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Change Letter – Revision No. 01 – August 30, 2013

The *Concrete Technology Manual* is available on the Structure Construction (SC) Intranet site *Technical Manuals*: http://onramp.dot.ca.gov/hq/oscnet/sc_manuals/

Revisions

SECTION	REMOVE	DATED		SECTION	INSERT	DATED
ENTIRE MANUAL	DRAFT CONCRETE TECHNOLOGY MANUAL	January 2011		ENTIRE MANUAL	CONCRETE TECHNOLOGY MANUAL	June 2013

Background

The *Concrete Technology* Manual was first issued in September 1968, and has not been published since December 1992.

Structure Construction staff received a draft copy this edition of the *Concrete Technology Manual* at the 2011 Winter Training sessions. Since then, suggested edits have been reviewed and incorporated. The *Concrete Technology Manual* includes:

- Basic components and practice of concrete technology introduced in the first edition; and recent technological advances as outlined in the nine chapters:
    1. Structure Concrete Characteristics.
    2. Concrete Construction Materials.
    3. Review of Concrete Mix Designs.
    4. Proportioning, Mixing and Transporting.
    5. Concrete Construction.
    6. Structure Concrete Repair and Rehabilitation.
    7. Caltrans Advancements/High Performance Concrete.
    8. Material Sampling and Testing.
    9. Job Control Sampling and Testing.

Refer to the Preface of this manual for additional background information. Also included is the original preface issued in September 1968.

ROBERT A. STOTT
Deputy Division Chief
Division of Engineering Services
Structure Construction

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California Department of Transportation

Concrete Technology Manual

Division of Engineering Services | Structure Construction

June 2013

Preface June 2013

The evolution of concrete technology has probably seen greater advances in the last three decades than in the previous two millennia. Of the products on the earth useful to the life of man, concrete may have the most significant impact to modern civil society.

While the basic components and practices may not seem to have differed much from the last few generations, recent technological advances have pushed high performance concrete and the resulting design and durability advantages to new heights. In the practice of California's bridge engineering, we have realized economies for the public we serve resulting from higher strengths, that allow longer spans, fewer supports, less disturbance to the environment, and many other cost effective benefits such as the ability to achieve "Crack-Less" concrete bridge decks.

With the ubiquitous use of supplementary cementitious materials, admixtures to attain peak performance, changes to the Department's 2010 Standard Specifications, and with exciting opportunities developing from recent advances in technology like Shrinkage Reducing Admixtures and Self Consolidating Concrete, it was deemed necessary to completely rewrite the Department's Concrete Technology Manual. This effort was in conjunction with the Department's training course in Concrete Technology commissioned by Robert A. Stott, Deputy Division Chief, Structure Construction, and directed under the leadership and guidance of Dennis Wilder, Area Construction Manager, San Diego, Structure Construction.

This manual was the culmination of the brow-sweat contributions and peer review of practicing, Caltrans Bridge, Transportation, and Materials Testing Engineers, of which I am grateful for their time and efforts; some of whom are recognized here: Ric Maggenti, Craig Knapp, Matt Solano, Jeremy Peterson-Self, Mohammad Fatemi, Roberto Luena, Rod Murray, Patricia A. Shields, Arnel Gomez, and especially Tom Collins, our indispensable engineer-editor with his advanced degree in Rhetoric.

Past as Prologue: While a completely new manual is bound herein, it is desired to acknowledge the passionate efforts and studies of those that came before us that formed the foundation for this manual, as well as the backbone of the California Highway System built in the heydays of the mid 60s through the mid 70s. The First Edition Preface (1968), by my predecessor L. Edwin "Ed" Dunn, is included herein in furtherance of this desire, and to memorialize a significant piece of the Bridge Department heritage.

Norman A. "Sonny" Fereira, PE
Senior Area Bridge Construction Engineer
Structure Construction, Caltrans
Chief Editor
June 2013

PREFACE
(First Edition 1968)

Over the past decade the average value of bridge concrete items included in State highway projects has amounted to nearly one-half of the total value of all bridge construction performed. During 1967 for example, nearly $120 million was spent on bridge items. Of this total, more than $63 million was spent on structure concrete and concrete products, including 875,000 cubic yards of cast-in-place structure concrete, over 1,000,000 linear feet of cast-in-place concrete piling, nearly 70 miles of concrete bridge railing, 250,000 linear feet of CIDH piling from 16″ to 72″ in diameter, and precast products of all types from foundation piling to bridge girders.

In terms of engineering costs, approximately 50 cents out of every dollar budgeted by the Bridge Department for construction engineering at the project level is spent for field and office activities directly related to concrete and concrete products. In other words, about one-half of our field personnel are at all times engaged in administration and inspection of concrete construction. In view of this, the importance of a thorough and complete understanding of concrete as a structural building material is obvious and requires no elaboration.

This manual was originally intended as a textbook for use in conjunction with the Bridge Department's training course in basic concrete technology. During the editing process it was broadened in scope and content until it now includes all aspects of concrete production and concrete construction practices, as well as materials testing and control procedures. It has been the editor's goal to cover in detail each step of the concrete manufacturing process, with particular emphasis on the use of concrete in bridge construction on State highway projects.

Concrete technology is a rapidly changing field. New techniques in concrete production and construction practices are being introduced at an increasingly rapid rate; hence, some of the information included in this manual may soon be outdated – even obsolete. However, the sampling, testing, inspection and control procedures described herein are founded upon basic principles which will remain valid regardless of technological advances or changing production techniques; consequently, CONCRETE TECHNOLOGY will be a valuable reference manual for Bridge Department field personnel for many years to come.

Sacramento, California
September, 1968

L. Edwin Dunn
Editor, CONCRETE TECHNOLOGY

CONCRETE TECHNOLOGY MANUAL

Table of Contents

Preface .... i & ii

Chapter 1 Structure Concrete Characteristics

Structure Concrete Characteristics 1-1

Workability and Consistency 1-3
Consolidation 1-6
Bleed Water 1-6

The Hardening Process 1-7
Initial Set 1-7
False Set 1-8
Final Set 1-9
Heat of Hydration 1-9

Strength Development 1-9
Compressive Strength 1-10

Effect of Water-Cement Ratio on Concrete Strength 1-11
Curing 1-12
Strength of Concrete in Existing Structures 1-12

Volume Changes and Deformations 1-14
Creep 1-14
Dry Shrinkage 1-17
Shrinkage and Swelling Due to Moisture Change 1-19
Thermal Expansion and Contraction 1-19

Durability 1-20
Carbonation 1-21
Sulfates 1-21
Chlorides 1-22
Resistance to Freezing and Thawing 1-25
Alkali Aggregate Reaction 1-26

Delayed Ettringite Formation 1-30
Abrasion 1-30

References 1-31

Chapter 2 Concrete Construction Materials

Concrete Construction Materials 2-1

Cementitious Materials 2-1
Portland Cement 2-4
Portland Cement Types 2-5
Blended Portland Cements 2-6
Portland Cement Manufacturing Process 2-7
Particle Size and Appearance 2-8
Specific Gravity 2-8
Hot Cement 2-8
Supplementary Cementitious Materials 2-8
Fly Ash 2-11
Metakaolin 2-12
Ground Granulated Blast Furnace Slag 2-13
Silica Fume 2-14
Rice Hull Ash 2-15
Specialty Cementitious Materials 2-16
Type I, Type II or Type III (White) Portland Cement 2-16
Magnesium Phosphate Cements 2-16
Calcium Aluminate Cements 2-18
Aggregate 2-18
Contamination 2-19
Moisture 2-19
Shape and Texture 2-20
Gradation 2-20
Alkali-Silica Reactivity 2-20
Water 2-24
Concrete Admixtures 2-26
Type AE, Air-Entraining Admixtures 2-28
Air-Entrainment Advantages 2-29
Air-Entrainment Disadvantages 2-29
The Air-Entrainment Process 2-30
Dosage 2-31
Water Content 2-32

Aggregate 2-32
Other Admixtures 2-33
Mixing 2-33
Concrete Temperature 2-34
Vibration 2-34
Type A – Water Reducing Admixtures 2-34
Type B – Retarding and Type D – Water Reducing and Retarding Admixtures 2-35
Type C – Accelerating and Type E – Water Reducing and Accelerating Admixtures 2-36
Type F – High Range Water Reducers and Type G – High-Range Water Reducer and Set Retarder 2-37
Type S – Specific Performance Admixtures 2-39

Chapter 3 Review of Concrete Mix Designs

Introduction 3-1

Design of Concrete Mixes 3-1

Definitions 3-2

Supplementary Cementitious Materials 3-3

Aggregate 3-3

Grading Limits 3-5

Maximum Density and Balanced Grading 3-5
General Information 3-6

Workability and Gradation 3-6

Particle Shape and Surface Texture 3-6

Water 3-7
Admixtures 3-8

Mix Design Procedure (Absolute Volume Method) 3-9

Review of Concrete Mix Designs 3-13
Initial Submittal Review 3-14

Checking Submitted Cementitious Material Requirements and Proportioning................ 3-17

Checking Submitted Aggregate Gradations 3-20

Checking Amount of Water and Penetration 3-21

Checking Submitted Chemical or Air-Entraining Admixtures.. 3-21

Minor Concrete Binary Mix... 3-48

References 3-52

Chapter 4 Proportioning, Mixing and Transporting

Introduction 4-1

Proportioning.. 4-1
Aggregate Storage and Handling 4-2
Cementitious Material Storage and Handling 4-7
Weighing and Measuring Equipment 4-8
Manual and Automatic Batching...4-11
Moisture Meters 4-12
Chemical Admixtures.. 4-14
Controlling Batch Temperature 4-16
Cooling Concrete 4-16
Heating Concrete... 4-19

