largely governed by the properties of the coarse aggregate. Increasing the size of coarse aggregates or using stiffer coarse aggregates with a higher modulus of elasticity increases the modulus of elasticity of the concrete. Being a composite material composed of paste and aggregate, the modulus of elasticity of concrete in compression is closely related to the mechanical properties of the paste relative to that

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Figure 4.2 As compressive strength increases, failure takes on an increasingly explosive mode. Courtesy of CTLGroup.

of the aggregate particles. It should be noted that while stiffer or denser aggregates improve the elastic modulus of the concrete, they are also capable of introducing stress concentrations at the transition zone and subsequent microcracking at the bond interfaces, thus reducing the ultimate compressive strength capacity of the concrete.

As the elastic moduli of paste and aggregate particles approach each other, the resulting concrete tends to exhibit a more linear stress–strain relationship and increased brittleness (Neville, 1997). Two models representing the two boundaries of behavior of composite materials are discussed (Hansen, 1958). The first model, an *ideal composite hard material*, has filler particles of a low modulus of elasticity bound together by an elastic phase matrix having a high modulus of elasticity. The second model, an *ideal composite soft material*, has filler particles of high modulus of elasticity bound together by an elastic phase matrix having low modulus of elasticity. Of the two idealized models, high-strength concretes would more closely

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fit the first model, whereas, conventional-strength concretes would more closely fit the second.

A significant difference in behavior with respect to the early strength of high-strength concretes is in the relationship of compressive strength to other mechanical properties. Typically, compressive strength increases at a faster rate than does the bond strength at the interfacial transition zone. This will lead to proportional differences in the modulus of elasticity and tensile strength at early versus later ages. Therefore, the proportionality of mechanical properties to later age compressive strength (28 days or later) of high-strength concrete cannot be expected to apply as it does with conventional-strength concretes.

Myers (1999) investigated various methodologies for increasing modulus of elasticity. Higher modulus of elasticity values are typically achieved using coarse aggregate sizes larger than what would produce an optimum compressive strength. Larger sized aggregate allows for the use of higher coarse aggregate volumes, a key parameter for modulus of elasticity, without sacrificing workability, which could suffer if similar volumes of small-sized aggregate were used. In such cases, trade-offs become necessary in order to achieve acceptable mechanical performance. Larger-sized aggregate, though yielding lower compressive strength, could provide a higher modulus of elasticity. Extremely high elastic modulus concretes have been produced using high volumes of stiff coarse aggregate bonded to dense, low W/B ratio paste.

The modulus of elasticity of conventional-strength concrete generally increases proportionally to the square root of the compressive strength. While many empirical equations for predicting modulus of elasticity have been proposed, few equations predict the modulus of elasticity of high-strength concrete as accurately as they do for conventional-strength concrete. ACI Committee 363 reports that the following equation has generally proven to be a reliable lower bound expression for normal density high-strength concrete based on most high-strength concrete data collected:

$$E_c = 40{,}000\,(f_c')^{0.5} + 1{,}000{,}000 \quad (\text{for } 3000 \text{ psi} < f_c' < 12{,}000 \text{ psi})$$
$$E_c = 3320\,(f_c')^{0.5} + 6900 \quad (\text{for } 21 \text{ MPa} < f_c' < 83 \text{ MPa})$$

However, based on recent studies (Gross and Burns, 1999; Myers and Carrasquillo, 1999), the Committee cautions that when this expression is used, significant underestimations can occur. The measured modulus of elasticity is highly sensitive to the moisture content of the test specimen. It is believed that this is due to the effect of drying at the interfacial transition zone. For a given concrete, the modulus of elasticity of specimens tested in a wet condition is about 15 percent higher than specimens tested dry.

Investigators with the Research Committee on High-strength Concrete of the Architectural Institute of Japan (AIJ) performed multiple regression analyses on over 3,000 data where compressive strength and unit weight

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(density) were taken as the explanatory variables and modulus of elasticity as the target variable (Tomosawa and Noguchi, 1995). The compressive strength of the investigated normal density concretes ranged from 20 to 160 MPa (3000 to 23,000 psi). Based on the results, the following equation was proposed:

$$E = k_1 * k_2 * 3.35 * 10^4 * (\gamma/2.4)^2 * (\sigma B/60)^{1/3}$$

where,

k_1 = coarse aggregate correction factor
k_2 = mineral admixture correction factor
γ = unit weight (density), kg/m^3
σ_B = measured compressive strength, MPa.

Figures 4.3a and 4.3b present the measured elastic moduli for the six commercially available high-strength concretes studied by Burg and Ost (1992). In general, the measured modulus of elasticity fell between the values predicted by the equations in ACI 318 and ACI 363. Figure 4.4 displays the 91-day results for cylindrical specimens cured under varying conditions.

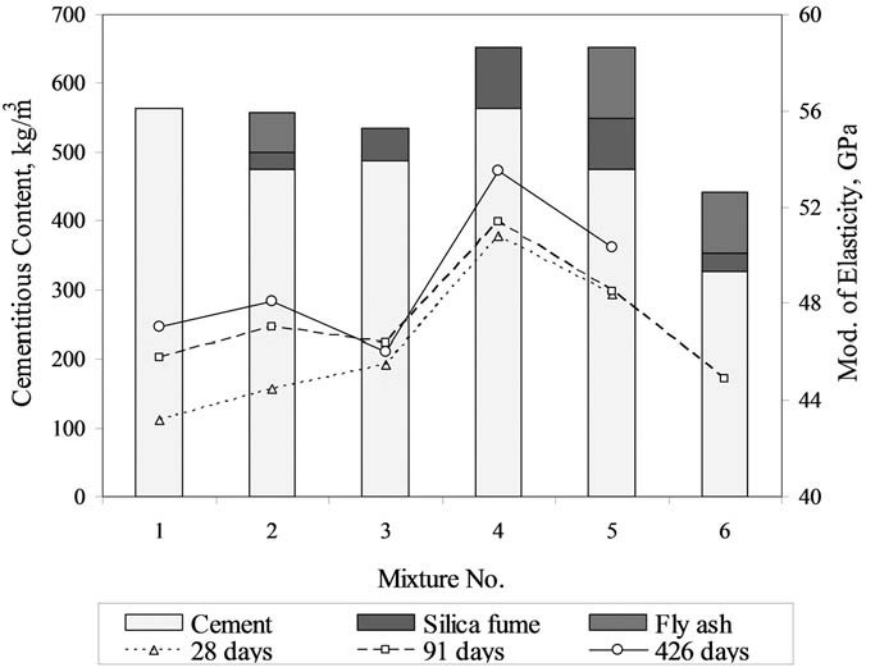

Figure 4.3a (SI units) Measured modulus of elasticity at 28, 91, and 426 days from Burg and Ost (1992) for moist cured 150 × 300 mm cylinders.

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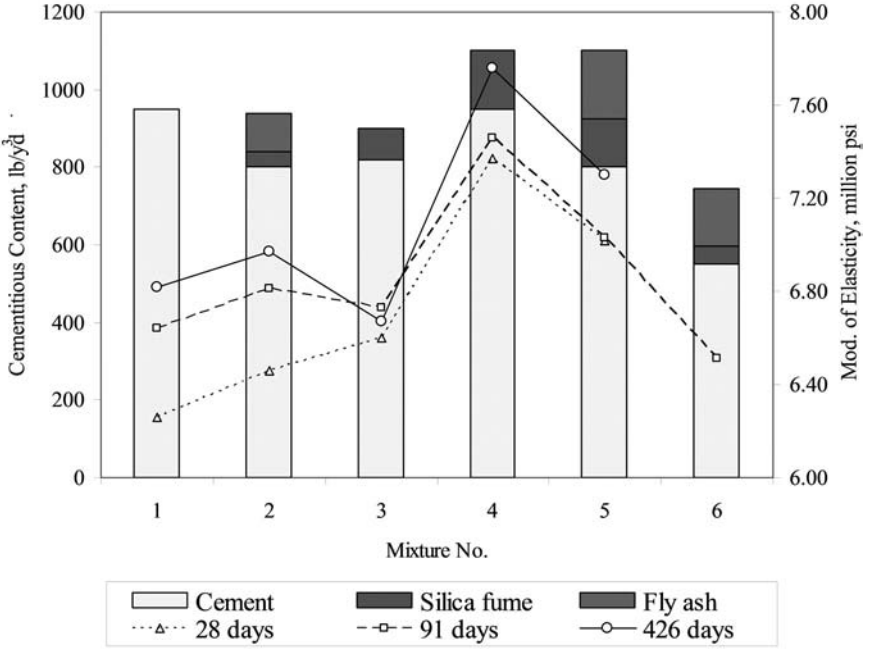

Figure 4.3b (inch-pound units) Measured modulus of elasticity at 28, 91, and 426 days from Burg and Ost (1992) for moist cured 6 × 12 in cylinders.

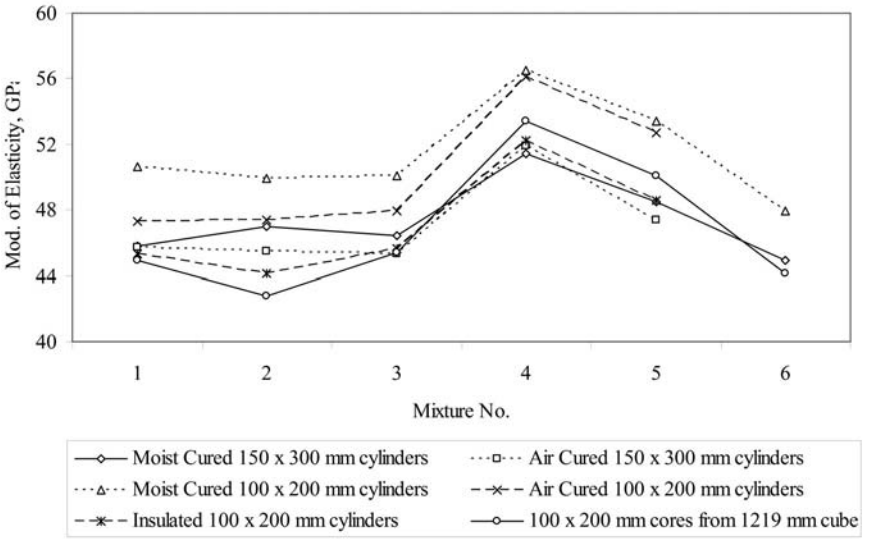

Figure 4.4 Measured modulus of elasticity at 91 days from Burg and Ost (1992) for various size cylindrical specimens cured under varying conditions.

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The nominal maximum aggregate size used in mixtures. 1–5 were 12 mm (½ in) and 25 mm (1 in) in mixture no. 6. Mixtures 1–5 contained 1068 kg/m^3 (1800 lb/yd^3) of crushed dolomitic limestone. Mixture 6 contained 1121 kg/m^3 (1890 lb/yd^3).

At the current time there is little consensus in regards to the applicability of one universal methodology that could accurately predict the modulus of elasticity of high-strength concrete. For structures requiring an accurate knowledge of modulus of elasticity, direct measurement using locally available materials and mix designs is still the best approach. The modulus of elasticity should be determined as early as possible in the design phase; either through a field trial evaluation program or based on previously documented field performance data.

Dynamic modulus of elasticity

Little information is available on the dynamic modulus of high-strength concrete. As Zia *et al.* (1997) described, the measurement of dynamic modulus corresponds to a very small instantaneous strain. The difference between the static and dynamic moduli is due in part to the fact that heterogeneity of concrete affects each differently. For low, medium, and high-strength concretes, the dynamic modulus is generally 40 percent, 30 percent, and 20 percent respectively higher than the static modulus of elasticity (Mehta, 1986). Nilsen and Aïtcin (1992) used the pulse velocity test to predict the static modulus of elasticity of high-strength concrete.

Poisson's ratio

Poisson's ratio under uniaxial loading conditions is defined as the ratio of transverse strain to the corresponding axial strain resulting from uniformly distributed axial stress below the proportional limit of the material. Based on the limited data on values for high-strength concrete, the Poisson's ratio of high-strength concrete in the elastic range of strain seems similar to values for conventional-strength concretes. In the inelastic range, the relative increase in lateral strains is less for higher-strength concrete than for concrete of conventional strength, suggesting less internal microcracking higher-strength concretes. Perenchio and Klieger (1978) reported values for Poisson's ratio of 0.20 to 0.28 for normal-weight high-strength concretes with compressive strengths ranging from 55 to 80 MPa (8000 to 11,600 psi). They concluded that Poisson's ratio tends to decrease with increasing water–cement ratio. Kaplan (1959) found values for Poisson's ratio of concrete determined using dynamic measurements to be from 0.23 to 0.32 independent of coarse aggregate properties, test age, and strength for concretes having compressive strengths ranging from 17 to 80 MPa (2500 to 11,500 psi). Setunge *et al.* (1990) suggested that Poisson's ratio of very high-strength concrete increased with an increase in compressive strength.

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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 that affect the strength are the quality of fine and coarse aggregate, the cement paste and the paste-aggregate bond at the interfacial 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, profoundly influence measured strength and are discussed in Chapter 9.

Compressive strength

The strength development characteristics of high-strength concrete are different from those of conventional-strength concrete. Tests by Wild *et al.* (1995) showed that high-strength concrete with a W/B ratio of 0.35 (without silica fume) had a 7-day compressive strength that averaged 86 percent of the 28-day strength when cured at 20°C (68°F). This same ratio for conventional-strength concrete was in the range 60 to 70 percent. When silica fume was added to the high-strength concrete in the range 12 to 28 percent mass fraction of cement, the average ratio of the 7-day to the 28-day strengths was 76 percent when cured at 20°C (68°F). When the curing temperature was increased to 50°C (122°F), this ratio increases significantly to 97 percent, indicating that high curing temperatures can be very beneficial to early strength development in silica-fume high-strength concrete (Meeks and Carino, 1999). Typically, strength gain in compression is much faster than strength gain in the transition zone bond. Changes in the strength of high-strength concrete over time are driven by two opposing factors—hydration and self-desiccation. Provided free moisture is available to unhydrated cementing particles, they will continue to form hydration products, and strength will continue to increase. Conversely, systems absent of free moisture may self-desiccate, in which case, measured strength over time could conceivably decrease. Conventional-strength concretes, being produced at significantly higher W/B ratios than high-strength concretes commonly continue to increase in strength over time, provided free moisture is present and losses in strength due to self-desiccation is not an issue. Actual decreases in measured long-term strength are not terribly common with high-strength concrete either. Long-term loss in measured strength due to self-desiccation usually becomes a concern only in very high-strength concretes with target compressive strengths of 100 MPa (14,500 psi) or higher. It is not the strength of the concrete per se, but rather the characteristics of the paste that influence the potential for long-term strength loss. On several occasions, the author has observed decreases in measured

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compressive strength about 10 percent from 56 days to one-year ages with concretes produced at W/B ratios below 0.29 and consisting of extremely fine cements or cementing materials.

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, in fact, does not appear to be appreciably larger than conventional concretes. This is probably due to the increase in stiffness of the stronger mixes.

Tensile strength

In addition to influencing the structural properties of concrete, the tensile strength is a major factor affecting concrete's susceptibility to cracking, thus playing a critical role with respect to durability. There are three distinct methods of determining the tensile strength of concrete, either by direct tension or indirectly by splitting tensile or flexure (modulus of rupture). The direct application of a pure tensile force, free of any eccentricities, is difficult to achieve, and as a result, only limited and often conflicting data is available (Zia *et al.*, 1997). No standard tests have been adopted for direct determination of the tensile strength of concrete.

The most commonly used tests for estimating the indirect tensile strength of concrete is splitting tensile (ASTM C 496)[1] and modulus of rupture (ASTM C 78).[2] Both the splitting tensile strength (f_{ct}) and the modulus of rupture (f_r) are related to the compressive strength by the following general expression:

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

For design purposes, the tensile strength of concrete is frequently taken to be 10 percent of the compressive strength; however, the tensile strength of high-strength concrete may not be quite so proportionally high. Dewar (1964) studied the relationship between the splitting tensile strength and the compressive strength of concretes having measured compressive strengths of up to 84 MPa (12,000 psi) at 28 days. He concluded that at low strengths, the splitting tensile strength may be as high as 10 percent of the compressive strength but at higher strengths, it may reduce to 5 percent. He observed that the tensile splitting strength was about 8 percent higher for crushed-rock-aggregate concrete than for gravel-aggregate concrete. In addition, he found that the indirect tensile strength was about 70 percent of the flexural strength at 28 days (ACI 363R-92, 2007).

Strength retrogression

Concretes with different composition and microstructure do not follow the same drying pattern when exposed to air-drying for the same period. In

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some cases, as in the case of a high-strength, silica-fume concrete, a significant moisture gradient can develop at the surface that can result in a compressive stress field within the specimens used to test concrete compressive strength, while in other concrete, this moisture gradient has completely disappeared at the same age. The transient phenomenon affects any type of concrete; however, for a given concrete at a particular time, the intensity of the developed stresses depends on the severity of the drying conditions and on the permeability of the concrete.

De Larrard and Aïtcin (1993) report that it can be demonstrated that when some apparent strength regression is found, the maximum strength regression that can be estimated from this proposed mechanism is equal to twice the tensile strength of concrete. This value is in good agreement with the experimental strength losses reported by some authors.

Shrinkage and creep

Cracking occurs when the tensile stresses developed within concrete exceeds the tensile strength. Aside from overloading, concrete structures can crack due to conditions that induce volumetric changes. Hydraulic cement concrete can change volume with or without the influence of environmental factors. With the exception of concretes containing special shrinkage-compensating cements or additives, cracking due to volumetric expansion is less problematic than cracking due to volumetric reduction.

In addition to instantaneous elastic deformations, concrete undergoes time-dependent deformations that must be considered in design. Creep is defined as the time-dependent strain resulting from an applied load. Shrinkage is the time-dependent strain that occurs in the absence of an applied load. The total strain occurring in a concrete member is the sum of elastic, creep, and shrinkage strains. Upon setting, shrinkage of concrete takes place in two distinct stages—early and later age. Even before standard shrinkage measurements traditionally begin (24 hours following specimen fabrication), volume reductions have already occurred. ACI Committee 209 on Creep and Shrinkage in Concrete suggests the following general equation for predicting shrinkage of concrete at any time[3]:

$$(\epsilon_{sh})_t = [t_a / (f + t_a)] * (\epsilon_{sh})_u$$

where:

$(\epsilon_{sh})_t$ = shrinkage strain at any time t ;
t = time in days;
a = constant, $(0.90 < a < 1.10)$;
f = constant, $(20 < f < 130$ days); and
$(\epsilon_{sh})_u$ = ultimate shrinkage strain, $(415 \times 10^{-6} < (\epsilon_{sh})_u < 1070 \times 10^{-6})$

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Committee 209 suggests the following general equation for predicting the creep coefficient (ratio of creep strain to initial elastic strain) of concrete at any time:

$$v_t = [t_Y/(d + t_Y] * v_u$$

where

v_t = creep coefficient at any time t;
t = time in days;
Ψ = constant, $(0.40 < \Psi < 0.80)$;
d = constant, $(6 < d < 30$ days); and
v_u = ultimate creep coefficient, $(1.30 < v_u < 4.15)$

In a comprehensive study examining the time-dependent properties of high-strength concrete, Mokhtarzadeh and French (2000) tested 268 specimens with 28-day compressive strengths ranging from 55 to 128 MPa (8000 to 18,600 psi). All of the specimens were cast with 445 kg/m^3 (750 lb/yd^3) of cementitious material with a W/B ratio of 0.30. Effects of variations in cementitious material combinations, coarse aggregate types, and curing procedures were included in the study. Findings of this study are discussed in the sections on Later-Age Shrinkage and Creep on pp. 113–114.

It is suggested that concrete shrinkage be categorized in the following manner:

- plastic shrinkage
- early-age shrinkage
- later-age shrinkage.

Plastic shrinkage

High-strength concretes bleed at a slower rate and exhibit less overall bleeding than most conventional concretes; therefore, they are inherently more susceptible to plastic shrinkage cracking. The American Concrete Institute defines plastic cracking[4] as "cracking that occurs in the surface of fresh concrete soon after being placed and while it is still plastic." Plastic shrinkage stresses develop due to the loss of water by evaporation from the surface and by suction when fresh concrete is in contact with absorptive materials, such as dry hardened concrete or a dry sub-base. Slabs are particularly vulnerable to plastic shrinkage given the high amount of exposed surface area in relation to total volume. When surface moisture evaporates at a faster rate than it can be replenished with bleed water, the surface will shrink more than the interior concrete. Susceptibility to plastic cracking is a function of two factors—evaporation rate and bleed rate, and both must be known to determine whether the concrete is at risk of cracking to a reasonable degree of accuracy.

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The forces that cause plastic shrinkage stresses to develop can be mitigated through good concreting practices. Provided proper measures are taken to prevent the dehydration-induced stresses that cause plastic shrinkage from occurring, plastic cracking, in principle should not occur.

Chemical shrinkage, which will be discussed in the next section, begins while concrete is in a still plastic condition. Unlike plastic shrinkage, chemical shrinkage stresses are not isolated to a particular location in the element, and occurs through setting and while the concrete is in a hardened condition. It occurs while concrete is "plastic," but is not considered plastic shrinkage per se. It is not cited as a cause of plastic shrinkage cracking, and therefore, is most appropriately discussed under early age shrinkage.

Early-age shrinkage

Concrete shrinkage is influenced by a number of factors, both internal and external, including environmental conditions, mixture characteristics, and curing practices. The long-term performance of concrete is highly dependant on the properties it develops at an early age and its ability to resist stresses acting upon it; therefore, controlling early-age shrinkage is essential for ensuring long-term durability. Consider early age shrinkage as the volume reduction that occurs during the first 24 hours from when water and cementitious materials come into contact; the approximate time between casting and mold removal of drying shrinkage test specimens when tested according to ASTM C 157.[5]

Upon demolding, initial (zero) readings are taken and length change monitoring commences; however, by that time, appreciable amounts of shrinkage may have already occurred. Early age volume changes have traditionally been disregarded by the designer because it is believed that the magnitude of early age shrinkage was much lower than that of later age shrinkage. Understanding the early age volume changes that can occur with high-strength concrete is of paramount importance. Failure to consider and deal with the stresses that can develop within the first 24 hours of high-strength structural elements can detrimentally affect long-term structural performance. This applies to both design and construction practices, and is a major reason why proper curing procedures are critically important. In addition to drying, other factors such as carbonation can contribute to later age shrinkage, but incorporated into the broader and somewhat misleading term *drying* shrinkage.

Upon the commencement of paste hydration, volume reductions attributable to the hydration reaction, a phenomenon known as chemical shrinkage occurs. Chemical shrinkage occurs because the products of paste hydration occupy less space than the sum of the constituents of the reaction. Most of the chemical shrinkage that occurs in concrete is not macroscopically measurable. The portion of chemical shrinkage that is macroscopically measurable is referred to as *autogenous shrinkage*. No other volume change

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mechanism differentiates conventional-strength concrete from high-strength concrete more than autogenous shrinkage. Autogenous shrinkage is a consequence of self-desiccation; directly influenced by the diameter of the capillary and nanopores in which menisci are developed in the concrete (Tazawa, 1999; Saric-Coric and Aïtcin, 2003). Autogenous shrinkage due to self-desiccation is perhaps more likely in concretes with very low W/B ratios, although there is little data outside indirect evidence with certain high-strength concrete research (Aïtcin and Laplante, 1990). Mather (2001) recognized the need for additional internal curing water in concretes having W/B ratios below 0.40.

Various factors related to the design, material properties, and construction practices influence the likelihood of shrinkage cracking in concrete structures. Unlike drying shrinkage, autogenous shrinkage can occur without evaporation. Autogenous shrinkage is associated with hydration alone and does not include environmental effects due to variations in moisture.

Even at early ages, high-strength, low-permeability concrete is significantly denser than conventional concrete. Therefore, it would be entirely impractical to believe that externally applied curing water alone can supply necessary quantities of moisture into a much denser concrete at a sufficiently fast rate to control early cracking due to self-desiccation. With the increasing interest in the use of concretes that may be at greater risk of early-age cracking, the concept of internal curing is steadily progressing. Internal curing is a mechanism where additional free water throughout the matrix of the paste is available to the hydrating paste. Internal curing can be extremely important in low W/B ratio concretes because it provides a source of available water at a time when it is most critically needed to prevent self-desiccation and subsequent autogenous shrinkage cracking. Internal water is typically supplied by using relatively small amounts of saturated, lightweight, fine aggregates or super-absorbent polymer (Jensen and Hansen, 2002). Benefits of internal curing include increased hydration and strength development, reduced autogenous shrinkage and cracking, reduced permeability, and increased durability (Geiker *et al*., 2004; Lam, 2005).

Chemical shrinkage occurs in the absence of drying; thus, it is impractical to believe water curing would be an effective method for controlling early cracking due to autogenous shrinkage. It would be incorrect to expect necessary amounts of curing water to effectively permeate into the body of dense concrete in sufficient amounts to mitigate cracking that occurs within hours of placement.

Concrete mixes having W/B ratios below 0.40, high volumes of cementitious material, or extremely fine cementitious material, are prime candidates for autogenous shrinkage. In general, for structures designed with compressive strengths of 35 MPa (5000 psi) or higher, autogenous shrinkage simply cannot be ignored. Whether or not it is correct to disregard the role of autogenous shrinkage in the performance of structures designed with conventional-strength concrete is an appropriate topic for future research.

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Autogenous shrinkage due to self-desiccation is perhaps more likely in concretes with very low W/B ratio, although there is little data outside indirect evidence with certain high-strength concrete research (Aïtcin and Laplante, 1990).

Later-age shrinkage

Of course, chemical shrinkage does not cease at 24 hours. Post 24-hour autogenous shrinkage, along with carbonation shrinkage, which occurs with the comingling of hydrated cement products with atmospheric carbon dioxide, are incorporated under the umbrella term *drying shrinkage*. Therefore, drying shrinkage, in the context that the term is often used, also includes other volume reduction mechanisms. Factors strongly affecting drying shrinkage include:

- Cementitious materials
  - — increasing fineness
  - — increasing C_3A content and reactivity
  - — increasing C_3S content and reactivity
  - — increasing alkali
  - — increasing sulfate content
- Aggregate properties
  - — decreasing coarse aggregate volume
  - — decreasing nominal maximum coarse aggregate size
  - — decreasing coarse aggregate stiffness
  - — increasing percentage of thin and elongated pieces
  - — increasing clay content.

Shrinkage is only one of several factors that can influence the potential for cracking, and, as the preceding section discussed, drying shrinkage is not the only form of shrinkage that needs to be addressed. Proportioning a mix design that meets a given specified value for drying shrinkage evaluated according to a test method such as ASTM C 157 will not assure that a greater magnitude of shrinkage will not occur.

Drying shrinkage of concrete is only a fraction of that of neat cement paste. With respect to shrinkage, the aggregate particles in concrete serve two purposes: to dilute the paste and to reinforce it against volume reduction. The elastic properties of aggregate determine the degree of available restraint. Although incorporating larger sized aggregates has its limitations with respect to the strength of high-strength concrete, incorporating better-graded aggregates can help in achieving lower shrinkage. Feldman (1969) observed that concretes low in shrinkage often contain quartz, limestone, granite or feldspar, and concretes containing some fine-grained sandstones, slate, basalt, trap rock and aggregates containing clay showed large shrinkage. Injurious effects on structures built with high shrinkage aggregates include excessive cracking, spalling, and abnormally large deflections.

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Mokhtarzadeh and French's (2000) findings suggest that for the cases studied, concrete compressive strength and composition of cementitious material had no significant effect on drying shrinkage of high-strength concrete mixtures. The drying shrinkage exhibited by the high-strength concrete reference mixtures made with crushed gravels was less than that of the companion mixture made with rounded gravel. Shrinkage strains after 380 days of drying were 565, 485, 469, 443, and 492 micro-strain for heat-cured reference mixtures made with round gravel, crushed river gravel, high absorption limestone, low absorption limestone, and granite coarse aggregates, respectively. Specimens heat-cured at lower temperatures [50°C (120°F)] had slightly higher drying shrinkage strains than companion specimens cured at higher temperatures [65°C (150°F)]. Table 4.1 lists coefficient of thermal expansion values for various aggregates used in structural concrete applications.

Based on the data collected, the following two equations were suggested for predicting the shrinkage strain of high-strength concrete:

Moist-cured concrete: $(\epsilon_{sh})_t = [t/(45 + t)]^*(\epsilon_{sh})_u$
Heat-cured concrete: $(\epsilon_{sh})_t = [t/(65 + t)]^*(\epsilon_{sh})_u$
where $(\epsilon_{sh})_u = 530$ micro-strain

Creep

Creep is the time-dependant strain of concrete under sustained loading. Creep is particularly important in structures where deflections or member shortening must be limited, or when prestress loss must be minimized. Limitations on creep may be imposed for mixture prequalification, but creep is rarely used for routine quality monitoring (Caldarone *et al.*, 2005). Creep testing is conducted on sealed or unsealed specimens. Sealed specimens with an applied stress have volumetric changes due to elastic deformation, basic creep, and autogeneous shrinkage. Sealed specimens without an applied

Table 4.1 Coefficient of thermal expansion of various structural concrete aggregates

	Coefficient of thermal expansion	
Aggregate	$\times 10^{-6}$/°C	$\times 10^{-6}$/°F
Granite	7.4	4.1
Basalt	6.5	3.6
Quartzite, greywacke	11.8	6.6
Quartzitic sandstone	11.8	6.6
Other sandstone	11	6.1
Limestone	5.0 to 11.5	2.8 to 6.4
Air cooled slag	5.5	5.5

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stress deform due to autogeneous shrinkage. Basic creep is the total deformation of a loaded, sealed specimen minus the elastic deformation and autogeneous shrinkage. Unsealed specimens are the most commonly used test method. Unsealed specimens without an applied stress have volumetric changes due to autogeneous and drying shrinkage. The total deformation of unsealed specimens is the result of an applied stress producing an elastic deformation, creep, and shrinkage. Creep includes both basic and drying creep. Shrinkage includes autogeneous and drying shrinkage. Drying creep of a loaded specimen is the total deformation minus the elastic deformation, basic creep, and shrinkage and requires the testing of both sealed and unsealed specimens. Therefore, creep is typically examined as the total of basic and drying creep (Vincent *et al.*, 2004).

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 (Zia *et al.*, 1997).

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. Studies by Collins (1989) on five mix designs having 28-day specified strengths ranging from 60 MPa to 64 MPa (8,700 psi to 9,300 psi) suggested that creep was somewhat less for mixtures with lower paste content and larger sized coarse aggregates. Carette *et al.* (1993) reported a study of high performance concretes with high volume fly ash from sources in the US. 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 (Zia *et al.*, 1997). Creep data for a group of commercially produced high-strength concretes was reported by Burg and Ost (1992). A collection of 13-year creep data on the concretes used for Water Tower Place in Chicago has been reported by Russell and Larson (1989).

Durability properties

Durability is by far the most important concern facing the concrete industry, and it is precisely for this reason that interest in high-performance concrete is steadily increasing. Concrete has traditionally been specified and purchased in terms of compressive strength, and for this reason, strength has been taken as the most important performance attribute of concrete (Wong and Kwan, 2005). It is ironic that given all of the attention paid to strength,

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when it becomes necessary to decommission, repair, or replace a concrete structure; it is usually the result of a durability-related rather than strength-related deficiency. Therefore, it is false to presume that strong concrete will necessarily be durable concrete.

Most durability problems are caused by the infiltration of one or more deleterious substances, such as water, salts, and sulfate-bearing compounds that, over time, cause internal expansions, cracking, and subsequent disintegration. Reducing the permeability of concrete to an effectively low level and restricting the ingress of harmful substances is the single most influential way of improving durability. As the W/B ratio of concrete decreases, so does permeability. Mindess and Young (1981) reported that the water-to-cement (W/B) ratio was the single parameter that had the largest influence on concrete durability.

An excellent review of the pore structure and its influence on the 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 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. 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 (gas) permeability, hydraulic permeability, and chloride ion permeability. A comprehensive review of different methods for measurement of permeation properties of concrete on site has been presented by Basheer *et al.* (1993).

Because of the extremely broad nature of the term "durability," there is no standardized method of measurement. Providing an effectively low coefficient of permeability to the ingress of injurious materials is a critically important first step, but low permeability alone does not always ensure satisfactory durability. The durability of a concrete structure depends on several factors, including the adequacy of the design, the durability potential of the concrete produced and delivered, and the construction practices employed, from initial placement through final curing. It is unlikely that high quality concrete alone can overcome design and construction-related deficiencies. For example, in an aggressive environment, low permeability offers considerable long-term durability potential, provided the member remains uncracked. Unanticipated cracking, whether design, construction, or material-related, can represent potentially serious breaches to long-term durability.

Permeability

Generally, three categories of concrete permeability are of interest: permeability to gases, liquids, and chloride ions. This book addresses only the latter two categories. Basheer *et al.* (1993) present a comprehensive review

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of various methods available for measurement of all three categories of permeation properties of concrete on site.

The rate of chloride ion penetration is usually determined by applying Fick's law, taking diffusivity as the age dependent variable. 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.

Resistance to freezing and thawing

Concrete's resistance to repeated cycles of freezing and thawing while critically saturated or in the presence of deicing chemicals is usually stipulated in terms of the total air content of the concrete based on maximum aggregate size and exposure severity. The resistance of low water-binder ratio high-strength concretes to freezing and thawing was investigated by Kashi and Weyers (1989). Specimens from 27 batches of air-entrained and non-air-entrained concrete with and without silica fume at W/B ratios of 0.25 and 0.32 were examined. Freeze/thaw tests were conducted in accordance with ASTM C 666[6] Procedure A.[7] To determine the influence of curing, a second set of identical specimens were moist cured for 28 days instead of the prescribed 14-day moist curing period. The results suggested that non-air-entrained high-strength concrete with W/B ratios of less than 0.30 was frost resistant regardless of the length of curing time. Non-air-entrained high-strength concrete with a W/B ratio of 0.32 was durable provided that silica fume was not used. The freeze/thaw resistance of non-air-entrained high-strength concrete produced at a 0.35 water-binder ratio and 10 percent silica fume (by mass 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 onset of freezing and thawing cycles. The findings similarly suggested that non-air entrained concrete with a W/B ratio of 0.35 and containing 10 percent silica fume were not resistant to rapid freezing and thawing when tested in accordance with ASTM C 666 (Procedure A), even when curing had been extended to 56 days (Zia *et al.*, 1997). Favorable resistance to freezing and thawing has been found when the supplementary cementing material metakaolin is used to produce high-strength concrete (Caldarone *et al.*, 1994). The results of research by Pinto and Hover (2001) on the freeze-thaw resistance of high-strength concrete

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indicated that air entrainment might not be necessary for mixtures with W/B ratios less than 0.35. The author is unaware of any documented problems involving freeze-thaw damage of non-air entrained high-strength exterior exposed vertical elements (i.e. columns and walls).

Freeze/thaw resistance and air entrainment go hand in hand. As noted in Chapters 2 and 3, air entrainment can profoundly impair the ability to achieve high strength. The proportional amount of strength loss that occurs in structural concretes with each incremental increase in air is not a constant. The rule of thumb that a 1 percent increase in air causes a 5 percent loss in compressive strength is simply not true. The loss in strength occurring with increasing air depends on the strength class of the concrete. For example, consider two concretes having 28-day target compressive strengths of 30 MPa (4000 psi) and 50 MPa (7000 psi). Both contain air-void systems with similar bubble size, distribution, and spacing, each with an initial total air content of 5 percent. Increasing the total air content of each mixture from 5 to 7 percent (using air-entraining admixture) could cause a 5 percent measured strength decrease, or 2 MPa (300 psi) for the conventional strength mixture and a 20 percent decrease, or 11 MPa (1600 psi) in the higher strength mixture. The consequences of the increased air content would be marginal with conventional strength concrete, yet devastating for high-strength concrete. The author has witnessed this phenomenon numerous times. In the previous example, it was mentioned that the air void characteristics (void size, distribution, and spacing) of each concrete were similar. They would have to be in order to do a true "apples to apples" comparison of the effects of air on strength. In fact, the characteristics of the air void system influence strength more than the total air content itself. It has been observed (Jana *et al.*, 2005) that many of today's newer-generation air-entraining admixtures produce smaller and more numerous bubbles, and thus significantly higher specific surfaces and significantly lower void spacing factors than those achieved with the more traditional air-entraining admixtures. The theoretical actual volume of air needed to accommodate water movement into the voids when concrete freezes are less than 1 percent of the concrete volume. It follows that effective air-void systems can be obtained at lower than the current minimum air content requirements when air-entraining admixtures that produce smaller more closely spaced voids are used. Potentially, both the upper and lower limits of air content could be reduced by 1 to 2 percent without jeopardizing durability.

Concrete is not critically saturated until the moisture content within the capillaries or pores exceeds 91.7 percent. In order to become critically saturated, concrete would have to be in direct contact with moisture for long periods. As concrete permeability decreases, the time to critical saturation increases. Periodic rain or snow against a vertical surface alone does not constitute conditions conducive to critical saturation. In 1982, Armand ("Gus") Gustaferro inspected 20 out of 50 concrete bridges constructed in Illinois in 1957. Of particular interest were the non-air

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entrained 35 MPa (5000 psi) prestressed (pre-tensioned) bridge girders. In 1957, 35 MPa (5000 psi) was considered high-strength concrete, and because of concerns that the design strength would not be consistently achieved using air entrainment, the decision was made to construct the girders without air entrainment. After 25 years under severe service conditions, the authors reported minimal freeze-thaw damage (Gustaferro *et al.*, 1983).

Scaling resistance

Scaling, the local flaking or peeling away of the near-surface portion of hardened concrete is usually the result of repeated application of deicing salts and freeze-thaw cycling. Scaling can also occur due to pre-existing delaminations below the surface caused by premature finishing or natural surface crusting while the concrete was still bleeding. 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 troweling and excessive bleeding should all be avoided.

Resistance to alkali-silica reactions

Two kinds of reactions can occur between potentially deleterious aggregates and the alkalis within concrete: (1) alkali-silica reactions (ASR), and (2) alkali-carbonate reactions (ACR). For deleterious expansion to occur, three mechanisms are necessary:

- alkali reactive aggregate;
- an effectively high quantity of alkalis in the concrete; and
- moisture.

ASR is significantly more prevalent than ACR, and discussions in this book will only be limited to ASR. Compared to all other constituents, the cementing materials, particularly Portland cement, usually introduce the largest quantity of soluble alkalis. Since high-strength concretes invariably contain higher quantities of cementitious material, particular attention is necessary to preclude alkali-aggregate related distress. It is normal for alkali-silica reactions to occur in most hydraulic cement concrete. However, whether or not the reaction is severe enough to cause visible cracking or a threat to long-term durability is another matter entirely.

Since high-strength concrete can be rich in cementitious material, and therefore have a potentially high alkali loading, caution should be exercised to prevent cracking due to ASR expansion. ASTM C 1260[8] is a 14-day mortar bar test used to evaluate the potential susceptibility of concrete aggregates to ASR. ASTM C 441[9] is a 14-day test used to evaluate the effectiveness of various combinations of cements and supplementary cementitious materials in preventing excessive expansion of concrete due to ASR.

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ASTM C 1567[10] is a 14-day mortar bar test for determining the potential ASR reactivity of combinations of cementitious materials and aggregates. The determination of ASR susceptible materials or combinations of materials using accelerated mortar bars are generally suitable for acceptance purposes; however, in light of the severity of exposure and potentially high percentage of false positives, the results of accelerated tests alone should not form a basis for rejection. In such cases, longer-term tests such as ASTM C 227[11] or C 1293[12] are suggested.

In most cases, the alkali-silica reaction can be effectively controlled by taking one or more of the following steps:

- Avoiding ASR susceptible aggregates: Local experience may show that certain types of aggregates contain reactive silica. ASR susceptible aggregates contain amorphous or poorly crystalline silica, and include siliceous gravel, siliceous-bearing limestone, chert, shale, volcanic glass, sandstone, opalines, and quartzite.
- Use of a sufficient quantity of an ASR suppressing pozzolan: By reacting with the calcium hydroxide in the cement paste, a pozzolan can lower the pH of the pore solution. Additionally, the silica contained in a pozzolan may react with the alkali in the cement.
- Use of low-alkali cement: Less alkali available for reaction will limit gel formation.
- Low water-binder ratio: The lower the water–cement ratio, the less permeable the concrete. Low permeability will help limit the supply of water to the alkali-silica gel.
- Use an ASR-inhibiting chemical admixture, such as lithium nitrate or lithium carbonate in an effectively high dosage.

Sulfate resistance

Deterioration resulting from reactions between sulfates, usually in soil or ground water, and concrete or mortar; the chemical reaction is primarily and components of cement paste. Sulfate is a naturally occurring mineral salt. Sulfate attack is a chemical breakdown that occurs when sulfate ions from an external source enter the concrete and attack components of the cement paste, resulting in the formation of ettringite or gypsum. Sulfate attack can occur when concrete is in contact with sulfate-bearing soils or water. When sulfate attack occurs, the result is irreversible deterioration, usually in the form of cracking or scaling. Stark (2002), concluded that the greatest resistance to sulfate attack can be achieved with low ratios of water to total cementitious materials. Thus, high-strength concrete can be a highly suitable material for resistance to sulfate attack.

Delayed ettringite formation (DEF) is viewed as a form of internal sulfate attack. A number of factors have been known to influence DEF, such as the composition of cementitious materials, curing conditions and exposure

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conditions. DEF is most significantly influenced by improper heat curing of the concrete, where the ettringite formation that occurs under normal circumstances is suppressed. The sulfate concentration in the pore liquid is high for an unusually long period in the hardened concrete. Eventually, the sulfate reacts with the calcium and aluminate phases in the cementitious paste and expansion occurs. Due to this expansion, cracks form around aggregates. The cracks may remain empty, later be coming partly, or even completely filled with ettringite. It is generally agreed that DEF can be precluded in most cases if the maximum temperature within the member does not exceed 160°F (70°C).

Corrosion resistance

Electrochemical induced deterioration causing oxidation of embedded steel reinforcement and the development of internal pressures and subsequent spalling and cracking of the concrete. The adequacy of the protection concrete provides against the corrosion of embedded steel reinforcement depends on several factors, including the amount of concrete cover over the steel, the properties of the concrete (particularly permeability) and the degree the concrete is exposed to chlorides. High-strength concrete, a material of inherently low permeability has the potential to provide excellent protection against corrosion, if adequate attention is given to raw material selection, mixture proportioning, design, and construction. ACI 201.2R provides an extensive discussion of specific deterioration mechanisms of concrete, the recommended requirements for individual components, quality considerations for concrete mixtures, and construction practices. For further information on the durability properties of high-strength concrete, refer to ACI 363R.

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, not only because concrete is a composite material whose components have different thermal properties, but because its properties also depend on moisture content and porosity. Early research on the effects of elevated temperature on concrete material properties and performance in large measure was in support of the development of prestressed concrete pressure vessels for nuclear power plant designs.

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

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of thermal expansion (Farny and Panarese, 1994). 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 9.4 and 12.3μm/m/°C (5.2 to 6.8 μ-in/in/°F).

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 18° and –157°C (65° and –250°F). Cumulative drops in compressive and splitting tensile strengths were measured after each of five gradual freeze-thaw cycles ranging from a high of 18°C (65°F) to two low temperatures of –40° and –73°C (–40° and –100°F). That was done to evaluate the concrete's frost durability at liquefied petroleum and natural gas temperatures, respectively. As expected, moist concrete exhibited larger dilation and residual strains than air-dried concrete.

Fire resistance

One of the greatest advantages hydraulic cement concrete has over alternative construction materials like structural steel or wood is its superior fire resistance, and thus, capability of fulfilling the principal task of protecting the public from safety-related hazards. Fire resistance is defined as the ability of a structural element to maintain its load-carrying capacity when exposed to fire conditions.

As the use of high-strength concrete in columns continues to increase, concern has developed with respect to its fire resistance properties, particularly with respect to spalling. Explosive, fire-induced spalling is presumed to be caused by the build up of pore pressure during heating (Diederichs *et al*., 1995; Kodur and Lie, 1997). High-strength concrete's susceptibility is principally due to its significantly lower permeability compared to the permeability of conventional-strength concretes. Because of the significantly lower capillary porosity of high-strength concretes, residual free moisture within the concrete can become entrapped. Extremely high water pressures generated during fire exposure is unable to readily escape due to high-strength concrete's high paste density, and this pressure often reaches the saturation vapor pressure. At 300°C (500°F), the vapor pressure reaches approximately 8 MPa (1200 psi), almost twice the tensile strength of the concrete (Phan *et al*., 1997). In addition to strength, Kodur (2000) pointed out that spalling is also attributed to aggregate type, load intensity, reinforcement configuration and layout. Studies conducted on full-scale structural members found that the fire resistance rating of high-strength concrete columns could be improved by adding synthetic fiber reinforcement to the concrete (Kodur, 2000).

Consequently, vapor pressures that would normally be relieved in higher porosity concrete, results in a more rapid increase in internal tensile stresses, subsequently leading to greater spalling. One method that has been identified

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to overcome this shortcoming is to add polypropylene fibers to the high-strength concrete. Polypropylene fibers melt at approximately 160°C (320°F), and therefore are capable of creating moisture transport channels that can effectively control internal pressures.

Olsen (1990) evaluated the explosion risk of heat induced high-strength concrete as compared to normal-strength concrete. Cylinders of 100 × 200 mm with measured compressive strengths in the range of 30 MPa to 90 MPa (4300 to 13,000 psi) were cured in the following two ways:

- Condition No. 1: 7 days in water followed by 21 days in the laboratory environment [20°C (68°F) and 60 percent relative humidity).
- Condition No. 2: 7 days in water followed by 21 days sealed with plastic aluminum foil.

Thirty-six cylinders were heated in an electrical oven at a heating rate of 2.5°C (4.5°F) per minute until reaching a temperature of 600°C (1112°F). After 2 hours at this temperature, the cylinders were cooled at a rate of up to 1°C (1.8°F) 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 no higher than for normal-strength concrete especially for concrete cured under Condition No. 1 (Zia *et al.*, 1997).

Abrasion resistance

Abrasion resistance refers to the ability of a surface to resist being worn away by rubbing and friction. Principal factors influencing abrasion resistance include aggregate properties, surface finish, surface toppings such as dry-shake or liquid hardeners, and adequacy of curing. Concrete surfaces can abrade for numerous reasons, including hydraulic erosion, scraping, and grinding. From the standpoint of safety, satisfactory abrasion resistance is essential for pavements and bridge decks. Effective abrasion resistance is also important in spillways in order to withstand damage due to attrition and cavitation. High quality dense paste and hard aggregates are necessary to produce abrasion resistant concrete. ASTM C 779[13] covers three methods for determining the relative abrasion resistance of horizontal concrete surfaces; Procedure A, revolving discs; Procedure B, dressing wheels; and Procedure C, ball bearings. ASTM C 779 is principally intended to characterize the variations in surface properties that can occur because of changes in factors affecting abrasion resistance. Such factors include changes to the constituent materials or proportions of a concrete mix design, construction practices (placement, consolidation, finishing, curing), and surface treatments. The test is *not* intended to provide a quantitative measurement of the length of service that may be expected from a surface based on given conditions.

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The use of silica fume high-strength concrete with low W/B 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 (1991). It was shown that silica fume offers potential for improving many properties of concrete that are particularly beneficial in repair of hydraulic structures. When compared with a high quality asphalt pavement, the abrasion resistance of a very high-strength concrete pavement represents an increase in the service life by a factor of nearly ten (Zia *et al.*, 1997)

Abrasion resistance of high-strength concretes containing chemical admixtures and supplementary cementitious materials 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. The compressive strength of the concrete varied from 60 to 110 MPa (9000 to 16,000 psi) and the W/B ratio varied from 0.24 to 0.42. The concrete mixtures contained silica fume, fly ash or natural pozzolan, with and without a high-range water-reducing admixture, with consistency held fixed. The test results suggested that the abrasion resistance of concrete generally varies inversely with the W/B ratio, the porosity, and the paste volume of the concrete. Therefore, by using a high-range water-reducer to decrease the W/B ratio, the abrasion resistance of concrete could conceptually be improved greatly. Introducing mineral admixture without using superplasticizer would reduce the abrasion resistance of concrete since more water would be needed to maintain a constant consistency. 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 percent of ordinary concrete (Zia *et al.*, 1997).

Constructability properties

Constructability refers to the properties that are necessary for the mixture to be produced, delivered, placed, consolidated, finished, and cured, to achieve the required mechanical and durability properties. Typical constructability-related properties include consistency (slump or slump flow), workability, workability retention time, pumpability, finishability, and setting time.

Characterizing consistency

The slump test is defined in ASTM C 143[14] and is generally a relevant test for concrete having measured slump values below 7.5 inches (190 mm). Rather than measuring the distance of vertical subsidence, a more relevant way of characterizing the consistency of flowing and fluidized concretes would be by measuring the diameter of horizontal spread using ASTM C 1611.[15]

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Bleeding

Bleeding[16] is a form of segregation, and there are both advantages and disadvantages associated with it. Whether bleeding is a good thing or a bad thing depends largely on who you ask. When they develop, bleed channels essentially become express highways for the transportation of deleterious substances into the concrete. In theory, any amount of bleeding is detrimental to concrete durability; therefore, bleeding is a property that has no place with high durability concrete. However, in practice, bleeding is viewed by many concrete finishers as not only a good thing, but also a necessary constructability property for finished surfaces. Concretes designed for high strength or high durability tend to be sticky and bleed very little, therefore, surfaces tend to dehydrate rather rapidly and the concrete takes on the appearance that it is not bleeding at all. Air-entrained concrete is significantly less susceptible to bleeding than non-air entrained mixtures. Concretes produced with poorly graded aggregates have higher water demands and tend to exhibit bleeding.

Rheology

The rheology of fresh concrete can be mainly described by its yield point and plastic viscosity:

- The yield point describes the amount of force needed to put the concrete into motion.
- Plastic viscosity describes the resistance of a concrete to flow under external stress.

Balancing the yield point and the plastic viscosity is fundamental to obtaining suitable rheological concrete properties. Materials that modify concrete's viscosity, such as various non-reactive or low-reactivity powders, or viscosity modifying admixtures, change the rheological properties of concrete by increasing the plastic viscosity. Viscosity modifying materials usually increase the yield point. High-range water-reducing admixtures, which decrease the yield point, are often used in conjunction with viscosity modifying materials to optimize the yield point.

High-strength concrete consists of larger amounts of cementitious materials and chemical admixtures, lower water-cementitious materials ratios, and smaller coarse aggregates. As a result, the rheology of high-strength concretes can be quite different from that of conventional-strength concrete. Unless the rheological properties are properly addressed in the mixture development process, high-strength concretes are likely to be stickier and may be more prone to early stiffening, making placement, consolidation and finishing more difficult. The setting characteristics and heat development of a high-strength concrete mixture may make it more vulnerable to cracking caused

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by changes in moisture and temperature. Therefore, with high-strength concrete, greater attention needs to be paid to the selection of constituents, particularly cementitious materials, and placement and curing practices.

Advancements in chemical admixture and supplementary cementitious materials technology have contributed significantly to the evolution of high-strength concrete, and have helped to overcome constructability-related issues that have been known to occur. For example, prior to the development of high-range water-reducing admixtures, high-strength concrete was typically placed at slump values no greater than 100 mm (4 in). Using newest-generation admixtures, it is possible to place high-strength concrete at virtually any level of consistency. Most properly designed self-consolidating concretes allow the concrete to be placed without the need for any additional forms of consolidation.

Notes

1 *Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens*.
2 *Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)*.
3 ACI 209R-92: *Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures* (Reapproved 1997).
4 ACI 116R-00 (Re-approved 2005).
5 *Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete*.
6 *Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing*.
7 Procedure A: Freezing and Thawing in Water.
8 *Standard Test Method for Potential Alkali Reactivity of Aggregates (Mortar-Bar Method)*.
9 *Standard Test Method for Effectiveness of Pozzolans or Ground Blast-Furnace Slag in Preventing Excessive Expansion of Concrete Due to the Alkali-Silica Reaction*.
10 *Standard Test Method for Determining the Potential Alkali-Silica Reactivity of Combinations of Cementitious Materials and Aggregate (Accelerated Mortar-Bar Method)*.
11 *Standard Test Method for Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar-Bar Method)*.
12 *Standard Test Method for Determination of Length Change of Concrete Due to Alkali-Silica Reaction*.
13 *Standard Test Method for Abrasion Resistance of Horizontal Concrete Surfaces*.
14 *Standard Test Method for Slump of Hydraulic-Cement Concrete*.
15 *Standard Test Method for Slump Flow of Self-Consolidating Concrete*.
16 Bleeding is also referred to as "sweating" and "weeping" in some parts of the world.

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5 Specifications

Introduction

Concrete specifications are typically written by Architects or Engineers on behalf of the Owner. The purpose of specifications is to communicate to the builder the guidelines necessary to ensure that the materials employed in the work satisfy the intent of the design, and therefore, the Owner's expectations. Satisfactory concrete construction and performance requires concrete with specific properties. This chapter is chiefly devoted to considerations when preparing specifications involving high-strength concrete.

Universally applicable, or "boilerplate" specifications are undesirable, cost inefficient, and in many cases, they inhibit the ability to achieve the properties most critically needed. In terms of high-strength concrete, boilerplate specifications probably will guarantee it. For example (Taylor and Bhide, 2005), a bridge deck in a cold region exposed to deicing salts needs effective resistance to chloride ion penetration in order to delay the onset of chloride-induced reinforcement corrosion. Freeze/thaw durability and scaling resistance would also be necessary. Depending on structural requirements, the concrete may need to have some minimum compressive strength; however, a compressive strength that is too high will have a proportionally high modulus of elasticity and might increase the tendency of the bridge deck to crack. Cracking is detrimental to durability, particularly in an environment conducive to corrosion. In such a case, the specifier might elect to include only the minimum strength requirement. Unless a specification involving high-strength concrete has been written with a particular design in mind, there is a good chance that it may not be adequately suited for the project. Most specifications become legally binding documents once a contract is awarded.

Prescriptive vs. performance-based specifications

Specifications for concrete can fundamentally be written in one of three ways: purely prescriptive, purely performance, or a combination of prescriptive and performance. Arguably, there are advantages and disadvantages with purely prescriptive and purely performance-based specifications.

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Prescriptive-based specifications stipulate the materials and mixture proportions to be used along with the production, delivery, placement, and curing methods to be followed. On the other side of the spectrum, performance-based specifications only stipulate the "end result" properties necessary to satisfy the design criteria without prescribing the manner in which they are to be attained. Combination specifications contain both prescriptive and performance requirements, that is, along with the material, proportioning, and procedural requirements, the end result properties are also stipulated. Prescriptive specifications usually include stated values for minimum cementitious materials content, maximum aggregate size, permissible slump range, and maximum ratio of water to cement or cementitious materials. To successfully produce and deliver high-strength concrete requires intimate knowledge of the following three factors:

- constituent materials
- mixture proportions
- material interactions.

It would be difficult to repeatedly produce quality concrete using prescriptive specifications. Prescriptive specifications can never adequately address any of the above items satisfactorily enough to produce consistent quality high-strength concrete. The quality of constituent materials, which drives mixture proportions, varies from market to market and day by day. Small variations in constituent material quality can have a pronounced effect with the performance of high-strength concrete. Without due consideration given to constituent material compatibility, unanticipated problems are significantly more likely to occur.

Modern concrete is a much more complex material than it was just 50 years ago, utilizing more constituent materials and a far greater variety of available materials. Although the amount of materials engineering knowledge required to produce high performance concretes has increased significantly between designers and concrete producers over the years, the fundamental nature of concrete specifications has not. Compared to their predecessors, many contemporary designers are less knowledgeable in materials engineering technology. As the structural engineering discipline become increasingly more specialized, less opportunity is available for students to learn about concrete as a material rather than merely as a design element. Conversely, many modern concrete producers on the other hand are more technically astute than producers were a century ago. Designers with a high degree of knowledge in contemporary materials engineering may feel more comfortable with a traditional prescriptive approach. On the other hand, designers with little materials engineering knowledge or experience would be better off specifying end result properties rather than defining the path necessary to attain end results. In any event, leaving the responsibility for selecting the materials and

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proportions for high-strength concrete in less qualified hands sets the stage for problems, as it could occur with any concrete. As the range of materials that can be combined to produce concrete increases, particularly with respect to supplementary cementitious materials and chemical admixtures, specifying concrete prescriptively becomes an increasingly arduous task for Engineers. Sirivivatnanon and Khatri (1997) describe a number of performance-based verification tests for specifying concrete for aggressive coastal and marine conditions. They include rapid chloride permeability (ASTM C 1202[1]), permeable voids (ASTM C 642[2]), and water sorptivity (ASTM C1585[3]).

Specifications calling for the achievement of end-results in a prescriptive manner can establish barriers in the ability to achieve the result. The degree to which such combination specifications can be problematic depends on the nature of what is specified. In some cases, the risks associated with supplying performance concrete in a prescriptive manner are low; however, when high performance properties are needed, prescriptions can be extremely problematic. For example, a producer's ability to supply 20 MPa (3000 psi) concrete having a maximum W/B ratio of 0.50 would not be difficult. Conversely, specifying 80 MPa (12,000 psi) in 56 days while disallowing the use of SCMs, or allowing for the use of SCMs with arbitrarily established limits can significantly reduce the chances for successfully achieving strength performance, economy, or both. As unusual as it may sound, concrete producers are like bakers. Experienced concrete producers and bakers are both in a better position to produce a quality product (and most likely in a more cost efficient manner) if their customers or third parties did not direct them on how to do their job. Specifying performance concrete on a prescriptive basis is like directing a baker to make a cake according to a prescriptive recipe and requiring the baker to ensure that the cake will still taste good.

Specifications prescribing the manner in which performance is to be achieved can become very problematic, particularly as the performance requirements of the material increases. Numerous disastrous outcomes have resulted when attempts were made to produce high-strength concrete in a prescriptive manner. Prescriptive specifications may not allow for the use of the more suitable material types, quantities, or proportions available in order to satisfy the project requirements in the most cost effective manner.

Once a contract is awarded, having a specification changed can be difficult. Some specifiers would embrace changes more readily than others would. In any event, sound rationale should always accompany requests for any contract modifications.

The pitfalls of arbitrarily established limits

Arbitrarily selected limits prescriptively imposed on constituent materials should be avoided. Although many of the limits prescriptively imposed on

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materials, such as fly ash or slag cement, are done so for "conservative" reasons, arbitrarily chosen limits can not only impede the performance of concrete, but also can reduce performance. Pessimistic performance can result if insufficient quantities of cementitious materials are used, particularly with respect to durability. For example, studies have suggested that using too low amounts of certain fly ashes, such as the commonly specified 15 percent replacement limit can lower concretes resistance to AAR rather than using none at all (Malvar *et al*., 2002). Similarly, using the wrong quantity or type of fly ash can reduce concretes resistance to sulfate attack (Tikalsky and Carrasquillo, 1993). When understood and used in satisfactory amounts, most fly ash can significantly improve concrete durability.

Chapter 4 presented three categories for classifying most concrete properties—mechanical, durability, and constructability related, and indicated how problems could arise unless each relevant property was identified. In principle, identifying relevant concrete properties is a logical and easily understood concept; however, in practice it is infrequently accomplished. There is often an inherent disconnect in the process of selecting the most appropriate concrete for the application. Designers and specifying authorities' primary concern is with the properties of the structure during its service life (mechanical and durability properties), whereas the contractor's primary focus is constructability-related properties. Unless the project is executed on a joint design–build basis, all relevant properties may not necessarily be considered during the mixture selection and submittal process, and the end-results might never satisfy the Owner's expectations.

The party responsible for material selections and mixture proportions should be provided with all mixture requirements, whether related to mechanical, durability, or constructability properties. In addition to the plans and specifications, the constructability needs of the builder should also be communicated. On most projects, the concrete producer is usually the party responsible for the materials and mixture proportions; however, in some markets it is common for concrete mixtures to be developed by independent testing laboratories and certified by licensed Engineers. To do so requires knowledge of how the concrete is to be placed, consolidated, and cured. Supplying concrete based on design requirements alone can and has led to problems during construction.

High-strength concrete is only one type of high performance concrete. Historically, concrete is a material that has been specified in highly prescriptive terms. Prescriptive requirements frequently include minimum cement content, maximum W/B ratio, slump range, and aggregate properties. Sometimes the properties prescribed for high performance concrete are not appropriate and can often be counterproductive in achieving favorable results. Examples of such parameters include fresh and hardened concrete properties, seasonal conditions, and construction methods. Time-related factors such as batching, delivery, and placement times are critically important. Therefore, while still in the preconstruction phase, it is strongly

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recommended that all necessary properties be clearly communicated prior to trial tests or actual use in the work.

The specifying authority should select properties that are relevant for the specific application. Specifying additional properties beyond what is needed is likely to increase cost, make it more difficult to meet the criteria that truly are important, and perhaps even lead to unanticipated problems. Hydraulic cement concrete is a complex and dynamic material. Fixing the proportions of materials having inherent variability guarantees variable mixture performance. Periodic, as-needed adjustments by the concrete producer can facilitate consistent performance and maintain the intent of a performance-based specification. Common adjustments include variations in dosage rates of chemical admixtures, such as high-range water reducers, or the inclusion of hydration-controlling admixtures when conditions warrant.

The relevancy of the slump test

The slump test is one of the oldest and most frequently used tests to measure the consistency of fresh concrete. Consistency refers to the relative mobility or ability of freshly mixed concrete to flow. Common terminology used to describe the consistency of fresh concrete include *stiff, plastic, normal, flowable*, and *fluid*. Workability refers to the relative ease at which freshly mixed concrete can be placed, consolidated, and finished. Though frequently used interchangeably, the terms consistency and workability are independent concrete properties. This misconception is most likely based on the false presumption that as the concrete slump increases, so does workability. Whether or not increasing slump improves or worsens workability depends on several factors, including aggregate grading, cementitious materials content, and W/B ratio.

By definition, slump is a measure of the relative stiffness, or consistency of fresh concrete. It is not a measure of workability, water content, or W/B ratio. Procedures for performing the slump test are described in ASTM C 143.[4] A very popular misconception within the industry is that a strong correlation exists between slump and water content. Slump is influenced by many factors in addition to water content. Even in concrete, where consistency is not produced with the aid of water-reducing admixtures, there is no reliable correlation between slump and water content. Other factors influencing slump, include, aggregate cleanliness and aggregate particle grading. For example, measures taken to improve aggregate grading will usually result in a reduction in water demand. If the same quantity of water were used to produce the concrete, the consequence of using aggregates having better grading uniformity would be an increase in measured slump. If, on the other hand, the water content was not adjusted, the increased slump might exceed the maximum prescribed limit when tested at the jobsite, and forming a basis for rejection, even though the W/B ratio remained unchanged.

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For the slump test to be relevant, the concrete must be of a plastic and cohesive consistency. Lean concretes often lack enough cohesion to prevent the slump test sample from shearing off to one side. The slump test is not suitable for measuring the consistency of very stiff or flowing and self-consolidating concretes. High-strength concrete is a cohesive material, and most modern high-strength concrete is placed at flowing or fluidized consistencies. High-strength concrete produced using well-graded aggregates usually do not exhibit segregation at measured slumps below 250 mm (10 in) or below. Measuring the diameter of spread of the slump sample rather than the vertical drop distance is a more relevant method for determining the consistency of flowing and self-consolidating concretes.

The slump test has little relevancy with superplasticized flowing concretes, and it is not recommended that the slump test be used as an acceptance test. If the slump test is used for these types of concrete, caution should be exercised when interpreting the results. Emphasis should be placed on controlling the W/B ratio, not slump.

Constituent materials

There are barriers within the industry that make it difficult to appreciate the potential benefits of supplementary cementing materials in high-strength concrete, both technically and economically. The limits commonly specified for supplementary cementing materials often fall short of the benefits that could be realized in high-strength concrete. As Chapter 2 touched on, after decades of use, there is still a mindset in the concrete industry that cementing materials such as fly ash, slag cement, and various natural pozzolans are merely cement replacements. It is perfectly understandable that this would be the case since these materials were originally treated as replacements for Portland cement.

Take for example a specification that limits the quantity of fly ash to no more than 20 percent by mass of total cementitious material. Given the ways in which fly ash and Portland cement are known to interact, the optimum strength at a given age in a mixture rich in cementitious material might exceed 20 percent of total cementitious material; a quantity disallowed by the specification.

Of course, all relevant parameters, not just strength, requires consideration when determining the most appropriate mixture to use. The point being made here is that prescriptively imposed, all-inclusive limits, can be counter productive when the objective is to achieve the highest level of performance in the most cost efficient manner.

Quality management plans

Once a contract is awarded to the builder, a thoughtfully planned and implemented Quality Management Plan (QMP), based on sound and

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reasonably attainable construction objectives, would be an essential instrument. Items addressed in a QMP should include:

- mix design properties;
- submittal requirements;
- conditions of sale;
- preconstruction conference agenda items;
- protocol for ordering concrete, including minimum permissible batch sizes;
- guidelines for constituent material handling;
- guidelines for production and delivery, including delivery ticket information;
- guidelines for hot and cold weather concreting;
- responsibility for jobsite acceptance and rejection of fresh concrete;
- placement, consolidation, finishing, and curing practices;
- protocol for constituent material sample retention;
- protocol for concrete sampling and testing; and
- protocol for handling non-conformant test results.

Producer qualifications

Concrete construction is not exempt from the saying "you get what you pay for." Most ready-mixed concrete is bought and sold as a commodity, and the producer is unfortunately treated accordingly. Many contractors shop for concrete based on price and ability of the producer to meet flexible delivery schedules, but not much else (Hester, 1989). Price should never supersede qualification when selecting a concrete producer. When considering price alone, there is no guarantee that the concrete producer selected will even be remotely capable of supplying high-strength concrete in the quantities and of the quality needed for the project. The concrete producer must first be able to demonstrate the ability to achieve the desired strengths consistently, and at a reasonable cost (Hester, 1989).

It is suggested that the project specifications require the submission of a Concrete Producer's Statement of Qualification. The Statement of Qualification should be available for review prior to the awarding of the contract, and should be in the builder's bid package.

It is further suggested that producers of high-strength concrete have their own QMP to establish the procedures for becoming prequalified as a supplier of high-strength concrete. Any project involving high-strength concrete should require that a QMP be included in the concrete producer's Statement of Qualification. The purpose of the QMP is to provide a reasonable degree of assurance that the producer is capable of supplying high-strength concrete of the consistency and quality needed for a successful project.

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Submittals and conditions of sale

It is recommended that proposed mix designs be submitted for review by the specifying authority at least 30, but preferably 60 days before first placement. The submittal should include the following minimum information:

- project identification (name, location);
- name and address of contractor and concrete production facility;
- mix design designation and description;
- specified strength including designated acceptance age;
- based on field strength results or results of trial mixtures, documentation indicating that the proposed concrete proportions will produce an average compressive strength equal to or greater than the required average compressive strength;
- results of laboratory trial batches conducted under normally anticipated and extreme anticipated environmental conditions. Results should address all fresh and hardened concrete properties specified in the contract; and
- method statement for site adjustments, if proposed.

The next list of items will depend on the conditions under which the concrete is being furnished. If the high-strength concrete is being supplied on a performance basis, the producer may elect to furnish all, some, or none of the following:

- Concrete mixture proportions, including the following for each mixture:
  - — listing of all constituent material types and quantities;
  - — the saturated surface dry mass of the coarse and fine aggregates;
  - — name and location of all raw material sources, including aggregates, cementitious materials, admixtures, and water; and
  - — certificates of compliance for cement, supplementary cementitious materials, aggregates, and admixtures.
- Raw material statistical summary for the previous 12 months for:
  - — cement mill certificate information;
  - — supplementary cementitious or pozzolanic material mill certificate information;
  - — aggregate gradation and cleanliness results.
- Raw material certificates.

Dedicated producers of high-strength concretes devote large amounts of time and resources in research and development. Once high-strength concretes have been developed, many producers are quite reluctant, and justifiably so, to disclose the materials and proportions to be used. In Chicago, MSC policy was that all high-strength concrete having specified compressive strength at

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or above 70 MPa (10,000 psi) was supplied as pure performance concrete. These mixtures were submitted only stipulating the performance criteria for which they were being furnished. Mixture proportions were not submitted. A quality control representative would be present at the jobsite to oversee the delivery at all times. The concrete was furnished on the condition that the supplier was the only party responsible for the acceptance or rejection of the concrete. There were occasions when MSC elected to reject their own concrete, but the author is unaware of any MSC-supplied high-strength concrete ever requiring removal and replacement that was furnished on a purely performance basis.

Testing laboratory qualifications

The task of sampling and testing concrete in strict accordance to project specifications and applicable industry standards is critically important to the success of any project. With the exception of high profile projects involving ultra high-strength concretes, the selection of an agency qualified to test high-strength concrete is unfortunately often overlooked.

Bickley (1993) suggests that testing agencies should be considered based on its past performance history on projects utilizing high-strength concrete and its ability to perform properly in future work. Determining the within-test variability from previous test records with high-strength concrete provides a measure of gauging the consistency of the agency. Résumés of laboratory and field technicians should be available to review the individual's qualifications and work experience. Laboratories failing to provide documentation showing compliance with ASTM C 1077[5] or similar standard should not be considered for use in the work. A Statement of Qualification to be submitted to the project Architect or Engineer should be required.

Preconstruction conferences

Being a highly perishable material, concrete construction requires considerable advanced planning, and the importance of preconstruction conferences cannot be over-emphasized with any concrete project. This is particularly true for high-strength concrete, since high strength can only be achieved through procedures that are controlled more closely than are required for conventional-strength concrete.

Once on site, there is little time to discuss whether the concrete meets the specifications or if it can be adjusted in a particular manner. Such details need to be worked out in advance so that all parties involved with the work mutually understand each other's responsibilities. The best way to accomplish this is by holding a preconstruction conference, preferably several weeks prior to the first scheduled placement of high-strength concrete. Therefore, it is suggested that requirements for a preconstruction conference

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be incorporated into the project specifications. A preconstruction conference is essential to clarify the roles of all parties. It is best to have the mix designs submitted and reviewed well in advance of the meeting. Every detail involving the installation of high-strength concrete should be covered well in advance of the first scheduled placement. The agenda for the meeting should be prepared by the principal contractor and distributed to the meeting invitees in advance. The meeting agenda should indicate the meeting date, time, and location, and a list of the topics that will be discussed. A sign-up sheet should be distributed showing the name of the attendee, who they represent, and their contact information. Detailed minutes should be taken during the meeting and promptly distributed within one or two days following the meeting. Preconstruction conferences should include representatives of all parties involved in the specification and production of the concrete: the concrete supplier, Contractor, inspection agency, Engineer, and the Owner. In some cases, the building official may also participate in preconstruction conferences. The topics discussed at preconstruction conferences will vary depending upon such factors as the scope of the project and local experience with design, production, and placement of high-strength concrete.

Chapter 7 addresses topics that are commonly discussed at preconstruction conferences. It may not be necessary to cover each of these topics, depending on the specific needs of the project, the requirements of the local building official, and the experience of the concrete production and placement team. Since the performance of high-strength concrete is more sensitive to material variations and requires more periodic adjustments, the responsibility for jobsite adjustments should be addressed and clearly identified.

Specifications for high-strength concrete should be predominantly performance-based. They should state the required properties of the hardened and fresh concrete clearly and understandably, and leave little or no room for interpretation. In addition, they should be free of unnecessary restrictions. This means that much of the responsibility for ensuring that these qualities are achieved lies with the supplier. This is appropriate, since the concrete supplier is producing concrete on a daily basis and therefore is likely to have much greater expertise relating to concrete production than any other party in the construction process.

Post-28-day designated acceptance ages

The traditional standard age for determining compliance with the design compressive strength has been 28 days. While 28 days is a reasonably acceptable age for conventional-strength concretes, its relevancy to high-strength concrete is highly questionable. In fact, continuing to select 28 days as the standard designated acceptance age can be counterproductive in the pursuit of satisfying important long-term properties.

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It is common for the selection of materials and mixture proportions for high-strength concrete to be based on a designated age of 56 or 90 days rather than the traditional 28 days. There are several sound reasons why specifying acceptance ages beyond 28 days can be important with high-strength concrete. For many applications of high-strength concrete, loading conditions are such that the design strengths are not needed until much later ages. In addition to taking advantage of post-28-day strength gain, choosing a designated age of 56 or 90 days allows for a reduction in the paste content of the mixture, which can be highly beneficial in reducing total shrinkage and improving long-term durability potential. The most common reason why specifying authorities have resisted in assigning 56 or 90 day designated ages is the amount of time it would take to determine whether the concrete is acceptable. This is truly not a valid reason. The designated age of concrete is not the age at which the minimum (specified) strength must be achieved.