Mixing and Transporting.. 4-20
Ready-Mixed Concrete 4-22
Central-Mixed Concrete.. 4-23
Transit-Mixed Concrete 4-24
Inspection of Mixing Equipment 4-27
Transporting Mixed Concrete 4-27
Time or Amount of Mixing 4-29
On-Site Mixing.. 4-30

References 4-31

Chapter 5 Concrete Construction

Concrete Construction.. 5-1

Quality Assurance: Pre-Construction Review **5-2**
Specifications 5-2
Standard Plans 5-2
Other Reference Documents 5-2
Technical Resources 5-3
Specific Pre-Construction Practices 5-3
Bridge Construction Surveying 5-3

Concrete Forms **5-4**
Specifications 5-5
Forming System 5-7
Form Fasteners 5-8
Form Liners 5-10
Miscellaneous Form Types 5-12
Steel Forms 5-12
Fiber Form Tubes 5-13
Metal Decking 5-13
Pre-Fabricated Forms 5-15
Form Workmanship 5-15
Structural Adequacy of Formwork 5-16
Formwork Design 5-16
Formwork Design Review 5-17
Deflection 5-17
Lateral Loading 5-17
Unit Weight of Concrete 5-18
Rate of Placement 5-18
Other Factors 5-19
Admixtures 5-19
Lateral Pressure Values (ACI 347-04) 5-19
Form Release Agent 5-21
Formwork Removal 5-22

Concrete Placement **5-23**
Prior to Placing Concrete 5-23
Concrete Delivery 5-24
Quality Assurance 5-25
Forms 5-25
Mix Consistency and Uniformity 5-25
Concrete Tickets 5-25
Concrete Placement Time 5-26
Transit Mixer Drum Revolution 5-26

Concrete Temperature 5-26
Certificates of Compliance 5-26
Weight Limits 5-27
Adding Water to Concrete 5-27
Delivery Rate 5-28
Conveyance of Concrete 5-29
Chutes 5-30
Concrete Buckets 5-31
Drop Chutes 5-33
Belt Conveyors 5-34
Concrete Pumps 5-35
Slick Line 5-36
Placing Concrete 5-37
Depositing Concrete 5-38
Segregation 5-39
Prevention of Concrete Defects 5-40
Protecting Concrete 5-41

Consolidating Concrete **5-42**
Vibrating Concrete 5-43
Internal Vibrators 5-43
External Vibrators 5-46
Consequences of Improper Vibration 5-47
Revibration 5-49

Finishing Plastic Concrete **5-49**
Bleed Water 5-49
Strike Off 5-50
Initial Finishing 5-51
Machine Finishing: Bridge Deck 5-54
Other Concrete Finishing Equipment 5-56
Finishing Air-Entrained Concrete 5-57

Curing Concrete **5-58**
Plastic Shrinkage Cracking 5-59
Specified Concrete Curing Methods 5-62
Water Method 5-62
Plastic Sheets 5-63
Burlene® 5-64
Forms-In-Place Method 5-66

Curing Compound Method ... 5-67
Fogging ... 5-69
Application ... 5-70
Quality Assurance ... 5-72
Safety ... 5-74
Waterproof Membrane Method ... 5-75
Curing Structures ... 5-75
Curing Minor Structures ... 5-76

Surface Finishes ... 5-76
Specified Concrete Finishes ... 5-76
Ordinary Surface Finish ... 5-77
Patching Snap-Tie Holes ... 5-78
Repair of Rock Pockets ... 5-79
Class 1 Surface Finish ... 5-79
Whip Blasting ... 5-80
Engineer's Roles and Responsibilities ... 5-81
Unacceptable Class 1 Finishes ... 5-82
Class 2 Surface Finish ... 5-82
Safety and Environment ... 5-82
Working Over Water and Environmentally Sensitive Areas ... 5-83
Working Over Traffic ... 5-83

Special Concrete Construction Practices ... 5-83
Specification Requirements ... 5-84
Hot Weather Construction ... 5-84
Control of Concrete Temperature ... 5-85
Mixing and Delivery ... 5-86
Placing and Finishing ... 5-87
Curing ... 5-88
Use Of Admixtures ... 5-88
Cold Weather Construction ... 5-89
Mixing And Delivery ... 5-89
Placing and Finishing ... 5-90
Curing ... 5-90
Measuring Concrete Temperatures ... 5-91
Placing Concrete Under Water ... 5-91
Seal Course Concrete ... 5-91
Concrete Placement ... 5-92
Water and Slurry Displacement Method ... 5-93

Bridge Construction Practices 5-94
Foundations 5-95
Concrete Piles 5-95
Footings 5-96
Columns and Walls 5-96
Bent Caps 5-97
Superstructure 5-97
Stem and Soffit-Box Girder 5-98
Stem/Deck Construction Joint 5-99
Bridge Decks 5-100
Barrier Rails 5-102
Emergency Construction Joints 5-102
Weight Limits 5-103

Chapter 6 Structure Concrete Repair and Rehabilitation

Structure Concrete Repair and Rehabilitation 6-1

Concrete Repair Process 6-3
Assess the Defect and Determine the Cause 6-3
Surface Imperfections 6-4
Cracking 6-5
Scaling 6-7
Spalling 6-8
Delamination 6-11
Surface Wear 6-12
Determine Limits of Defective Concrete 6-12
Determine If Repairs are Necessary 6-13
Choose the Repair Strategy 6-14
Cementitious Materials 6-14
Polymeric Materials 6-15
Thermoplastic/Thermoset 6-16
High-Molecular-Weight 6-17
Free Radical Polymerization 6-17
Key Variables 6-17
Methacrylate 6-19
Polyester Concrete 6-20
Urethane 6-21
Epoxy 6-22
Epoxy Mortar 6-23
Remove Defective Concrete 6-25

Place Repair Materials 6-27
Cure The Repair 6-27

Repairing Minor Construction Defects 6-27
Dry-Packing 6-28
Conventional Concrete Patching 6-28
Shotcrete 6-29
Commercial Patching Materials 6-29
Epoxy Mortar Patches 6-30
Repair Testing 6-30

Large Defect Repair - Concrete Replacement 6-30
Preparation 6-31
Placing 6-31

Repairing Defects In New Bridge Decks 6-32
Filling Shallow Voids 6-32
Filling Deep Voids 6-32
Bonding Agents 6-33
Small Area Repair (< Two Inches Deep) 6-34
Large Area Repair (> Two Inches Deep) 6-34
Filling Cracks by Epoxy Injection 6-35

Bridge Deck Rehabilitation 6-36
Surface Preparation 6-38
Low-Slump Portland Concrete Overlays 6-40
Latex-Modified Concrete Overlays 6-41
Conventional Portland Cement Concrete Overlays 6-43
Methacrylate Overlays 6-44
Polyester Concrete Overlays 6-47

Chapter 7 Caltrans Advancements/High Performance Concrete

Introduction 7-1

High Performance Concrete 7-1
High Early Strength 7-2
High Strength Concrete 7-7
Self-Consolidating Concrete 7-14
Mass Concrete 7-23
The Basics of Mass Concrete 7-23

Thermal Control Measures 7-24
Identifying Mass Concrete 7-26
Passive Control by Prescriptive Specifications, Caltrans Practice Prior to the 21st Century .. 7-28
Performance Based Requirements 7-29
The Use of Active Thermal Control by Caltrans 7-32
Noyo River Bridge 7-33
San Francisco-Oakland Bay Bridge East Spans 7-34
Benicia-Martinez Bridge Substructure and Foundation 7-37
Cast-in-Place Segmental Bridge Construction 7-38
Lightweight Concrete Segments and Mass Concrete in Elements with Dimensions as Small as 10 inches 7-38
Devil's Slide / Confusion Hill: A Laconic Approach to Mass Concrete Specifications 7-43
Acceptance of Active Controls And Return to Passive Control 7-45
Back to Prescriptive Passive Control Specifications & Mass Concrete for Concrete Piles 7-46
Mass Concrete Conclusions 7-49
Lightweight Concrete 7-49
Shrinkage Compensating Concrete 7-51
Durability 7-53
Fiber Reinforced Concrete 7-53
Reactive Powder Concrete 7-54

Chapter 8 Material Sampling and Testing

Introduction 8-1

Test Types 8-1
Initial Samples And Tests 8-2
Acceptance Tests 8-2
Independent Assurance Tests 8-2
FHWA Samples And Tests 8-2
Test Frequency 8-2

Sample Identification 8-3

Acceptance Testing and Sampling 8-3
Certification of Project Personnel 8-5

Sampling and Testing Aggregate 8-5
Test Methods for Aggregates 8-6
Sieve Analysis (California Test 202) 8-7
Percentage of Crushed Particles (California Test 205) 8-7

Resistance to Abrasion Loss (California Test 211) 8-7

Soundness and Durability (California Tests 214 and 229) 8-8

Sand Equivalent (California Test 217) 8-8

Surface Moisture (California Test 223) 8-9

Specific Gravity of Coarse Aggregate (California Test 224) 8-10

Specific Gravity of Fine Aggregate (California Test 225) 8-10

Moisture Content by Oven Drying (California Test 226) 8-10

Cleanness of Coarse Aggregate (California Test 227) 8-10

Alkali Reactivity of Aggregates 8-10
Short Method, ASTM C 1260 8-11
Long Method, ASTM C 1293 8-11