The designated age is the age at which the minimum strength must be achieved along with an over-design factor, the additional strength necessary in order to comply with the statistical requirements of the applicable building code. When the designated acceptance age is specified at later ages, such as 56 or 90 days, a relationship between early strength, such as 7 days, should be established so that problems can be identified and investigated early. For example, concrete with a specified compressive strength of 70 MPa (10,000 psi) at 56 days should not be expected to achieve any more than 70 MPa (10,000 psi) by the time standard strength specimens are 56 days old. Depending on the degree of control over the material and testing-related variables, such a mixture might be required to average 90 MPa (13,000 psi) by 56 days. What is important is the level of statistical confidence prior to the designated age that the concrete will achieve the necessary over-design factor. This can be achieved by specifying minimum target strength values prior to the designated age. For example, if the specified compressive strength of a high-strength concrete was 85 MPa (12,000 psi) at 56 days, the specifier might require no less than 75 percent and 85 percent of the specified strength be attained no later than 7 and 28 days, respectively. In the event that the target strength is not attained at these ages, remediation procedures would be required.

If a high-strength concrete has a specified compressive strength of 85 MPa (12,000 psi) at 56 days, the specifying authority could conceivably require no less than the specified strength be achieved at 28 days. Prior to actual use in the work, confirmation tests should be conducted to verify that the concrete is capable of attaining at least 85 MPa (12,000 psi) by 28 days, based on anticipated project conditions.

Notes

1 *Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration.*

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2 *Standard Test Method for Density, Absorption, and Voids in Hardened Concrete*.
3 *Standard Test Method for Measurement of Rate of Absorption of Water by Hydraulic-Cement Concretes*.
4 *Standard Test Method for Slump of Hydraulic-Cement Concrete*.
5 *Standard Practice for Laboratories Testing Concrete and Concrete Aggregates for Use in Construction and Criteria for Laboratory Evaluation*.

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6 Production and delivery

Introduction

The procedures and equipment for producing and transporting high-strength concrete are not much different to those used for conventional concrete; however, some changes, refinements, and emphasis on critical points are usually necessary. Had specialized equipment been necessary to produce high-strength concrete, its ascension into the mainstream industry probably never would have occurred. Depending on the condition and capacity of the production facility and transportation fleet, some adaption may be required. Producers that are already dedicated to supplying quality concrete routinely should have few difficulties producing and delivering high-strength concrete. However, as was discussed in the preceding chapter, expecting concrete producers to develop a highly sophisticated concrete, while imposing extraneous prescriptive requirements, can end up having counter-productive results. Prescriptive compositional requirements truly have no place with high-strength concrete. The control of high-strength concrete should be in the hands of the concrete producer, the party most familiar with the mixture ingredients and their interactions.

In the absence of project-specific requirements, most ready-mixed concrete in the US is produced according to ASTM C 94.[1] As is often mentioned in this book, the use of consistent, quality ingredients is critically important when making high-strength concrete. Raw material availability should be confirmed prior to the start of construction and materials should be available in sufficient quality and quantity throughout the duration of the project. Spot shortages of necessary materials could result in delays to the construction schedule. If shortages are unavoidable, mixtures using alternative materials should be developed in advance.

Production facilities, delivery equipment, contractor practices, testing equipment, inspection agency procedures, and environmental conditions, effective planning and a dedication to teamwork on the part of all involved parties is essential if high-strength concrete is to be used successfully. The successful production of high-strength concrete requires coordination of ordering, dispatching, production, and quality control personnel. Developing

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and implementing an internal Quality Assurance Manual is one of the best ways to begin. When producing and delivering high-strength concrete, having a formal Quality Assurance Manual should not be thought of as a luxury, but rather, a necessity.

High-strength concrete should be produced to the design W/B ratio, not consistency. Consistency should only be adjusted using water-reducing or high-range water reducing admixtures. With the exception of controlled and pre-compensated amounts of wash water, no water whatsoever should be added to high-strength concrete once batched. All sampling and testing practices, whether for constituent materials or mixed concrete should be performed strictly according to applicable standards, which in most cases is stipulated in ASTM C 94, or similar standard specifications for ready-mixed concrete.

It is suggested that each project involving high-strength concrete have its own unique Quality Assurance Manual, stating the mix designs and constituent materials (by source and type).

Order taking

Orders for high-strength concrete should be placed at least two days in advance of scheduled placements in order to allow ample time to inventory raw materials, and coordinate equipment and personnel. When producing high-strength concrete, it is advisable to have a back-up batching facility available in the event a breakdown occurs at the primary facility.

When taking an order for high-strength concrete, the following minimum information should be obtained:

- size of placement
- starting time
- mix design
- delivery rate
- truck staging location
- truck washout locations.

Regardless of the manner in which it happens within an organization, once an order for high-strength concrete is received, quality control personnel should always be notified. In addition, measures should be taken to ensure that the production facility has the necessary resources available on the day of the placement.

Dispatching

Verification tests are usually performed by quality control technicians before a batch of high-strength concrete can leave the plant. It is the responsibility of the dispatcher to ensure that when scheduling high-strength concrete

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deliveries, ample time is given to quality control representatives to conduct all necessary tests and admixture adjustments before the truck departs the batching plant. It is the dispatcher's responsibility to ensure that only trucks pre-qualified for delivering high-strength concrete are used and that the trucks have working communication equipment. When dispatching high-strength concrete, avoid indiscriminately cross-shipping batches from different plants unless prior arrangements have been made in advance.

On larger projects, it is good practice to "flag" trucks containing high-strength concrete, especially when multiple mix designs are being simultaneously delivered. A common method of identifying trucks delivering high-strength concrete would be to place different colored flags on the trucks or signs in the windshield. Whether or not trucks are flagged, the purchaser of the concrete should always review the delivery ticket information prior to discharge.

Small batches of concrete delivered in large-scale drums should be avoided, as the proportions charged into the drum will be different from the proportions discharged. A notable amount of concrete, primarily from the mortar fraction, ends up adhering to the drum lining and inner drum workings from the mixer *buttering* process. The effects of mixer buttering on a large batch of concrete mixed or hauled in an originally clean drum is negligible. For practical purposes, the proportions going in are the same as those coming out and considered tolerable; however, with very small batches, the mostly mortar fraction that could be retained by an originally clean mixer could appreciably increase the ratio of coarse aggregate to total volume of concrete. In such occurrences, the batch might appear harsh and exhibit poor workability. Therefore, every effort should be made to divide the quantity of concrete produced and delivered into equal size batches. Doing so will help to ensure both uniformity and consistency. For example, if 7.5 m^3 (10 yd^3) of high-strength concrete is ordered, and the mixing drum has a rated mixing capacity of 7m^3 (9 yd^3) each, it would be more practical to batch two 3.5m^3 (5 yd^3) batches rather than one 7 m^3 (9 yd^3) batch and one 0.5 m^3 (1 yd^3). In general, when delivering high-strength concrete, batches smaller than 3m^3 (4 yd^3) should be avoided.

Lastly, no assumptions should be made as to whether or not drivers have a clear understanding of where the project is located. Dispatchers should always ensure drivers know where the job is located and advise the best route given current traffic conditions.

Quality control

Whether precast or ready-mix, the Quality Control Department lies at the focal point of all concrete operations. Quality control staff members regularly interact with customers, sales representatives, dispatchers, plant personnel, testing laboratory personnel, and occasionally with engineers, architects, general contractors, and owner's representatives. Therefore,

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maintaining strong communications with the Quality Control Department is essential within the concrete producer's organization.

It is a misconception to believe that quality control is expensive and not worth the investment. Well-structured and implemented quality control programs require investments of time and money; however, in the end, they lead to net savings through mix design cost optimization, avoiding the need for follow-up testing, and avoiding litigation. Believing that contracts are won merely based on price alone is another misconception. Quality is a major consideration to many purchasers of concrete, and many are willing to pay more to get it.

The degree of plant inspection needed depends on the strength level of the concrete being produced and the producer's experience in making high-strength concrete. For example, an experienced supplier might produce 55 MPa (8000 psi) concrete routinely and may only perform full-time plant inspection for greater strengths. Whenever possible, when producing high-strength concrete, it is always good practice to sample the first batch of the day for routine fresh concrete testing and, perhaps most importantly, to check the visual appearance and density of the high-strength concrete. Improperly batched high-strength concrete often has a distinctly different appearance (Detwiler, 1992). Other routine fresh tests include slump or slump flow, temperature, and air content.

One of the responsibilities of the Quality Control Department is to formally train order takers, dispatchers, plant personnel, and drivers about the basic aspects associated with concrete quality control. Taking a few hours to do so is unquestionably worth the time and effort.

The frequency of testing constituent materials and concrete depends largely on the uniformity of materials, plant throughput rate of materials, and producer's experience making high strength. Initially it is advisable to make tests several times a day, but as work progresses, the frequency can often be reduced.

One of the keys to attaining high strength is by slowing down the rate of hydration. All else equal, as the temperature of fresh concrete increases, the rate of hydration will increase, water demand will increase, and concrete strength will decrease. Hydration can be effectively controlled by physically lowering the temperature and slowing down the rate of the reaction or it can be chemically controlled using retarding or hydration-stabilizing admixtures, in which case the temperature of the concrete will remain unchanged. The temperature of fresh concrete can be lowered using ice, chilled water, or liquid nitrogen. Prior to the start of the work, the performance of mix designs intended for placement during cold weather should be verified with trial batches replicating anticipated job conditions.

If hot weather conditions are expected during the course of the work, the performance of the high-strength mix design should be verified using trial batches representative of job conditions. Supplementary cementitious materials such as low calcium (Class F) fly ash, natural pozzolans, and slag

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cement can be particularly beneficial for reducing the rate of heat liberated, the total amount of heat liberated, and the chances for premature stiffening. Caution should be exercised when high calcium (Class C) fly ash is considered. High calcium fly ash liberates heat at a much faster rate than low calcium fly ash and runs a greater risk of creating aluminate-sulfate imbalance.

Plant operations

Unless governing specifications exist, concrete should be produced according to the provisions of ASTM C 94 or a similar standard specification for ready-mixed concrete. The production facility and transportation equipment used to produce high-strength concrete should conform to the certification requirements of the National Ready-Mixed Concrete Association, or similar standard. When producing high-strength concrete, batch plants having a stationary "central" drum integral to the plant are preferable over "transit-mix" facilities that introduce the materials into a truck-mounted drum that provides all of the mixing action. This is not to say that high-strength concrete cannot be successfully produced at a transit mix facility, just that a greater amount of batch-to-batch variability should be anticipated. Central mix plants have one mixing drum operated by one individual. The number of mixing drums and operators at a dry-mix plant depends on the number of trucks and drivers the producer has available for that day. At most dry-mix facilities the truck driver is the person responsible for ensuring that the concrete is uniformly mixed. Some truck drivers perform this important task in a consistent and conscientious manner, whereas others may not. Factors that can influence batch-to-batch consistency when producing high-strength concrete from a dry mix facility, include differences in mixing and agitation speed, number of revolutions during mixing and agitation, and mixer efficiency. Factors influencing the mixing efficiency of concrete drums include blade configuration, drum geometry and size, cleanliness, internal wear, and mixing capacity.

Saucier (1968) and Strehlow (1973) reported that high-strength concrete can be produced in common types of mixing drums. High-strength concrete can be produced in plants with manual, semi-automatic, or fully automatic batching systems, although, for achieving the best batch-to-batch consistency, fully automated batching systems are preferred. The production facility should be equipped with an automated moisture-measuring device having the capability of continually measuring the moisture content of the fine aggregate. The batching system should have the capability of automatically compensating, in real time, the amount of free water required based on the fine aggregate moisture content. Since the weights and measures used to produce the concrete is usually based on the mix design in a saturated, surface dry moisture condition, the moisture meter should indicate free moisture (total moisture less absorbed moisture) rather than total moisture.

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Calibration checks of moisture probes should be performed at least once a month or at anytime the accuracy of the device is questioned. Prior to the start of each day's production, and anytime there is reason to believe that a notable change in moisture content has occurred, the moisture content of coarse aggregates should be manually determined according to ASTM C 566[2] or a similar standard. Even with automatic moisture meters in place, the moisture content of fine aggregates should be manually determined at least weekly or anytime that the accuracy of the meter is questioned. To avoid contaminations during delivery, cementitious material fill pipes, admixture tanks, and aggregate stockpiles should be clearly identified.

When producing high-strength concrete, it is essential to ensure that thorough mixing takes place prior to departure to the jobsite. Concrete should be mixed for a period necessary to comply with applicable concrete uniformity requirements in Annex 1 of ASTM C 94 or similar standard for mixing uniformity. As a rule of thumb, concrete containing high-range water-reducing admixture should be mixed at least 30 to 40 percent more. For example, if a minimum of 70 revolutions at mixing speed is used for conventional concrete, a minimum of 100 revolutions is suggested.

Central mixers can be used in one of two ways. Either the material introduced into the drum can be mixed to full uniformity within the stationary drum, or it can be partially mixed in long enough for the loose constituents to recombine, then mixed to uniformity in the truck-mounted drum, a procedure known as "shrink mixing." To use central mix plants most efficiently, it is common practice in the ready-mixed concrete industry for producers to use the latter method. When producing high-strength concrete at central mix facilities, it is strongly suggested that the concrete be mixed to full uniformity in the plant mixer prior to being discharged into the truck-mounted mixer. Doing so may double or even triple the time that the material remains in the mixer, but it will greatly ensure that the batch has been satisfactorily mixed and it will improve batch-to-batch consistency. To ensure that more efficient mixing occurs, it is often beneficial to reduce the batch size by 10 to 15 percent below the drums rated mixing capacity, especially when using high-range water reducers. When using fine cementitious materials such as silica fume, metakaolin, or fine blended cements, special attention should be paid to the charging sequence to ensure uniform mixing.

Whether added at the batch plant, en route, or at the jobsite, many low-strength investigations involving high-strength concrete have been traced back to the addition of higher than desired quantities of water. Trucks should be completely emptied of all previous material, including wash water prior to receiving a batch of high-strength concrete. In principle, trucks that delivered the same high-strength mix design from the preceding load need not be rinsed out, provided they are completely empty and carry no wash water. In order to maintain consistency within the operation, the author's preference is to establish a policy that all truck-mounted drums

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are rinsed out prior to receiving a batch of high-strength concrete. Rinsing of truck collection hoppers and any internal drum workings (blades, fins) after batching should not be permitted unless carried out under the supervision of trained plant supervisory or quality control personnel. If performed, minimal amounts of water should be used and water introduced into the drum should be compensated for during batching. For example, if it has been found that 8 L (2 gal) of water is needed to rinse the collection hopper, the amount of batch water programmed into the plant should be reduced by 8 L (2 gal) per batch. This may seem like a negligible amount of water, and practically speaking, it probably is. However, the importance of maintaining strict control over the water content used in the manufacture and placement of high-strength concrete cannot be over-emphasized.

Moisture content determination should be completed in advance of batching. A record of aggregate moisture content determination should be maintained by the concrete producer, available at all times throughout the course of the work, and retained for at least two years following the completion of the project or in accordance with the producer's document retention policy, whichever is later.

If ice is used, it should be measured on a mass-basis and included as a portion of the mixing water. During cold weather periods, heated mixing water, if used, should be available in a sufficient quantity to provide consistent batch-to-batch temperatures. Concrete temperatures should be closely monitored when steaming aggregates, especially if the time between batches varies. The maximum tolerable temperature during cold weather will depend on the usage of the concrete, member size, and ambient conditions; however, in general, batches produced at temperatures exceeding 32°C (90°F) should be discarded.

Most chemical admixtures respond more robustly when introduced after pre-wetting the cement for several seconds. The effectiveness of delaying the addition of an admixture can vary substantially depending on the specific admixture and cement involved. Admixtures comprised of naphthalene or melamine condensates are most effective and produce the most consistent results when introduced at the end of the mixing cycle after all other ingredients have been introduced and thoroughly mixed. Newer-generation polycarboxylate polymers are less sensitive to timing of their introduction and can often be added earlier into the charging process. Admixtures should be introduced separately and never commingle individually until after they are introduced into the batch. Most chemical admixtures are either injected into the water line or water weigh box.

Charging materials into the mixing drum is a more critical process in a dry batch plant than in a central mix plant. This is principally due to the configuration of the mixing blades. Most truck-mounted drums have blades configured in a screw pattern where the materials fold over while simultaneously traveling forward and backward along the longitudinal axis of the mixer. Attempting to charge cementitious material rapidly, allowing

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cementitious materials to commingle with water as the materials are charged, or batching with exceptionally wet sand can result in the development of "dry cement balls." In central mix drums, the constituents are physically impacted by the mixer blades as it folds over thus inhibiting the development of dry cement balls. Therefore, the charging sequence at central mix plants is of a less critical nature; however, the uniformity of concrete produced in central mix drums is generally enhanced by ribbon loading the aggregate, cementitious material, and water simultaneously. Cement balling rarely occurs in central mix plants provided the correct batching procedures are in place. Regardless of the batching process used, under no circumstances should air-entraining admixtures commingle with any other admixtures during charging. Air entraining admixtures can be introduced with water or dispensed on the sand.

Whether stationary or truck-mounted, mixer drums should be checked routinely for blade condition and degree of buildup. As an alternative to visual inspection from within the drum itself, an equally effective and far safer method of drum inspection would be to insert a small security camera mounted onto a sufficiently long telescoping pole. The inner drum can be viewed from a closed-circuit camera and, if desired, visually recorded.

Depending on the demands for high-strength concrete in a given market, the procurement of additional plant components may be well worth the investment. If a production facility has only two cementitious material silos, serious consideration should be given to installing a third. Having the ability to produce ternary concretes gives the producer significantly better flexibility, and may expand the different types of concrete that can be supplied. Plants unable to batch smaller coarse aggregates or coarser sands, while at the same time housing the aggregates needed for producing conventional concrete, might not be able to service their customers efficiently.

It is a fact of life that periodic plant adjustments will be necessary in order to maintain consistent performance. Allowable adjustments of approved mix designs should be identified prior to the start of the work and should be based on the results of trial batches.

Delivery

High-strength concrete is usually delivered in truck-mounted mixer drums capable of both mixing and agitation. On most high-strength concrete projects, it should be presumed that jobsite admixture adjustments will be needed and should be planned for accordingly. It would be foolish to begin a project with the belief that site adjustments will never be necessary. Therefore, use of equipment incapable of mixing, such as dump trucks and trucks with agitator tubs should never be used to deliver high-strength concrete.

High-strength concrete should always be kept agitated en route to the project site and while waiting to discharge. Early stiffening is more likely

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to occur without continual agitation. In order to avoid the necessity to make site adjustments, haul times, waiting times, and discharge times should be kept to a minimum. Extending the times of each will only invite problems. During the preconstruction conference, it should be communicated that delivery tickets will be filled out completely, and therefore will not be surrendered by drivers until completion of the discharge of the high-strength concrete. Delivery tickets should contain the following minimum information:

- name of the concrete producer,
- name of the concrete purchaser,
- project address,
- location where the concrete was batched,
- date delivered,
- serial number of the delivery ticket,
- mix design serial number,
- truck number,
- time of batching,
- load number,
- batch volume,
- volume of concrete ordered,
- fields for entering status times,
- plant departure time,
- jobsite arrival time,
- discharge start time,
- discharge end time,
- jobsite departure time,
- field for entering quantities of added water and time added, and
- comments/site observations.

Mixer drivers should record all pertinent information regarding the load of concrete they are delivering, including truck status times (arrival time, discharge start time, discharge end time), adjustments made to the batch and the times that the adjustments were made. If the batch of high-strength concrete they are delivering is sampled, drivers should record the time sampled, and record if any discrepancies in observed sampling or testing procedures occurred.

Additional information relating to the production and delivery of high-strength concrete is available in the ACI Committee 363 Report on High-Strength Concrete.

Notes

1 *Standard Specification for Ready-Mixed Concrete*.
2 *Standard Test Method for Total Evaporable Moisture Content of Aggregate by Drying*.

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7 Placement, consolidation, and finishing

Introduction

The performance of high-strength concrete is highly dependant on jobsite practices. Regardless of the strength or performance classification of the concrete, good materials, proportions, and production alone can never serve as a substitute for good jobsite practices. Even the most carefully designed high-strength concrete mixtures can perform poorly if good placement, consolidation, and, when necessary, finishing practices are not followed. Many problems that have occurred with high-strength concrete have been traced to poor jobsite control, particularly retempering practices and prolonged waiting times. Coordination and communication between all involved parties is essential for successful construction with high-strength concrete.

Concrete is a perishable material when first produced, and is highly vulnerable to abusive construction and testing practices. Given the temperature sensitivity of the hydration reaction, in general, as the temperature of fresh concrete increases, the material stiffens, sets, and hardens at a faster rate. Factors influencing early stiffening are addressed in Chapter 10. Prolonging the time it takes to place concrete increases the chances that it will become too stiff and increases the likelihood that the consistency and workability necessary for satisfactory placement will be lost. Without a practical means of controlling hydration during placement and finishing, the likelihood for early loss of consistency and workability increases. In addition to controlling hydration as it relates to strength and other hardened mechanical properties, set retarding and hydration stabilizing admixtures also play a critical role in prolonging the constructability properties of high-strength concrete. Important factors influencing early stiffening and setting are discussed in Chapter 10.

Tempering refers to the early addition of water in order to increase workability. For conventional concretes, ASTM C 94[1] allows for jobsite tempering, provided the water is added only once, either upon arrival to the site or during a reasonable (and defined) period thereafter in order to increase slump to within the specified range (provided the maximum W/B

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ratio is not exceeded). Retempering is the practice of adding water to concrete for restoring workability that has been lost. The practice of retempering can have a profoundly negative effect on the quality of all structural concretes, particularly strength. For high-strength concrete, the practice of adding *any* site water at all can be profoundly damaging.

Preconstruction conferences

There can often be very fine lines separating success from failure. In concrete construction, the importance of communication cannot be overstressed. The following example (Aberdeen, 1985) presents a good example of why effective communication is essential. The project is fictional, but the problems described are all too real and occur on major projects:

> Construction of the concrete frame for a mid-rise office building had just begun. Midmorning on the day of the first large concrete placement the architect's representative arrived and saw a worker adding water to a truckload of concrete. He immediately rejected the load, and an argument with the truck driver and job superintendent ensued. If a preconstruction conference were not held in advance of the work by all involved parties, once the work commences, it might be difficult to answer the following questions:
>
> 1. Who has the authority to reject a concrete delivery?
> 2. For what reasons may concrete deliveries be rejected?
> 3. Who will receive test reports and when?
> 4. Who is responsible for cylinder storage and curing?
> 5. What are the acceptance criteria for strength?
> 6. Under what circumstances is additional testing required?
> 7. Who pays the costs of additional testing?

Preconstruction conferences review and clarify contractual requirements, construction means and methods, and testing and inspection procedures. Following are a few more questions that would be appropriate agenda items at a preconstruction conference involving high-strength concrete:

1 If multiple mixtures are concurrently delivered, how will they be distinguished at the jobsite?
2 What will be the policy for slump or slump flow adjustments, if needed? Will a chemical admixture be stored on the job for such adjustments?
3 If a chemical admixture is to be added at the jobsite, what criteria will determine the dosage? Who would perform this task? How will they be trained?
4 Who will be the designated contacts for the contractor, engineer, testing agency, concrete producer (dispatchers, quality control staff), owner, and architect?

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5 How much lead-time will the concrete supplier need prior to each placement?
6 What will be the minimum and maximum batch sizes for high-strength concrete?
7 What size test specimens will be used? How will the specimens be initially cured under normal and extreme weather conditions?

In Chapter 5, it was suggested that preconstruction conferences should be incorporated into the project specifications, and therefore, the builder's responsibility to execute. Invitees to a preconstruction conference should include, but not be limited to, the engineer, architect, concrete producer, and testing laboratory.

Preparation

As is the case with all concrete, delaying the placement of high-strength concrete can result in a greater loss in workability over time, therefore delivery of the concrete to the site must be scheduled so it will be placed promptly upon arrival. Coordination of delivery between the producer and the contractor is essential. Equipment for placing the concrete must have adequate capacity to perform its functions efficiently so that placement delays can be kept to an absolute minimum. Equipment breakdowns occur from time to time; therefore, the need for back-up equipment should be anticipated. For example, provisions should be made for an adequate number of standby vibrators. ACI Committee 363 recommends that at least one standby vibrator should be available for every three vibrators in use. A high-strength concrete placing operation is in serious trouble, especially in hot weather, when vibration equipment fails and the standby equipment is inadequate.

Sufficient amounts of water should be available at the project site for cooling formwork and reinforcement prior to concrete placement.

Placement

High-strength concrete should be delivered so that it can be placed with minimal amounts of waiting time. By delaying the placement of high-strength concrete, there is a greater chance that the concrete will stiffen beyond the point that it can be properly placed, and may subsequently lead to jobsite retempering. Regardless of when it is introduced, jobsite added water can be extremely detrimental to the integrity of the high-strength concrete, and therefore should never be permitted. The author strongly recommends that all necessary adjustments to workability be made using high-range water reducer.

When steps are taken to satisfactorily control the rate of hydration of the mixture, the permissible time from batching to placement can usually be limited to 90 minutes in most cases. Limiting the allowable placement

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time to a shorter period, such as 60 or 75 minutes might be necessary under hot weather conditions. Time limitations should not prevent older concrete from being used, provided it is still of a placeable consistency without a need to introduce additional high-range water reducer. Under no circumstances should stiff concrete be used if the temperature appears to be increasing.

Placing must be done so that segregation of the various constituent ingredients is avoided and full consolidation is achieved with all entrapped air voids eliminated. The slump test should not be used as a basis for acceptance or rejection if a high-range water-reducer is being used, provided there are no indications that the concrete is segregated. Over-plasticized concrete that has segregated should never be used. Stopping the drum and allowing the concrete to rest for 10 to 20 minutes often results in a sufficient loss of plasticity that allows the concrete to be placed without further segregation.

The pump method is an attractive means for placing concrete since it can be placed in a generally continuous manner at a relatively high rate of speed. Whenever possible, pumps should be positioned so that the next truck in line can immediately begin discharging concrete into the hopper, thereby minimizing breaks in the placement sequencing. Direct communication is essential between the pump operator and the concrete placing crew. Chapter 9 of ACI 304R-00[2] provides guidance for the use of pumps for transporting high-strength concrete. Pump lines should be laid out with a minimum of bends and firmly supported.

Concrete should be deposited at or near its final position in the structure. Buggies, chutes, buckets, hoppers, or other means may be used to move the concrete as required. In applications where concretes having two different strengths are being used simultaneously, such as high-rise buildings, place the high-strength concrete at the prescribed locations before the normal-strength concrete.

There is a recognized and justified need to occasionally add site water to conventional-strength concrete in order to increase workability, and the provisions for doing so are laid out in ASTM C 94; however, under no circumstances should additional water ever be used to increase the workability of high-strength concrete. In the event that the need to increase workability arises, a high-range water-reducing admixture of the same brand and type as used at the concrete production facility should be used. Site-added high-range water-reducing admixture should be added to the batch by means of a pipe or wand that can introduce the product to the center of the drum using an automated metering device. Only trained personnel should be allowed to add high-range water-reducer. A method statement by the contractor for the site addition of high-range water-reducer should be submitted and a record of jobsite additions should be maintained and available at the project site at all times.

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Transmission of column loads through floor slabs

ACI 318 states that when the specified compressive strength (f_c') of columns does not exceed the floor concrete strength by more than 40 percent, no special precautions need to be taken. However, if f_c' of the column is greater than 1.4 times that of the floor slab, ACI 318 requires that transmission of load through the floor slab shall be provided by placing two different concrete mixtures in the flooring system (Figure 7.1). This procedure is done so using a placement method known as "puddling" or "mushrooming" as shown.

Placement of high-strength concrete in this manner can be a tedious endeavor. It requires careful coordination of the concrete deliveries and finishing procedures involving concretes likely to have differing setting characteristics. To avoid creating cold joints in these high shear locations, the lower-strength slab mixture has to be placed while the higher-strength concrete is still plastic and should be adequately consolidated to ensure the section is monolithic. To prevent the inadvertent placement of the lower-strength slab mixture in the column area, it is important that the high-strength concrete be placed first. From the time the high-strength concrete is placed, precautions should be taken to prevent dehydration and surface crusting. High-strength concrete used for puddling purposes should be of a relatively stiff, but workable consistency.

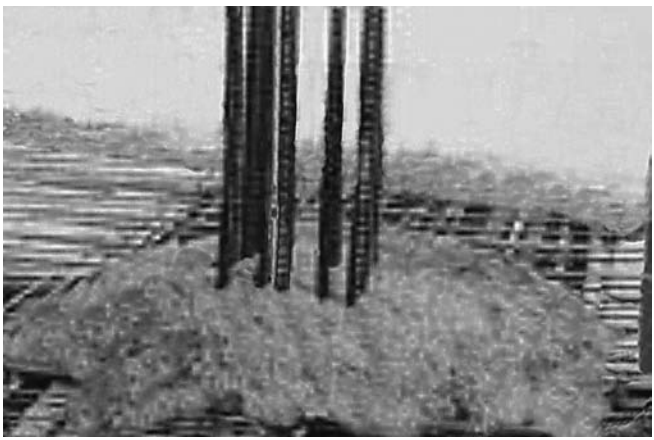

Figure 7.1 Placement of high-strength concrete simultaneously with conventional-strength concrete requires careful planning and coordination. Courtesy of CTLGroup.

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Hot weather placement

Hot weather can be considered as any period of high temperature in which special precautions need to be taken to ensure proper handling, placing, finishing, and curing of concrete. ACI 305R-99[3] states that hot weather concreting problems are most frequently encountered in the summer, but the associated climatic factors of high winds and dry air can occur at any time, especially in arid or tropical climates.

The primary concerns relating to hot weather concreting include increased water demand, premature stiffening, loss of workability, faster setting times, loss of entrained air, plastic shrinkage cracking, decreased later-age strength, excessive hydration temperatures, and larger thermal gradients leading to cracking (Caldarone, *et al*., 2005). Most high-strength concrete exhibits little or no bleeding; thus, it is particularly vulnerable to plastic shrinkage cracking.

Prolonged mixing in hot weather conditions accelerates cement hydration, thus causing greater workability loss and the likelihood for retempering. This in turn leads to increased permeability, increased shrinkage, and lower strength. Even under normal conditions, high-strength concrete is sensitive to the effects of retempering. High-strength concrete is profoundly more vulnerable to retempering in hot weather. It is possible to offset the deleterious effects of high temperature as it relates to strength and other mechanical concrete properties; however, admixtures alone might not be satisfactory for massive elements or applications requiring high durability. In such cases, temperature control used in conjunction with chemical admixtures may be necessary.

Cold weather placement

The principal concerns when placing concrete in cold weather is slow setting, reduced rate of strength development, thermal-induced cracking, and non-recoverable distress caused by premature freezing. Ideally, concrete should not be placed when the temperature of the air at the site or the surfaces on which the concrete is to be placed are less than 5°C (40°F). Table 3.1 in ACI 306R-88[4] lists the recommended minimum concrete temperature as mixed for indicated air temperature, minimum concrete temperature as placed and maintained, and maximum allowable gradual temperature drop in first 24 hours after end of the protection period.

It is generally accepted in the industry that in-place concrete should attain a minimum compressive strength of about 3.5 MPa (500 psi) prior to freezing. Note that since the actual mode of distress due to premature freezing is expansive in nature, it is the developed tensile strength, not the compressive strength, which is the critical property governing resistance to early freezing.

Admixtures purposely containing chlorides should not be used in high-strength concrete, or in any concrete application where there is a risk of corrosion distress.

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Consolidation

Thorough consolidation is required for high-strength concrete to achieve its full potential. Inadequately consolidated concrete will have reduced strength and durability, and a less pleasing visual appearance. As W/B ratios decreases, the consistency of concrete becomes increasingly stickier, making the need for effective consolidation important even in high slump concretes produced at flowing consistencies. For highly effective consolidation, flowing or self-consolidating concrete is recommended. Self-consolidating concretes are gaining in popularity, especially in precast and prestressed applications; and in most applications require no additional means of consolidation. However, in certain circumstances, such as tall, thin reinforced walls, small amounts of added vibration is sometimes necessary. Therefore, the term "self-consolidating concrete" should not be interpreted to mean that additional forms of mechanical consolidation should not be permitted. In fact, the author has observed well-proportioned self-consolidating mixtures highly resistant to segregation when subjected to prolonged mechanical vibration. High-strength concrete mixtures requiring vibration should be vibrated as quickly as possible after placement into the forms. Blick (1973) found that high-strength concretes produced with coarse graded sands exhibited better workability than mixtures produced with finely graded sands. Consolidation methods are detailed in ACI 309R.[5]

Because of its inherently cohesive nature, most high-strength concretes have little difficulty withstanding pumping pressures. However, there can be circumstances when a high-strength concrete behaves as a dilatant (shear thickening) material. Such mixtures appear to exhibit favorable flow characteristics, but strongly resist flowing if attempts are made to move them too rapidly. In such cases the concrete would take on more solid-like than fluid-like rheological properties. Reducing the rate of energy input (shearing rate) has been found to resolve such problems. Reducing the pumping rate usually eliminates problems of this nature. It is normal for the air content exiting pump lines to be less than the air content entering. It is extremely rare for concrete to gain air during the pumping process.

Early stiffening during hot weather is one of the most difficult challenges to both producers and purchasers of concrete, since it often leads to harmful retempering practices.

Finishing

Most high-strength concrete is used in vertically formed applications; therefore, finishability is usually not a necessary constructability property. However, when high-strength concrete requires finishing, modified procedures may be necessary, particularly if a hard trowel finish is required. Increases in the paste fraction, cementitious materials content, or fineness, or decreases in the water-binder ratio will generally cause concrete to

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become more cohesive and bleed less. From the time concrete is placed to the time finishing is complete, initial curing methods are often needed and should be planned. Given their high cementitious material contents and low W/B ratios, high-strength concretes inherently bleed at rates much lower than conventional concretes. Initial curing practices include misting/fogging or the application of evaporation retardants. If evaporation retardants are used in accordance with the product manufacturer's recommendations, the potential for plastic shrinkage cracking can be minimized. Evaporation retardants are meant to be applied *after* a given finishing operation, not before. Unless properly instructed, concrete finishers might apply them before the finishing operation, in which case they end up being used as finishing aids.

CASE STUDY: WHEN SELF-CONSOLIDATION IS NOT ENOUGH

The use of self-consolidating high-strength concrete is on the increase, and this case study, a popular misconception about alternative consolidation methods is reviewed.

Self-consolidating concrete[6] (SCC) is described as a type of concrete that does not require vibration for placing and compaction. There is somewhat of a misconception within the industry that SCC cannot or should not be consolidated using any other method than self-induced consolidation. When considering the name given to this type of concrete, it is not hard to see how such a misconception can nurture. "Vibrating self-consolidating concrete" is unquestionably an oxymoron. This discussion is not meant to conflict with the primary objective of SCC, but rather submit that special circumstances exists that represent limitations to the technology that may require procedural modifications.

It is true that in most applications involving a suitably proportioned SCC mixture, there should not likely be any need for mechanical consolidation. However, saying SCC should not require mechanical consolidation in most applications is one thing; believing that it could or should not be consolidated in any other manner is something else entirely, and is quite wrong. With respect to SCC, there is an exception to the rule. There are several occasions when alternative consolidation methods may be necessary.

This study began as an investigation into light to moderate honeycombing (Figure 7.2) that was occurring during the placement of a well-proportioned air entrained, self-consolidating mixture. The mixture was being used for the construction of deep and narrow with a double mat of moderate steel reinforcement. The walls were 6 m (20 ft) tall and 125 mm (5 in) wide. By definition, SCC is concrete that can be placed and consolidated without the need for vibration. In addition to its rheological properties, the ability

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Figure 7.2 Honeycombs in monolithic spandrel beams placed integrally with deep, thin walls. This element was placed with a well-proportioned and adequately produced SCC mixture. These honeycombs developed because of excessive obstructions encountered during free fall into the formwork. The occurrence of honeycombing could have been reduced had the element been detailed in such a way that the top portion of the beam had been gradually chamfered (tapered) into the wall face. Courtesy of CTL Group.

of concrete to self consolidate is also dependent on other factors, including entry velocity and self-weight (i.e. momentum) into the forms. In most SCC applications, no additional means of consolidation is necessary; however, under certain circumstances additional consolidating procedures may not only be warranted, but highly desirable. For example, placing SCC in locations congested with reinforcement or locations having complex geometries where it becomes difficult to place the concrete at the speed and fluidity necessary to ensure self-consolidation could increase the chances for segregation. Similarly, if the entry velocity of the concrete must be decreased, the fluidity of the mix will need to increase in order to ensure that the necessary flow characteristics are maintained. However, all else equal, as the fluidity of the mixture increases, the static or dynamic stability of the mix may decrease.

Tall thin walls can be more challenging elements to construct with SCC, primarily due to a high potential for honeycombing due to the combination of free fall and encountered obstructions. Honeycombing in SCC construction is generally caused by improper placement procedures, or inappropriate

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constituent materials or mixture proportions. Prime factors include the high vertical free fall drop and the increased likelihood for encountering obstructions during placement.

Excessive entrapped air voids deposited on formed surfaces (i.e. bugholes) can occur with any types of improperly placed concrete, whether vibrated or not. In the absence of excessive bugholes, honeycombing would be a direct indication that the concrete is encountering obstructions and segregating. Vibration tests conducted on small mockup sections that were saw-cut the next day indicated the mixture had exceptionally good dynamic stability, even when subjected to extended periods of vibration. Based on the favorable results of the vibration tests, it was determined that subjecting the concrete to internal or external vibration would not cause segregation. Other detailing and construction-related modifications included:

- not subjecting the concrete to free fall in the pump line and maintaining the pump line full of concrete at all times;
- carefully directing the flow of concrete straight downward, avoiding reinforcement, wall ties, forms, or any other obstructions that can cause deflections to occur; and
- eliminating sharp corners that introduce obstructions and require changes in the direction of concrete flow.

Notes

1 *Standard Specification for Ready Mixed Concrete.*
2 *Guide for Measuring, Mixing, Transporting, and Placing Concrete.*
3 *Hot Weather Concreting.*
4 *Cold Weather Concreting* [ACI 306R-88 (Reapproved 2002)].
5 *Guide for Consolidation of Concrete.*
6 Also referred to as "Self-compacting" concrete.

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8 Curing

Introduction

ACI 308R-01[1] describes curing as a process during which hydraulic-cement concrete develops hardened properties through the hydration of the cement in the presence of water and heat. Curing allows hydration to occur so that the intended mechanical and durability properties of the concrete may develop. Hardened cement paste reacts as a porous matrix that bonds aggregates. As hydration continues, the strength of the inter-particle bonding increases, and the inter-particle porosity decreases. The rate and extent of the hydration depend on the availability of water.

Curing is a more critical process for high-strength concrete than it is for conventional-strength concrete; therefore, attention to proper curing practices is essential if high-strength concrete is to develop its intended properties (Kosmatka, *et al*., 2002). What is considered "effective" curing depends on several factors, including the element under consideration, particularly the ratio of exposed surface area to total volume of the element; the thermal and moisture-related properties of the concrete, environmental conditions and serviceability requirements of the structure. In general, as the surface-to-volume ratio of the element and the cementitious materials content of the concrete increases, so does the need for curing. Without effective curing, significant breaches to long-term durability can result. Distress typically takes one of several forms, including visible cracking, microcracking, and weak wearing surfaces.

Consideration for curing should be given the moment that concrete is placed, not as a final step after the completion of placement and finishing. Once placed, plastic concrete is extremely vulnerable to destructive volume changes caused by changes in moisture and temperature prior to hardening. Unanticipated volume changes prior to the development of intended hardened properties can have devastating effects on long-term serviceability. Inadequate curing generally affects only the outer 20 to 50 mm (0.8 to 2 in) of an element, but that critical zone is exposed to the environment, and it provides protection and passivation to steel reinforcement (Detwiler and Taylor, 2005). High-strength concretes typically have very dense paste

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matrices; therefore, some curing methods that have worked favorably with conventional concretes may be less effective for high-strength concrete. The ACI 308 Guide to Curing Concrete discusses many acceptable methods for curing concrete.

Strength loss notwithstanding, inadequate curing can result in distress in the form of shrinkage cracking, spalling, scaling, paste erosion, and increased carbonation rates. Meeks and Carino (1999) found no consensus on the curing requirements of high-strength, high-performance concrete, and there is no agreement on whether it requires special considerations compared with conventional concretes. Some conclusions have been contradictory. Possible reasons cited by the authors for these contradictions by different investigators include the use of different materials and techniques to study the influences of curing methods.

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

When environmental conditions and concrete properties are such that no significant drying or thermally induced stresses develop on the concrete structure, minimal curing practices may be satisfactory. Because of the high ratio of exposed surface area to total volume, slabs and pavements rarely are in this class of concrete. For example, merely keeping formwork in place for 2 to 3 days might be an effective form of minimizing moisture loss for small to moderately sized vertical elements.

Although more internal heat is retained when elements are wrapped with insulation materials, such as Styrofoam or heat-retaining blankets, doing so can effectively reduce the magnitude of the temperature gradients, the principal cause of thermal cracking. Provided the peak temperature and chemical properties of the paste is conducive to avert the threat of delayed ettringite formation, insulation can be a very effective means of curing. When considering curing concrete in this manner, the period that elements must remain insulated should also be determined. Premature removal of the insulation could cause the concrete to crack, and doing so will completely negate the time and expenses put forth to prevent such cracks from occurring.

Curing materials include:

- moisture retention
- water sprinklers

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- polyethylene sheeting
- liquid curing compounds
- wet burlap
- heat retention
- insulated blankets
- straw or hay.

The most effective but seldom used method of water curing consists of total immersion of the finished concrete unit in water. Ponding is an excellent method, wherever a pond of water can be created by a ridge or dike of impervious earth or other material at the edge of the structure. When considering the benefits of water curing vertical surfaces versus the logistical implications water curing represents, the author feels that the cost would outweigh the benefits. Since nearly all of the water applied to a vertical surface would runoff, there will be little if any benefit derived from this method of curing.

Fog spraying or sprinkling with nozzles or sprays provides satisfactory curing when immersion is not feasible. Lawn sprinklers are effective where water runoff is of no concern. Intermittent sprinkling is not acceptable if drying of the concrete surface occurs. Soaker hoses are useful, especially on surfaces that are vertical. Burlap, cotton mats, rugs, and other coverings of absorbent materials will hold water on the surface, whether horizontal or vertical. Liquid membrane-forming curing compounds retain the original moisture in the concrete but do not provide additional moisture. Monomolecular film-forming compounds have been effectively employed for interim curing before deployment of final curing procedures for surfaces exposed susceptible to drying during finishing.

Curing is probably the most essential element when working with high-strength concrete, especially concretes containing fine sized supplementary cementitious materials such as silica fume, metakaolin, and ultra-fine fly ash.

Moisture requirements

Proper curing is vitally important, especially as concrete undergoes its transition from a plastic to a hardened material. When the bleeding rate of high-strength concrete is exceeded by the evaporation rate, interim-curing measures such as fog sprays or evaporation retardants should be used to prevent plastic shrinkage cracking. Freshly placed concrete becomes increasingly vulnerable to plastic shrinkage as portions of the placed element are subjected to dehydration. The need for interim curing will depend on the characteristics of the concrete being used and the environmental conditions. Whether or not interim curing is needed for a particular placement, contingencies should always be in place to employ interim curing practices. Concrete's vulnerability to plastic cracking increases as the setting time of the concrete increases. When the predominant cause of surface evaporation is wind-induced, the cracks generally form in a direction perpendicular to

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the wind direction. Unlike wind-induced cracking, plastic cracking caused by drying conditions such as low humidity or high air temperature generally are oriented in a more random direction.

The development of plastic shrinkage stresses, and the resultant cracking that may occur can be eliminated by preventing dehydration while concrete is still in a plastic condition.

Plastic shrinkage cracking (Figure 8.1) may take the form of relatively large, parallel, well-spaced cracks that begin shallow but may penetrate deeply into the concrete. In other cases, plastic shrinkage cracking may take the form of a fine pattern of map cracks that penetrate only 15–30 mm into the concrete. These are difficult to see on textured or tined pavements. These types of cracks do not seem to cause problems in some situations, but in other cases, they provide an entry for deicing salts and may contribute to freezing and thawing damage.

Figure 8.1 Severe plastic shrinkage cracking caused by ineffective interim curing. Cracks of this nature start out relatively shallow, but subsequently are capable of behaving as contraction joints, causing the cracks to propagate deeper into the section. Courtesy of CTL Group.

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Unless effective measures are taken to preclude premature dehydration until the concrete sufficiently hardens, the likelihood of plastic shrinkage cracking remains. Concrete sections such as slabs are particularly vulnerable to the development of plastic shrinkage stresses due to the high ratio of surface area-to-total volume. When moisture loss (evaporation) occurs while the concrete is still plastic, shrinkage stresses will develop. Since plastic concrete has little or no strength to resist these stresses, when the tensile strength of the fresh concrete is exceeded, stress relief will occur in the form of localized tearing (cracking).

The length, depth, width, orientation, and number of plastic cracks depend upon the magnitude and direction of the plastic shrinkage stresses. The degree to which plastic shrinkage cracking occurs depends on three primary factors:

- the setting rate of the concrete;
- the bleeding rate of the concrete; and
- the rate of evaporation from the surface.

Plastic shrinkage stresses principally develop due to the loss of water by evaporation from the surface, but may also develop when fresh concrete is in contact with absorptive materials, such as dry hardened concrete or a dry sub-base.

High-strength concretes usually do not exhibit much bleeding, and without protection from loss of surface moisture, plastic shrinkage cracks have a tendency to form on exposed surfaces. Curing should begin immediately after finishing, and in some cases, other protective measures should be used during the finishing process. Curing methods include fog misting, applying an evaporation retarder, covering with polyethylene sheeting, or applying a curing compound (ACI 363).

In general, as the ratio of exposed surface area to total volume increases, the significance of curing intensifies. The importance of curing cannot be over-emphasized; however, employing the same curing procedures for all elements is both difficult and highly impractical. The need for effective curing increases as durability requirements increase. Curing has more significance on the long-term performance of exterior exposed elements such as bridge slabs compared to interior elements that will be maintained under near constant moisture and temperature conditions while in service. Curing vertically cast elements such as columns and walls in the same manner as horizontally finished elements such as bridge slabs is impractical, and other curing methods should be employed, such as leaving the forms in place. Since high-strength concrete elements contain higher quantities of cementitious materials, formwork removal may need to be delayed in order to preclude the incidence of thermal cracking. Alternatively, additional insulation might be needed so that the concrete has sufficient tensile strength

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to resist thermal stresses resulting from early formwork removal, especially in cold weather.

To prevent premature drying, upon finishing, concrete surfaces should be kept continuously moist for a period usually ranging from 3 to 7 days or sealed with a liquid curing membrane.

If water curing is employed, it should be done on a continuous basis throughout the curing duration. Intermittent water curing that allows concrete to undergo cycles of wetting and drying can be more detrimental than no curing whatsoever.

Moist curing enhances both strength development and permeability. As the moist curing period is increased, the strength development will increase and the permeability will be lower (Neville, 1971). Cast-in-place high-strength concrete should be cured at an early age since partial hydration may make the capillaries discontinuous. On renewal of curing, it will be quite difficult for water to be able to enter the interior of the concrete and further hydration would be arrested (Neville, 1996). Sprinkling on a continuous basis is suitable provided the air temperature is well above freezing. The concrete should not be allowed to dry out between soakings, since alternate wetting and drying can cause more distress than no moist curing whatsoever.

Ponding water onto a slab is an excellent method of curing. To avoid thermally shocking the concrete, the water should be tepid, preferably no more than about 10°C (20°F) cooler than the surface temperature of the concrete. An ample supply of water should be readily available at the jobsite if fogging or water curing is planned, or subgrade moistening is necessary. Burlap or cotton mats and rugs used with a soaker hose or sprinkler. Care must be taken not to let coverings dry out and absorb water from the concrete. The edges should be lapped and the materials weighted down so that they are not blown away.

Curing compounds are liquids that can be applied as a coating to the surface of newly placed concrete to retard the loss of water or, in the case of pigmented compounds, also to reflect heat to provide an opportunity for the concrete to develop its properties in a favorable temperature and moisture environment. Liquid membrane-forming compounds should be applied at the rate specified by the manufacturer. Do not apply to concrete that is still bleeding, or has a visible water sheen on the surface. While clear liquid may be satisfactory, white pigments are suggested since they will give reflective properties. A single coat may be adequate but where possible a second coat, applied at right angles to the first, is desirable for even coverage. If the concrete is to be painted, or covered with vinyl or ceramic tile, then a liquid compound that is non-reactive with the paint or adhesives should be used, or a compound that is easily brushed or washed off. On floors, the surface should be protected from the other trades with scuff-proof paper after the application of the curing compound.

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It should be noted that curing and sealing are different and that curing compounds are not sealers and vice versa. The principal purpose of curing compounds is to prevent loss of internal moisture, whereas the purpose of sealers is to prevent moisture ingress. These materials are usually sprayed or rolled on the surface. When dry, they form a thin film that restricts moisture evaporation from the surface. Timing is most important when using a curing compound. Curing compounds should be applied as soon as final finishing is complete. Otherwise, they could mar the concrete's surface.

Plastic sheeting and waterproof paper should be laid in direct contact with the concrete surface as soon as possible without marring the surface. The edges of the sheets should overlap and be fastened with waterproof tape and then weighed down to prevent the wind from getting under the material. Discoloration caused by a "greenhouse effect" may occur at wrinkled locations where the cover material is not in contact with the surface. When it occurs, discoloration of this nature is difficult to mitigate. For this reason, plastic sheeting and waterproof paper should not be used on concrete surfaces where appearance is important.

Internal curing

Due to the inherently low permeability of the matrix, internal curing is especially beneficial in concrete with a low W/B ratio, where external curing has little effect on hydration in the internal portion of the concrete. If the W/B ratio is below about 0.36, these mixtures can also self-desiccate because the amount of water included in the mixture is not enough to completely hydrate the cementitious materials (Villarreal and Crocker, 2007).

Internal moist curing is a method in which additional moisture for cement hydration is provided from within the concrete with no effect on the initial W/B ratio. Internal moist curing can be accomplished by the use of water saturated coarse, intermediate, or fine-sized lightweight aggregate or super absorbent polymers (Duran-Herrera *et al.*, 2007). Introduction of these materials into concrete will provide a source of water within the concrete matrix and better hydration of cement particles. Additional moisture in concrete becomes available through the slow release of water absorbed by the pores within the lightweight aggregate. In principle, any material that is used to provide a source of internal moisture has to be effectively saturated before it can be introduced into concrete mixture (Pyc *et al.*, 2007).

The benefits of internal curing were demonstrated through the reduction of autogenous shrinkage in cement mortars with w/cm ratio of 0.35 and 8 percent silica fume replacement by the use of saturated low-density fine aggregate or saturated super-absorbent polymer (Geiker *et al.*, 2004). Previous research also demonstrated that the most beneficial mechanism for internal curing would be a well-dispersed system of lightweight fine aggregate (Bentz and Snyder, 1999).

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Temperature requirements

Controlling the temperature of concrete prior to and after placement has long been debated. During hot weather, placement temperatures exceeding 35°C (95°F) should only be considered if it can be demonstrated that placement at higher temperatures would not be detrimental to the specified concrete properties. Concrete produced prior to the advent of retarding or hydration-controlling admixtures was significantly more vulnerable to the effects of elevated temperatures. If concrete placed under hot weather conditions is exposed to rapid temperature drops, thermal protection should be provided to protect the concrete against thermally induced cracking. Finally, curing materials should be readily available at the project site to permit prompt protection of all exposed surfaces from premature drying upon completion of the placement.

In winter, the ambient temperatures may be so low that it is necessary to take measures to ensure that the concrete temperature is maintained at a suitable level during the initial stages of curing. Exposure to below-freezing temperatures should be avoided at all cost. The cast concrete can be insulated against loss of heat generated by the hydration process.

Thermally induced cracking is commonly associated only with large-scale, mass elements; however, concrete elements do not have to be large in order for thermal cracking to be a concern. Thermal cracking can occur even in relatively thin slabs provided effectively large temperature gradients develop, particularly when high-strength concrete is involved. Thermally induced cracking should be a concern any time the developed tensile strength of the concrete is insufficient to resist stresses caused by temperature gradients at any given moment in time.

High-strength concrete has a higher potential for heat development than conventional-strength concretes, therefore, special attention should be given to curing in order to control the development of thermal gradients, which, if uncontrolled, may lead to cracking. In mass concrete, the difference between the warmest and the coolest portion of the member should not exceed approximately 20°C (35°F) unless it can be predetermined through thermal modeling that it is not detrimental to the structure. Similarly, unless it can be predetermined through modeling that higher internal temperatures will not result in DEF, the maximum developed internal temperature should be limited to 70°C (160°F).

Curing high-strength precast concrete

The use of accelerated curing in the production of precast/prestressed elements has been an industry practice for many years. With the greater use of high-strength concrete in precast/prestressed concrete, however, some of the traditional practices for accelerated curing need to be reassessed.

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Initial set

The Prestressed Concrete Institute (PCI) Manual (PCI, 1999) states that accelerated curing shall be started after the concrete has attained initial set as determined in accordance with ASTM C 403.[2] Since high-strength concrete can contain various combinations of cementitious materials and chemical admixtures, measuring the time of initial set for the specific materials used, and not relying on rules of thumb, is essential. Application of heat at too early a time may have a detrimental effect on long-term strength and durability. Conversely, applying heat after a long delay period may be less effective and will slow down production.

Rate of temperature rise

Since high-strength concrete usually contains much larger amounts of cementitious material than conventional-strength concrete, particular attention should be given to curing high-strength concrete in order to control the development of internal temperature differentials that, if uncontrolled, could lead to cracking. The PCI Manual states that the heat gain of the concrete shall not exceed 20°C (36°F) per hour. High-strength concrete generally contains a high quantity of cementitious materials. Consequently, the concrete is capable of achieving a higher peak temperature than that in the enclosure due to its own heat of hydration. Therefore, with high-strength concrete, monitoring the temperature of the concrete, not the temperature of the enclosure, is critical.

Maximum temperature

The PCI Manual states that the maximum temperature should not exceed 82°C (180°F) measured at the portion of the unit that is likely to experience the maximum temperature during curing. The Commentary also stipulates that if a known potential for alkali-silica reaction or delayed ettringite formation exists, the maximum curing temperature should be reduced to 70°C (158°F). For practical and economical reasons, the maximum temperature should not be greater than necessary to attain the minimum release strength in the required amount of time. Accelerating the early strength gain beyond that needed to achieve the release strength can make achievement of later age strengths more difficult with high-strength concrete.

When curing precast/prestressed concrete without the introduction of supplemental heat, the element needs to be enclosed to retain moisture on any exposed surfaces and to retain the heat. In colder climates, the use of insulated blankets may be appropriate. Without supplemental heat, the rate of temperature rise is not likely to exceed the values specified when curing with an external heat source. The principle concern under such conditions becomes maximum temperature, which is dependent on the concrete

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temperature at time of placement and the concrete temperature rise after placement. If the maximum concrete temperature is likely to exceed the specified maximum temperature, temperature control measures may need to be employed. Lowering the temperature of the concrete will reduce the rate of early strength gain, but provide a higher strength at later ages.

Notes

1 *Guide to Curing Concrete*, ACI 308R-01.
2 *Standard Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance*.

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9 Quality control and testing

Introduction

The preceding chapters have stressed the increased sensitivity of high-strength concrete to testing variables. Many of the test methods for high-strength concrete are not much different than for conventional concrete; however, with respect to strength and other mechanical properties, certain concerns become raised about the adequacy of current testing standards. Carino *et al.* (1994) demonstrated that the measured compressive strength of concrete is more sensitive to testing conditions as the target strength of the concrete increases. As the W/B ratio decreases (i.e. as strength increases), concrete becomes much less forgiving of inconsistent or improper sampling and testing procedures; therefore, the significance of good quality control procedures becomes paramount if high-strength concrete is to be tested and evaluated accurately. Discrepancies in sampling and testing procedures having negligible effects on conventional concrete can profoundly influence high-strength concrete.

In a robust and progressive performance concrete market, it would not be unusual to eventually reach a point where the advancements made in high-strength concrete technology exceed the industry's evaluation capabilities. In the early 1990s, after several years of successful research and development in commercially supplying high-strength concrete with a specified compressive strength of 110 MPa (16,000 psi) at 56 days, Material Service Corporation's main concern was not their ability to successfully produce the concrete, but rather, the testing industry's ability to evaluate it in a reproducible and reliable manner. Rosenbaum (1990) described how the rapid development of concrete with increasingly higher strength had outpaced the updating of testing practices to ensure reliable results. Pielert (1994) maintains that the key to competent testing is the development and implementation of an effective and comprehensive system by the laboratory involving both quality assurance and quality control activities.

ACI Committee 214 recognizes that discrepancies in measured test values for any material property can be traced to two fundamentally different sources—variability inherent to the concrete itself, which includes factors

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related to constituent materials, production, and handling; and variability inherent to the methods used to test the material. As the compressive strength of concrete increases, variability in measured test values also increases. This is not strictly due to an inherent variability in the quality of the material per se, but rather, the sensitivity of the material to variability associated with testing. It is difficult to assess the relative importance of these factors; in any event, their importance will vary for different regions and different construction projects (Mindess and Young, 1981).

With the continual transition that occurs between the relative mechanical properties of paste and aggregates as W/B ratios decrease, two major things happen which significantly influences concrete's sensitivity to strength testing. With increasing strength, modulus of elasticity, the slope of the elastic stress–strain relationship increases, and the magnitude of inelastic post-peak strain capacity decreases. Stated differently, as the physical strength of the material increases, it becomes increasingly brittle, and as a result, failure takes on more of an explosive nature. The mechanical properties of compression machines is just one of numerous factors responsible for potentially greater variability associated with high-strength concrete.

Strength is not an intrinsic property of concrete. Numerous variables influence the magnitude of strength results; including specimen size, geometry, age, moisture content, moisture distribution, loading rate, and testing equipment parameters. When defining strength and other mechanical properties, it is necessary to specify the test used to determine the value. This is precisely why standard test methods are developed and why it is important that they be strictly enforced. Organizations responsible for writing concrete-related standards include:

- American Society for Testing and Materials (ASTM)
- British Standards Institute (BSI)
- Canadian Standards Association (CSA)
- European Committee for Standardization (CEN)
- German Institute for Standardization (DIN)
- International Organization for Standardization (ISO)
- Standards Australia Limited
- Standardization Administration of China (SAC).

Even though the terminology and specific test methods may vary considerably from one standard writing organization to another, the fundamental objectives of standardization is universal; that is, concrete properties can only be reliably measured by samples made, cured, and tested under standardized, reproducible conditions.

Testing variables influencing compressive strength

Measurement of compressive strength during construction is by far the most common method of quality control or quality assurance, and it provides the

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most fundamental information needed to evaluate whether the concrete is capable of complying with the intended design requirements. The concrete industry relies heavily on the results of concrete compressive strength tests to determine the adequacy of as-delivered or in-place concrete, and momentous decisions have been based on measured strength (Richardson, 1991). As the target strength of concrete increases, it becomes increasingly more sensitive to variations related to both materials and testing, thus the magnitude of the standard deviation, the overall gauge of variability relating to both the material and the testing practices increases. Planning for inspection and testing of high-strength concrete involves giving attention to personnel requirements, equipment needs, test methods, and the preparation and handling of test specimens. Given this pronounced sensitivity, testing and acceptance standards developed for and applicable to conventional-strength concrete are not always appropriate for high-strength concrete. For this reason, elevated standard deviations should be anticipated as the target strength of concrete increases. If acceptance standards are not changed to compensate for this natural consequence, high-strength concrete performing well might be inappropriately viewed as performing quite poorly.

The consequences of deviating from some standardized test procedures may have a negligible influence on the outcome of the test; however, the consequences of others can be considerable. Initial curing test specimens at elevated temperatures and subjecting non-immersed specimens to prolonged initial curing periods in an air environment are two of the most egregious testing deviations. Each will be addressed in this chapter.

Mechanical properties such as elastic modulus, tensile strength, or modulus of rupture are frequently expressed in terms of compressive strength. Except for pavements and airport runways, compressive strength is the common basis for the design of most concrete structures. Compression tests assume pure uniaxial loading. In actuality, this is not the case due to frictional forces between the compression machine platens and the specimen surface, which restrain the specimen from lateral deformation. As the ratio of length to diameter (l/d) decreases the end effects become increasing more influential, resulting in artificially higher measured compressive strength. Testing variables can considerably influence the measured strength of high-strength concrete.

Sample representation

Proper sampling requires a well-planned and implemented sampling program, which should include considerations for the sampling frequency, sample size, sampling locations, and locations where tests are to be performed and samples are to be cured. Although seemingly a simple concept to comprehend, proper batch representation by a sample is often not achieved (Richardson, 1991). Samples for high-strength concrete, or any concrete for that matter, should *never* be obtained by untrained personnel, such as

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laborers working for the contractor. When sampling concrete, use every precaution that will ensure that the samples obtained truly represent the nature and condition of concrete used in the work. ASTM C 172[1] specifies composite samples are to be collected from *two or more* portions taken at regularly spaced intervals during discharge of the *middle* portion of the batch. Samples should never be obtained from the first or last portion of concrete discharging from the mixer drum. Upon procurement, the composite samples are then to be remixed to ensure sample uniformity. Sets of test specimens should always be fabricated from samples taken from the same batch. Test samples should be appropriately identified and a record should be maintained of the location that the concrete was placed, the time it was sampled, and the fresh test results.

Normal weight concrete is most commonly sampled as the concrete is delivered from the mixer to the conveying vehicle used to transport the concrete to the final point of placement, especially when the concrete is non-air entrained. When concrete is placed by pump method, some would argue that the only meaningful place to obtain samples is at the discharge end of the hose. Of course, the most logical place to sample concrete would be closest to its final point of placement; however, doing so can be both dangerous and unnecessary. Obtaining grab samples on ladders or scaffolding can be extremely dangerous and more often than not, simply not worth the risk. Depending on the rheological properties of the concrete or placement method used, changes to the slump or air-void characteristics could be great enough to warrant obtaining samples closer to the point of placement. For example, it might be more appropriate to do so when pumping lightweight concrete or air-entrained normal weight concrete. If air checks are desired at the discharge end of a pump line, take note that the pump configuration should not change in order to suit the sample procurement method. Changing the configuration can influence the air content of the concrete exiting the pump line and result in a non-representative sample being obtained. For example, if positioning the pump boom in a near vertical configuration is necessary to obtain a grab sample, much larger than actual amounts of air could be lost in the process of providing the sample. Increasing the dosage of air-entraining admixtures based on erroneous test results has resulted in the complete removal and replacement of bridge decks and parking structure slabs—enormously expensive remediation measures.

Specimen consolidation

Unless the concrete is of a self-consolidating consistency, most test specimens are consolidated by rodding or vibration. The method of specimen consolidation should match the consistency of the concrete. ASTM C 31[2] specifies the permissible methods of consolidation based on the measured slump of the concrete. Note also that the specific consolidation method used also depends on aggregate shape. Rodding mixtures containing excessive amounts

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of thin and elongated aggregate can orient the coarse aggregate pieces in a non-representative vertical position, and measured strength could be reduced by as much as 40 percent (Mercer, 1951). In such cases, vibration would be a more representation method of consolidation, regardless of the consistency of the concrete. The results of a limited study by Richardson (1989) suggest that using a piece of reinforcing bar with a flat end causes a decrease in measured strength. In the case of the materials involved, a strength decrease of 2 percent was reported.

Specimen size and shape

Many studies have been conducted to investigate the effect of specimen size and shape on compressive strength (Gonnerman, 1925; Kesler, 1954; Mindess and Young, 1981; Neville, 1981; Tanigawa *et al.* 1990; Baalbaki *et al.* 1992; French *et al.* 1993; Aïtcin *et al.* 1994). Comparisons were usually made between the compressive strength of 100 × 200 mm (4 × 8 in) cylinders to that of 150 × 300 mm (6 × 12 in) cylinders. Generally, 100 × 200 mm (4 × 8 in) cylinders exhibit higher strengths than 150 × 300 mm (6 × 12 in) cylinders. The difference may vary from 2 to 10 percent with a common value being about 5 percent, with the difference being lower for higher-strength concrete. Burg and Ost (1992) reported that the strength of 100 × 200 mm (4 × 8 in) cylinders was within 1 percent of the strength of 150 × 300 mm (6 × 12 in) cylinders. Specimen shape significantly affects the measured strength of concrete. Test results obtained from 150 × 300 mm (6 × 12 in) cylinders are commonly about 75 to 85 percent of those obtained from 150 mm (6 in) cubes. However, the difference in strengths obtained from cylinders and cubes decreases as the concrete strength level increases (Carrasquillo, 1994).

ASTM C 31 requires that the diameter of cylindrical specimens for compressive strength or splitting tensile determination be at least three times the nominal maximum size of the coarse aggregate. Therefore, since high-strength concretes are usually produced with smaller sized aggregates, many can reliably be tested using 100 × 200 mm (4 × 8 in) cylinders compared to the traditional 150 × 300 mm (6 × 12 in) cylinders. For concrete of a given strength, 100 × 200 mm (4 × 8 in) cylinders require compression machines with less than 50 percent of the force capacity required for 150 × 300 mm (6 × 12 in) specimens. The use of smaller test cylinders is acceptable provided the strength is determined in accordance with ASTM C 39 or similar standard. Burg *et al.* (1999) reported similar results for both specimen sizes when rigid upper test platens were used. Although smaller cylinders generally yield higher strengths, the range of test results can be more variable. This would increase the calculated standard deviation causing the required average strength to increase.

On average, 100 × 200 mm (4 × 8 in) cylinders will have measured compressive strengths approximately 2 percent higher than 150 × 300 mm

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(6 × 12 in) cylinders when testing high-strength concrete. The small magnitude of difference between the two specimen sizes suggests that 100 × 200 mm (4 × 8 in) cylinders can be used for acceptance purposes. However, due to the larger standard deviations exhibited by the smaller size cylinders, it may be both necessary and prudent to test more than the current code-specified two specimens to obtain a representative value of compressive strength.

Specimen moisture content and distribution

Both the moisture content and manner in which moisture is distributed at the time of testing can largely influence measured concrete strength. In general, uniformly moist specimens yield lower compressive strength than uniformly dry specimens. Upon drying, capillary forces acting on the specimen's outer surface generate lateral compressive forces that oppose the lateral forces developed during loading. Therefore, the apparent compressive strength of specimens with moist interiors and dry exteriors increases. Similarly, the apparent compressive strength of specimens with greater exterior moisture decreases. The strength of saturated specimens can be 15 to 20 percent lower than that of dry specimens. Parrott (1990) reviews the effects of sampling volume, sample geometry, pore fluid composition, and moisture gradients on methods for determining the moisture condition of concrete.

Mold material

The rigidity and watertightness of molds can significantly affect measured compressive strength. Compared to rigid steel molds, cardboard molds have been found to cause more than a 10 percent reduction in measured compressive strength (Blick, 1973), therefore they are not recommended for use with high-strength concrete. Carrasquillo and Carrasquillo. (1988) observed 150 × 300 mm (6 × 12 in) plastic molds yielding slightly lower strength than steel molds and 100 × 200 mm (4 × 8 in) plastic molds gave negligible difference with steel molds (ACI 363R-92).

Plastic single-use molds are generally considered satisfactory for testing high-strength concrete. Stiff steel molds may result in slightly higher measured compressive strength in ultra-high-strength concretes with target strengths exceeding 125 MPa (18,000 psi); however, in practical terms, the magnitude of the increase is not considered of practical importance (Burg *et al.*, 1999).

Initial curing conditions

The author cannot overemphasize how profoundly important proper field handling is when initially curing concrete test specimens. Being significantly smaller than full-scale elements, the measured strength specimens are

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strongly influenced by the changes in temperature and moisture experienced during the initial curing period. Elevated initial curing temperatures result in accelerated setting and high early strength, but reduced later-age strength. Low initial curing temperatures can have a reverse effect. Regardless of the age of the test, both cases will yield erroneous results. High-strength concrete frequently may contain hydration-controlling admixtures. Being much smaller in mass compared to the actual element, the rate of hardening and strength development of high-strength concrete specimens can be slower; therefore, handling and transporting high-strength concrete specimens the next morning may cause damage to occur. Care should be exercised when terminating the initial curing of high-strength concrete.

Figure 9.1 illustrates the effect of high temperature curing on measured 28-day compressive strength for concrete with a specified compressive strength of 35 MPa (5000 psi) at 28 days. In order to comply with the acceptance criteria of ACI 318, an average 28-day target strength of approximately 44 MPa (6400 psi) was needed. Three sets of test cylinders were cast from the same sample, yet cured under the following three different initial curing conditions:

- Set No. 1—Field cured for one day at 22°C (72°F) (compliant with ASTM C 31), then standard cured for 27 days.

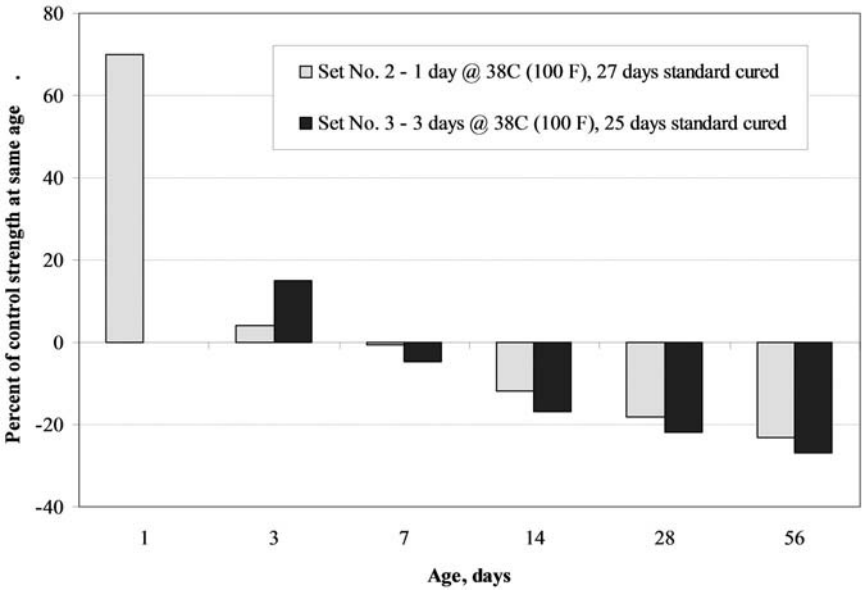

Figure 9.1 Effect of high temperature initial curing on compressive strength of 150 × 300 mm (6 × 12 in) cylinders.

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- Set No. 2—Field cured for one day at 38°C (100°F) (non-compliant with ASTM C 31), then standard cured for 27 days.
- Set No. 3—Field cured for three days at 38°C (100°F), then standard cured for 25 days.

As Figure 9.1 shows, at 28 days, the test cylinders that were subjected to industry standard initial curing conditions (Set No. 1) attained the desired target strength. However, both sets of test cylinders subjected to high initial curing temperatures had substantially lower 28-day measured strengths. Comparing the results of Set No.s 2 and 3 suggests that just one day of elevated temperature curing was enough to cause a profound reduction to later-age strength. If non-compliant testing of this nature regularly occurred, the results would suggest that this moderately high-strength concrete was incapable of achieving the 44 MPa (6400 psi) target strength necessary for code compliance. Unfortunately, occurrences of this nature are all too common. Immersing specimens in saturated limewater maintained between 15 and 27°C (60 to 80°F) is, in the author's view, the most practical and effective method for maintaining specimens within standard temperature and humidity conditions during the initial curing period (Figure 9.2). When initially curing specimens via limewater immersion, specimen capping is optional provided the specimen tops are completely immersed. For cylinder caps, Pistilli, *et al.* (1992) suggests that caps should have a clearance of at least 13 mm (½ in).

Figure 9.2 Initial jobsite curing by immersion in lime-saturated water. Courtesy of CTLGroup.