Material Sampling 8-11
Sampling Cementitious Materials 8-11

Sampling Water 8-13

Sampling Admixtures 8-13

Sampling for Polyester Concrete 8-14

Chapter 9 Job Control Sampling and Testing

Introduction 9-1

Sampling Procedure 9-2

Field Control Tests 9-2

Ball Penetration Test (California Test 533) 9-4
Lightweight Concrete 9-4
Calibration 9-4

Test Procedure **9-5**

Density Of Fresh Concrete (California Test 518) **9-5**

Air Content by Pressure Method (California Test 504) **9-7**

Air Content by Volumetric Method (California Test 543) **9-8**

Proportion of Coarse Aggregate (California Test 529) **9-9**

Tests for Compressive Strength (California Test 521) **9-9**
Number of Test Cylinders 9-10
Frequency of Sampling 9-10

Making and Handling Test Specimens (California Test 540) **9-10**
Identification of Test Cylinders 9-12
Shipping Test Cylinders 9-13
Reporting Test Results 9-15
Safety 9-16
Concrete Pour Records 9-16
Trial Batches 9-19
Concrete Compressive Strength by Maturity Method 9-19
Sampling and Testing of Self-Consolidating Concrete 9-21
Slump-Flow Test (ASTM C1611) 9-22
Static Segregation Test (ASTM C1610) 9-22
Passing Ability Test (J-Ring, ASTM C1621) 9-23

References **9-24**

CHAPTER 1
STRUCTURE CONCRETE CHARACTERISTICS

Table of Contents

Structure Concrete Characteristics **1-1**

Workability and Consistency **1-3**
Consolidation 1-6
Bleed Water 1-6

The Hardening Process **1-7**
Initial Set 1-7
False Set 1-8
Final Set 1-9
Heat of Hydration 1-9

Strength Development **1-9**
Compressive Strength 1-10

Effect of Water-Cement Ratio on Concrete Strength **1-11**
Curing 1-12
Strength of Concrete in Existing Structures 1-12

Volume Changes and Deformations **1-14**
Creep 1-14
Dry Shrinkage 1-17
Shrinkage and Swelling Due to Moisture Change 1-19
Thermal Expansion and Contraction 1-19

Durability **1-20**
Carbonation 1-21
Sulfates 1-21
Chlorides 1-22
Resistance to Freezing and Thawing 1-25
Alkali Aggregate Reaction 1-26
Delayed Ettringite Formation 1-30
Abrasion 1-30

References **1-31**

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1 STRUCTURE CONCRETE CHARACTERISTICS

Throughout the history of the California Department of Public Work's Division of Highways and now the California Department of Transportation (Caltrans), concrete has been a major material for California's transportation facilities. Around the world, as in California, constant research into materials and methods has lead to gradual improvements in concrete. Over time, the incremental changes have resulted in significant quality differences. Advancements have occurred in:

- The manufacture of cementitious materials
- Use of chemical admixtures
- Mix design procedures
- Batching, mixing, and placing
- Testing
- Concrete compositions that expand the uses of concrete beyond the traditional definition of Portland cement

Portland cement concrete, a 19th Century material, remains an effective material that meets the complex engineering needs of 21st Century bridges and transportation facilities. Bridge engineers having the opportunity to design, build, and maintain California's vast inventory of concrete structures should appreciate this evolutionary advancement and continue to be a part of it.

When Jose de Navarro built the first modern cement mill in 1896 at Roundout, New York, the definition of "good" concrete was advanced. Over 100 years later, hydraulic cements, manufactured with methods developed by de Navarro, are used to bind aggregate into a "good" concrete that is used to construct engineered structures that are as modern today as they were then. Although the definition has changed with new needs and technological advancements, producing "good" concrete has always been a major engineering objective.

"Good" concrete needs to be designed. Duff Abrams, the discoverer of the rule of water-cement ratio, defined "design" in a paper delivered to a Portland Cement Association (PCA) conference in 1918 which still holds true today:

> "The term 'design' is used since it is the intention to imply that each element of a problem is approached with a deliberate purpose in view, which is guided by a rational method of accomplishment."

But what is good concrete? There are several qualities associated with concrete from mixing and placing to its final use as a structure or at least portions of a structure. The design engineer specifies a concrete that meets design strength and counteracts known deleterious environmental factors. Selection of ingredients is important to long-term durability. While concrete is being transported and placed or poured and surface finished into the design engineer's envisioned hardened shape, consistency and workability is important. Measures taken immediately after placement and during the early stages of the hardening process are important to concrete performance as a structure or structural element.

Contractors want a workable mix for placement (or pouring) and finishing. Workability generally refers to the ease that materials can be mixed into concrete and the subsequent handling, transporting, and placing with minimum loss of homogeneity. Measure of bleed water is a workability measurement since bleed water results in a lack of homogeneity at the surface. Several workability tests are used to measure the ability of a flowing or self-leveling concrete to resist segregating into its component parts while flowing from a point of discharge over a relatively large distance of travel within the form.

Consistency is related to workability. Consistency is a measure of the geometric stability of fresh concrete. Slump and slump flow tests measure the movement of concrete from the static state of an unconfined cone shape. A Kelly or Penetration Ball measures the distance a sphere shape of a given size and weight sinks into a flat surface of fresh concrete. A drop table is used to measure consistency of mortar. An unconfined cone of mortar is placed on a flat brass disk that is dropped to a sudden stop several times. The flow of the mortar is determined by measuring the diameter of the resulting pancake. Efflux times of grout flowing through an orifice are consistency measurements. Field engineers check concrete mixes for consistency and sometimes workability characteristics when overseeing the placement process.

Plasticity refers to a mix being plastic, which is its ability to be molded into a shape without crumbling or flowing as a liquid. Generally plasticity is not tested though sometimes it is equated with consistency. Mixes that have very stiff or very fluid consistencies are not plastic. Concrete mixes approaching initial set are said to have lost plasticity.

The methods used for placing and consolidating concrete, controlling temperatures during hardening, finishing and curing concrete are important in the selection of the material combinations used for the mix design. Inattentive workmanship and inappropriate selection of materials can lead to placement difficulties, less than optimum concrete strength, inadequate durability and performance, or irregular appearance. Fresh concrete must be placed such that the finished structure concrete meets the specified grade, cross-section, surface smoothness and frictional performance requirements. After hardening, concrete in a chemically stable environment will continue strength development for years. The inherent volume changes of concrete needs to be minimized and accounted for in the design of the structure. The concrete must be able to withstand the stresses and strains to which it will be subjected throughout its lifetime.

After construction, maintenance engineers inspect structures for changes in deflection, cracking and other signs of deterioration. Durable concrete requires periodic inspection and repair, to maintain a serviceable structure throughout its lifespan.

Good concrete is therefore a concrete that can be cast as easily and efficiently as possible, and upon hardening have appearance, shape, strength, durability, and surface qualities as engineered.

Workability and Consistency

Workability, a performance term, refers to the ease with which fresh concrete may be produced, transported, placed, consolidated and finished without segregating into component parts. Workability is affected by:

- Economic considerations
- Water content
- Types of cementitious materials
- Aggregate gradation
- Aggregate shape and texture
- Aggregate moisture content
- Admixture use
- Temperature
- Placement conditions
- Forming
- Reinforcing steel layout
- Finishing requirements

Workability is dependent on job considerations. The rheological aspect of workability is also referred to as fluidity of flow. Fluid flowing mixtures are efficient when placed from trucks or buckets directly into the forms or when concrete is placed under water, as in wet pile construction, or when the structural elements are densely packed with reinforcing steel. A stiff mixture might be usable when there is wide or open placement, when slip or jump forms are used, or when containment of the concrete within forms is difficult such as combined stem walls and soffit placement.

Economic considerations also have an effect on workability. The most economical concrete mix is the one that meets all required specifications and:

- Balances the costs of materials and placement.
- Maximizes the proportion of aggregate to cement; the optimal aggregate gradation will allow use of the minimum cementitious material specified while producing concrete of standard consistency, meeting fluidity requirements may require use of chemical admixtures that enhance fluidity.
- Minimizes the water-cement ratio, concrete strength will increase as the quantity of concrete increases and as the water-cement ratio decreases.

In general, consistency testing is an indication of workability. However this is not always true. Concrete having the same consistency measurement value may exhibit vastly different finishing characteristics. One mix may be "harsh," "sticky," or "bony" compared to another having the same slump. A slump flow value does not indicate whether or not a concrete will segregate. High percentages of crushed coarse aggregate, with elongated, angular, flat surfaces can bind against each other and result in a mix that does not flow around reinforcing steel or into tight spaces. The slump of such a mix can be the same for a mix with well-rounded aggregate that flows easily around reinforcing steel and into tight spaces. Thus workability may be different for the same specified consistency limits.

The ideal workable mixture, from a consistency standpoint, is dependent on the placement parameters. An ideal consistency for concrete placed with a moving form, or where forming is awkward such as the surface having a large grade, or at a wall rising from a horizontal surface placed monolithically, is a mixture that can be readily molded but will resist changing shape if its mold or form is removed prior to initial set. Concrete of this ideal consistency flows sluggishly without segregation or not at all after vibration ceases. Paving a slab, such as a pavement, with a slip form paver requires a stiff mix that sometimes appears to be "crumbly" when dispatched from dump trucks or end dumps. The concrete is restricted to 0 - 1 inch penetration (0 - 2 inches slump). This is constructible because of the energy a slip form paver is able to apply to the mixture. It is desirable because no forms are used and the vertical edges can be as much as 12 inches high. The edges need to maintain the flat vertical plane without forms. Roller compacted concrete is stiffer still as compaction is by the weight of machines used to compact AC or soils. In contrast very fluid, self-leveling mixtures will produce quality concrete even when placed under water or into tight strong forms. Very fluid concrete can only be used where the top surface is level. Self-leveling flowing mixtures may be necessary where access for discharge is limited or areas with highly congested steel reinforcement.

Consistency is roughly proportional to the water content for a given aggregate source with standardized gradation and proportions. Consistency is affected by aggregate gradation, proportions and characteristics and also by the characteristics of admixtures in use. Accordingly, for a given combination of aggregates and cement content, changes in consistency from batch to batch are usually the result of variations in the amount of free water or admixture use.

The recognized ASTM standard and traditional measure of concrete consistency is the Slump Test (CTM 556), shown in Figure 1-1. The slump test is performed by filling a frustum-shaped metal form with fresh concrete and then measuring the subsidence of the concrete mass after the form is removed. The subsidence, or slump, of the concrete gives an indication of the relative consistency of the concrete mixture. A high slump indicates a fluid mix useful for such elements as CIDH piles. Self consolidating concrete flowability is similarly measured with flow cone and "J ring" measurements.

Figure 1-1. Slump Test.

The Ball Penetration Test (CTM 533), sometimes referred to as the Kelly Ball Test, is a standard consistency test with Caltrans. There is a correlation between the Kelly Ball and Slump Tests. Kelly Ball reports the results as the inches actually displaced by the hemisphere. Slump Test values are typically twice the size of a ball penetration measurement. Thus a concrete with a consistency measurement of 1 inch penetration would have a consistency measurement by Slump Test of 2 inches. The Kelly Ball Test is easier and faster to perform than the traditional Slump Test, and gives uniform results for stiffer mixes. Leveling a portion of the fluid concrete mass, placing a standard spherical weight on the leveled surface, and measuring the depth of penetration of the weight into the concrete constitutes the Kelly Ball Test. The depth of penetration is an indication of the relative consistency of the mixture.

Consolidation

Ideally the in-place concrete needs to be consolidated to make it free of aggregate segregation and air spaces. Cement paste should be in intimate contact with the coarse aggregate, interior faces of the formwork, reinforcing steel and other embedded objects. The consistency of the mix is the key to consolidation. The more water in the paste and the higher the paste content the higher the fluidity. Therefore consolidation of newly placed concrete is associated with the workability, which includes consistency, and the method of consolidating fresh concrete. Manual tamping, rodding and power vibration are consolidation methods. Concrete flows more easily into tight spaces in formwork and reinforcing steel during vibratory consolidation. Vibration reduces entrapped air, helping to eliminate air bubbles and rock pockets, resulting in a denser, more compact concrete. However, over-vibration could cause segregation. (Refer to Chapter 5 for additional information on consolidation.)

With viscosity modifying admixtures, high range water reducer admixtures, and additional fine aggregate in the mix, a self consolidating concrete can be designed that needs little if any vibration or tamping for dense compacted concrete free of segregation and free of voids around steel or in tight spaces.

Bleed Water

As defined by ACI and ASTM C 125 bleeding is "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…" The presence of bleed water in concrete is a function of water-cement ratio, cementitious contents, aggregate gradations, admixture use, concrete thickness and forming. As concrete solids settle into final position, water segregates and rises to the surface as bleed water. Looked at another way, the aggregate and paste settle and push water to the surface.

Note: If water can flow through the bottom surface of formwork, as in the case where a concrete slab is poured on low saturated sand, gravel, concrete permeable base, or any highly permeable material, bleed water will not appear and there will not be a workability issue.

As the bleed water collects and rises to the surface, micro-channels are cut in the fresh concrete. Concrete strength is lower because micro-channels remain after concrete hardens. The micro-channels also serve a pathway for corrosive chemicals, hastening concrete deterioration and reinforcing steel corrosion. Bleed water may also be trapped in on the lower side of aggregate and reinforcing steel. The voided spaces signify reduced concrete strength, increased permeability and reduced durability.

Proper finishing methods for floor slabs require the bleed water sheen to disappear before working the final steel float and finish. If finish floating is done prior to this, undesirable water is sealed into the top surface of the concrete, resulting in cracking and crazing and premature maintenance and repair costs.

The Hardening Process

The change in concrete state from fluid to solid occurs as a result of a chemical process between cement and water, referred to as hydration. The hydration process begins as soon as the cement contacts water, and ends when the concrete reaches its ultimate strength or water is no longer available. Hydration occurs in three phases, initial set, final set, and progressive hardening. The main strength compounds of cement paste are formed during hydration.

Long term strength gains through the pozzolanic reaction are made possible by adding supplementary cementitious materials (SCMs) to the concrete mix. SCMs are usually made from amorphous silica and alumina oxides. The pozzolanic reaction, is the result of silica or alumina combining with calcium hydroxide, which is a product of the Portland cement hydration process. When SCMs such as fly ash are used as replacement of Portland cement, the development of strength over time maybe slower, extending over a longer time. For this reason the traditional time of 28 days for strength development of mix designs has been increased. On a project that required 8,000 psi or more, the time was extended to 56 days[1]. When fly ash use exceeds 35% replacement as a method of controlling temperature in mass concrete the time has been extended to as much as 90 days. For strength requirements from 4,000 to 7,000 psi, 42 days have been allowed for strength development.

Initial Set

The first phase, initial set, is marked by a decrease in consistency as the concrete begins to stiffen considerably. Loss of plasticity can occur gradually as the cement hydrates and the cement paste begins to harden. Or set can occur quickly, a flash set, as with fast setting hydraulic cement. Initial set is when it is no longer possible, with standard placement tools, to place fresh concrete onto the setting concrete, or consolidate setting concrete along the surfaces of a break or crack. The Vicat needle (ASTM C 191) or the Gillmore needle (ASTM C 266), can be used to identify initial set. Both tests measure the penetration, or lack of penetration of a standard diameter needle under standard pressure.

[1] Strength development time is extended to 56 days for any concrete meeting new specifications (2010 Standard Specification, Section 90-1.01D(5)(a))

Initial set is by chemical design. Initial set must not occur before there has been time to place and finish the concrete. Initial set for the vast majority of placements is typically designed to be within two hours of placement. In the case of CIDH piles placed in wet holes the set time for the first concrete discharged into the hole must be for the duration of the entire pour; this can be as much as 12 hours for very large CIDH piles. On the other side of the spectrum where a very rapid strength gain is desired, set time can be only minutes after placing. In this procedure, timing, machinery, and staffing are necessary to get the concrete in place and finished without delay.

As cement particles decrease in size, the hydration rate increases and initial set occurs faster. The set time increases as the ratio of water-cement increases. Set time decreases as temperature increases, and increases with falling temperatures. Admixtures affect set time by design, either hastening or delaying set time. The setting time of Portland cement is regulated by the amount of gypsum added to the clinker prior to final grinding. Without any gypsum, cement will set quite rapidly. The setting time increases roughly proportional to the amount of gypsum added. Although individual cement manufacturers can vary the percentage, the use of gypsum to modify set time typically requires about 3% gypsum (by weight) to the clinker to meet the requirements of ASTM C 150. For rapid-setting hydraulic cement, setting times have been regulated with accelerating admixtures, citric acid and borax.

False Set

Occasionally, a Portland cement concrete mixture will stiffen almost immediately after mixing cement and water without the generation of hydration heat. The phenomenon is known as false set. This can sometimes be a problem when using “mobile mixers” since the contact time during the “mixing” of the water and cement is measured in a few seconds. Stiffening associated with false set can be overcome with additional mixing or when additional water is permitted, by adding water.

If the temperature of the gypsum and clinker becomes excessive during the final grinding stage of the cement manufacturing process, the gypsum may transform chemically into plaster of Paris. When water is added for concrete mixing, plaster of Paris hardens immediately, causing premature stiffening of the mix. Cooling clinker prior to grinding and cooling the grinding mill are methods of reducing the use of plaster of Paris. False set may be caused by excessive temperature when cement temperature exceeds 140°F at the time of mixing. False set may occur in cold weather as a result of over-heating of the water and/or aggregates, or when the heated water mixes directly with the cement. Temperature-related false-set can be reversed by additional mixing, without adding water to restore plasticity. Additional mixing should allow the impacted concrete to set in a normal manner without detrimental effects.

Final Set

The second phase is the final set. Final set time is variable. By design, the time could be less than 1 hour or up to 24 hours later. During final set, the concrete is a soft solid without appreciable surface hardness but will retain shape without formwork support. It will support light loads without indentation; however, it is easily abraded and its surface can be scored or roughened with little effort. The heat from hydration increases significantly at final set. The length of time required for final setting is a function of the cement's characteristics and various mix design factors, i.e., cement content, water-cement ratio, and admixture use. Final set usually occurs in less than 1 hour, but may occur over 24 hours after initial set.

Heat of Hydration

Heat of hydration refers to the heat produced by the exothermic chemical reaction between Portland cement and water. Heat is usually not of major concern when it escapes into the surrounding atmosphere. The temperature rise due to the heat of hydration can be beneficial during periods of cold weather, since it helps maintain favorable curing conditions.