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In cold weather, specimens should be stored in boxes, and when conditions warrant, the curing boxes should be insulated and heated (Figure 9.3). The maximum and minimum temperatures developed will depend on the heat retaining characteristics of the curing box, the number of specimens present, and the heat liberating properties of the concrete. Without control measures, maintaining temperatures within the permissible range will be difficult.

In any event, correlating the compressive strength of small test specimens with full-scale elements will lead to high inaccuracies. Maturity and match curing is more suitable for assessing early in-place strength.

Following initial curing, high-strength concrete test specimens are generally more vulnerable during transportation back to the laboratory for final curing. Damage often occurs in the form of microcracks not readily visible to the eye. By placing test specimens in transportation boxes, such as the one shown in Figure 9.4, the opportunity for damage can be greatly reduced.

Final curing conditions

High-strength concrete compressive strength test specimens can be adequately cured in a moist room. Although a simpler method of curing, there is no particular need for underwater limewater curing for high-strength concrete test specimens. Underwater curing can be a relatively simple and effective means of standard curing for test laboratories located at the

Figure 9.3 Temperature controlled jobsite-curing box. Courtesy of CTLGroup.

Figure 9.4 Transportation boxes (right) can significantly reduce the chances specimens will be damaged during transportation. Courtesy of CTLGroup.

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jobsite. If the storage capacity of test specimens is insufficient for the number of specimens produced, conditions such as the one shown in Figure 9.5 may develop.

End preparations

The effect of end preparation depends on the strength level of the concrete. Carino *et al*. (1994) investigated the significance of sulfur capping and grinding using concrete with strength levels of 45 MPa (6500 psi) and 90 MPa (13,000 psi). No strength difference due to the method of end preparation was observed for the 45 MPa concrete, but for the 90 MPa concrete, grinding resulted in as much as 6 percent greater strength. Lessard *et al*. (1993) found a commercially available "high-strength" capping compound to be satisfactory when used for testing concrete with strengths up to 120 MPa, provided the capping layer is less than 3 mm thick.

The appropriateness of capping compounds depends largely on the cap thickness provided. Certain capping materials appear be suitable for testing high-strength concrete; however, the compressive strength of the capping compound alone should not form the sole basis of selection. The most suitable means to judge the adequacy of a particular capping compound is by performing comparative testing with cylinders having surface ground ends.

Pistilli and Willems (1993) compared traditional sulfur caps with unbonded neoprene pads in compressive strength testing of concrete with

Figure 9.5 Improper storage of test cylinders at field laboratory.

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strengths ranging from 20 to 125 MPa (3000 to 18,000 psi) and compared sulfur caps with specimens having ground and lapped surfaces within the range of 90 to 138 MPa (13,000 to 20,000 psi). Significantly lower within-test variability occurred with neoprene pads compared to the sulfur caps for strengths above 55 MPa (8000 psi). The ratio of 100 × 200 mm to 150 × 300 mm (4 × 8 in. to 6 × 12-in) cylinder strengths ranged from 0.96 to 1.06. The strength differences due to cylinder size did not appear to be of practical significance for concretes with actual measured strengths ranging from 28 to 62 MPa (4000 to 9000 psi). Grinding the ends of cylinders with measured strengths ranging from 83 to 138 MPa (12,000 to 20,000 psi) showed promise as an improved test procedure for end preparation. Provided the finished surfaces are smooth, neoprene pads appear to be a satisfactory alternative for concretes with strengths within the range of 90 to 138 MPa (13,000 to 20,000 psi).

Testing machines

Use of compression machines having adequate load transfer capability and appropriate stiffness (longitudinal and lateral) for the strength level of concrete being tested is critically important if high-strength concrete is to be accurately tested. Most concrete testing laboratories are equipped with compression machines inadequate for reliably testing 150 × 300mm (6 × 12in.) specimens of high-strength concrete. The results of an interlaboratory test program (Burg *et al*. 1999) conducted to determine the effects of selected variables on the measured compressive strength of high-strength concrete suggests that the requirements for test platen (spherical bearing blocks) are insufficient for concrete with compressive strengths exceeding 70 MPa (10,000 psi). Furthermore, the results of the study suggested that some test platen designs that meet current industry requirements result in nonuniform load transfer from the test machine to the test specimen, potentially resulting in a reduction of measured compressive strength of more than 10 percent when testing concrete with a compressive strength of 124 MPa (18,000 psi).

Loading rate

In general, compressive strength increases with increasing loading rate. ASTM C 39 specifies a permissible loading rate range for cylindrical specimens of between 0.14 to 0.34 MPa/sec (20 to 50 psi/sec). The compressive strength of cylinders tested at the high-end load rate limit has been determined to be approximately 3 percent greater than cylinders tested at the low load rate limit. As much as a 20 percent increase in compressive strength of high-strength concrete has been possible when loading rates exceed the limits in ASTM C 39 (Gedney, 2005). Under impact loading, strength may be as much as 25 to 35 percent higher.

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Standard cured vs. field cured specimens

A common misconception encountered in the field is that test samples should cure under conditions representative of the placed concrete rather than according to standardized testing conditions. Since subjecting test specimens to actual field conditions rather than artificial conditions would appear logical when the objective is to assess the quality of the construction, it is understandable why such practices frequently occur. Evaluating the quality of field placed concrete is important, particularly when information about the early properties of in-place concrete is needed, such as for formwork removal or to apply prestress forces; however, using specimens designated for standardized testing is inappropriate. Obla *et al.* (2005) described the following differences between standard curing and field curing:

> *Standard curing*: Subjecting the test specimens to standard temperature and humidity conditions and the strength results are primarily used for concrete acceptance and quality control.
>
> *Field curing*: Subjecting the test specimens to the temperature and humidity that the actual structure experiences and the strength results are primarily used for determining whether a structure is capable of being put in service and scheduling formwork removal.

The purpose of standardized tests is only to measure the properties of the concrete itself, which could not be possible if the test conditions vary. Standard tests are tools strictly used to evaluate the concrete itself, not the conditions upon which the in-place concrete is subjected. Standard tests are important for ensuring proper batching and revealing problems that may be related to the constituent materials or mixture proportions. *Standard tests must always supersede field tests*. If less than desired performance occurs when testing field-cured specimens, without having specimens handled under standard conditions, how would it be possible to determine whether the source of the problem was inherent to the concrete or the potentially extreme conditions the samples were subjected? Test specimens, being of a much smaller mass than the placed concrete, are much more vulnerable to changes in moisture or temperature. Field cured specimens would hydrate at a different rate than the in-place concrete, thus yielding inaccurate data. Under hot weather conditions, small specimens could yield higher than actual strength early strength results. Conversely, under cold weather conditions, small specimens could yield lower than actual early strength results.

When field-cure testing is conducted, it is imperative that field tests be conducted in unison with standard tests. The results of field-cured tests alone are an invalid basis for the acceptance of concrete. ACI 318–05 has established that the procedures for protecting and curing in-place concrete shall be improved when the measured strength of field-cured (job-cured) cylinders at the designated acceptance age test age for determination of f_c'

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is less than 85 percent of that of companion laboratory-cured cylinders. The code currently states that if the field-cured strength exceeds f_c' by more than 500 psi, the 85 percent limitation shall not apply. The 85 percent limitation has been based on research showing that cylinders protected and cured to simulate good field practice should test not less than about 85 percent of standard laboratory moist-cured cylinders. The comparison is to be made between the measured strengths of companion job-cured and laboratory-cured cylinders, not between job-cured cylinders and the specified value of f_c'. However, results for the job-cured cylinders are considered satisfactory if the job-cured cylinders exceed the specified compressive strength by more than 500 psi, even though they fail to reach 85 percent of the strength of companion laboratory-cured cylinders.

The importance of adhering to established methods of testing is critically important for high-strength concrete. With few exceptions, most deviations from standardized methods of testing will result in a decrease in measured strength. One exception would be to initially cure test specimens in a cooler than specified environment. Provided the specimens are not subjected to freezing temperatures or transported prematurely, the slower rate of hydration that comes from cooler temperature curing could artificially increase later age measured strength. Whether or not testing deviations cause increases or decreases in measured strength is irrelevant. Departures from standardized testing methods introduce inaccuracies, and the results will not reflect the true manner in which the concrete can be expected to perform.

In-place evaluation

Drilled cores

The ACI 318 Code provisions require that the average strength of three drilled cores extracted from suspect locations meet or exceed 85 percent of the specified strength (f_c') and no single core be less than 75 percent of f_c'. The relationship between the compressive strength of 150 × 300 mm (6 × 12 in.) cylinders and drilled cores from a column was studied by Cook (1989) for concrete with a specified compressive strength of 69 MPa (10,000 psi). It was concluded that the 85 percent criterion specified in the ACI Building Code could also be applicable to high-strength concrete. The study also confirmed that field cured specimens were *unreliable* indicators of the in situ strength (ACI 363R-92).

In another study, Akers and Miller (1990) evaluated the relationship between 150 × 300 mm (6 × 12 in.) cylinders, 100 × 200 mm (4 × 8 in) cylinders and drilled cores. The results showed that the strengths obtained from drilled cores were greatly influenced by three factors: (1) their tested orientation relative to that in the structure; (2) the elevation of the core in the structure; and (3) the type of pre-test conditioning. A comparison of the core and cylinder compressive strengths indicated that the acceptance

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criteria of the ACI 318 Code might 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. Burg and Ost (1992) reported on 100 mm (4 in) cores drilled from 1220 mm (4 ft) cubes of concrete with compressive strength in the range of 70 to 140 MPa (10,000 to 20,000 psi) (ACI 363.2R-98).

In an evaluation of engineering properties of six commercially available high-strength concrete mixes in the range of 70 to 140 MPa (10,000 to 20,000 psi), Burg and Ost (1992) reported that the core compressive strength tested at 91 days and 14 months was only slightly lower than the strength of companion insulated cylinders. In the study, all but one mixture exceeded 85 percent of the specified design strength of the concrete. It was further reported that no significant strength difference was found between cores taken from near the surface and the center of large-sized cubes (ACI 363.2R-98).

Maturity method

Curing temperatures profoundly affect the strength development properties of concrete elements and test specimens. Accurate determination of early in-place strength is critically important when pre-tensioning, post-tensioning, or removing formwork. The Maturity Method (ASTM C 1074[3]) has long been used as a generally reliable indicator of in-place strength development. The concept is because temperature is a critical factor in the progress of cement hydration and thus of strength development of concrete, especially at early ages. The maturity of concrete is determined by multiplying an interval of time by the internal temperature of the concrete in question. This product is summed over time, and concrete maturity is equal to the sum of these time-temperature products (Kehl *et al*., 1998). One major drawback of the Maturity Method is the underlying assumption that the concrete in the full-scale member is comprised of the same materials and mixture proportions as the concrete that was used for the establishment of the original strength-maturity curve; an assumption that is not always true. For this reason, the match-cure method is more preferred.

Match-curing method

Match-cure technology takes the maturity concept to the next plateau by curing the test specimens in a heat environment identical to that of the in-place concrete. Testing a match-cured specimen would be very similar to testing a virtual sample extracted from the concrete member itself.

Match curing is a system in which a standard-strength specimen is cured at the same temperature as that measured in a concrete element. The system includes a temperature sensor in the member, a controller, a special insulated

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cylinder mold with a built-in heating system, and a temperature sensor in the mold. A reference sensor is located in the member to obtain the temperature of the freshly placed concrete. The reference sensor and the sensor from the cylinder mold are connected to the controller. The controller continuously compares the reference temperature with the temperature of the cylinder mold. When the reference sensor temperature exceeds the cylinder temperature, the controller activates the heater on the cylinder until the cylinder temperature and reference temperature are equal. One controller can be used with several molds. The controller can be replaced with a personal computer that can also record temperature versus time. As research data indicate, a match-cured cylinder produces a compressive strength that more closely matches the strength of the concrete in the member than the strength measured using other curing methods. This is particularly true at early ages (FHWA/NCBC, 1999).

Rebound number

Rebound numbers are obtained using a spring-driven steel device, commonly referred to as a "rebound hammer" (Figure 9.6). The rebound hammer is often used as a means to characterize in place concrete when the strength comes into question; most commonly when standard cured test specimens fail to achieve required strength. The essence of the test involves correlating surface hardness to compressive strength, provided the device has been

Figure 9.6 Rebound number determination using a "rebound hammer." Courtesy of Portland Cement Association.

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calibrated to concrete of a known compressive strength, such as cylinders and cubes. Standard methods for rebound number testing includes ISO/DIS 8045, ASTM C 805, BS-1881–202, and EN 12 504–2.

Most rebound hammers provide hardness-to-strength correlations from about 14 to 55 MPa (2000 to 8000 psi); therefore, their applicability with high-strength concrete is quite limited. Rebound numbers are highly sensitive to near-surface characteristics; therefore, caution should be exercised when interpreting test results. For example, erroneously high hardness values are obtained from surfaces that have carbonated. When performed on rough surfaces, the results might indicate lower than actual hardness. Rebound number testing is suitable for comparing the relative surface hardness of different locations but the test should not be used for acceptance purposes.

Penetration resistance

The penetration resistance test method[4] can be used for in-place testing of the surface hardness and strength of conventional- and high-strength concrete. Unlike the rebound number method, which is appropriate for use up to about 55 MPa (8000 psi), the penetration resistance method has been found to be suitable for testing concrete with compressive strength up to about 110 MPa (16,000 psi). In addition to its use in low-strength investigations, the penetration resistance test is also used to measure early in-place strength for formwork stripping. The penetration resistance test (Figure 9.7) involves driving a steel probe through a template. The zone and depth of penetration by the probe are then correlated to the surface compressive strength of the concrete. Silver probes should be used for testing normal weight concrete

Figure 9.7 Evaluating surface hardness by means of penetration resistance method (left). Three hardened alloy-steel pins of specified hardness are driven into the surface (right). The average exposed pin length is determined by placing a triangular base plate over the pins. Courtesy of Portland Cement Association.

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with densities exceeding approximately 2000 kg/m^3 (125 lb/ft^3). Gold probes should be used for testing lightweight concrete with densities below approximately 2000 kg/m^3 (125 lb/ft^3). Standard test methods include ASTM C 803[5] and BS 1881–207.[6]

Profiling constituent materials in the laboratory

Distinguishing cements suitable for use in high-strength concrete in a local market is best accomplished in the concrete laboratory using local materials. The process of identifying "high-strength cement" does not have to be exhaustive; it can be accomplished in a relatively straightforward manner. Tables 9.1a and 9.1b presents an example set of concretes that the author has previously used for profiling the strength characteristics of cement in concretes of varying composition and strength grades. In this example, the author's objective is to evaluate the performance of six different Portland cements in four concrete mixtures of varying strength and/or composition. The following mixtures were selected for the study:

- Mix A: Conventional strength (plain).
- Mix B: Conventional strength (with SCMs and chemical admixtures).
- Mix C: Moderately high strength (with SCMs and chemical admixtures).
- Mix D: High strength (with SCMs and chemical admixtures).

Mixtures A and B represent conventional-strength concretes having a specified compressive strength of 28 MPa (4000 psi) at 28 days. Mixture A is "plain" concrete proportioned without chemical admixtures or supplementary cementing materials. Mixture B contains a conventional (Type A) water-reducing admixture and a high calcium fly ash, both materials being locally available and used at dosages representative of local market conditions. Mixture C represents a moderately higher-strength concrete having a specified compressive strength of 40 MPa (6000 psi) at 28 days. Mixture C is proportioned with the same water-reducer and fly ash as used in mixture B. Mixture D represents high-strength concrete with a specified compressive strength of 70 MPa (10,000 psi) at 56 days, containing the same fly ash, along with high-range water-reduce and a retarding water-reducer. Keeping in mind that mixtures A, B, and possibly even mixture C could conceivably be subject to jobsite retempering, the slump of Mixtures A, B, and C were adjusted using water. For mixture D, the W/B ratio would be held fixed. Adjustments to the consistency of mixture D would be achieved through the addition of high-range water-reducer. The jobsite addition of water to even a moderately high-strength concrete, such as mixture C, should be discouraged; however, in the case of mixture D, both tempering and retempering should be expressly prohibited. A program of this scope can provide key information about the relative rate of strength development,

Table 9.1a Example mixtures used for laboratory evaluation of various cement samples (SI units)

			Kg/m³					*l/m³*		
Mix designation	*Specified compressive strength, MPa*	*Target slump, mm*	*Portland cement*	*High calcium fly ash*	*Fine aggregate*	*Coarse aggregate*	*Water*	*WRA*	*Retarder*	*HRWR*
A	28 @ 28 days	100	333	0	856	1033	Varied	0	0	0
B	28 @ 28 days	100	250	47	885	1033	Varied	0.50	0	0
C	40 @ 28 days	100	347	74	779	1033	Varied	0.68	0	0
D	70 @ 56 days	200	472	118	620	1003	165	0	1.24	Varied

Table 9.1b Example mixtures used for laboratory evaluation of various cement samples (inch-pound units)

			lb/yd³					*oz/yd³*		
Mix designation	*Specified compressive strength, psi*	*Target slump, in*	*Portland cement*	*High calcium fly ash*	*Fine aggregate*	*Coarse aggregate*	*Water*	*WRA*	*Retarder*	*HRWR*
A	4000 @ 28 days	4	564	0	1450	1750	Varied	0	0	0
B	4000 @ 28 days	4	423	80	1500	1750	Varied	12.7	0	0
C	6000 @ 28 days	4	588	125	1320	1750	Varied	17.6	0	0
D	10,000 @ 56 days	8	800	200	1050	1700	280	0	32	Varied

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stiffening rate, water demand, and setting characteristics of the cements evaluated.

Figures. 9.8 through to 9.11 present the results of compressive strength tests performed at ages 3, 7, 28, and 56 days. The data is presented representing the relative deviation from the average strength for each mixture at each age. Based on review of these data, of the six cements evaluated, only cements 2 and 6 appear to warrant further consideration for use in high-strength concrete. It was also noted that the W/B ratio decreased, the relative strength of cement 2 generally increased. A laboratory study of this nature can reveal useful, practical information about the relative performance of cements in a given geographic region in both conventional and high-strength concretes. Of course, as it has been emphasized throughout this book, strength should not be the only consideration when proportioning high-strength concrete. Other important properties that may need to be examined include setting characteristics, strength performance, and workability retention at extreme hot or cold temperatures, shrinkage, and durability potential. Similar laboratory studies can additionally be used for profiling any concrete constituents, including aggregates, supplementary cementitious materials, and chemical admixtures.

CASE STUDY: JOBSITE CURING IN LIMEWATER

For concrete to be properly evaluated, it is essential that testing practices be conducted in strict accordance with the required standards. Most deviations from standard test methods involving compressive strength will result in a decrease in measured strength, and some deviations will influence measured strength vastly more than others will. Concrete cylinders and cubes, being much smaller in mass than full-scale elements, are significantly more vulnerable to the effects on hydration due to elevated curing temperatures. This case study addresses one of the most influential and destructive practices influencing the later-age measured strength of concrete—subjecting test specimens to elevated initial curing temperatures.

The work involved construction of large public works building in a region experiencing moderately cool temperatures during the winter, but very hot, moderately dry conditions in the summer. The highest-strength concrete specified for the project had a specified compressive strength (f_c') of 41 MPa (6000 psi) at 28 days. It was specified for use in interior lower level columns.

When tested under standard conditions, the 41 MPa (6000 psi) concrete exhibited satisfactory 28-day strength performance. Column placements commenced in March when the air temperature ranged from an average daily low of 3°C (38°F) to an average daily high of 14°C (58°F). Using 150 × 300 mm (6 × 12 in) cylinders, Average 28-day compressive strength during that period was approaching 55 MPa (8000 psi). By early July, daily

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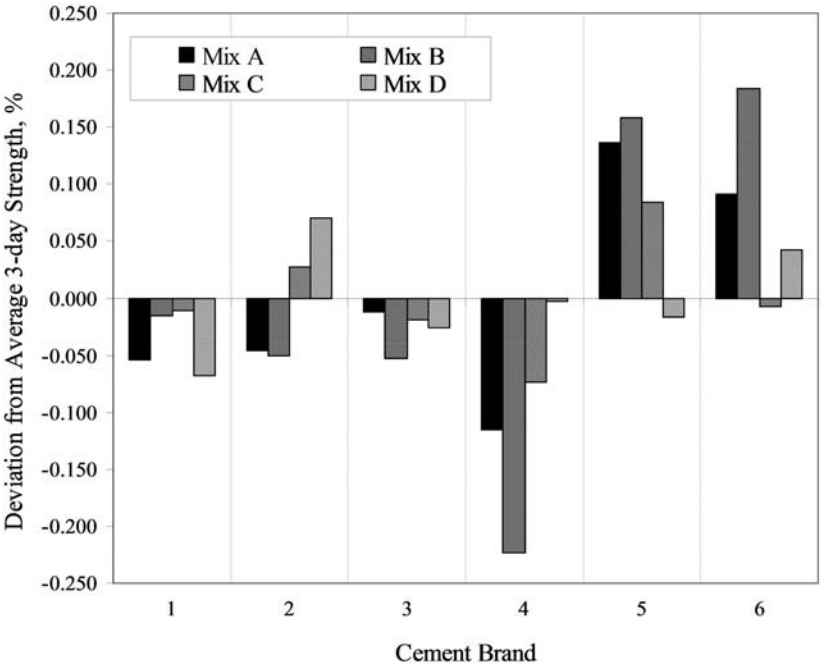

Figure 9.8 Deviation from average strength at 3 days.

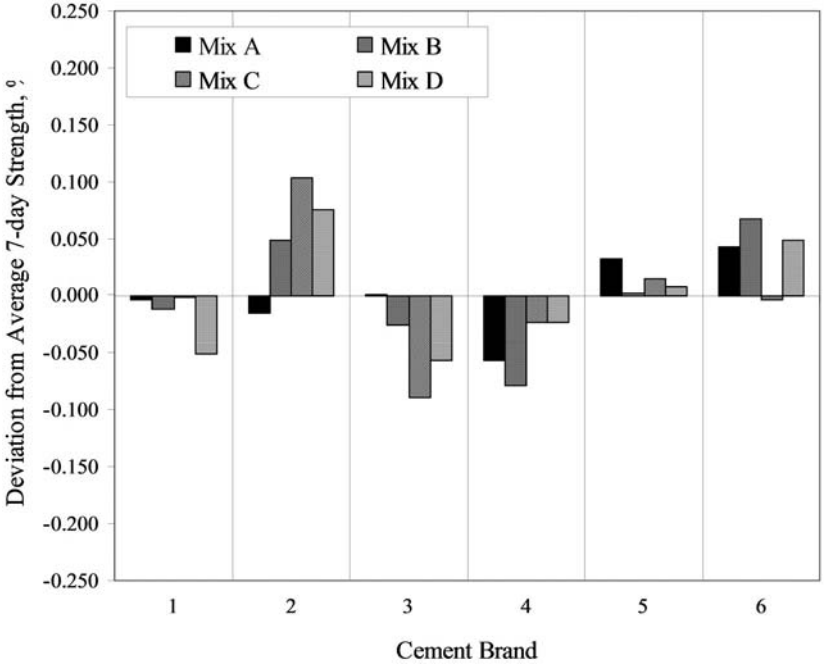

Figure 9.9 Deviation from average strength at 7 days.

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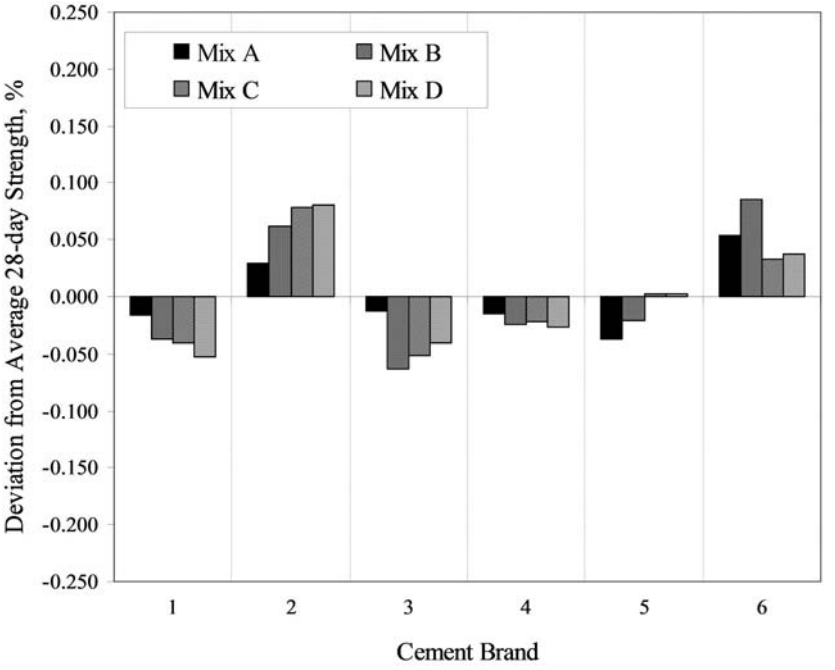

Figure 9.10 Deviation from average strength at 28 days.

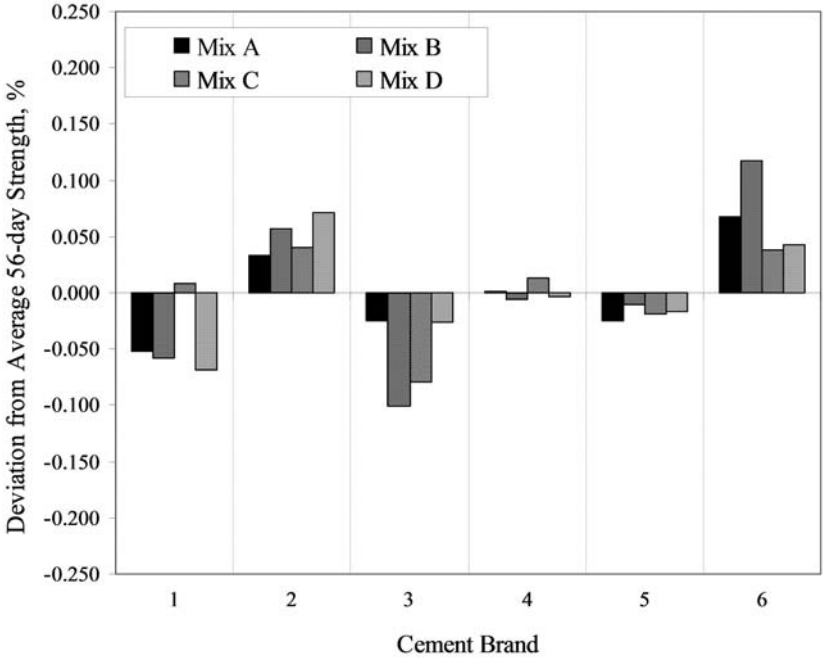

Figure 9.11 Deviation from average strength at 56 days.

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temperatures were ranging from a low of 21°C (70°F) to a high of 33°C (92°F) and climbing rapidly. By the end of July, numerous 28-day strengths below 6000 psi were beginning to be reported. Within a few more days the concrete producer was called into a jobsite meeting and informed by the owner and project engineer (via the contractor) that they would be held responsible for all costs associated with resolving the problem, including liquidated damages in the event of scheduling delays.

The investigation into the low strength complaint involved review of production records, delivery tickets, weather records, notes from daily field logs, and a site visit to review placement and testing practices. The production and delivery records indicated no discrepancies that would account for the low strengths. However, during the site visit, it was noted that the test cylinders, upon fabrication, were being covered with plastic bags and stored in the shade below a parked trailer. It was further noted that cylinders cast three to four days earlier had not yet been moved. Even though the specimens were placed in the shade, a check of the air temperature in the immediate proximity of test cylinders cast earlier in the day indicated 42°C (107°F). Suspecting that the initial curing conditions were a major factor contributing to the problem, the concrete supplier made immediate arrangements for the delivery of a 1150 L (300 gal) metal water tank (Figure 9.12) for the specimens to be initially cured in limewater rather than in air. Though apprehensive at first, thinking that limewater immersion

Figure 9.12 Initial jobsite curing by immersion in lime-saturated water. Cylinders in background represent previous, problematic initial curing conditions.

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amounted to "cheating," the project engineer and owners testing laboratory agreed that future specimens would be immediately immersed in the limewater without plastic bags or cylinder covers. The water temperature was checked daily and adjusted accordingly by adding ice when the temperature was found to exceed 27°C (80°C).[7] Changing nothing else, beginning with the next scheduled placement, the low strength problem immediately disappeared. Seven-day compressive strength increased by approximately 7 MPa (1000 psi). 28-day strength rose by approximately 10 MPa (1400 psi). The concrete producer never received back charges for the strength problem.

Authors note: As this case study demonstrated, curing fresh concrete test specimens in saturated limewater can be profoundly beneficial for ensuring accurate representative material testing. Increases in 7 and 28-day compressive strength on the order of magnitude described above was not only limited to high-strength concrete. The benefits of limewater curing during hot weather concreting can be just as beneficial with conventional strength concretes.

Notes

1 *Standard Practice for Sampling Freshly Mixed Concrete.*
2 *Standard Practice for Making and Curing Concrete Test Specimens in the Field.*
3 *Standard Practice for Estimating Concrete Strength by the Maturity Method.*
4 Also known as the "Windsor Probe" test.
5 *Standard Test Method for Penetration Resistance of Hardened Concrete.*
6 *Testing Concrete Part 207: Recommendations for the Assessment of Concrete Strength by Near-to Surface Tests.*
7 ASTM C 31 states that the permissible temperature range during initial curing shall be 16° to 27°C (60° to 80°F).

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10 Problem solving

Introduction

Successful problem solving is essential for maintaining the competitiveness of concrete with alternative construction materials. Problems occasionally occur with all construction materials, and it is imperative that they are resolved knowledgeably if they are to be prevented. It is natural for persons with extensive experience to sense causation early into an investigation; however, it would be a mistake on the investigator's part to prematurely arrive at conclusions without gathering all available information and thoughtfully considering all possible factors. Concrete problems are often the result of multiple factors, not just one, and solutions can often be arrived in multiple ways. One of the easiest ways for concrete investigators to lose credibility is by jumping to conclusions without eliminating possible extenuating circumstances.

Incompatibility[1]

The term "incompatibility" refers to undesirable interactions occurring between acceptable constituent materials resulting in unanticipated and objectionable performance. The advancements that have been made in the field of concrete materials technology have vastly increased the feasible realm of concrete applications. Without modern chemical admixtures, cements, and supplementary cementing materials, most high performance concrete would simply not be possible. Nevertheless, with increased performance demands comes increased risk. Inclusion of greater amounts of more complex materials means that concrete mixtures are progressively becoming more sensitive to conditions that in the past would not have been problematic. Due to the increased complexity of modern concrete, practices such as substituting one cement for another with the presumption that the substituted cement should "work about the same" as it has in the past, may now lead to poor performance (Roberts and Taylor, 2007). Understanding the fundamental nature of incompatibility problems is critically important if high-strength concrete is to be successfully produced.

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It is important to recognize that incompatibility problems are not attributable to only one material. Incompatibility problems involve two or more materials, which individually may not be problematic, yet when combined in certain proportions or dosages can adversely affect performance. The most common problems resulting from adverse material interactions include premature loss of workability (early stiffening), erratic setting behavior (rapid set or extended set), poor strength development, and poor quality air-void system characteristics. In some cases, concretes produced with incompatible materials have normal workability and setting characteristics when plastic, yet perform abnormally when in a hardened state. Many different mechanisms can contribute to incompatibility problems. The mechanisms causing such problems can be highly complex and are often interrelated. Often there is a very fine line between normal behavior and incompatible behavior, and there is usually no simple method of reliably determining the risk of incompatibility. It is precisely for this reason that trials should be conducted using candidate materials under actual job conditions.

In 1946, William Lerch published what is still considered by many to be the most comprehensive study on the optimization of sulfate in cement. Using isothermal calorimetry, Lerch showed that the magnitude of the silicate hydration peak (Figure 10.1), associated with hydration of the silicate phases in cement (tricalcium silicate (C_3S) and dicalcium silicate (C_2S) on mill certificates), depends on having enough sulfate present at the appropriate time.

For "normal" hydration to occur, the following order of events should take place:

1. C_3A initially hydrates.
2. SO_3 then takes control of the system and renders C_3A dormant for a limited period.
3. Re-hydration of C_3A proceeds again.

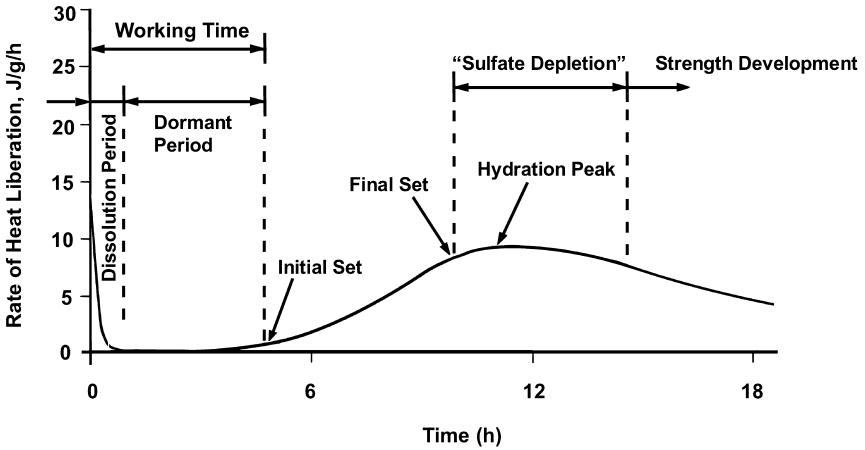

Figure 10.1 Profile of normal paste hydration.

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The interactions occurring between C_3A and sulfate during the early stages of cement hydration forms the basis of many incompatibility problems. Cement hydration in the first 15 minutes is a very delicate balance between the C_3A in the cement and sulfate in solution. The results of an extensive study by Tang (1992) strongly suggest that the very early aluminate hydration reactions can profoundly affect paste flow and ultimately strength development. If sulfate is supplied to the hydrating system at the appropriate rate (Figure 10.2), the hydration of the C_3A will be effectively controlled, and the concrete should stiffen and set without incident. However, if there is insufficient sulfate in solution (Figure 10.3), the C_3A begins to react immediately to form calcium aluminate hydrate, which causes immediate and unrecoverable stiffening commonly referred to as "flash set." C_3A hydrates

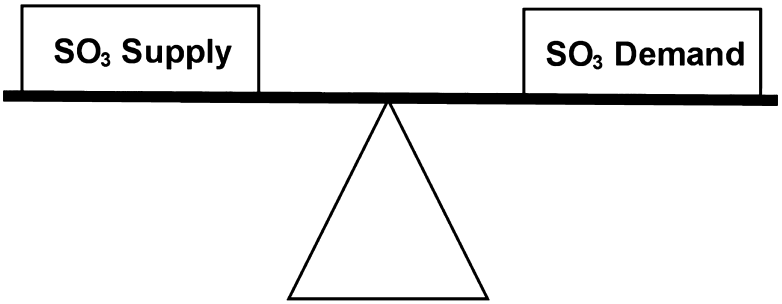

Figure 10.2 Illustration of an ideally balanced paste during the early stages of cement hydration. The system is being supplied sulfate at the same time it is needed to control C_3A reactivity.

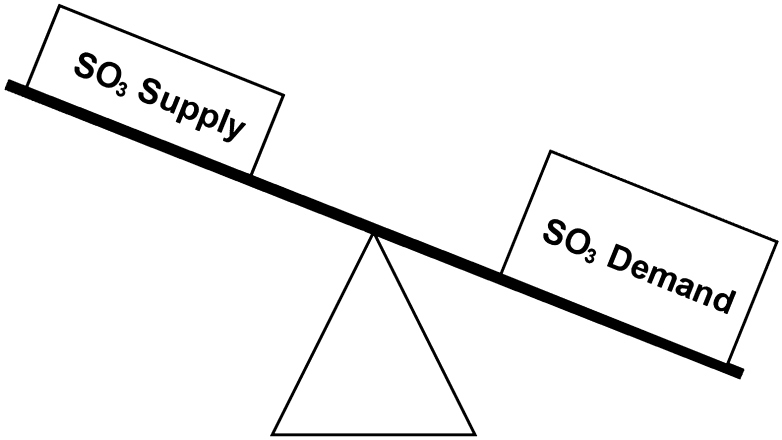

Figure 10.3 Illustration of an "under-sulfated" paste. When under-sulfated during the early stages of hydration, flash set may occur.