In the case of mass concrete pours, heat expansion and subsequent cooling contraction of the partially hydrated (green) concrete, can result in severe stresses and appearance of thermal cracks. When hydration temperatures exceed 160°F, the hydrates that are formed are unstable and expansive. When one dimension of a concrete placement exceeds 7 feet, a mitigation plan for excessive heat is usually required. In unique situations, the concrete materials or placement methods could create higher temperatures during hydration than normally expected. For example, in the Benicia cast-in-place segmental bridge, the soffit and deck were only 1 foot thick but the high Portland cement content and lightweight aggregate would have produced dangerously high temperatures had preventive measures not been taken.

Strength Development

Concrete gains strength rapidly after placement; then the rate of gain grows progressively slower, but may continue for years. Traditionally, 100% Portland cement concrete mixes with 564 to 658 pounds of Portland cement per cubic yard (pcy) and water-cement ratios of 0.5, required 28 days for strength attainment. The rule of thumb for early breaks was 65% of 28-day strength at 7 days and 80% at 14 days. When high strength concrete was specified with water-cement ratios below 0.4, the 7-day strength was expected to be 80% of the 28-day strength.

Now, with the current practice of using SCMs, non-Portland hydraulic cements, and specialized set control (retard and accelerate) admixtures, the only similarity with the traditional strength versus time graph is that the rate of strength development decreases with time. Some (not those with ≤ 3,600 psi compressive strength) concrete mixes with SCMs are allowed 42, 56 and sometimes 90 days to attain design strength because the rate of strength increase maybe slower than 100% Portland cement. Concrete with fast setting hydraulic cement can be designed to have strength in hours instead of days. Using fast setting hydraulic cement can result in rapid strength gains exceeding 5,000 pounds per square inch (psi) in 2 hours; this represents 50% of ultimate strength which will develop progressively for years.

During hydration of Portland cement, water combines with dicalcium and tricalcium silicates, tricalcium aluminates and tetracalcium aluminoferrite to form calcium silica hydrates, calcium alumina hydrates and calcium hydroxide. Water is necessary for hydration of any hydraulic cement and subsequent pozzolanic reactions when SCMs are used. SCMs combine in a pozzolanic reaction with calcium hydroxide, the hydration byproduct, in the presence of water to form calcium silica hydrates. Cementitious materials in concrete do not gain significant strength by drying; they gain strength when water is available for the hydration (chemical) reaction. Therefore once water is unavailable, strength development peaks.

Compressive Strength

Compressive strength is the most universally accepted measurement of concrete quality, because concrete structural elements are designed using a specified concrete compressive strength and strength can be easily tested. During construction, concrete strength is typically determined by breaking standard concrete cylinder samples (6-inch diameter × 12-inch height) that have been moist cured for 28 days at 70°F. After construction, cores can be taken from a structure at any age and tested for compressive strength.

ASTM allows breaking 4-inch × 8-inch cylinders, except a test is considered to be three not two cylinders. Cylinders can be cured at temperatures specifically controlled to simulate the temperature of the actual concrete in the structure element. This can be as simple as leaving cylinders on or near the structure or placing them into environmental containers. For the latter, temperature sensors such as a thermocouple are placed inside the structural concrete during placement. The container temperature is regulated to match the temperature of the in-place concrete. When the cylinders are tested, the strengths are comparable to the in-place concrete at the time of the test.

Since the hydration process is exothermic, a fairly accurate estimate of compressive strength can be obtained by integration of the time temperature curve of a curing cylinder. The accumulation of the area under the curve at any point in time can be related to the strength of the concrete at that time. This is known as the Maturity Method. Concrete temperature histories are correlated with strengths in lab tests. This data is used to produce a temperature/time/strength relationship. The temperature/time/strength relationship is unique to the given concrete tested.

When the Maturity Method has been allowed in Caltrans projects, a verification test was also required. For the Bay Area segmental bridge projects, a nondestructive test, such as a Capo Pull-Off or a Windsor Probe was required. For the Capo Pull-Off test, a ring is cast into the concrete. The force required to release the ring from the concrete was correlated to a compressive strength. A Windsor Probe shoots a pin into the hardened concrete. The depth of penetration of the pin corresponds to a compressive strength. If the nondestructive test indicated a significant difference in strength than the Maturity Method, then a cylinder test was required to verify concrete compressive strength. The cylinder test was assumed to be conservative. If cylinders were stored in an environmental container, engineering judgment was needed.

Effect of Water-Cement Ratio on Concrete Strength

The primary factor, besides the quality of the cementitious materials, influencing strength and overall quality of any concrete is the water-cementitious material ratio. Increasing cementitious content lowers water-cementitious material ratio for any given consistency. A secondary but important factor is the quality of the aggregate. Often a 'high quality' aggregate is such because it reduces the amount of water needed for a given slump. Thus, well rounded rock and sand make good concrete because it requires less water than a crushed angular rock to produce a desired consistency.

The effects of the water-cementitious material ratio on strength and permeability are shown in Figure 1-2. As the amount of water increases, strength will decrease and permeability will increase.

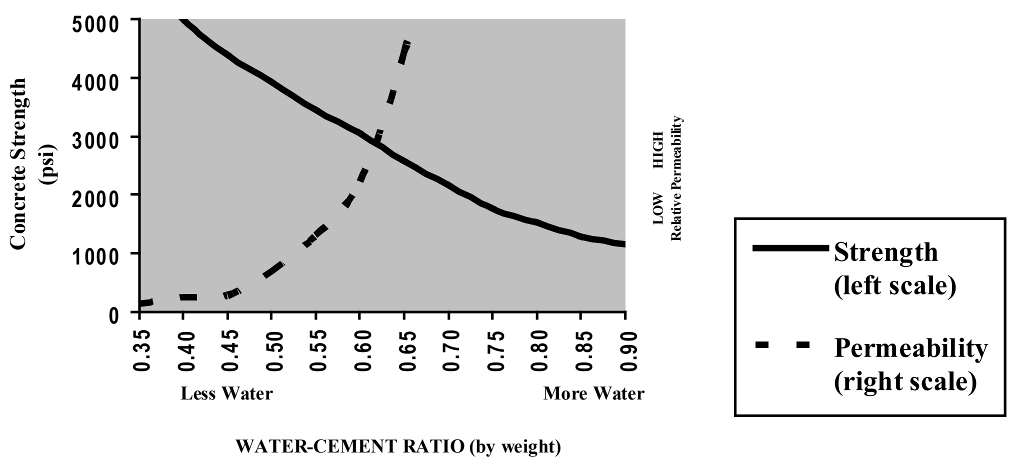

Figure 1-2. Typical Concrete Strength and Permeability Versus Water-Cementitious Material Ratio.

Curing

Obtaining optimum strength depends greatly on the extent to which the cement hydrates during the hardening process. Curing is a process by which a concrete mixture is kept in a moist state following initial setting. It is intended to ensure hydration, as well as to prevent the formation of surface cracks due to a rapid loss of water while the concrete is plastic and low in strength.

Under field conditions, concrete can become partially dry within only a few days after controlled curing concludes. Thereafter, hydration and pozzolanic reactions may occur slowly and intermittently over a period of many years using moisture occurring with rainy weather or prolonged periods of high humidity. For slabs and footings in contact with moist earth, moisture will continually wick through the concrete and hydration may continue indefinitely.

At a minimum, the paste must be kept saturated, or nearly so. If the paste is not kept saturated, the hydration process will cease when all the free water has evaporated from the paste. Thus, water must be added during the curing process to compensate for surface evaporation and for water consumption during hydration.

Strength of Concrete in Existing Structures

When core strengths are not readily available, 5,000 psi can reasonably be assumed for the current strength of concrete in existing concrete structures built between 1930 and the turn of the century. This assumes the construction was proper, the service loads have been within design assumptions, and the concrete is not subject to any durability issues such as alkali-silica reaction, delayed ettringite formation, sulfate attack, freeze-thaw damage, carbonation or other materials related distress.

Most if not all concrete during the time period between 1930 –1965 was based on a working stress (service stress) design. Per this method the allowable working stress (f_c) at the outer fiber of concrete was limited to a fraction of the 28-day ultimate compressive strength (f_c') of the concrete. The 1936 ACI code used 0.40 as the coefficient limiting the calculated stress to a fraction of the specified 28-day compressive strength of the concrete (unless it was adjacent to supports of continuous beams then it could be increased to 0.45). In 1986, the AASHTO Standard Specifications for Highway Bridge Designs still had the coefficient at 0.40. Thus we can assume the maximum designed stress for a bridge was limited to 40% of the 28-day ultimate compressive strength of the concrete ($f_c = 0.40\ f_c'$) for bridge structures built between the mid-1930s through the mid-1980s using working stress design method. Typical f_c, the maximum compression extreme fiber stress or allowable working stress, through this period ranged from 1,000 psi to 1,300 psi, with 1,200 and 1,300 being common values. Therefore, this means before leaving the engineer of record, f_c', the

28-day ultimate concrete strength used in the calculations, ranged from 2,500 psi to 3,250 psi. Using a value for f_c of 1,200 for calculating the beam stresses, we can safely say the mixes were intended to be 3,000 psi at 28 days. An f_c of 1,300 makes f_c' 3,250 psi. (For Ultimate Strength designs, 3,250 psi for f_c' was a common value used on bridge plans produced into the 1990s.)

The practice used then by the California Department of Public Works / Division of Highways to ensure actual minimum concrete strengths, was governed by the class of concrete required by materials specifications. The important concrete material requirements were the minimum sacks of cement per cubic yard along with a limit on total water. The 1954 California Division of Highways Standard Specifications required "Class A Concrete" for structures; this was a 6-sack mix, limiting water to no more than 52 pounds per sack (W/C ratio = 52 pounds/94 pounds = 0.55). This was the same practice for most of the Bay Bridge built 20 years earlier, though there were additional requirements for the Bay Bridge including a lower water limit of 47 pounds per sack in a 6-sack mix design.

The 6-sack requirement for bridge structures was used from about 1970 until the mid-1990s (with the exception of decks which were increased to 7 sacks to enhance abrasion and corrosion protection). The 6-sack requirement continued until units were changed to metric, whether designed by working stress or ultimate stress, for any design assuming 28-day compressive strengths less than 3,500 psi. (Currently, where f_c' is not above 25 MPa or about 3,600 psi, even with the current metric conversions and the use of only Ultimate Stress design methods, the standard specifications are very similar to the 1954 and older standards.) Using the typical cements of that time period, the 6-sack mix specifications usually resulted in concrete near 4,000 psi. When minimum quality aggregates and air entrainment were used, concrete strength was still above 3,000 psi.

Any Portland cement concrete from the early 1950s through 1960s that neared 4,000 psi in 28 days should have reached 6,000 psi by now, while those near 3,000 psi located in areas having lower quality aggregates and air requirements should still exceed 5,000 psi by now. If there are reasons to doubt the 5,000 psi assumption, compressive cylinder cores should be taken and tested to verify strength.

An example supporting the assumed strength is the Cypress Street Viaduct, designed and built in the 1950s (BR. #33-178). After the 1989 Loma Prieta Earthquake caused the Viaduct to collapse, 59 cores were taken and tested. The average strength of concrete from the fallen structures was over 6,600 psi, while only 3 cores tested below 5,000 psi (4,520, 4,870 and 4,890).

Cement manufacturing technology has gradually changed and concrete strengths rose. Ultimate Strength design makes the f_c' the same value as used for the design assumption with the safety factor applied to the loads. But f_c' was still typically no lower then than 3,250 psi and the concrete was still by the same class requirements. So a safe estimate for structures after 1970 can still be 5,000 psi. It should be noted that currently this 5,000 psi value is mostly used as an estimate in flexural analysis. For shear capacity estimates, 3,250 psi is recommended per Caltrans Seismic Design Standards (SDC) 3.6.1.

Volume Changes and Deformations

Not only does concrete strength and elasticity change with time after hardening; the dimensions of concrete structural elements also change. A characteristic of all materials is the volumetric change that occurs immediately with changes in temperature, moisture content, and applied loads. There are also time dependent plastic volume changes resulting from dry shrinkage and time dependent plastic deformation or "creep" caused by continuing applied loads. Concrete shrinks upon drying and swells upon wetting, but swelling is never equal to the original volume so there is an irreversible or a plastic (not to be confused with "plastic shrinkage") deformation with shrinking. As with strength, volumetric changes due to temperature, moisture content, creep and irreversible dry shrinkage are dependent also on the selection of materials in the mix design.

Even when loads are in the elastic range of concrete there is still some plastic deformation referred to as hysteresis. Hysteresis is primarily due to micro and macro cranking of the concrete as it goes through load cycles. Hysteresis is considered when predicting the behavior of an element during a seismic event as there could be multiple load cycles during the event. Hysteresis that occurs with temperature change is not considered in design practices.

Creep

Just like strength, creep and shrinkage occur rapidly at first and then gradually slow to almost nothing after a few years. The ultimate deflection (creep) for a concrete structure decreases with increased concrete maturity at the time of loading. Cement type and content, aggregate type and size, member shape and size, and reinforcement amount and type (mild or stressed) all affect creep. Generally creep magnitude is proportional to member stress, and measured in the direction of the applied load. As much as 75% of ultimate creep can take place by three months and is assumed to essentially cease after 4 years. Bridge Memo to Designers 7-10, doesn't address creep specifically, but recommends assuming one-half the total anticipated shortening should be out of the structure at 11 weeks when calculating the movement rating. Although the volumetric changes may appear small (measurements are taken by dial indicator reading to the nearest 0.00001 inch for a 1 foot length), the changes are significant to the expectations of structural behavior. Setting camber for a bridge is a result of the ability to predict creep.

Prior to 1958 camber design practices assumed creep would be four times as great as instantaneous deflection and the ultimate deflection of structures closely approximated the assumed deflection. In 1958, our specification for the minimum time before falsework removal changed from 21 days to 10 days. As structures built under the 1958 specifications entered service, many of them began developing sagging decks. It became apparent that the camber design was inadequate for structures built under the 1958 specifications. It was also observed that when falsework remained in place for the previously specified 21 days period that the structure met long-term deflection expectations. Falsework remained in place for 4 months on one structure, the Fields Landing Overhead on SR-1 in Humboldt County and when it entered service, the anticipated deflection never occurred. Adjustments in camber design and construction practices have since been made.

Although predicting the long term shape and volume changes of concrete structures due to creep and shrinkage is an important part of bridge design in general, more precise quantitative prediction of the gradual volume changes in concrete can be vital to the design and performance of segmental concrete bridges. There is a smaller margin for error in long-term volume change for balanced cantilever segmental construction. As segments are placed in opposite directions toward a closure point, the final grades are dependent on volume change and stresses due to the volume changes. Significant stresses can occur in both the superstructure and in the piers due to segments changing volume with time if not properly accounted.

At the level of prediction needed for segmental design and specifications, a reasonable database for at least the shrinkage characteristics of potential aggregates is needed. The first US segmental bridge was Pine Valley done by Caltrans in 1974. Prior to design of this structure, shrink and creep tests were performed on the anticipated mix, which included the aggregate source. The same was true for the quasi-segmental bridge done by Caltrans, Napa River Bridge, in 1976 (quasi since it was not by cantilevered traveling forms but on moving the falsework for each segment).

There have been some notable consequences when volume change is not adequately accounted. One famous example is the Parrotts Ferry Bridge, a segmental bridge built by the Army Corps of Engineers in 1979, where there was a 22-inch deflection in the center span (See Figure 1-3). Other segmentally-constructed bridges in the world with inadequate creep assumptions had more catastrophic results including collapse.

Figure 1-3. Parrotts Ferry Bridge, New Melones River, 22 Inch-Deflection[2].

[2] http://highestbridges.com/wiki/index.php?title=Parrotts_Ferry_Bridge (visited June 30, 2010).

Dry Shrinkage

Unlike creep, which is in the direction of load, dry shrinkage is not affected by load direction. Tensile stresses will develop in concrete structural elements when they are restrained from shrinking. Cracking of concrete structures has been investigated for decades, with particular interest in deck cracks caused by shrinkage. Curing practices developed over the last century have been effective in preventing early cracking caused by plastic shrinkage while stress cracking can be mitigated by proper design of the element. However dry shrinkage cracking has been accounted for at best by crack control such as engineering the crack to predetermined locations or spreading them over a large area.

A cast-in-place concrete deck on precast girders adds to the inherent stresses due to dry shrinkage. The 1958 ACI Journal Proceedings, "Tentative Recommendations for Pre-stressed Concrete" by ACI/ASCE Joint Committee 323, was quoted in the March 2004 Concrete International Magazine:

> In structures with a cast-in-place slab supported by prestress beams, the differential shrinkage tends to cause tensile stresses in the slab and in the bottom of precast beams....When cracking load is significant, such stresses should be added to the effects of the load. (Section 212.4.6 "Shrinkage Stresses")

Various strategies to mitigate dry shrinkage stresses have been proposed over the years including using shrinkage compensating cements such as Type K, reducing the cement content while reducing water as much as possible, using fibers, and even slowing down the hydration process significantly by replacing up to 70% of the Portland cement with fly ash and waiting months for strengths typically achieved in days. Shrinkage compensating cement initially expands upon curing and thus dry shrinkage results in near net zero shrinkage. To be effective this initial expansion needs to be restrained so that the accompanying dry shrinkage releases stress rather than volume changes. Several decks using Type K or similar cements have been tried over the last few decades. Success of Type K at reducing cracking has been marginal at best. In the 1980s, one method was proposed where concrete was put under vacuum after placement to literally suck water from the concrete; this method was never fully developed.

During the construction of the San Mateo/Hayward Bridge widening project completed in 2002, deck cracking appeared to be due to the cast-in-place deck being placed over precast girders and precast deck bottom panels. Paying more attention to curing details solved some early initial problems with plastic shrinkage. But as time went on cracking was noted in the deck after several weeks. This cracking correlated with the hypothesis that the deck was undergoing a volume change due to shrinkage but was restrained by the "preshrunk" precast girders and precast deck bottom panels. As noted in the 1958 ACI Journal quote referenced above regarding induced shrinkage stresses, this type of cracking would be typical to this type of construction. The cracks were repaired with methacrylate to bond and seal the cracks at the end of the job.

Toward the end of the San Mateo/Hayward Bridge project a similar phenomena was taking place on a series of bridges on I-80, near the town of Truckee in the Sierra Nevada Mountains, being built as part of a realignment and replacement project. These bridges consisted of placing cast-in-place concrete decks on precast Bulb Tee Girders. Again cracking was noted several weeks after deck placement. It was decided that since there were several deck placements and some lead time before the next placements Shrinkage Reducing Admixtures (SRA) could be evaluated as a possible solution. Between August 2001 and May 2002 the problem of cracking was addressed during stage construction on six bridges on this project located on I-80 in the Sierra Nevada Mountains. Though perfect control of the environment and mix parameters were not fully achieved since this was an ongoing construction project, adequate assumptions could be made and more than reasonable conclusions were drawn. There was a dramatic reduction in cracks when SRA was used (see Figure 1-4). This was repeated on the Angels Crest Bridge in 2008. To date the decks are crack free.