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at a more controlled rate in the presence of sulfate to form the trisulfate commonly known as "ettringite" (calcium trisulfoaluminate) while there is sulfate in solution. When the sulfates are consumed, ettringite continues to react to form monosulfate. Conversely, too much sulfate in solution (Figure 10.4) may precipitate out as gypsum, causing the formation of weak binding platelets and a temporary set that can be recovered merely through additional mixing—a phenomenon commonly referred to as "false set." Again, these reactions form the basis of many "incompatibility" problems.

The amount of sulfate in solution is dependent not only on the amount of sulfate ion in the cement, but the mineral phase in which it occurs. Cement that has overheated in the mill may contain excess amounts of relatively fast dissolving plaster ($CaSO_4 \cdot \frac{1}{2}H_2O$). Cement manufacturers will normally target a balance of plaster and gypsum ($CaSO_4 \cdot 2H_2O$) suitable for the reactivity of a given clinker type and cement fineness, and they will optimize the sulfate content to balance the setting time of the concrete. The fineness of the cement will also influence the reaction rates. As the fineness of cement increases, so does the risk of uncontrolled C_3A reactions with other ingredients in the concrete. Some chemical admixtures will interfere with C_3A hydration and the solubility of calcium and sulfate in the pore solution; thus, they may significantly affect the workability of the concrete in the first few minutes. Stiffening may result when water-reducing admixtures containing lignosulfonate or triethanolamine (TEA) are used in combination with some cements and high calcium fly ash, particularly in hot weather. Some chemical admixtures may reduce early slump when used with some cementitious combinations (Taylor *et al.*, 2006). Lignosulfonates tend to accelerate aluminate hydration and retard silicate hydration. Note

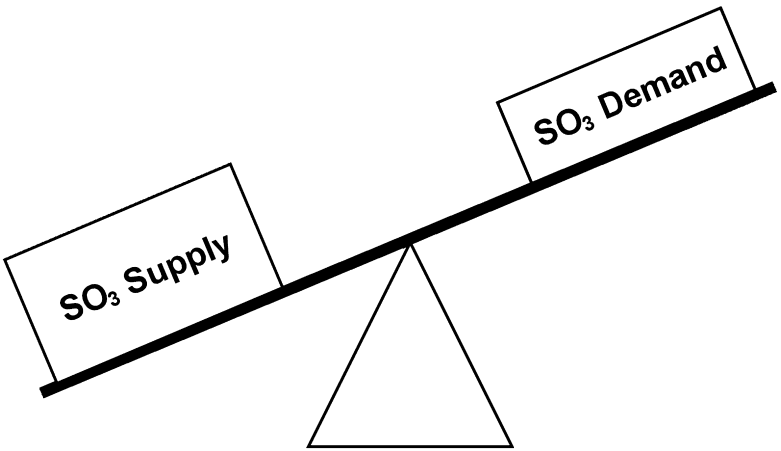

Figure 10.4 Illustration of an "over-sulfated" paste. When over-sulfated during the early stages of hydration, false set may occur.

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also that the rate of C_3A reactivity varies by cement source and type; therefore, two cements having equal amounts of C_3A might require quite different amounts or forms of sulfate.

The solubility and reactivity of all of the compounds are also strongly influenced by temperature, with higher temperatures generally increasing solubility (except calcium) and accelerating reaction rates. These changes can affect the balance of the system and change stiffening rates and setting times. This is true, even though the basic role of sulfate is to control the aluminate phases C_3A and C_4AF by forcing reaction to form ettringite. When the available sulfate level drops below the stability level, a secondary hydration peak is seen. This has been attributed to conversion of the trisulfate ettringite to the monosulfate form and, in some cases, to direct hydration of the aluminate phases. This will be referred to as the sulfate depletion peak, but it is not meant to imply that the sulfate is entirely gone. While some sulfate is usually present in the cement clinker as it exits the kiln, more is added during the cement grinding process, usually in the form of gypsum. The amount of sulfate added is limited by the ASTM C 150 requirement that allows more to be added when the C_3A content of the cement is higher. When the sulfate is low, the silicate phase reaction is suppressed. This can result in slow setting and, in some cases, little or no early strength development. When more sulfate is added, the silicate peak improves to the full hydration level, while the sulfate depletion peak occurs later. Current sulfate levels are based on achieving the maximum 1-day strength in 50 mm (2 in) mortar cubes mixed at laboratory temperature with no admixtures present (Roberts and Taylor, 2007).

Sulfate's role in hydration

Hydration begins as soon as cementitious materials are exposed to water. The cementitious particles partially dissolve, and the various components start to react at various rates. In high C_3A cements, the C_3A begins to react extremely rapidly to form calcium aluminate hydrate if there is insufficient sulfate in solution. If uncontrolled, this can cause immediate and permanent stiffening characterized by the liberation of large amounts of heat (flash set). Flash set does not normally occur in low C_3A cements, but rapid hydration of the C_4AF is possible, leading to slow silicate hydration. C_3A hydrates in the presence of sulfate to form ettringite at a more controlled rate. This control occurs because the ettringite forms around the C_3A grains and limits access of C_3A to water, but too much sulfate in solution may precipitate out as gypsum, causing stiffening without the liberation of heat (false set). False set is only temporary provided the concrete can continue to be mixed.

Chemical admixtures and supplementary cementitious materials can alter the amount of sulfate needed to control the aluminate reactions. If a supplementary cementing material containing additional calcium aluminates,

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such as high calcium fly ash is added to the mixture, the balance between aluminates and sulfates can be compromised because there is insufficient sulfate for the C_3A in the system, causing the problems discussed earlier. Likewise, if an admixture disperses the cement grains (thus exposing more surface area to water) or changes the solubility rate of calcium sulfates, the balance can be altered (Roberts and Taylor, 2007).

Early stiffening depends on several factors, including C_3A content and reactivity; alkali content; and the form, content, and distribution of sulfates in the cement. C_3A hydrating in the presence of sulfate ions forms ettringite on its surface. The ettringite acts as a barrier, limiting further reactivity.

The solubility and reactivity of all of the compounds are also strongly influenced by temperature, with higher temperatures generally increasing solubility (except for calcium sulfate) and accelerating reaction rates. These changes can affect the balance of the system and change stiffening rates and setting times. Another confounding factor is cement fineness, which also influences the reaction rates. After a dormant period of 1 to 3 hours, calcium becomes supersaturated in the pore solution, and the silicates (C_3S and later C_2S) start to hydrate and form solid compounds resulting in progressive stiffening, hardening, and strength development. If there is insufficient calcium in solution because it has been consumed in early C_3A hydration, silicate hydration will slow or stop, leading to retardation of the concrete or failure to set. A system may experience rapid stiffening in the first few minutes because of uncontrolled aluminate reactions. These reactions consume calcium, thereby significantly retarding setting.

Influence of chemical admixtures

Tuthill *et al.* (1961) described an example of a simple lignosulfonate based water reducing and retarding admixture that induced extended set and related the phenomenon to the sulfate level in the cement. Minor changes in the sulfate level that had little discernable effect on the setting of the cement alone caused the concrete to gain strength very slowly. As a result, concrete placed in the ceiling of a tunnel came down with the forms when they were stripped after the normal 10-hour interval. Because the admixture was needed due to placement conditions, increasing the sulfate level of the cement was required to solve the problem. In 1978, similar results were found by Khalil and Ward (1978) who used isothermal calorimetry on CSA Type 10 and 50 cements. Large retardations occurred with an ordinary lignosulfonate retarding admixture at normal dose when the low C_3A Type 50 cement was under-sulfated. In 1980, Meyer and Perenchio (1980) showed that these effects could be related to admixture components. For instance, although the triethanolamine in water-reducing admixtures typically reduces setting time and improves early strength at normal doses, it was shown to severely delay setting time when overdosed.

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High calcium fly ash

High calcium (Class C) fly ash contains aluminate phases that can cause early stiffening. Some chemical admixtures, particularly conventional water reducers, may disturb this balance. The exact causes of problems associated with certain combinations of portland cement and high calcium fly ash are not known (Gress, 1997); however, the possibilities include available alkalis, CaO, and aluminates. Free lime increases water demand, contributing to the loss of workability of the fresh concrete. Alkalis, CaO, and aluminates all affect setting. Alkalis accelerate the hydration reactions of the cementitious materials. Cement contains sulfates to control the hydration of the aluminates. However, if fly ash is added at the batch plant, the sulfates in the cement alone may not be sufficient to control the hydration of the additional aluminates from the fly ash. Some chemical admixtures, particularly conventional water reducers, may additionally disturb this balance (Taylor *et al*., 2006). In Figure 10.5, data from Cost (2006) are plotted to

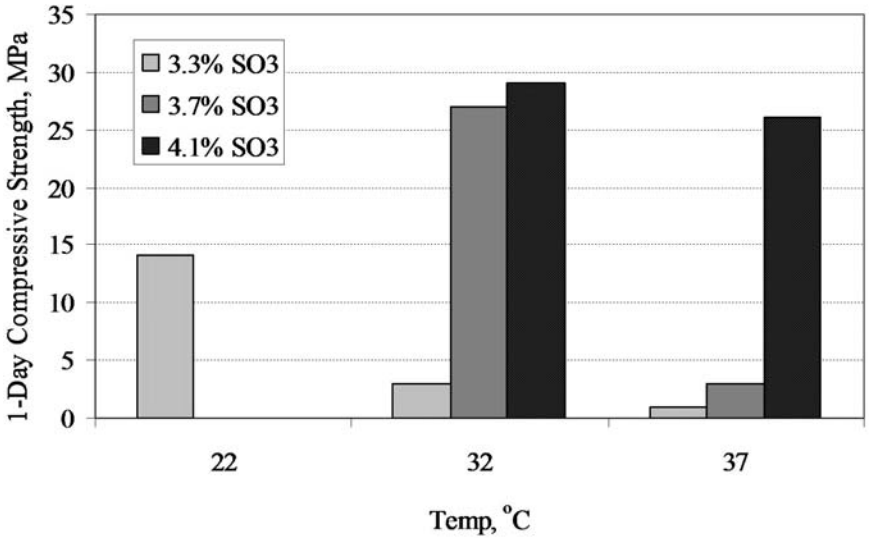

Figure 10.5 Data extracted from Cost (2006) showing 1-day strength values for mortars made using cement with various levels of sulfate, Class C fly ash used as a 25 percent cement replacement, and a carbohydrate-based water-reducing admixture. In addition to the odd behavior of decreased strength with increases in temperature, the data show the beneficial effect of additional sulfate. In this case, although the 4.1 percent sulphate resolved the strength issue, the cement was not able to pass the required ASTM C 1038 test for dimensional stability. Thus, at high temperature, the system of this cement, Class C fly ash, and this admixture placed a demand for sulfate that the cement could not supply without exceeding specification limits (after Roberts and Taylor, 2007).

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emphasize the critical effect of temperature. This is another case of interaction with high calcium fly ash where the 1-day strength was severely depressed at higher temperatures. The effect was again relieved by providing higher levels of sulfate in the cement. Unfortunately, the 4.1 percent sulfate level needed to achieve satisfactory performance at high temperature would not satisfy the maximum expansion limits in ASTM C 150 for the results of ASTM C 1038[2] tests. While increasing the sulfate content of the cement can sometimes solve these problems, the cement producer is constrained by the requirements of ASTM C 150 to optimize cement without the presence of either SCM or admixtures, and thus may be unable to increase the sulfate content enough to solve the problem. No cement producer can be expected to make cement immune to these potential problems, as the range of SCMs and admixture types, dosages, combinations, and temperatures is so great. Likewise, no admixture or SCM producer can produce products immune from problems if the system is pushed enough. Therefore, the party responsible for selecting the materials and proportions of the concrete should execute appropriate due diligence to ensure problems of this nature are avoided.

Useful tests

Roberts and Taylor (2007) discuss several useful laboratory tests that could be helpful in identifying potential incompatibility problems. Some test methods are suitable for identifying the risk of problems during the first 30 minutes because of aluminate/sulfate balance issues. Other tests are suitable for detecting later silicate hydration problems. They include:

- isothermal calorimetry and semi-adiabatic field calorimetry;
- a modified version of the early stiffening test per ASTM C 359[3] using supplementary cementing material doses replicating the field mixtures and admixtures added at various times; and
- the mini-slump test as described below.

The mini-slump test

The mini-slump test (Figure 10.6) was developed to assess the early stiffening of cement paste in the first 30 minutes of hydration (Taylor *et al.*, 2006). Cementitious paste is mixed at high speed in a high shear blender equipped with a cooling system that controls the final paste temperature. Chemical admixtures can also be added at various times to simulate field-batching procedures. After following a standard mixing schedule, samples of the paste are tested, using a small slump cone, at 2, 5, 10, and 30 minutes (and later if required) after the water is added to the cement. The area of the paste pat formed after the mini-slump cone is lifted is an indication of the cement paste

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Figure 10.6 The "mini-slump" test performed on paste samples. Courtesy of CTLGroup.

workability (Figure 10.7). An index of early stiffening is calculated by dividing the area of the pat for the test at 30 minutes by the area for the test at 5 minutes. Typically, a value less than 0.85 has been considered an indicator of rapid stiffening. Another value used to assess the stiffening rate is the average pat areas at 5 and 30 minutes. Low values (less than 14,800 mm^2 [23 in^2]) indicate a stiff mixture, likely caused by a high water requirement of the system.

Mini-slump is convenient to use in conjunction with isothermal calorimetry, as the same mixture can be used for both tests. Pastes can be somewhat more sensitive to compatibility issues than concrete, meaning that a system indicated as potentially problematic in the mini-slump test may be satisfactory in the field—therefore, care should be exercised in their interpretation.

Proposed version of mini-slump cone test:

- Cement: 500g
- W/B ratio: 0.50
- Mixing water temperature: 22 ± 1°C
- Mixing Schedule: 0.5 min. mix—2 min. rest—1.5 min. mix
- Mixing rpm: 13,000
- Testing Schedule:—2,5; Remix—15, 30, 45 min.
- Remixing: 2 min. prior to test 1,200 rpm for 1 min.

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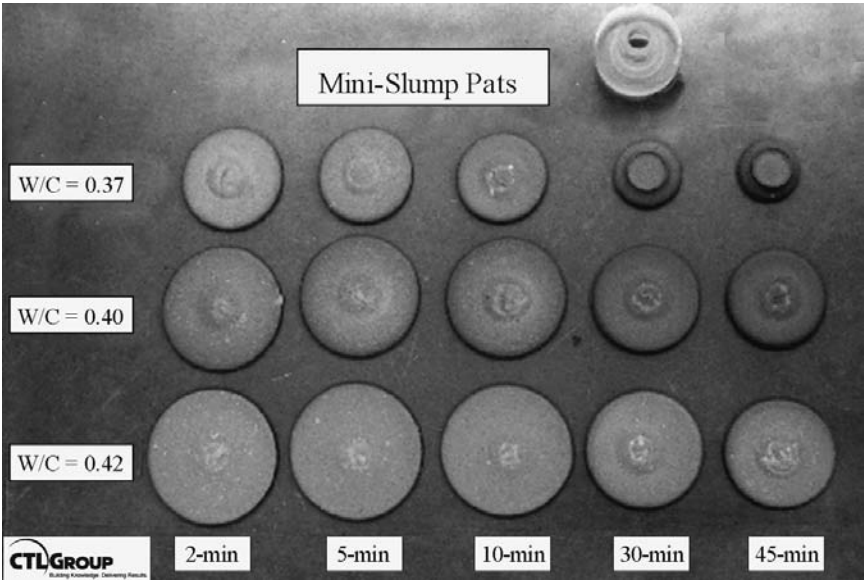

Figure 10.7 Mini-slump pats taken at 2, 5, 10, 30, and 45 minutes for mixtures produced with W/B values of 0.37, 0.40, and 0.42. Note the rapid reduction in size in the top row (W/B = 0.37), indicating rapid stiffening. Less stiffening is noted in the mixtures made with the same materials at higher W/CM values. Courtesy of CTLGroup.

Some tests are low cost and more appropriate for field use, but they tend to be less sensitive than more precise laboratory-based tests. Many of these tests take a long time to conduct, which is problematic for field applications where an answer may be required in a few hours. It also has been observed that in many of the tests, no threshold clearly indicates incompatibility with any given system; therefore, the greatest value of many of the field tests is in monitoring the uniformity of a system over time, such as using control charts. A marked change in a test result would indicate potential problems and necessitate investigation by other means. Such tracking would need to be based on knowing the acceptable ranges of that system for the environment where it is used.

This protocol has been developed on the premise of obtaining as much information as possible during a preconstruction phase. This work would include calibrating the more sensitive central laboratory tests with the equivalent field tests, using materials that are likely to be used in the field and environments similar to field conditions. This protocol also includes preparing alternative mix proportions and practices to accommodate changes in environment or in materials sources. Field tests developed for this protocol

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would be more rugged and conducted regularly, primarily to monitor the uniformity of the materials and the final mixture.

While most of the tests in this protocol are valuable, the extent of preconstruction and field testing depends on the availability of equipment and the relative cost of testing compared to the cost and risk of failures. A typical example is in selecting a method to determine setting time.

Reducing problems

To reduce the possibility of incompatibility problems, several precautions can be taken during the design of concrete mixtures containing portland cement, SCMs, and admixture combinations:

- Avoid excessively high doses of admixtures or SCMs.
- When high doses are needed, test beyond the expected levels to find both the nature and severity of any potential problems.
- Recognize that lower C_3A Type II and especially Type V cements have naturally lower sulfate contents, as required by specifications. They may have less free sulfate to contribute when an extra demand is placed on the system by SCMs or admixtures.
- When SCMs are used, the admixture dosage per unit of cementitious material may have to be reduced. This is especially true with mixtures with low W/B material ratios, with high cementitious material contents, or for high-performance concrete mixtures.
- Test mixtures over the temperature ranges to be encountered. If higher water-reducing admixture doses are expected to be used in hot weather, test at the higher dose and the relevant temperature before the mixtures must be used.
- Do not switch components of a mixture without pretesting. If unexpected outages unavoidably require such substitution, be especially careful with high admixture dose, high SCM content mixtures.
- Examine the interaction of mixture materials in advance using laboratory techniques including isothermal calorimetry, a modified version of the early-stiffening test per ASTM C 359, mini-slump, and others.

Whether used independently or together, admixtures and SCMs can raise the sulfate level required for proper early hydration. These effects are dose dependent, so higher SCM replacement levels and higher admixture doses are more likely to result in problems. In some cases where the admixture dose is very high, a slight, entirely normal variation in cement composition can lead to extreme variation in setting behavior. These effects are also temperature dependent, with higher temperatures usually causing greater problems. Thus, the seemingly unusual result of higher temperatures causing excessively long setting times and slow strength development can occur

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because the heat accelerates the aluminates, which limits the silicate reactions. In such systems, increasing the admixture dosage to control slump loss in hot weather can be exactly the wrong thing to do.

Generally, testing at higher doses of both SCM and admixtures is advisable so that the potential failure mode is understood. These interactions can be systematically investigated by a series of straightforward laboratory techniques that can help mixture designers understand the sensitivities of their chosen combination of materials.

A typical example is with determining the setting time, which can be measured by up to six different techniques, any of which are acceptable; selecting from among these different techniques, therefore, should be based on other project requirements and conditions. A relatively simple suite of the following field tests, conducted regularly, will help to ensure that the concrete mixture is performing satisfactorily or provide a warning of undesirable variability or potential incompatibility (Taylor *et al.*, 2006):

- foam index;
- foam drainage;
- density (unit weight);
- consistency (slump or slump flow) loss;
- semi-adiabatic temperature monitoring;
- setting time;
- chemistry of reactive materials.

To detect significant changes in composition or proportions, it is a good idea to track mill certificates and supplier's data sheets for changes in chemistry of all the reactive systems, which could indicate potential problems. Pay special attention to variations in reported sulfur trioxide (SO_3), C_3A, C_3S, fineness, setting time, and equivalent alkali content (Na_2Oeq).

Problems with unexpectedly long setting times, slump loss, and poor early strength development are frequently related to the cementitious system not having enough sulfates to control the early aluminate reactions. When these reactions are out of control, excessive slump loss, depression of early silicate reaction, or both, can occur. This effect was described by William Lerch over 50 years ago. The fundamental relationship, which bears repeating, is that when the aluminates react too quickly, the silicates are in danger of reacting too slowly.

Early stiffening and erratic setting

Early stiffening during hot weather is one of the most difficult challenges to both producers and purchasers of concrete, since it often leads to harmful retempering practices. Stiffening and setting are not always inclusive properties. Concrete stiffens in the course of setting; however, stiffening is not necessarily an indication of setting. As the previous section discussed,

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early stiffening and erratic setting can be the result of constituent material incompatibility, particularly, factors resulting in aluminate/sulfate imbalance. Aside from incompatible materials, other influential factors include:

- high temperature;
- low water-binder ratio;
- dry aggregates;
- excessively fine cementitious materials;
- inadequate mixing time; and
- inadequate mixing efficiency.

Several practical methods exist for evaluating the performance of material combinations at various temperatures. They include hydration profiling of paste samples, prepared and cured under adiabatic conditions in a conduction calorimeter, and ASTM C 359, a test for determination of early stiffening in hydraulic-cement mortars. However, oftentimes, the most effective admixture type and dosage is determined by trial and error during trial evaluations.

Poor strength development

When the measured strength of concrete is less than anticipated, the first step is to determine if the problem is real or perceived, that is, determining whether the problem is principally related to the material itself or discrepancies in the manner in which the material was evaluated. Chapter 9 discussed how discrepancies in measured test values could be traced to two fundamentally different sources—variability inherent to the material itself and variability inherent to the testing methods used. There are countless reasons that could cause low measured strength in high-strength concrete, but one of the most common material-related strength problems is the result of excessive W/B ratio due to water added after batching.

- Material-related problems:
  - — variations in constituent material quality;
  - — switching to lower quality constituents without prequalification testing;
  - — high levels of entrained air;
  - — air void clustering;
  - — over-yield;
  - — variations during batching:
    - measuring materials: weight/volume;
    - charging sequence;
  - — inadequate mix design for the application;
  - — prolonged delivery time;
  - — excessive jobsite waiting time; and
- jobsite added water.

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- Non-representative testing, improper:
  - — sampling;
  - — specimen molding;
  - — initial curing conditions;
  - — initial curing periods;
  - — transporting;
  - — final curing; and
  - — testing.

Aesthetic defects

Plastic shrinkage cracking

As Chapter 4 described, high-strength concrete is more vulnerable to plastic shrinkage cracking than conventional concrete. Plastic shrinkage stresses develop as a result of two mutually exclusive factors: (1) moisture loss, a function of environmental conditions, and (2) moisture replenishment (i.e. bleeding), a property of fresh concrete. When the rate of moisture loss exceeds the rate of moisture replenishment, plastic shrinkage stresses develop. Plastic concrete will crack when the magnitude of these shrinkage stresses exceed the relatively small magnitude of tensile strength. Factors influencing concretes propensity to plastic shrinkage cracking include:

- relative humidity;
- wind velocity;
- air temperature;
- concrete temperature;
- sub-base absorption; and
- setting time.

Concrete was traditionally considered highly vulnerable to plastic cracking when the evaporation rate exceeds 0.12 kg/m^2/hr (0.2 gal/ft^2/hr). A rule of thumb of this nature has no relevancy to high-strength concrete. Concretes containing high amounts of cementitious material, low W/B ratios, or finely divided cements or supplementary cementitious materials can show no signs of bleeding whatsoever.

The direction and orientation of plastic shrinkage cracks principally depend on the factors causing the loss of moisture from the concrete. For example, wind-induced plastic shrinkage cracks are generally oriented parallel to each other and perpendicular to the direction of the wind; whereas cracks caused by high temperature or low relative humidity are generally oriented in random directions. Cracks caused by surface moisture loss usually develop earlier than cracks caused by absorptive bases materials.

Bleeding should not be considered a necessary property of high-strength concrete unless it can be shown that other needed properties would not be

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breached, especially when high durability is required. Therefore, preventive measures should nearly always be planned. Methods to prevent plastic shrinkage cracking include:

- applying a fog spray to the surface;
- applying an evaporation retardant to the surface;
- decreasing the setting time; and
- setting up wind breaks.

Thermal cracking

It is common for thermal induced cracking only to be associated with large-scale, mass elements. Concrete elements do not have to be massive in order for thermal cracking to be a concern. Most cracks of this nature occur within the first few days following placement. Regardless of the design strength of the concrete, thermal-induced cracking should be a concern any time the developed tensile strength of the concrete is insufficient to resist stresses caused by temperature gradients at any given moment in time. Thermal cracking can occur in thin members, such as slabs-on-grade made with both conventional and high-strength concrete, whenever effectively large temperature gradients develop. Slabs-on-grade, especially those with poorly located or few contraction joints, are particularly vulnerable during transitional seasons, such as spring and fall, when differences between daily high and low temperatures are at their highest.

Maintaining favorable temperatures within the element through proper curing is critically important given the large amount of heat that could potentially develop even in moderately sized elements constructed with high-strength concrete. Although more internal heat is retained when elements are wrapped with insulation, doing so can effectively reduce the magnitude of the temperature gradients, the principle cause of thermal induced cracking. Insulation can be a very effective means of curing provided the peak temperature and chemical properties of the paste are conducive to avert the threat of delayed ettringite formation. When considering curing concrete in this manner, the period that elements must remain insulated should also be determined through thermal modeling. Premature removal of the insulation can cause the concrete to crack, and doing so will completely negate the time and expenses put forth to prevent such cracks from occurring.

Crazing

Crazing is the development of a network of unsightly fine random cracks on the surface of concrete caused by shrinkage of the immediate surface. Crazing is also referred to as *craze cracking, shallow map cracking,* and *pattern cracking*. Crazing cracks are rarely more than 3 mm (0.1 in.) deep and are quite a bit more noticeable on smooth, steel-troweled surfaces

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compared to rougher surfaces. Often they are not readily visible until the surface has been wetted and beginning to dry. The irregular hexagonal areas enclosed by the cracks are typically no more than about 40 mm (1.5 in.) across. Generally, cracks of this nature usually develop within the first few days after placement and are more prone to develop because of poor quality curing. The author has observed crazing to be more prevalent during times of the year when extreme air temperatures occur, particularly in cold weather. Crazing does not affect the structural integrity of concrete and they would rarely be expected to influence long-term durability in all but the most severe cases. Factors influencing crazing cited by the National Ready-Mixed Concrete Association (NRMCA) include:

- Poor finishing practices:
  - use of a "jitterbug" or similar tool that depresses the coarse aggregate, thus causing a concentration of paste and sand fines at the surface;
  - finishing while there is bleed water on the surface or the use of a steel trowel at a time when the smooth surface of the trowel brings up too much water and cement fines; and
  - sprinkling cement on the surface to dry up the bleed water concentrates fines on the surface, and is a frequent cause of crazing.
- Poor curing practices:
  - subjecting the surface to intermittent cycles of wetting and drying;
  - delaying the onset of curing; and
  - subjecting the surface to air drying and carbonation.

Steel troweled high-strength concrete surfaces are more prone to crazing than conventional concretes that are produced at higher W/B ratios containing less cementitious material; however, when proper finishing and curing practices are followed, crazing should not be anticipated.

Honeycombs and bugholes

Honeycombs are usually caused by improper placement or consolidation practices. The concrete mix should be designed to provide a workable mix for the type of consolidation that will be used on the job. When honeycombing occurs, do not just add water to the mix to correct the trouble. That will decrease the strength and durability of the concrete. The mix should be redesigned to provide improved workability or the procedure for consolidating the concrete should be improved. When concrete is consolidated by hand the puddling sticks should be pushed through the entire layer of freshly placed concrete. Concrete along the forms should be thoroughly spaded. The use of vibrators will consolidate a stiffer mix than can be consolidated by hand. The entire depth of a new layer of concrete should be vibrated. The systematic spacing of the points of vibration should be such that no part of the concrete is missed.

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Removing the defective concrete and replacing it with new concrete is the only effective method for correcting a honeycomb surface. If the area to be repaired is large in relation to its depth, it may be filled with pneumatically placed mortar or concrete. For this method of repair the surface should be sloped outward. When hand placed mortar or concrete is used, the edges should be sharp and straight and all portions of the area should be at least one inch deep. The surface of the old concrete should be thoroughly scrubbed to remove dust or dirt, and should be damp but not overly wet when the new concrete is placed to secure a good bond. The new concrete should have a color matching that of the older adjoining concrete. The new concrete should be adequately moist cured.

The occurrence of air pockets on the formed surfaces can be prevented by the proper use of form oil, the use of a well-designed mix and proper placing procedures. The use of excessive amounts of form oil will cause the air bubbles to stick to the surface more tenaciously. The use of an over-sanded mix makes it more difficult for the air bubbles to escape upward through the mortar. Placing the concrete in successive layers with a maximum depth of about 3 ft with adequate consolidation of each layer and with proper spading along the forms should remove the air pockets. The air pockets are less likely to occur when the concrete is consolidated by vibration than when it is consolidated by hand.

Some engineers are of the opinion that the use of air-entrained concrete increases the number and size of air bubbles on the formed surfaces of concrete. There is much evidence to the contrary, except when the concrete is placed under a sloping form.

Bugholes are small regular or irregular cavities, usually not exceeding 15 mm (0.6 in) in diameter, resulting from entrapment of air bubbles in the surface of formed concrete during placement and compaction.

Scaling and mortar flaking

Scaling is a scabrous condition where the surface mortar has peeled away, usually exposing the coarse aggregate. It is usually the result of a physical action caused by water freezing within the concrete and creating hydraulic pressure, which exceeds the tensile strength of the concrete. Scaling can be caused by lack of an adequate amount of entrained air in the surface paste for durability during freezing and thawing cycles. However, even well air-entrained concrete can scale if other factors are involved. After curing, several weeks of air drying greatly increases concrete resistance to freezing and thawing in the presence of deicers. Use of deicing salts will promote more moisture to accumulate prior to freezing due to the lower coefficient of freezing created

Any finishing operation that increases the W/B ratio of the surface such as finishing bleed water, inclement weather, or addition of water to the surface as a finishing aid will increase the permeability and significantly reduce

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durability. Saturated concrete is much more susceptible to deterioration than drier concrete; therefore, properly slope the concrete to inhibit water ponding onto the surface. Cure promptly with wet burlap or other materials to maintain moisture for a minimum of 7 days or until 70 percent strength has been achieved. Curing compounds may be used in spring and summer placements.

Surface scaling can be of two types. One is a relatively thin, sheet scaling caused by improper finishing and interim curing operations. The other is the scaling of non-air-entrained concrete caused by freeze-thaw damage and the application of salts for snow and ice removal.

The materials, finishing procedures, and curing methods that cause dusting of concrete surfaces also cause thin surface scaling. The procedures used to prevent dusting will prevent this type of surface scaling.

Scaled concrete surfaces can be repaired by applying a thin resurfacing of concrete properly bonded to the underlying old concrete. All defective concrete must be removed from the surface, by scarifying or scrubbing with hydrochloric acid, before applying the new concrete. A thin layer of neat cement paste should be brushed into the damp surface of old concrete just before the new concrete is placed to secure a good bond. The new concrete is placed, finished, and cured by normal procedures. Any relief joints present in the old concrete should be carried through to the new resurfacing.

Mortar flaking is a form of scaling that occurs over coarse aggregate particles and is often mistaken for a popout. Mortar flaking is a loss of surface paste and usually does not result in freshly fractured aggregate particles and there are fewer, if any, conical voids such as those found in popouts. Aggregate particles with flat surfaces are more susceptible than round particles to this type of defect. Mortar flaking occasionally precedes more widespread scaling but its presence does not necessarily lead to more extensive scaling. Moisture loss is accentuated over the coarse aggregate particles near the surface because the shape of the particles precludes the overlying surface paste from being replenished upon drying out by bleed water. In other words, the relatively long, flat shape of the particles trap the water underneath and the paste does not hydrate as well as the surrounding paste.

Blistering

Blistering is the irregular rising of a thin layer of placed mortar or concrete at the surface during or soon after completion of the finished operation. Blisters occur when bubbles of entrapped air or water rising through the plastic concrete are trapped under an already sealed airtight surface. Mixtures comprised of excessive fines, such as high-strength concretes, are prone to blistering unless proper precautions are implemented.

Blistering can be caused by either excessive amounts of entrapped air, insufficient vibration of concrete during placement or finishing the surface too soon—before the air has had a chance to escape. Blistering can be

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reduced by delaying steel troweling until after bleed water has evaporated, avoiding excessively high air contents, and avoiding overworking the surface.

Discoloration

Many factors affect discoloration, including calcium chloride admixtures, cement alkalis, hard-troweled surfaces, inadequate or inappropriate curing, variation of the water–cement ratio at the surface and changes in the concrete mix. Discoloration from these factors appears very soon after concrete placement. Discoloration at later ages may be the result of atmospheric or organic staining. Calcium chloride will have a retarding effect on the ferrite phase of the hydration process. The ferrite phase gets lighter with hydration. Retardation of the phase causes the concrete to be darker in color.

"Over-working" or "burning" the surface of the concrete—attempting to hard-trowel finish after it has become too stiff, can decrease the W/B ratio causing the surface to become very dark.

Petrography

Petrography is the examination of concrete and related building materials using methods and techniques derived from geology, metallurgy, and ceramics. Petrography is applicable to aggregates, mortar, grout, plaster, stucco, terrazzo, and similar portland cement mixtures. Evaluating concrete with petrographic methods yields valuable information about its composition, physical condition, and potential performance. A petrographic examination, performed in accordance with the nationally accepted standard, ASTM C 856, often yields the most cost-effective initial analysis when material properties are in question (PCA, 2003).

Petrography can help identify or rule out possible causes of a variety of concrete-related problems, and may suggest directions for further testing. Results of a petrographic examination are presented in a report that includes the detailed observations, photographic documentation of the important features, and a summary of the findings.

Petrographic examination can be a stand-alone tool to solve a specific problem, part of a comprehensive engineering evaluation, or support in litigation proceedings. Concrete petrography requires the careful preparation and examination of samples by highly trained specialists (Figure 10.8).

Samples are prepared by sectioning with diamond saws, cutting and polishing surfaces with lapping equipment, and preparing "thin sections" by mounting a selected portion of the concrete on a glass slide and grinding it thin enough for light to pass through. The samples are examined using stereo and petrographic microscopes and, if necessary, a scanning electron microscope. Petrographic examination describes the composition and properties of concrete and can determine:

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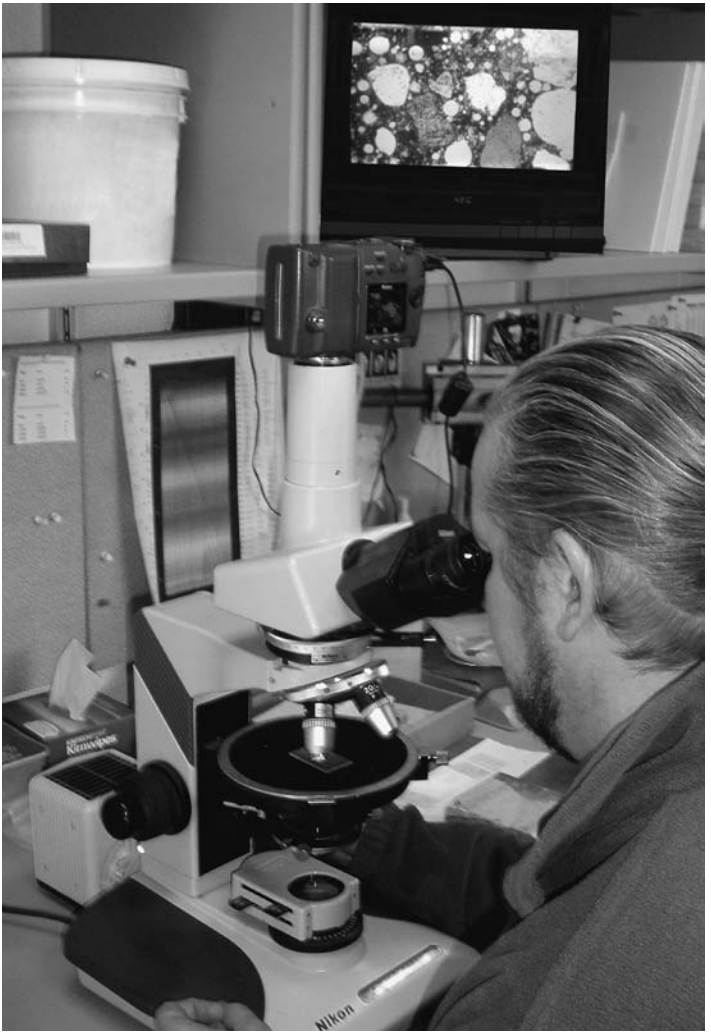

Figure 10.8 Examination of thin sections using a polarized-light microscope at magnifications up to 400×. Courtesy of CTLGroup.