At about this same time testing was being done on SRA being used to control shrinkage in the Skyway portion of the east Spans of the San Francisco-Oakland Bay Bridge. The main purpose of the SRA on this structure was to control geometry of this precast segmental bridge structure. It was apparent from the testing for the Skyway and in the literature that dry shrinkage could be significantly reduced by use of these chemical admixtures.

Figure 1-4. Deck Cracks Before and After Use of SRA.

Shrinkage and Swelling Due to Moisture Change

During the curing period, concrete is maintained in a continually saturated or nearly saturated condition. When curing ceases, the free water within the concrete mass soon evaporates and, as the concrete dries, reversible and irreversible shrinkage occurs. While dry shrinkage may continue for several years, about one-third of the total expected drying shrinkage occurs within the first month and about 90% within the first year. Dry shrinkage ranges from about 600 to 800 millionths per unit of length (approximately 1 inch per 120 feet).

The most important factor influencing drying shrinkage is the water content. Other factors influencing swelling and drying shrinkage are types of cement and aggregates used, and amount and type of reinforcement used.

Non-reinforced concrete shrinks and swells more than reinforced concrete. The reinforcement restricts shrinkage and swelling but does not prevent it. Typical drying shrinkage in reinforced concrete is approximately 200 to 300 millionths per unit of length (or about one-third of the shrinkage for a comparable mass of non-reinforced concrete).

When concrete dries it shrinks more rapidly at the surface than within the mass. This produces stresses, which may cause the formation of a network of cracks extending a short distance inward from the surface (called "crazing" or "map cracking"). Unrestrained, thin slabs, such as concrete paving panels, may warp or curl due to non-uniform drying.

After the initial drying process occurs, the amount of moisture remaining in a concrete mass is proportional to the relative humidity of the surrounding air. A decrease in humidity causes cement paste within the concrete mass to lose moisture and shrink, while an increase will cause it to gain moisture and swell. These cycles produce alternating states of internal compressive and tensile stresses. Overall, these incremental volume changes and their resulting stresses are small and of little consequence for most concrete.

Thermal Expansion and Contraction

Concrete expands and contracts as temperatures rise and fall. The following equation is used to estimate the length change for a concrete mass resulting from a known temperature change:

$$\Delta L = \alpha L_1 (T_2 - T_1)$$

Where: ΔL = Estimated length change
L_1 = Original concrete mass length in inches
T_1 = Original concrete mass temperature (°F)

T_2 = Final concrete mass temperature (°F)
α = Thermal expansion/contraction coefficient (1/°F)

The magnitude of the thermal expansion/contraction coefficient is influenced by several factors including the type of aggregate, the amount of cementitious materials in the mix, the water-cementitious material ratio, the age of the concrete, and the relative humidity. Of these factors, aggregate type has the greatest influence.

Aggregate thermal expansion/contraction coefficients vary from 3.8×10^{-6}/°F for limestone to 6.6×10^{-6}/°F for quartz. Typically, a value of 5.5 ×10-6/°F is used. For a 100-foot concrete span that has undergone a 77°F temperature change this equates to 1/2 inch change in length.

Durability

Durability refers to the ability of concrete to withstand the adverse effects of environmental weathering and transportation demands during its service life without serious deterioration. Durability involves the consideration of a number of related factors, such as resistance to weathering, abrasion, resistance to chemical attack, and physical deterioration.

Permeability can have a large impact on durability. Permeability is the rate (distance/time) that molecular-sized particles migrate through porous substances like concrete. Water and other liquids, liquid borne ionic compounds such as chlorides and sulfates, and gases such as carbon dioxide are examples of materials that permeate through concrete and react with concrete components in the process. For concrete, permeability is controlled by density of cementitious paste and porosity of aggregate, the ratio of paste to aggregate and the bond between paste and aggregate. As permeability increases, durability decreases.

Susceptibility of concrete and reinforcing steel to chemical attack is a function of design, cementitious materials, water-cementitious material ratio, admixtures, and curing processes. However, the most significant factor is the water to cementitious material ratio. When the water-cementitious material ratio is lower than 0.4, permeability is reduced, outer chemical attack threats are reduced, and durability is greatly enhanced. Chemical ions (primarily sulfates and chlorides) advance through concrete as a function of permeability, low permeability equates with more resistance to ion flow. Surface treatments that seal concrete surfaces can also be effective in reducing chemical caused deterioration. Silane has been shown to be an effective sealer in this regard.

Carbonation

Carbon dioxide and water begin combining with calcium hydroxide and to a lesser extent with calcium silica hydrates in concrete after it reaches final set. Although carbonation increases concrete strength, it also leads to corrosion of reinforcing steel. As hydration occurs in fresh concrete, hydroxides form as a byproduct. The presence of hydroxides raises the pH of concrete to 12. As the pH passes 11.5, reinforcing steel develops a passivating layer of iron that protects the steel from further corrosion. Carbonation reduces the alkalinity of concrete. When the pH drops below 11.5, the passivating layer is destroyed and corrosion can proceed.

Sulfates

The greatest danger from external sulfates occurs when sulfates and ground water are present. The sulfates usually are found in the form of magnesium, sodium, potassium or ammonium sulfate. Sulfates may also come from sulfuric acid produced by industrial operations or decay of organic matter. Sulfate ions will either cause expansion and cracking of concrete or deterioration of the concrete as the concrete loses bonding strength. External sulfates are countered by quality concrete with low water-cementitious material ratio and low permeability. In high sulfate environments, the concrete's water-cementitious material ratio should be no more than 0.4.

For 57 years the Portland Cement Association and the Division of Highways/Caltrans evaluated concrete embedded in a sulfate rich soil water basin maintained at Caltrans Laboratory facilities. (From 1940 until 1958 the water basin was at the laboratory complex on H Street. The laboratory complex moved to the current location on Folsom Boulevard in 1958.) Samples were placed in the sulfate rich soil so only the bottom half of each sample was covered. The basin was then flooded and allowed to dry. There were 10 to 15 wet dry cycles per year; the wetting and drying cycles in the sodium rich solution provided for the most severe sulfate exposure. Over the years the concrete variables tested included: water-cement ratios; cement content; air entrainment; cement fineness, varying cement chemistry within a type of cement; SCMs such as slag, fly ash, silica fume and calcined shale; and curing techniques such as water cure, stream curing and curing compounds.

After 57 years it was found that the reduction in water-cement ratios was "the most effective means of improving the performance of concrete in sulfate exposure conditions, regardless of cement type or composition" (Portland Cement Association's "Concrete Technology Today", David Stark, Volume 18, #2, July 1997). The type of cement and SCM constituency is of greater importance only when water-cement ratio exceeded 0.45. In those cases, Type V cement resists sulfate attack best because of the lower percentage of calcium aluminate. Also, Silane sealer proved to be effective in mitigating sulfate damage to concrete having water-cement ratios in the 0.45-0.55 range.

Chlorides

Chloride ions associated with deicing chemicals and salt water are carried by water as it permeates through concrete, eventually coming into contact with reinforcing steel. As ions reach the reinforcing steel, an electrochemical process, a corrosion cell will form and the steel will corrode into iron oxide. Rust occupies four to six times the volume of steel, resulting in expansive stresses on concrete and surface spalling and diminishing reinforcement cross-sectional area. A typical spall is shown in Figure 1-5.

Figure 1-5. Corrosive Girder Spalling.

When the chloride/water concentration in a concrete element reaches the corrosion threshold (the concentration at which corrosion activity can occur), a corrosion cell will be established along a bar of reinforcing steel. Corrosion begins as a single anodic area forms on the bar; the rest of the bar will be cathodic. As the chloride/water concentration increases, another anodic area will form, either on the same bar or on an adjacent bar. Development of anodic areas continues as the chloride concentration increases, until most of the steel is anodic and the entire structural element is filled with numerous, battery-like corrosion cells.

Early attempts at rehabilitation of corrosion damaged bridge decks were driven by the conventional wisdom of the time, which focused on the removal and replacement of all concrete in areas where visible and undersurface delamination were evident. This procedure left large areas of chloride-contaminated concrete still in place, and even though the repaired deck was covered with a protective membrane (and in some cases a concrete overlay) to inhibit further moisture penetration, corrosion, along with subsequent development of corrosion induced fracturing, continued, and for some structures, continues to this day. Corrosion persists in repaired concrete for two reasons. First, when an anodic area is disturbed by removing the chloride-contaminated, fractured concrete and replacing it with relatively salt-free concrete, the electric potential is reversed; that is, steel in the undisturbed portions of the deck, formerly cathodic, becomes anodic, whereas steel within newly patched areas, formerly anodic, becomes cathodic. As long as moisture and oxygen are present, the corrosion process will continue unabated. Second, the membrane systems used during the 1970s and early 1980s were not 100% impermeable.

The concrete overlays installed during the same time period also experienced chloride penetration. As a consequence, the barrier systems were not effective in preventing further chloride contamination. Of greater importance, however, is the fact that even with a totally impermeable deck protective system, moisture in amounts sufficient to fuel the corrosion process will migrate upward from the unprotected underside of the deck, or inward from the exposed lateral surfaces of the deck and girder stems.

Today, it is understood that the only certain cure for a chloride-contaminated structural element is total replacement. However, replacement is not currently a viable option for most of the structures where corrosion continues to occur. In view of this, it is evident that rehabilitation will remain a necessary maintenance effort for many years to come.