- condition of material;
- causes of inferior quality, distress or deterioration;
- compliance with project specifications; and
- potential for future performance.

Petrographic examination may assess several features of the material in question, such as:

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- estimated hardened air content;
- estimated water–cement ratio;
- degree of cement hydration;
- extent of corrosion of reinforcing steel;
- extent of paste carbonation;
- potential causes of stains or discoloration;
- evidence of freeze-thaw deterioration;
- evidence of improper finishing;
- evidence of early freezing;
- presence of harmful alkali-aggregate reaction, sulfate attack, or other chemical attack;
- aggregate type;
- presence or absence of supplementary cementing materials (e.g. silica fume, fly ash, ground granulated blast furnace slag); and
- presence or absence of other additions such as fibers and pigment.

Figure 10.9 shows air-void clustering along the periphery of a coarse aggregate particle using stereo microscopy. Figure 10.10 shows a thin-section photomicrograph showing concrete damaged by expansive alkali–silica reaction. Petrographic examination is often supplemented with X-ray fluorescence analysis, X-ray diffraction analysis, air-void analysis, physical testing, and scanning electron microscopy.

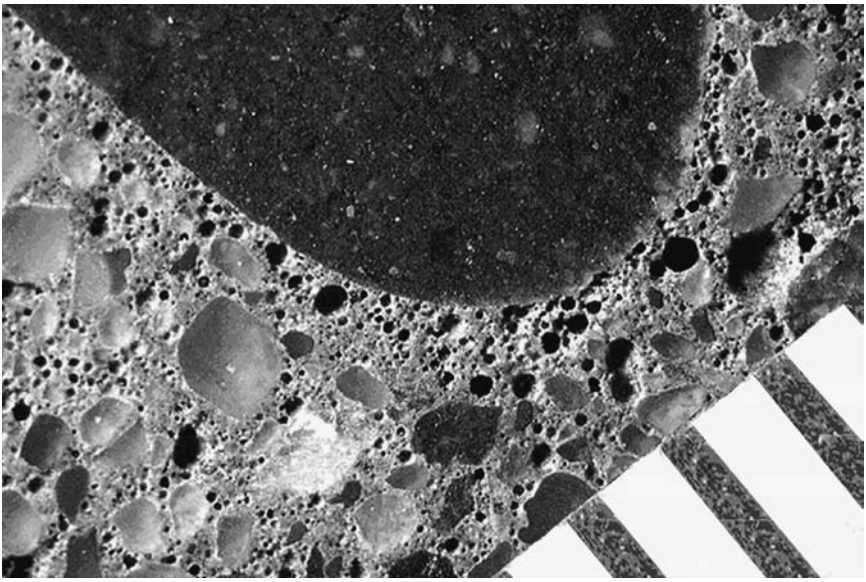

Figure 10.9 Air voids cluster along the periphery of a coarse aggregate particle. Scale in bottom right is in millimeters. Courtesy of Portland Cement Association.

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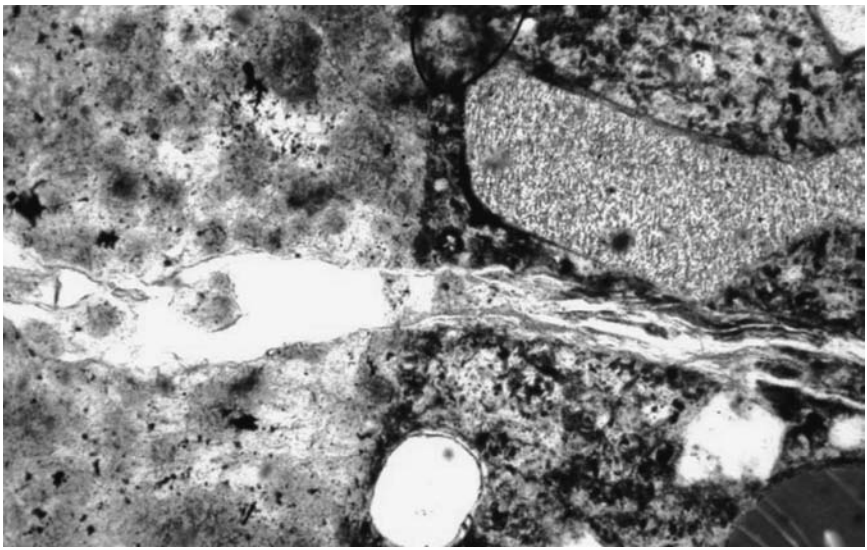

Figure 10.10 Thin-section photomicrograph showing concrete damaged by expansive alkali–silica reaction (ASR). The microcrack radiating from a reactive volcanic rock is partially filled with ASR gel. The field of view, left to right, is approximately 0.8 mm. Plane polarized light. Courtesy of Portland Cement Association.

The scanning electron microscope with energy dispersive X-ray spectroscopy (SEM/EDS) is an important tool for examination and analysis of micro structural and micro chemical characteristics of materials. SEM provides high resolution imaging at high magnifications with a greater depth of field, enhancing morphological and textural characteristics of the material; EDS provides elemental microanalysis of particles or areas of a sample or can "map" distribution of elements within a sample. SEM/EDS can supplement the petrographic examination in the following applications, among others:

- analyzing for surface contamination or stains;
- evaluating paints or coatings;
- evaluating corrosion products; and
- identifying and measuring microscopic features.

CASE STUDY: WHEN COLOR BECOMES A CONCERN

Chapter 7 addressed a situation when the ACI 318 Code requires that the transmission of column loads through the floor slab shall be provided by

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placing two different concrete mixtures in a building's flooring system—a placement method often referred to as "puddling" or "mushrooming." Due to various factors, such as differences in W/B ratio or inclusion of color altering constituents such as silica fume, the color of high-strength concrete can be markedly different from that of conventional-strength concretes. In many cases, color differences are not an issue since the concrete will not be exposed to view. This case study addresses one case where the concrete was going to be exposed and the steps that were taken to maintain a favorable aesthetic appearance.

The first five stories and the basement columns of 225 W. Wacker in Chicago[4] contain 96 MPa (14,000 psi) concrete produced with silica fume. The silica fume, combined with a very low W/B ratio of 0.28 created a mixture with a markedly darker appearance compared to the light color of the mixtures that were used in the slabs (Figure 10.11). For aesthetic reasons, the project architect requested that the same color be provided where the slab and column concretes were visible, as in the case in the parking garage portion of the structure. To satisfy this requirement, a 96 MPa (14,000 psi) mixture containing no silica fume was delivered for 18 columns. After investigating the feasibility of producing concrete of this strength level using adjusted quantities of high-strength Portland cement, fly ash, and chemical admixtures, it subsequently became necessary to extend the designated acceptance age from 56 days to 90 days—a proposal

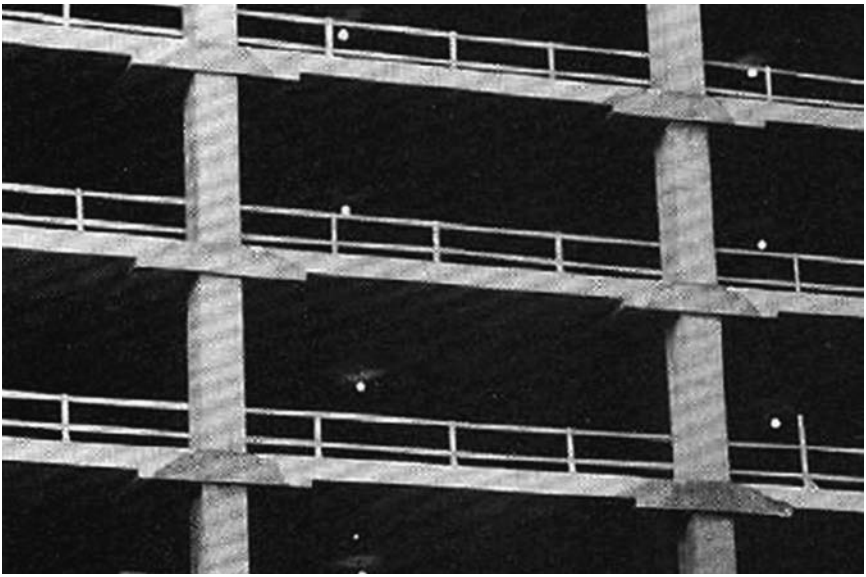

Figure 10.11 Color contrast of darker column concrete (containing silica fume), with slab concrete. Courtesy of American Concrete Institute.

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raised by the concrete producer and accepted by the project architect. With silica fume, the "normal" 96 MPa (14,000 psi) mixture attained an average compressive strength of approximately 114MPa (16,500 psi) at 56 days. Without the silica fume, the "special" 90-day mixture only averaged about 102 MPa (14,500 psi).

CASE STUDY: AN AUTOGENOUS SHRINKAGE CRACKING INVESTIGATION

The author considers the following case study as a textbook example of autogenous shrinkage cracking. This cracking problem occurred during the summer construction of a multi-story parking structure. At the heart of the problem was a moderately high-strength air-entrained concrete with a specified compressive strength (f_c') of 41 MPa (6000 psi) at 28 days. The mixture was proportioned at a W/B ratio of 0.36, contained a finely ground high-strength cement combined with silica fume. Other than an air entrainment, the only other admixture used was a high-range water-reducer. No retarding or hydration-stabilizing admixture was used.

Structurally, concrete with an f_c' of 34 MPa (5000 psi) would have been sufficient. Although permeability was not a specified concrete property, a higher-strength concrete was specified over concerns related to chloride permeability, not strength. Steel reinforcement was located in the beams, but not in the flat slabs.

The location and orientation of the cracks were consistent with drying shrinkage, the type of cracks that would be expected to develop several weeks or even months after placement; however, it was reported that the cracks involved in this investigation were consistently developing within 24 hours after placement. The cracks did not appear to have developed while the concrete was in a plastic condition, but rather after hardening. The cracks were generally oriented perpendicular to the post-tensioned beams, and they were only developing in the larger central slab bays having the largest ratio of surface area to volume. In most cases the cracks were developing across the full width of the flat slabs. The timeliness of the cracking suggested that they resulted from self-desiccation rather than moisture loss. The length and depth of the cracks suggest that they were the result of stresses that developed after the concrete hardened. Good interim curing practices were being employed during the placements and there were no signs of plastic shrinkage cracking. Upon completion, a white pigmented curing compound was being generously applied.

In an attempt to resolve the cracking problem as quickly as possible, several modifications were immediately undertaken, including:

- increasing the water content of the mixture by 8 kg/m^3 (14 lb/yd^3), increasing the W/B ratio from 0.36 to 0.38;

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- increasing the coarse-to-fine aggregate percentage of the mix design from 58:42 to 60:40 (by mass); and
- introduction of a retarding water-reducing chemical admixture into the mix.

The incidence of early-age cracking had reduced significantly beginning with the next scheduled slab placement. It is difficult to identify the solution to a problem when multiple steps are simultaneously taken. In this case, three things were changed at once. However, in light of the timeliness of the cracking and mixture characteristics, it is believed that the most influential of the three modifications was the increased amount of mix water provided. In addition to benefitting strength and other mechanical properties, increasing the coarse aggregate content would assist in restraining paste shrinkage. Adding the retarding admixture, although highly beneficial for achieving favorable long-term strength, at best would have imparted marginal benefits with respect to precluding self-desiccation.

The occurrence of autogenous shrinkage cracking does not represent a breach to structural integrity. It is anticipated that many of the cracks will seal because of post-tensioning width reduction, and continuation of hydration and the deposition of hydration products. Cracks that do leak with time can be addressed by conventional garage-deck crack repair methodology.

The propensity for autogenous shrinkage is significantly influenced by the material properties, including the water content, water-to-cementitious materials ratio and the chemical and physical properties of the cementitious materials, particularly the fineness of the cementitious material and the most early reactive phases: C_3A, C_3S, alkalis and sulfates. As the water content of the concrete decreases, the amount of internal moisture available for hydration decreases, and stresses induced due to self-desiccation increase; therefore, the water content of the concrete with extremely fine cementitious materials and/or low water-to-cementitious materials ratios are generally more vulnerable to autogenous shrinkage cracking.

Notes

1 The author is most grateful for the guidance provided by Larry Roberts, Peter Taylor, and Fulvio Tang in preparing this section.
2 *Standard Test Method for Expansion of Hydraulic Cement Mortar Bars Stored in Water*.
3 *Standard Test Method for Early Stiffening of Hydraulic Cement (Mortar Method)*.
4 This project was also discussed in Chapter 1.

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11 Summary

High-strength concrete is one variety of concrete categorized under the much broader term "high-performance concrete," or "HPC." As this book has frequently emphasized, concrete should only be considered as "high performance" provided it has satisfied all necessary performance requirements, not just a few. Though its name might imply bias towards strength, high-strength concrete, like most all structural concretes must satisfy all necessary mechanical, durability and constructability properties in a reasonably economical manner to compete with alternative construction materials. The achievement of satisfactory strength alone does not guarantee favorable long-term durability. In general, stronger concrete may possess better durability potential due to the lower permeability that comes with higher strength; however, depending on the durability property under consideration, steps taken to increase strength may in fact be harmful to long-term durability.

The fundamental principles of proportioning high-strength concrete are a result of the reversal in relative mechanical properties of paste and aggregate. Material properties, principally those mechanical in nature are fundamentally derived from the relative similarities (or differences) in the properties of the aggregate and paste. For this reason, the laws governing the selection of materials and proportions of concrete are not static. The most influential factor affecting the strength and largely influencing the durability of concrete is the W/B (W/C) ratio.

The achievement of high strength alone must never serve as a surrogate to satisfying other important mechanical or durability-related properties. It would seem logical that strong concrete would be more durable, and in many respects, the lower permeability that comes along with higher strength does improve concrete's resistance to certain durability-related distress, but unlike strength, the prerequisites for durability are not easily defined. In fact, depending on the manner in which higher strength is achieved, the durability of high-strength concrete could actually diminish.

In the last 40 years, the compressive strength of commercially produced concrete has nearly tripled, from 35 MPa (5000 psi) to 95 MPa (14,000 psi). This unprecedented escalation in strength was largely made possible because of the following factors:

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- advancements in chemical admixtures technology;
- availability of mineral admixtures (supplementary cementing materials); and
- increased knowledge of the principles governing higher-strength concretes.

Though naturally viewed as a single material, hydraulic cement concrete is, in reality, better understood when viewed as a composite material comprised of two fundamentally different materials—filler (i.e. aggregate) and binder (i.e. paste).

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The principles applicable to proportioning structural concrete are primarily driven by the relative mechanical properties of paste and aggregate. For this reason, proportioning guidelines that might be viewed as "best practice" for one strength level might be quite inappropriate for concrete of a different strength class. The selection of suitable cementitious materials for concrete structures depend on the exposure conditions, the type of structure, the characteristics of the aggregates, material availability, and method of construction. As the target strength of concrete increases, it increasingly becomes less forgiving to variability, both material and testing-related. Compared to conventional concrete, variations in material characteristics, production, handling, and testing, will have a more pronounced effect with high-strength concrete. Therefore, as target strengths increase, the significance of control practices intensifies.

Hydraulic cement concrete is a composite material comprised of two inherently different materials—paste and aggregate.

Portland cement is indisputably the most widely used binder in the manufacture of hydraulic-cement concrete. Selecting Portland cements having the chemical and physical properties suitable for use in high-strength concrete is one of the most important, but frequently overlooked considerations in the process of selecting appropriate materials for high-strength concrete. Cements should be selected based on careful consideration of all performance requirements, not just strength. To avoid interaction-related problems, the compatibility of the cement with chemical admixtures and other cementing materials should be confirmed. Cements can vary widely in the manner in which they perform in concrete. Cements that perform exceptionally well in conventional-strength concrete may not necessarily perform as well in high-strength concrete. Conversely, the strength efficiency of some cements increase as the cement content and W/B ratio decreases.

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Fly ash and slag cement are usually the supplementary cementitious materials chosen first for high-strength concrete. When combined with a high-strength Portland cement, these materials have been used for economically producing binary concretes with specified compressive strengths of at least 70 MPa (10,000 psi). For higher strength, ternary mixtures containing very fine, paste densifying pozzolans such as silica fume, metakaolin, or ultra-fine fly ash can be quite advantageous.

When identifying fresh and hardened properties, whether or not the paste constituents are classified as hydraulic or pozzolanic is of little relevance. More emphasis should be placed on what comes out of a system (i.e. performance) rather than what goes in (i.e. prescription). What matters is the rate binder is produced and the binding capacity of the system (performance characteristics) rather than what goes in (prescriptive requirements). Portland cement has traditionally been and remains at the heart of hydraulic cement concrete, and high-strength concrete is no exception. When making high-strength concrete, significantly better performance is achievable with SCMs. SCMs are critically important materials for high-strength concrete, and they should routinely be viewed as necessary mixture constituents. In conventional-strength concretes, fly ashes typically comprise 15 to 30 percent by mass of cementitious material. In high-strength concrete, higher percentages are common, particularly when using high calcium fly ash. With respect to strength, for a given set of cementitious materials, the optimum quantity of fly ash in concrete depends largely on the target strength level desired, the age at which the strength is needed, and the chemical and physical properties of the fly ash and other cementitious materials used. Slag cement is exceptionally desirable for use in high-strength concrete. At a given W/B ratio, higher long-term compressive strength can be expected with concretes incorporating slag cement compared to Portland cement-only concretes.

No single material has been responsible for opening the gateway to the achievement of ultra-high strength more than silica fume. When used correctly, silica fume is an extremely effective material for producing very high strengths and significant decreases in permeability. Because of its chemical and physical composition, silica fume is highly effective for achieving high strength at both early and later ages. The silica fume content of concrete generally ranges from 5 to 10 percent of the total cementitious materials content, though in very high-strength concretes having target strengths exceeding 100 MPa (14,000 psi), higher amounts have been used. Metakaolin is a highly reactive aluminosilicate with the capability of producing mechanical and durability-related properties similar to silica fume.

Among the most important parameters affecting the performance of concrete are the packing density and corresponding particle size distribution (gradation) of the combined aggregates used. Efficient aggregate packing improves important engineering properties, including strength, modulus of

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elasticity, creep, and shrinkage, while generating savings due to reductions in paste volume. When inappropriate aggregates are first selected, it is ironic that once everything is said and done and the high-strength mixture has been developed, the mixture cost is higher than it would have been had suitable aggregates been selected in the first place. When selecting aggregates for high-strength concrete, the ability to satisfy a strength requirement should never constitute the sole basis of selection. Aggregates that are considered suitable for conventional-strength concrete are not necessarily well suited for high-strength concrete. Aggregates should be selected considering all necessary properties and not just strength. The important parameters of coarse aggregate are its shape, texture, grading, cleanliness, and maximum size. Since the aggregate in conventional strength structural concretes is usually stronger than the paste, aggregate strength is not a critical factor; however, aggregate strength becomes increasingly important as strength increases, particularly in the case of high-strength lightweight aggregate concrete.

As target strength increases, the properties of aggregates as they relate to water-demand become less relevant and the properties that relate to interfacial bond become more important. Even though the water demand of smaller size coarse aggregates is higher, having greater surface area (and correspondingly greater interfacial bonding potential), smaller aggregates become more desirable as the target strength increases. For high-strength concrete, aggregate particles should be generally cubical in shape and should not contain excessive amounts of flat and elongated pieces. The optimum gradation of fine aggregate for high-strength concrete is determined more by its effect on water demand than on particle packing. High-strength concretes typically contain high volumes of cementitious sized material. As a result, fine sands that would be considered acceptable for use in conventional concretes may not be well suited for high-strength concrete due to the sticky consistency they may impart.

If used in excessive quantities, water represents concrete's greatest single enemy. Equally true, for high-strength concrete to attain it desired fresh and hardened properties, a certain minimum quantity of water is necessary. If it is not used in sufficient quantity, having not enough water can also be an enemy of concrete.

A common practice when producing high-strength concrete is to use a high-range water reducer (superplasticizer) in combination with conventional retarder or hydration-stabilizing admixture. The high-range water-reducer gives the concrete adequate workability at low water–cement ratios, leading to concrete with greater strength. The water-reducing retarder slows the hydration of the cement and allows workers more time to place the concrete. Combining high-range water-reducing admixtures with water reducing or retarding chemical admixtures has become common practice in order to achieve optimum performance at lowest cost.

Entrained air can significantly reduce the strength of high-strength concrete, and in addition, increases the potential for strength variability as

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air contents in the concrete varies; therefore, extreme caution should be exercised with respect to its use.

There are two critical points to keep in mind when developing high-strength concrete, both related to the W/B ratio. As the target W/B ratio progressively decreases:

- the proportioning principals that were appropriate with conventional concrete progressively become less relevant; and
- some of the constituents that worked well with conventional concrete become less appropriate.

Strength is usually not the most important consideration when developing high-strength concrete. The achievement of a mechanical property, such as strength, is relatively simple and straightforward, provided the principles of material selection and mixture proportioning are well understood and followed. Matters related to durability and constructability usually supersede strength. The true challenge is attaining high mechanical properties while still satisfying constructability and durability requirements.

The process of proportioning concrete is not a means to an end, but rather a means to a beginning. It is a process that, when completed, ends up at a starting point. Once a trial evaluation process has been conducted, first in the laboratory, and subsequently in the field, there is a greater than not chance that some adjustments to materials or mixture proportions will be necessary. As is the case with all concrete, before a high-strength concrete mixture can be proportioned, it is essential that all relevant fresh and hardened properties have been identified. Careful consideration should be given to the mixture properties needed during both construction and while in service. As obvious as identifying relevant properties may seem, this point is emphasized because it does not happen nearly as often as it should. How concrete properties are classified is *insignificant* compared to the importance of identifying and dealing with the properties that are truly relevant. Failure to consider only a few necessary properties, or centering a disproportionate amount of attention on only a few properties could impair performance in both the fresh and hardened state. Concrete mixtures can be developed to meet an array of different properties. Identifying and disregarding properties of little importance is equally as important as recognizing those that are important. Attempting to satisfy irrelevant properties might make it difficult to satisfy the ones that truly are important.

When developing mixture proportions for high-strength concrete, three fundamental components must be considered in order to produce a mix design satisfying its intended property requirements:

- mechanical properties of the aggregates;
- mechanical properties of the paste; and
- bond strength at the paste-aggregate interfacial transition zone.

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A common mistake when first attempting to produce high-strength concrete is to apply proportioning principles that would be more appropriate for conventional-strength concrete. Despite the fact that the principles of proportioning high-strength concrete have been identified and validated, nonetheless, it is an all too common occurrence. The objective of this section is to identify principal factors to consider when proportioning high-strength concrete.

Important properties to consider when proportioning high-strength concrete include:

- water-binder ratio (W/B);
- paste density;
- particle distribution;
- aggregate characteristics;
- water contained in admixtures;
- air entrainment; and
- workability.

Increasing the cementitious materials content merely to achieve an arbitrarily imposed 28-day strength requirement can be counterproductive to both the long-term mechanical and durability properties, including creep and shrinkage.

Once a mixture has been proportioned, a laboratory trial-batch program is a highly effective method for determining concrete properties and establishing mixture proportions. Careful attention is required during the trial-batch program to assure that materials and proportions selected will perform satisfactorily under field conditions. Trial batches should be conducted at temperatures representative of the work. This is particularly important for mixtures containing combinations of cementing materials and chemical admixtures to identify the presence of incompatible materials. Trial conditions should reproduce the mixing, agitating, and delivery time conditions anticipated during the work. Consistency (slump or slump flow), setting time, and batch temperature should be monitored for the duration of the testing period. Laboratory trial batches do not perfectly replicate field conditions. Fresh and hardened properties achieved in the laboratory are sometimes different from those achieved in full-scale production. Therefore, after the work has been completed in the laboratory, production-sized batches are recommended.

As an alternative to evaluating concrete simply on a trial and error basis, several, more efficient practical methods exist for evaluating the compatibility of material combinations at various temperatures, including hydration profiling of paste samples in a conduction calorimeter, and early stiffening of lab prepared mortars using the method prescribed in ASTM C 359.[1] Oftentimes, the most effective admixture type and dosage is determined through trial and error, therefore, it is suggested that the proposed

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combinations of cementitious materials and chemical admixtures be evaluated prior to their actual use.

Being a two-component composite material consisting of paste and aggregate, it is understandable that the mechanical properties of concrete are highly dependent on the relative properties of these two materials. Overall, this and the manner in which bond at the interfacial transition zone is affected is probably the most important, but still underestimated characteristics influencing the service life of most concrete structures.

Mechanical concrete properties such as tensile strength, shear strength, modulus of rupture, bond strength, and stress–strain relationships are normally expressed in terms of compressive strength. Since the laws governing the different mechanical properties of concrete vary, extreme caution should be exercised when attempting to extrapolate relationships that work well for conventional-strength concrete to high-strength concrete. The stress–strain behavior of concrete is primarily influenced by the relative stiffness of the paste and aggregates, and the bond strength at the interfacial transition zone. All else equal, higher interfacial bond strength is achieved using rough as opposed to smooth textured aggregate. Although it is common to think about the elastic modulus of concrete as a single concrete property, in actuality, concrete has *two* elastic moduli—the elastic modulus of paste and the elastic modulus of aggregate.

The modulus of elasticity of concrete is largely governed by the properties of the coarse aggregate. Increasing the size of coarse aggregates or using stiffer coarse aggregates with a higher modulus of elasticity increases the modulus of elasticity of the concrete. Being a composite material composed of paste and aggregate, the modulus of elasticity of concrete in compression is closely related to the mechanical properties of the paste relative to that of the aggregate particles. It should be noted that while stiffer or denser aggregates improve the elastic modulus of the concrete, they are also capable of introducing stress concentrations at the transition zone and subsequent microcracking at the bond interfaces reducing the ultimate compressive strength capacity of the concrete.

Universally applicable, or "boilerplate" specifications are undesirable, cost inefficient, and in many cases, inhibit the ability to achieve the properties most critically needed. To successfully produce and deliver high performance concrete requires intimate knowledge of the following three factors:

- constituent materials;
- mixture proportions; and
- material interactions.

It would be difficult to repeatedly produce quality concrete using prescriptive specifications. Prescriptive specifications can never adequately address any of the above items satisfactorily enough to produce consistent quality high-strength concrete. The quality of constituent materials, which drives mixture

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proportions, varies from market to market and day by day. Small variations in constituent material quality can have a pronounced effect with the performance of high-strength concrete. Without due consideration given to constituent material compatibility, unanticipated problems are significantly more likely to occur. Preconstruction conferences are essential to clarify the roles of all parties. It is best to have the mix designs submitted and reviewed well in advance of the meeting. Every detail involving the installation of high-strength concrete should be covered well in advance of the first scheduled placement. Detailed minutes should be taken during the meeting and promptly distributed within one or two days following the meeting. Preconstruction conferences should include representatives of all parties involved in the specification and production of the concrete: the concrete supplier, contractor, inspection agency, engineer, and the owner. Specifications for high-strength concrete should be predominantly performance-based. They should state the required properties of the hardened and fresh concrete clearly and understandably and leave little or no room for interpretation. In addition, they should be free of unnecessary restrictions. This means that much of the responsibility for ensuring that these qualities are achieved lies with the supplier. This is appropriate, since the concrete supplier is producing concrete on a daily basis and therefore is likely to have much greater expertise relating to concrete production than any other party in the construction process.

Continuing to select 28 days as the standard designated acceptance age for high-strength concrete can be counterproductive in the pursuit of satisfying important long-term properties. It is common for the selection of materials and mixture proportions for high-strength concrete to be based on a designated age of 56 or even 90 days rather than the traditional 28 days.

The procedures and equipment for producing and transporting high-strength concrete are not much different to that of conventional concrete; however, some changes, refinements, and emphasis on critical points are usually necessary. Had specialized equipment been necessary to produce high-strength concrete, its ascension into the mainstream industry probably never would have occurred. Expecting concrete producers to develop sophisticated concretes, while imposing extraneous prescriptive requirements, can end up having counterproductive results on the success of the project. Prescriptive compositional requirements truly have no place with high-strength concrete. The control of high-strength concrete should be in the hands of the concrete producer, the party most familiar with the mixture ingredients and their interactions.

The successful production of high-strength concrete requires coordination of ordering, dispatching, production, and quality control personnel. Developing and implementing an internal Quality Assurance Manual is one of the best ways to begin. When producing and delivering high-strength

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concrete, having a formal Quality Assurance Manual should not be thought of as a luxury, but rather, a necessity.

High-strength concrete should be produced to the design water-binder ratio, not consistency. Consistency should only be adjusted using water-reducing or high-range water reducing admixtures. The Quality Control Department lies at the focal point in the production and delivery of concrete. Quality control staff members regularly interact with customers, sales representatives, dispatchers, plant personnel, testing laboratory personnel, and occasionally with engineers, architects, general contractors, and owner's representatives. Therefore, maintaining strong communications with the Quality Control Department is essential within the concrete producer's organization.

When producing high-strength concrete, batch plants having a stationary "central" drum integral to the plant are preferable over "transit-mix" facilities that introduce the materials into a truck-mounted drum that provides all of the mixing action. High-strength concrete can be produced in plants with manual, semi-automatic, or fully automatic batching systems, although, for achieving the best batch-to-batch consistency, fully automated batching systems are preferred. When producing high-strength concrete, it is essential to ensure thorough mixing takes place prior to departure to the jobsite.

Whether added at the batch plant or at the jobsite, many low strength investigations involving high-strength concrete have been traced back to the addition of higher than desired quantities of water. On most high-strength concrete projects, it should be presumed that jobsite admixture adjustments will be needed and should be planned for accordingly.

Many of the problems that have occurred with high-strength concrete have been traced to poor jobsite control, particularly retempering practices and prolonged waiting times. Coordination and communication between all involved parties is essential for successful construction with high-strength concrete.

In concrete construction, the importance of communication cannot be overstressed. Preconstruction conferences review and clarify contractual requirements, construction means and methods, and testing and inspection procedures.

High-strength concrete should be delivered so that it can be placed with minimal amounts of waiting time. By delaying the placement of high-strength concrete, there is a greater chance that the concrete will stiffen beyond the point that it can be properly placed, and may subsequently lead to jobsite retempering. Regardless of when it is introduced, jobsite added water can be extremely detrimental to the integrity of the high-strength concrete and therefore, should never be permitted. The author strongly recommends that all necessary adjustments to workability be made using high-range water reducer.

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Placing must be done so that segregation of the various constituent ingredients is avoided and full consolidation is achieved with all entrapped air voids eliminated. The slump test should not be used as a basis for acceptance or rejection if a high-range water-reducer is being used, provided there are no indications that the concrete is segregated. There is a recognized and justified need to occasionally add site water to conventional-strength concrete in order to increase workability, and the provisions for doing so are laid out in ASTM C 94.[2] However, under no circumstances should additional water ever be used to increase the workability of high-strength concrete.

Curing is a process during which hydraulic-cement concrete develops hardened properties through the hydration of the cement in the presence of water and heat. Curing allows hydration to occur so that the intended mechanical and durability properties of the concrete may develop. What is considered "effective" curing depends on several factors, including the element under consideration, particularly the ratio of exposed surface area to total volume of the element; the thermal and moisture-related properties of the concrete, environmental conditions and serviceability requirements of the structure.

Consideration for curing should be given the moment that concrete is placed, not as a final step after the completion of placement and finishing. High-strength concretes typically have very dense paste matrices; therefore, some curing methods that have worked favorably with conventional concretes may be less effective for high-strength concrete. When environmental conditions and concrete properties are such that no significant drying or thermally induced stresses develop on the concrete structure, minimal curing practices may be satisfactory. Because of the high ratio of exposed surface area to total volume, slabs and pavements rarely are in this class of concrete.

Measurement of compressive strength during construction is by far the most common method of quality control or quality assurance, and it provides the most fundamental information needed to evaluate whether the concrete is capable of complying with the intended design requirements. The concrete industry relies heavily on the results of concrete compressive strength tests to determine the adequacy of as-delivered or in-place concrete, and important decisions have been based on measured strength (Richardson, 1991). As the target strength of concrete increases, it becomes increasingly more sensitive to variations related to both materials and testing, thus the magnitude of the standard deviation, the overall gauge of variability relating to both the material and the testing practices increases. Planning for inspection and testing of high-strength concrete involves giving attention to personnel requirements, equipment needs, test methods, and the preparation and handling of test specimens.

The consequences of deviating from some standardized test procedures may have a negligible influence on the outcome of the test. The consequences

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of others can be considerable. Initial curing test specimens at elevated temperatures and subjecting non-immersed specimens to prolonged initial curing periods in an air environment are two of the most potentially destructive testing deviations, and each will be addressed in this chapter.

The advancements that have been made in the field of concrete materials technology have vastly increased the feasible realm of concrete applications. Without modern chemical admixtures, cements, and supplementary cementing materials, most high performance concrete would simply not be possible. Nevertheless, with increased performance demands comes increased risk. Inclusion of greater amounts of more complex materials means that concrete mixtures are progressively becoming more sensitive to conditions that in the past would not have been problematic. Due to the increased complexity of modern concrete mixtures, practices such as substituting one cement for another with the presumption that the substituted cement should "work about the same" as it has in the past, may now lead to poor performance (Roberts and Taylor, 2007). The term "incompatibility" refers to undesirable interactions occurring between acceptable constituent materials resulting in unanticipated and objectionable performance.

The most common problems resulting from adverse material interactions include premature loss of workability (early stiffening), erratic setting behavior (rapid set or extended set), poor strength development, and poor air-void system characteristics. The interactions occurring between C_3A and sulfate during the early stages of cement hydration forms the basis of many incompatibility problems. The mechanisms causing such problems can be highly complex and are often interrelated. Often there is a very fine line between normal behavior and incompatible behavior, and there is usually no simple method of reliably determining the risk of incompatibility. It is precisely for this reason that trials should be conducted using candidate materials under actual job conditions.

Notes

1 *Standard Test Method for Early Stiffening of Hydraulic Cement (Mortar Method)*.
2 *Standard Specification for Ready-Mixed Concrete*.

References

Richardson, D.N. (1991) "Review of Variables that Influence Measured Concrete Compressive Strength," *Journal of Materials and Civil Engineering,* American Society of Civil Engineers, May, pp. 95–112.

Roberts, L.R. and Taylor, P.C. (2007) "Understanding Cement-SCM-Admixture Interaction Issues," *Concrete International*, Vol. 29, No. 1, American Concrete Institute, pp. 33–41.

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Glossary

Absolute volume The displacement volume of an ingredient of concrete or mortar; in the case of solids, the volume of the particles themselves, including their permeable or impermeable voids but excluding space between particles; in the case of fluids, the volume which they occupy in concrete.

Air void A space in cement paste, mortar, or concrete filled with air; an entrapped air void is characteristically 1 mm or more in size and irregular in shape; an entrained air void is typically between 10 μm and 1 mm in diameter and spherical (or nearly so).

Average daily air temperature The mean of the highest and the lowest temperature occurring during the period from midnight to midnight.

Binary cement A term for cement containing two main constituents.

Blast furnace slag A nonmetallic product consisting essentially of silicates, aluminosilicates of calcium, and other compounds developed in a molten condition simultaneously with iron in an iron blast furnace.

Bleeding The autogenous flow of mixing water within, or its emergence from, newly placed concrete or mortar; caused by the settlement of the solid materials within the mass; also called *sweating* and *water gain*.

Blended cement A term for cements having more than one main constituent; combinations of portland cement and granulated blast-furnace slag, portland cement and pozzolan, or portland blast-furnace slag cement and pozzolan, or granulated blast-furnace slag and hydrated lime.

Builder See **Contractor**.

Calcium sulfate In cement manufacture, a material composed essentially of calcium sulfate in one or more of its hydration states: anhydrite ($CaSO_4$), gypsum ($CaSO_4 \cdot 2H2O$), or calcium sulfate hemihydrate ($CaSO_4 \cdot ½H2O$).

Carbonation A reaction between carbon dioxide and a hydroxide or oxide to form a carbonate, especially in cement paste, mortar, or concrete; the reaction with calcium compounds to produce calcium carbonate.

Cementitious materials Materials having cementing value when used in concrete, either by itself or in combination with pozzolans (e.g., fly ash, slag cement, silica fume, metakaolin, volcanic ash, and calcined clay).

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Cold weather A period when, for more than three consecutive days, the following conditions exist: (1) the average daily air temperature is less than 5°C (40°F) and (2) the air temperature is not greater than 10°C (50°F) for more than one-half of any 24-hr period.

Compressive strength The measured maximum resistance of a material specimen to uniaxial compression loading.

Consistence A term now used in the UK in place of *workability*.

Contractor An individual, corporation, or joint venture with whom the Owner enters into agreement for construction of the work under the contract documents.

Creep Time-dependent increase in strain of hardened concrete under sustained load.

Creep, specific Strain due to creep divided by the applied stress.

Consistency The relative mobility or ability of fresh concrete, mortar, or grout to flow.

Curing The maintenance of satisfactory moisture and temperature during concretes' early stages allowing desired properties to develop.

Delayed ettringite formation (DEF) A form of internal sulfate attack caused by the suppression of normal ettringite formation during early hydration.

Dilatant material A material in which viscosity increases with the rate of shear (also termed shear thickening). The opposite of a dilatant material is a pseudoplastic material.

Engineer The registered engineer designated by the Owner as the accepting authority responsible for issuing the project specification or administering work under the contract documents.

Evaporation retardant A long-chain organic material, which when spread on a water film on the surface of concrete retards the evaporation of bleed water.

Fineness modulus An index of the fineness of an aggregate—the higher the fineness modulus (FM), the coarser the aggregate. Determined according to ASTM C 125.

Flowing concrete Concrete that is characterized by a slump greater than 190 mm (7.5 in) while remaining cohesive.

Heavyweight aggregate Aggregate having an oven-dry particle density of at least 3000 kg/m^3 (190 lb/ft^3).

Heavyweight concrete Concrete having an oven-dry density greater than 2600 kg/m^3 (160 lb/ft^3).

High-strength cement Portland or blended hydraulic cement suitable for producing high-strength concrete.

Hot weather A period when, for more than three consecutive days, the following conditions exist: (1) the average daily air temperature is greater than 25°C (77°F) and (2) the air temperature for more than one-half of any 24-hr period is not less than 30°C (85°F).

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Hydraulic cement Cement that sets and hardens by reacting chemically with water.

Inspector The Engineer's or Owner's authorized representative who is assigned to make detailed inspections of the quality of the work and its conformance to the provisions of the Contract.

Lightweight aggregate Aggregate of mineral origin having a loose oven-dry bulk density not exceeding 1200 kg/m^3 (75 lb/ft^3).

Lightweight concrete Concrete having an oven-dry density not less than 800 kg/m^3 (50 lb/ft^3) and not more than 2000 kg/m^3 (125 lb/ft^3), produced using lightweight aggregate for all or part of the total aggregate.

Mass concrete A volume of concrete with dimensions large enough to require that measures be taken to cope with the generation of heat and temperature gradients from hydration of the cementitious materials, and attendant volume change.

Metakaolin A highly reactive aluminosilicate pozzolan produced by low temperature calcination of kaolinite clay.

Modulus of elasticity, dynamic The modulus of elasticity computed from the size, weight, shape, and fundamental frequency of vibration of a concrete test specimen, or from pulse velocity.

Modulus of elasticity, static The slope of the elastic part of the stress–strain curve in tension or compression. Also referred to as Young's Modulus.

Modulus of rupture the maximum surface tensile stress in a bent beam at the instant of failure. Also referred to as *rupture modulus* and *rupture strength*. Modulus of rupture is a property strictly applicable to brittle materials.

Natural cement Extensively used in nineteenth and early twentieth century construction, hydraulic cement produced by mining natural deposits of limestone and clay with a specific chemical composition within a narrow range. When heated in a kiln, and ground to a fine powder, sets and hardens when mixed with water through chemical reactions.

Normal-weight aggregate Aggregate with an oven-dry particle density greater than 2000 kg/m^3 (125 lb/ft^3) and less than 3000 kg/m^3 (190 lb/ft^3).

Owner The public or private agency or entity taking possession of the work upon completion.

Poisson's ratio The ratio of transverse strain to the corresponding axial strain resulting from uniformly distributed axial stress below the proportional limit of the material.

Porosity The quality of having pores, one of the factors that contributes to the permeability of concrete.

Portland cement A powder formed by the calcination of limestone, clay, and shale that hardens and becomes cementitious when it reacts with water.

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Proportional limit The greatest stress that a material is capable of developing without any deviation from proportionality of stress to strain.

Pseudoplastic material A material in which viscosity decreases with increasing rate of shear (also termed shear thinning). The opposite of dilatant material is a pseudoplastic material.

Quality assurance The planned activities and systematic actions necessary to provide adequate confidence to the Owner and other parties that the products or services will perform their intended functions.

Quality control Actions related to the physical characteristics of the materials, processes, and services that provide a means to measure and control the characteristics to predetermined quantitative criteria.

Rheology The study of the deformation and flow of matter.

Rheopecty A reversible increase in viscosity at a particular shear rate. Upon shearing, rheopectic concrete mixtures appear to thicken and resist movement, for example, when pumped or rodded.

Self-consolidating concrete (SCC) Highly fluidized, non-segregating concrete that can spread into place, fill the formwork, and encapsulate the reinforcement under its own weight without any mechanical consolidation.

Self-desiccation The removal of free water by chemical reaction to leave insufficient water to cover the solid surfaces and cause a decrease in the relative humidity of the system.

Slump test A commonly used measure of the consistency of freshly mixed concrete in which a conical metal mold is first filled with fresh concrete, and then lifted off the concrete.

Slump The vertical distance the concrete settles during the slump test.

Slump flow The average horizontal spread diameter the concrete settles during the slump test.

Specific heat The amount of heat required to raise the temperature of 1 kg (2.20 lb) of matter by 1°C (1.8°F).

Strength test The average of two or more test specimens of the same age taken from a single batch of concrete.

Sulfate attack A deleterious reaction between concrete and sulfates from the soil, ground water or other sources.

Supplementary cementitious materials Cementitious materials other than Portland cements used in concrete (e.g. fly ash, slag cement, silica fume, metakaolin, volcanic ash, and calcined clay).

Tensile strength The measured maximum resistance of a material specimen to uniaxial loading in tension. The tensile strength of brittle composite materials like hydraulic cement concrete is difficult to determine with a high degree of statistical confidence, and thus is rarely determined by direct measurement.

Thermal conductivity, coefficient of The rate at which heat is conducted through a solid under steady state temperature conditions.

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Thermal expansion, coefficient of The thermal strain per change in unit temperature.

Thixotropy A reversible decrease in viscosity at a particular shear rate (the opposite of rheopexy). Shearing causes a gradual breakdown in gel structure over time. The thixotropy is a measure of applied work needed to break down the structure.

Water-binder ratio (W/B) The ratio of the mass of water to the mass of all cementitious materials in the concrete.

Workability The relative ease at which freshly mixed concrete can be placed, consolidated, and finished.

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Institutes and standard writing organizations

American Concrete Institute (ACI)
38800 Country Club Drive
Farmington Hills,
Michigan 48331 USA
Tel: +01 248–848–3700

ASTM International (ASTM)
100 Barr Harbor Drive, PO Box C700
West Conshohocken, Pennsylvania,
19428–2959 USA
Tel: +01 610–832–9500
Website: www.astm.org

Canadian Standards Association (CSA)
5060 Spectrum Way
Mississauga,
Ontario, CA
L4W 5N6
Tel: +01 416–747–4000
www.csa.ca

European Committee for Standardization (CEN)
36 rue de Stassart, B-1050 Brussels
Tel: + 32 2 550 08 11
Website: www.cen.eu

German Institute for Standardization (DIN)
DIN Deutsches Institut für Normung e.V.
Öffentlichkeitsarbeit
Burggrafenstrasse 6
10787 Berlin, Germany
Fax: +49 30 26 01–12 63
Website: www.din.de

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International Organization for Standardization (ISO)
1, ch. de la Voie-Creuse, Case postale 56
CH-1211 Geneva 20,
Switzerland
Telephone +41 22 749 01 11
Website: www.iso.org

Standards Australia Limited
286 Sussex Street, Sydney, NSW, 2000
GPO Box 476, Sydney,
NSW, 2001
Tel: +61 2 8206 6000
Website: www.standards.org.au

BSI British Standards
389 Chiswick High Road
London W4 4AL, UK
Tel: +44 (0) 20 8996 9001
Website: www.bsi-global.com/en

Standardization Administration of China (SAC)
Zhichun Road No. 4,
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Beijing, CN 100088
Tel: +86–10–62000675
Website: www.sac.gov.cn

National Institute of Standards and Technology (NIST)
100 Bureau Drive, Stop 1070
Gaithersburg, Maryland 20899–1070
Tel: +01 301–975–6478
Website: www.nist.gov

National Ready-Mixed Concrete Association (NRMCA)
900 Spring Street
Silver Spring, Maryland 20910
Tel: +01 301–587–1400
Website: www.nrmca.org

The Concrete Society
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Fax: +44(0)1344 466008
www.concrete.org.uk

www.technobetar.ir

Center for Advanced Cement Based Materials (ACBM)
Northwestern University
2145 Sheridan Road, Suite A130
Evanston, IL 60208
Tel: +01 (847) 491–3858
Fax: (847) 467–1078

Expanded Shale, Clay and Slate Institute
2225 E. Murray Holladay Road—Suite 102
Salt Lake City, UT 84117
Tel: (801) 272–7070
Fax: (801) 272–3377

Indian Concrete Institute
Ocean Crest New #79 (Old #35) 3rd Main Road
Gandhi Nagar, Adyar, Chennai-600 020, India
Phone: 044–24912602/24455148 Fax: 044–24455148

Portland Cement Association (PCA)
5420 Old Orchard Road
Skokie, IL 60077
Phone: (847) 966–6200 Fax: (847) 966–8389

Precast/Prestressed Concrete Institute (PCI)
209 W. Jackson Blvd, Suite 500
Chicago, IL 60606–6938
Phone: (312) 786–0300 Fax: (312) 786–0353

RILEM
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F-92220 Bagneux, France
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Email: admin@concretesociety.co.za

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Index

Please note that any page references to Figures or Tables will be in *italic* print

AAR (alkali-silica reactions), 41, 134
Abrams, Duff, 3, 70, 72
abrasion resistance, 123–124
absolute volume method, 65, 68–69, *95*
accelerated curing, 42
accelerating of admixtures, 57
acceptance age, designated, 84–85; post 28-day, 140–141, 233
ACI (American Concrete Institute), ix; on air entrainment, 83; Building Code, x, 67, 85–86, 164, 187, 221–222; committees, 7, 8, 68–69, 103, 109, 151, 155, 174–175; on consolidation, 159; on corrosion resistance, 121; on creep and shrinkage, 109; on curing, 164, 165; on definition of high-strength concrete, 7; on high-performance concrete, 10; on modulus of elasticity, 103; on plastic cracking, 110; State-of-the-Art Report on High-Strength Concrete, xi–xii, 7–8; and statistical variability, 67; Tables, 67, 81, 82
ACR (alkali-carbonate reactions), 119
additives, concept, 3
admixtures: accelerating, 57; air-entraining, 59–60; chemical *see* chemical admixtures; combined, synergistic effects of, 59; concept, 3; conventional water-reducing, 55–56; high-range water-reducing *see* HRWR (high-range water-reducing chemical admixture); hydration stabilizing, 56; mineral, 4; set retarding, 56; viscosity modifying, 57–58; water contained in, 82
aesthetic defects, 213–218; blistering, 217–218; bugholes, 216; crazing, 214–215; discoloration, 218; honeycombs, 215–216; plastic shrinkage cracking, 213–214; scaling, 216–217; thermal cracking, 214
aggregate blending, 52
aggregates, 48–53; characteristics, 76–81; coarse *see* coarse aggregates; composition of concrete, 69; drying shrinkage, 113; fine, 50; heavyweight, 49; in hydraulic cement concrete, 2; lightweight, 49; normal-weight, 49; packing density, 49; particle packing, 44, 53; size, 51; water, 51, 53–54
AIJ (Architectural Institute of Japan), Research Committee on High-strength Concrete, 103–104
air content, estimation in proportioning, 90–91, 93
air entrainment, 82–84, 229–230; air entraining admixtures, 59–60; and bleeding, 125; and freeze/thaw resistance, 118
air voids, 11, 69, 162
Aïtcin, Pierre Claude, xi, 51, 71, 106, 109
Akers, D.J., 187
Albinger, John, xi
alkali-aggregate reactions (AAR), 41, 134
alkali-carbonate reactions (ACR), 119
alkali-silica reactions (ASR), 119–120
American Concrete Institute *see* ACI (American Concrete Institute)
American Society for Testing and Materials *see* ASTM (American Society for Testing and Materials)
Amirjanov, A., 53
anthracite coal, 34
Architects, concrete specifications written by, 131

www.technobetar.ir

Architectural Institute of Japan (AIJ), Research Committee on High-strength Concrete, 103–104
ASTM (American Society for Testing and Materials), ix; on abrasion resistance, 123; on alkali-silica reactions, 119; on blended cements, classification, 29; on fly ash, 207; on loading rate, 185; on maturity method, 188; on metakaolin, 47; on penetration resistance, 191; on petrography, 218; on placement, 156; on Portland cement, 27–28; precautionary statement of, 29; on production of ready-mixed concrete, 143; on sample representation, 177; on shrinkage, 111; on specimen consolidation, 177; on specimen size and shape, 178; on standardized test methods, 28; on tempering, 153–154; on water, 54
autogenous shrinkage, 111, 112, 113; case study, 223–224
axial compression, 115
axial stress versus strain, 100

Balayssac, J.P., 117
Basheer, P.A.M., 116–117
Berntsson, L., 76
Bickley, J.A., 139
binary cement/concretes, 39, 42
binders, estimated content (proportioning application example), 91, 94
bituminous coal, 33–34
blast furnace slag, 4, 39
bleeding, 43–44, 125, 213–214
blended cements, 29
Blick, R.L., 159
blistering, 217–218
Bogue compounds, 29
bridges, high-strength, 13, 15–16, 17
brittle materials, 4
bugholes, 216
Building Code Requirements for Reinforced Concrete (ACI 318–56) *see under* ACI (American Concrete Institute)
buildings, 16; tall *see* tall buildings
Burg, R.G., 178, 188
Burj Dubai skyscraper, 17, *18*, 19
buttering process, 145

calcium hydroxide (CH), 32, 44
calcium silicate hydrates (CSH), 44, 74
calcium sulfate ($CaSO_4$), 26
calibrating consistency, 82
calorimeter tests, 26
capillary system, 165
capping materials, end preparations, 184
carbon black, 45
Carino, N.J., 165, 184
Carrasquillo, R.L., 68
case studies: autogenous shrinkage investigation, 223–224; color concern, 221–223; jobsite curing in limewater, 193, 196–197; self-consolidating, 160–162
cement: binary, 39, 42; blended, 29; high-strength, identifying, 27–30; hydraulic cement concrete *see* hydraulic cement concrete; Portland *see* Portland cement; samples, laboratory evaluation, 191, *192*; slag *see* slag cement
cementitious materials, 21–48; bulk specific gravity, *22*; drying shrinkage, 113; Portland cement *see* Portland cement; *see also* cement
CEN (European Committee for Standardization), ix
central mix plants, 147, 148, 150
CH (calcium hydroxide), 32, 44
chemical admixtures: constituent materials, 54–59; high-range-water-reducing *see* HRWR (high-range water-reducing chemical admixture); and incompatibility, 205; increased use, 13; mini slump test, 207; production, 149; and sulfate, 204; *see also* admixtures
chemical shrinkage, 111, 112
Chicago, xi, 12; historic building, rehabilitation of, 11; Outer Drive East Condominium Project (40-story), x, 13; ready-mixed high-strength concrete, development (1960–1990), x, 13; Water Tower Place, 115; West Wacker building project, x, 13, *14*, 222
Chidiac, S.E., 84
china clay, 47
chloride ion penetration, permeability, 117
coal, 33–34
coarse aggregates, 50–52; characteristics/sizes, 76–81; composition of concrete, 69; and concrete paste, 44; content, estimating in proportioning example, 94; estimating volume, 81–82; and modulus of elasticity, 101
cold weather, 149, 182; placement, 158
Collins, T.M., 115

www.technobetar.ir

colloidal silica, 58
column loads, transmission through floor slabs, 157
columns, use of high-strength concrete in, 122
compressive strength: aggregate characteristics, 80–81; design compressive, and specified compressive, 6; escalation in, 12, 226–227; and failure in compression, 4; final curing conditions, 182, 184; high-strength concrete versus convention strength concrete, 1, 107–108; initial curing conditions, 179–182, *183*; loading rate, 185; machines, 185; measuring, 235; mechanical properties, 99–100; mold material, 179; mortar-cube tests, 29; Portland cement, 28; and proportioning, 66; sample representation, 176–177; specimen consolidation, 177–178; specimen moisture content and distribution, 179; specimen size and shape, 178–179; testing of variables influencing, 175–187; and W/B ratio, 11, 70–71
computer simulation algorithm, 53
concrete: air-dried, 122; composition of, 69–70; conventional strength *see* conventional strength concrete; fresh, rheological properties, 84; high-performance *see* high-performance concrete (HPC); high-strength *see* high-strength concrete (HSC); high-strength precast, curing of, 171–173; historical background, 12–15; hydraulic cement *see* hydraulic cement concrete; identifying relevant properties, 65–67; low permeability, 112; normal weight, 177; perishable nature of, 153; plastic, 164; self-consolidating *see* self-consolidating concrete (SCC); thermal properties, 121
condensed silica fume *see* silica fume
consistency: adjusting, 144; batch-to-batch, 147, 234; calibrating, 82; characterizing, 124; shipment-to-shipment, ensuring, 37
consolidation, 159
constituent materials: aggregates, 48–53; cementitious, 21–48; chemical admixtures *see* chemical admixtures; profiling in laboratory, 191, *192*, 193, *194*, *195*; specifications, 136
constructability properties, 124–126
conventional strength concrete: aggregates, 49; versus high-strength concrete, viii, ix, 1, 8–9, 99, 107; inelastic strain, capable of, 4–5; and low permeability concrete, 112; modulus of elasticity, 103; terminology, 8–9
Cook, J.E., 80
corrosion, chloride-induced, 41
corrosion inhibiting, 58
corrosion resistance, 121
cracking: defined, 109; early-age, 224; and moisture requirements, 166–167; plastic shrinkage, 167–168, 213–214; thermal, 165, 171, 214
craze cracking, 214–215
crazing, 214–215
creep, 109, 114–115; *see also* shrinkage
CSH (calcium silicate hydrates), 44, 74
cube testing, 28–29
curing, 164–173; accelerated, 42; compressive strength, final curing conditions, 182, 184; compressive strength, initial curing conditions, 179–182, *183*; defined, 164; of high-strength precast concrete, 171–173; inadequate, 165; internal, 112, 170; jobsite curing in limewater, case study, 196–197; jobsite-curing box, 182; match-curing method, 188–189; materials, 165–166; moisture requirements, 166–170; and sealing, 170; silica fume, 44; specimens, standard versus field cured, 186–187; temperature requirements, 171; water, 166, 169
curing compounds, 169
cylinders, test, 178, 179, *184*, 187

de Almeida, I.R., 124
de Larrard, F., 65, 80, 109
dehydration, precluding, 168
Delayed Ettringite Formation (DEF), 120–121, 171
delivery, 150–151
Design of Concrete Mixtures (Abrams), 70
design strength, and target strength, 5
designated acceptance age *see* acceptance age, designated
Detwiler, R.J., 45
Deutzer Bridge, Germany, 16
dicalcium silicate (C_2S), 25, 201
discoloration, 170, 218
dispatching, 144–145
Domone, P.L.J., 65
drilled cores, 187–188
drums, plant operations, 147

www.technobetar.ir

drying shrinkage, 108, 113
Dubai, Burj Dubai skyscraper, 17, *18*, 19
durability: and permeability, 11; prerequisites for, 1; problems, 116; properties, 115–124; and strength, 11, 226; and water-binder ratio, 2

early-age shrinkage, 111–113
elastic modulus *see* modulus of elasticity
end preparations, 184–185
ettringite, 26, 203; Delayed Ettringite Formation, 120–121, 171
European Committee for Standardization (CEN), ix, 27
European Federation for Specialist Construction Chemicals and Concrete Systems, 57–58

Feldman, R.F., 113
ferrous and ferric oxide, corrosion inhibiting, 58
field-cure testing, 186–187
fine aggregates, 50
fineness modulus (FM), 50, 80
finishing, 159–160
fire resistance, 122–123
fly ash, 33–39, *40*; advantages and disadvantages, 37; blended cements, 29; chemical composition, 36; high calcium, 147, 206–207; lightweight, 122; low and high calcium, 34, 36; micrographs, *40*; prescriptive specifications, 35; spherical shape of particles, 37; as supplementary cementitious material, 4, 31, 228; ultra-fine *see* UFFA (ultra-fine fly ash); and water-binder ratio, 3
FM (fineness modulus), 50, 80
fog spraying, curing, 166
freeze/thaw resistance, 117–119
French, C., 110, 114
frost cycles, cryogenic, 122
Fuller, W.B., 53

gel pores, 74, 165
gel-space ratio, 70
Germany, Deutzer Bridge in, 16
Gilkey, H.J., 72
Gustaffero, Armand, 118–119
gypsum, 26

hammers, rebound, 189, 190
heavyweight aggregate, 49
hemihydrate (plaster), 26
Hester, W. T., 30
higher-strength concrete, terminology, 7
high-performance concrete (HPC), 9, 10–11
high-strength concrete (HSC): applications, 15–19, 122; versus conventional strength concrete, viii, ix, 1, 8–9, 99, 107; cost issues, 16–17; developing on trial and error basis, 64, 88; and high-performance concrete, 9, 10–11; proportioning, example, 88–95; ready-mixed, development, x; strength values, 7; terminology, 6–8
Hoff, G.C., 49
Holm, T.A., 49
honeycombs, 160, *161*, 215–216
Hooke's law of elasticity, 101
hot weather, 146, 158
Hover, K.C., 117–118
HRWR (high-range water-reducing chemical admixture): advantages of use, 229; air entrainment, 83; description of, 57; estimated water contributed by, 91–93; and metakaolin, 48; proportioning of high-strength concrete, example, 89; water, estimating in proportioning example, 94
hydration: controlling of rate, 64–65, 87, 146, 155; "normal", 201; of Portland cement, 44; sulfate's role in, 204–205; water consumed by, measuring, 73–74
hydration stabilizing admixtures, 56
hydraulic cement concrete: blended cements, 29; chemical admixtures, 55; description of, 227; failure in compression, 4, 5; international standardization system, lack of, 27; stresses, resistance of, 4; as two-element composite material, 2; volume changes, 109

inch-pound measurement units, 2, 93–95
incompatibility, 200–211; chemical admixtures, influence, 205; fly ash, high calcium, 206–207; and interfacial transition zone, 75; sulfate's role in hydration, 204–205
in-place evaluation: constituent materials, profiling in laboratory, 191, *192*, 193, *194*, *195*; drilled cores, 187–188; match-curing method, 188–189; maturity method, 188; penetration resistance, 190–191; rebound number, 189–190
interfacial transition zone, 75–76, 80, 99, 100
internal curing, 112, 170

www.technobetar.ir

Japan National Railway, high-strength concrete bridges, 13

K (efficiency factor), 44
kaolin clay, 47
Khalil, S.M., 205
Khayat, K.H., 122
King, Art, xi
Kinzua Dam, stilling basin at, 124
Klieger, P., 106
Kwan, A.K.H., 65

laboratory qualifications, testing, 139
later-age shrinkage, 113–114
length/diameter ratio (l/d), 176
Lerch, William, 201
lightweight aggregate, 49
lignosulfonates, 203, 205
lime, 34
loading rate, compressive strength, 185
Los Angeles River, 124
Louetta Road Overpass, US, 16
low-strength, concept, 8

match-curing method, 188–189
Material Service Corporation (MSC), x, xi, 13, 138–139, 174
Mather, K., 49
maturity method, in-place evaluation, 188
maximum density theory, 65
McDonald, J.E., 124
measurement units, 2, 90–95
mechanical properties, 99–115
Meeks, K.W., 165
Mehta, P.K., 45, 51
metakaolin, 31, 32, 47–48
Metha, P.K., 38
microsilica *see* silica fume
Mid-Continental Plaza, Chicago (52-story), 13
mill certificates (certificates of compliance), 29–30
Miller, R., 187
Mindess, S., 116
mineral admixtures, 4, 13
mini slump test, 207–210
mix proportioning *see* proportioning
mixing drums, 147, 149, 150, 234
modulus of elasticity, 232; and compressive strength, 176; conventional strength concrete, 103; defined, 101; dynamic, 106; increasing, 103; measured, *105*; mechanical properties, 99; North American buildings, 13–14; static, 100–106, *105*; tall buildings, 17
modulus of rupture, 99, 100, 108, 176
moisture probes, calibration checks, 148, 149
moisture requirements, curing, 166–170
Mokhtarzadeh, A., 110, 114
mold material, compressive strength, 179
Moreno, Jaime, x
mortar, 69
mortar flaking, 217
mortar-cube tests, 28–29
MSC (Material Service Corporation), xi, x, 13, 138–139, 174
mushrooming, placement method, 157, 222
Myers, J.J., 68

Naaman, DR Antoine, xi
National Ready-Mixed Concrete Association (NRMCA), 147, 215
Neville, A., 99
Nilsen, A.U., 106
normal-weight aggregate, 49
Norway: bridges and highway structures, 16; silica fume, use of, 45
NRMCA (National Ready-Mixed Concrete Association), 147, 215

Olba, K., 186
Olsen, N.H., 123
order taking, 144
Outer Drive East Condominium Project, Chicago (40-story), x, 13
Owner, 131, 134

packing density, 49
parking structures, high-strength concrete use, 16, 17
Parrott, L.J., 179
particle distribution, 76
particle packing, 44, 53
particle size, *39*, 49
paste density, 72–74
pastes, 26; and coarse aggregate, 44; composition of concrete, 69; hydration profiling, 87; in hydraulic cement concrete, 2; "over-sulfated", *203*; pores, 165
paste-to-aggregate bond, 52
pattern cracking, 214–215
Paulson, K.A., 115
pavements, high-strength concrete used in, 16
PCI (Prestressed Concrete Institute) Manual, 172

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penetration resistance, 190–191
Perenchio, W.F., 106
permeability, 11, 16, 116–117
petrography, 218–221
Pinto, R.C.A., 117–118
Pistilli, M.F., 184
Place Victoria, Montreal, 13
placement, 155–158, 235; cold weather, 158; column loads, transmission through floor slabs, 157; hot weather, 158; pump method, 156, 177
plant operations, 147–150
plastic shrinkage, 110–111; cracking, 167–168, 213–214
Poisson's ratio, 75, 99, 106
polycarboxylate polymers, 149
polypropylene fibers, 123
ponding, curing, 166, 169
Popovics, S., 38
Portland cement: alkali content, 42; characteristics, 23–24; chemical composition, 23–24; clinker, 23, *24*, 26; hydration of, 44; interactions, unexpected, 59; lime in, 34; minerals, 31; oxide notation, 23, *24*; primary compounds, *24*, 25; production method, 23; specifications, 27; Type 1, *23*, 30; Type 2, *24*, 30; use of, 22, 227; and water-binder ratio, 3
Portneuf Bridge, Quebec, 16
Powers, T.C., 73–74
pozzolans, 4, 29, 31, 32; Strength Activity Index, 45
precast high-strength concrete, curing, 171–173; initial set, 172; maximum temperature, 172–173; rate of temperature rise, 172
preconstruction conferences, 154–155, 233; specifications, 139–140
preconstruction phase, 134–135
preparation, 155
prescriptive versus performance-based specifications, 131–133, 232
Prestressed Concrete Institute (PCI) Manual, 172
probes: calibration checks, 148, 149; penetration resistance tests, 190–191
problem solving, 200–224; aesthetic defects, 213–218; color concern, case study, 221–223; communication, 234; durability, 116; early stiffening/erratic setting, 211–212; incompatibility, 200–211; poor strength development, 212–213; reducing problems, 210–211; useful tests, 207–210
producer qualifications, 137
production, 143–150; buttering process, 145; dispatching, 144–145; order taking, 144; plant operations, 147–150; quality control, 145–147; trucks, flagging of, 145
proportioning, 68–84; aggregate characteristics, 76–81; best practice guidelines, 2, 227; calibrating consistency, 82; considerations, 68–84; high-strength concrete, example, 88–95; interfacial transition zone, 75–76; mix, changes in, 1; particle distribution, 76; paste density, 72–74; principles of, 1, 12, 227; process of, as means to a beginning, 65, 230; statistical variability, 67–68; water-binder ratio, 70–72; workability, 84
puddling, placement method, 157, 222
pump method, placement, 156, 177

Quality Assurance Manual, 144, 233
quality control, 145–147, 174–197; compressive strength, testing variables influencing, 175–187; dispatching, 144–145; end preparations, 184–185; Quality Control Department, 146, 234
Quality Management Plans (QMPs), 136–137

raw materials *see* constituent materials
rebound number, 189–190
reference sensor, match-curing method, 189
required average strength, 6, 90, 93
resistance: abrasion, 123–124; to alkali-silica reactions, 119–120; corrosion, 121; fire, 122–123; to freezing and thawing, 117–119; penetration, 190–191; scaling, 119; sulfate resistance, 120–121; thermal properties, 121–122
retrogression, strength, 108–109
rheology, 32, 84, 125–126
rodding mixtures, specimen consolidation, 177–178

sample representation, compressive strength, 176–177
Sandhornoya Bridge, Norway, 16
sands, 50, 80
Saucier, K.L., 147
scaling, 216–217
scaling resistance, 119
SCC (self-consolidating concrete), 160–162
Schmidt, William, 13

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SCMs (supplementary cementitious materials), 4, 30–33, 228; and fly ash, 207; metakaolin as, 47; minerals, 31; quality control, 146–147; and sulfate, 204
sealing, and curing, 170
Seattle: monorail track girder, specifications for, 12; Two Union Square, 13, *15*
self-consolidating concrete (SCC), 160–162
self-desiccation, 112, 113
SEM/EDS (energy dispersive X-ray spectroscopy), 221
set retarding, *56*
Setunge, S., 106
shallow map cracking, 214–215
shear strength, 100
Shideler, J.J., 49
shrink mixing, 148
shrinkage, 109–115; autogenous, 111, 112, 113; chemical, 111, 112; defined, 109; drying, 108, 113; early-age, 111–113; later-age, 113–114; plastic, 110–111, 167–168, 213–214; *see also* creep
SI measurement units, 2, 90–91
Siebel, E., 83
silica fume, 42–47; and abrasion resistance, 124; advantages of use, 42, 228; blended cements, 29; description of, 43; forms, 45; micrograph, *46*; as performance-enhancing additive, 31; in raw state, 46; as supplementary cementitious material, 4; use in Norway, 16; and workability, *65*
silicon dioxide (SiO_2), 42
slabs-on-grade, thermal cracking, 214
slag cement, 39–42, *41*; blended cements, 29; concept, 3; as supplementary cementitious material, 31, 228
slump, 84
slump flow, 84
slump test, 124, 135–136; mini slump test, 26, 207–210; relevance, 136
Sobolev, K., 53
Soutsos, M.N., *65*
specifications: arbitrarily established limits, pitfalls, 133–135; "boilerplate", undesirability of, 131, 232; combination, 132; constituent materials, 136; laboratory qualifications, testing, 139; preconstruction conferences, 139–140; prescriptive versus performance-based, 131–133, 232; for prestressed girders, 12; producer qualifications, 137; properties, selecting, 135; quality management plans, 136–137; slump test, 124, 135–136; submittals and sale conditions, 138–139
specified strength, meaning, *5*
specimens: compressive strength, 177–179; consolidation, 177–178; moisture requirements and distribution, 179; sealed, 114; size and shape, 178–179; standard versus field cured, 186–187
standardization bodies, international, ix, 175; *see also* ACI (American Concrete Institute)
standardized test methods, 28, 186
Statement of Qualification, concrete producer, 137, 139
State-of-the-Art Report on High-Strength Concrete (ACI), xi–xii, 7–8
statistical variability, 67–68
stiffening, early: and consolidation, 159; and delivery, 150–151; and erratic setting, 211–212; evaluation of concrete, 87; factors influencing, 205; and fly ash, 206; mini slump test, 207–208
stiffness, *50*, 135
strain, versus axial stress, 100
Strehlow, R.W., 147
strength: acceptability requirements, 85–86; bond, 100; compressive *see* compressive strength; and durability, 11, 226; poor development, 212–213; as relative material property, *5*, 6; required average, 6, 90, 93; retrogression, 108–109; specified, *5*; target, *5*, 51; tensile, 99, 100, 108, 122, 176; terminology, 4–5
stress flow, 4
stress-strain relationship, 100, *101*, 102, 175, 232
sub-butuminous coal, 33
submittals and sale conditions, 138–139
sulfate, 29, 120–121, 204–205
sulfate attack, 41
sulfur caps, end preparations, 184–185
supplementary cementitious materials (SCMs) *see* SCMs (supplementary cementitious materials)
surface area/total volume ratio, curing, 168
surface scaling, 217

tall buildings: Burj Dubai skyscraper, 17, *18*, 19; in Chicago, x, 13; elastic

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modulus, 17; self-consolidating, 161; specified strength, *5*
target strength, *5*, 51
TEA (triethanolamine), 203
temperature issues: curing, 171, 172–173; and fly ash, 37; frost cycles, 122; heat induced high strength concrete, 123; slag cement, 39–40; thermal cracking, 214; trial batching, 86–87; and weather, 149, 158
tempering (water addition), 153–154; retempering, 158
tensile strength, 100, 108, 122, 176; splitting, 99, 108
tetracalcium aluminoferrite (C_4AF), *25*
thermal cracking, 165, 171, 214
thermal properties, 121–122
Thompson, S., *53*
transition zone bond, 107
transportation boxes, curing, 182, *183*
trial batching, 68, 86–88, 231
tricalcium aluminate (C_3A), *25*, 26, 30, 201, 202, 203
tricalcium silicate (C_35), 25, 30
triethanolamine (TEA), 203
trucks, 145, 148
Two Union Square, Seattle, 13, *15*

UFFA (ultra-fine fly ash), 38–39, 228; particle size comparison, *39*; as performance-enhancing additive, 31; use of, 32
unit conversions, 2

vapor pressures, fire resistance, 122
Verbeck, K., *56*
vibration tests, 162
viscosity modifying admixtures (VMAs), *57*–58

Ward, M.A., 205
Washington State Highway Department, prestressed girder specifications, 12
water: addition of (tempering), 153–154; admixtures, contained in, 82; ASTM classification, *54*; combined, 54; from concrete production operations, 54; conventional reducing, *55*–56; demand for, and aggregates, 51; estimating, in proportioning example, 90–91, 94–95; and fine aggregate, 50; high-range water-reducing, *56*–57; hydration, consumed by, 73–74; mixing, 53–54; non-potable, 53, 54; potable, 54; and ultra-fine fly ash, 38
water curing, 166, 169
Water Tower Place, Chicago, 115
water-binder ratio (W/B): cementitious materials, 30; changes in, 1; and compressive strength, 11, 70–71; concept, 3; consolidation, 159; and durability of concrete, 2, 226; permeability, 11; proportioning, 71, 72–74; slag cement, 41; target, 64, 91, 94
water-cement plus pozzolan ratio (W/(C+P)), 3, 71
water-contentious materials ratio (W/CM), 3, 71
W/B *see* water-binder ratio (W/B)
W/(C+P) (water-cement plus pozzolan ratio), 3, 71
W/CM (water-contentious materials ratio), 3, 71
West Wacker building project, Chicago, x, 13, *14*, 222
Wild, S., 107
Willems, T., 184
workability, 27–28, *65*, 84, *135*, *156*

Young, J.F., 116
Young's Modulus *see* modulus of elasticity

Zia, Paul, 10–11, 106