Corrosion is an electro-chemical process. For an electro-chemical cell to function, three elements are necessary: an anode, where corrosion takes place; a cathode, which does not corrode but which maintains the ionic balance of the corrosion reactions; and an electrolyte, which is a non-metallic solution capable of conducting an electric current by ionic flow. When the chloride concentration within concrete reaches the corrosion threshold (the concentration at which corrosion activity can occur), a corrosion cell will be established along a bar of reinforcing steel. Initially, only a single anodic area will form on the bar; all of the other steel (on either side of the anodic area) will be cathodic. Subsequently, due to changes in moisture or chloride content, another anodic area will form, either on the same bar or on an adjacent bar. Development of anodic areas continues as the chloride concentration increases, until most of the steel is anodic and the entire element is filled with numerous, battery-like corrosion cells.

Concrete that is not contaminated with chlorine has an inherent resistance to corrosion attack because of its high pH value, caused by calcium hydroxide. The high pH inhibits the corrosion process, and is gradually neutralized by the presence of soluble chlorides. Once the corrosion threshold is reached, corrosion will occur if oxygen and moisture are present.

When a metal is placed in an electrolyte, a self-generated electro-chemical activity will occur if there is a potential difference between two areas of that metal. This activity (i.e., a flow of ions from the anodic to the cathodic areas of the metal) will result in corrosion at the anodic area. The process is not unlike the activity that occurs in a wet cell battery as it produces an electric current. That is, the potential difference between two electrodes in a battery, or between two areas along a single metal bar, causes a current to flow through the electrolyte from one electrode, or one area, to the other. In a battery the electrical circuit is completed through external physical connections; in a reinforcing steel bar, the circuit is completed through the bar itself.

Iron, since it is relatively high in the electromotive force series, can easily enter into solution, which liberates electrons at the anode.

Anodic reaction: $Fe = Fe^{2+} + 2e^-$

$Fe^{2+} + 2Cl \rightarrow FeCl_2$

To maintain chemical equilibrium, electrons must be consumed at the cathode, and provided oxygen and moisture are present, hydroxides will be formed.

Cathodic reaction: $O_2 + 2H_2O + 4e^- \rightarrow 4OH^-$

The free hydroxide replaces chlorine and ferrous hydroxide is deposited at the anodes, where it combines with water and oxygen to form ferric hydroxide.

$$2FeCl_2 + 2H_2O + O_2 \rightarrow 2Fe(OH)_2 + 2Cl$$

Once liberated, chlorine ions are free to cycle around and attack additional iron molecules.

$$4Fe(OH)_2 + 2H_2O + O_2 \rightarrow 4Fe(OH)$$

High quality aggregate and cement paste have the greatest effect on resistance to chloride intrusion. Aggregate particles should be sound (hard and strong) and of low porosity. The cement paste should be dense, watertight and completely embed the individual aggregate particles. Additionally, thorough mixing, optimum vibration to ensure maximum consolidation, and adequate curing are essential to the production of concrete that resists chlorides. Other standard preventive procedures include concrete cover (usually 2 - 2 1/2 inches or more in extreme exposure climates), low water-cement ratio (less than 0.40), use of supplementary cementitious materials such as silica fume that reduce permeability, and use of epoxy-coated reinforcement to keep chloride ions separate from reinforcement. Polyester concrete and similar topical coating overlays are also utilized to limit chloride-ion penetration.

Resistance to Freezing and Thawing

Of the many naturally occurring concrete disintegration mechanisms, repeated freeze-thaw cycles are the most severe. In normal concrete, both the hardened paste and the aggregate particles are vulnerable to the destructive effects of freeze-thaw cycles. If concrete is subjected to long periods of wet weather prior to freezing, it will almost certainly be damaged when freezing occurs. The cement paste is vulnerable because, when saturated paste is frozen, expansion of the free water in the paste produces a volume increase that exceeds the amount of expansion that the paste can withstand without damage. Porous aggregate particles may also absorb moisture. The absorbed moisture will produce expansive forces capable of bursting the aggregate particles if freezing occurs while the aggregate pores are filled (or nearly filled) with water. The rate of damage caused by concrete freeze-thaw cycling is directly related to the porosity of the cement paste.

Air-entrained concrete is used in areas subjected to freeze-thaw cycles. Air-entrained concrete contains billions of tiny air bubbles that are uniformly dispersed throughout the cement paste as the concrete is mixed. These air bubbles create a system of spherical air voids in the hardened concrete. The voids produced by entrained air reduce the permeability of the cement paste, which reduces the amount of free water present in the concrete. As the free water in the air-entrained concrete freezes, it expands through the void system and exerts pressure against the entrained air, which then compresses to relieve the pressure. When thawing occurs, the compressed air forces the water back into its original space. This process continues during repeated freeze-thaw cycles, providing long-term protection against deterioration. Entrained air protects only the cement paste; it provides no protection to the aggregate. Consequently, the ability of air-entrained concrete to resist freeze-thaw action will depend on the porosity of the aggregate particles and their degree of saturation. Typical before and after photos of freeze-thaw damage are shown in Figure 1.6.

Figure 1-6. Freeze-Thaw Damage, Before and After Photos[3].

Alkali Aggregate Reaction

In some areas the native aggregate contains silica that chemically reacts with the sodium and potassium oxides, Na_2O and K_2O, found in cement. All cement specified for use on State highway projects meets the requirements for "low alkali" cement. Additionally, aggregates that have not received an innocuous rating will require the use of more SCMs than innocuous aggregates. Such aggregate is commonly called reactive aggregate, and the chemical reaction that occurs is known as the alkali-silica reaction (ASR), which results in a degenerative expansion of concrete. The alkali-silica reaction produces a gel that absorbs moisture and swells. The expansive swelling results in surface deterioration, cracking, spalling, and eventually complete disintegration of the concrete, as shown in Figure 1-7.

[3] Investigation into Freezing-Thawing Durability of Low-Permeability Concrete with and without Air Entraining Agent, Kejin Wang, Gilson Lomboy, Robert Steffes, National Concrete Pavement Center, June 2009 (visited 9/10/10).

Figure 1-7. Concrete Deterioration Due to Alkali-Silica Reaction[4].

A similar reaction occurring between dolomitic limestone and alkali, called an alkali-carbonate reaction, also results in expansive aggregate through the breakdown of dolomitic limestone aggregates. At this time, the potential of alkali-carbonate reactions are recognized, but not considered relevant to Portland cement concrete.

Thomas Edison Stanton, shown in Figure 1-8, Materials and Research Engineer for the State Division of Highways, is credited with discovering ASR in 1940. Stanton published a paper in the proceedings of the American Society of Civil Engineers[5] stating that "excessive expansion of concrete may occur through chemical reactions between cements of relatively high alkali content and certain mineral constituents in some aggregates, such as shales, cherts, and impure limestones." Stanton implemented the use of low alkali cements.

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[4] Reactive Solutions — An FHWA Alkali–Silica Reactivity News Publication Volume 3, Issue 3.

[5] Stanton, T.E. (1940). "Expansion of concrete through reaction between cement and aggregate," *Proceedings of the American Society of Civil Engineers*, Vol. 66, No. 10, pp. 1781-1811.

Figure 1-8. Thomas Stanton.

From 1940 to 1985 alkali-silica reaction was addressed by requiring low alkali cements. Alkali content was limited by specifications to cement that contained no more than 0.6% (by weight) of total alkali, calculated as Na_2O plus $0.658K_2O$. In 1955, structures built with reactive aggregate and low alkali cement were inspected and found to be in good shape. In 1958 the Division of Highway's Materials Engineer, Bailey Tremper, reported in the Highway Research Board Highway Research Report C-18 that the sole use of low-alkali cement to prevent reactive aggregate damages "was fully satisfactory judging from fairly comprehensive field surveys." By 1960 all cement used in highway facilities was required to be low-alkali cement. Shortly after, it was discovered that in the Bishop area the lower limit needed to be 0.3%; this was referred to as "Low-Low Alkali" cement. The highly reactive Bishop aggregate requiring 0.3% alkali was an exception, and the 0.6% limit was thought to be adequate for other concrete. Later low-low alkali cement became unavailable and concrete using these aggregates required 15% fly ash in addition to low-alkali cement. This however did not appear to be adequate either.

In 1985, the Simi Valley Freeway was rapidly deteriorating due to reactive aggregate. After field investigations, bridges in the Santa Barbara and San Luis Obispo area were determined to have reactive aggregate damage. A Statewide field study revealed more structures damaged by reactive aggregate. In 1986, ongoing projects were required to add 15% fly ash to the cement content. Specifications were written so that all contracts required the aggregate to be tested per ASTM C 289 for reactivity or its potential. Any aggregate found to be reactive or potentially reactive would require 15% of cement to be replaced with fly ash. Aggregates known to be highly reactive required another 15% fly ash (30% total) by weight of the required cementitious content.

By 1992 as a result of a research project initiated at Caltrans Transportation Laboratory, it appeared the 15% replacement was not always adequate, sometimes even making the ASR more detrimental (see Figure 1-9). In 1996, when aggregate was found by ASTM C 289 to be potentially reactive, an additional 15% was required, making the total fly ash requirement 30%. Given the variability of results of ASTM C 289 for many aggregates the concrete industry proposed having a universal requirement for fly ash in all concrete. In 1997, specifications required all Caltrans cast-in-place concrete to be at least 25% fly ash. There were alternatives available in certain situations, such as 10% silica fume in corrosive environments or if the fly ash contained less than 2% calcium oxide, then only 15% fly ash was required.

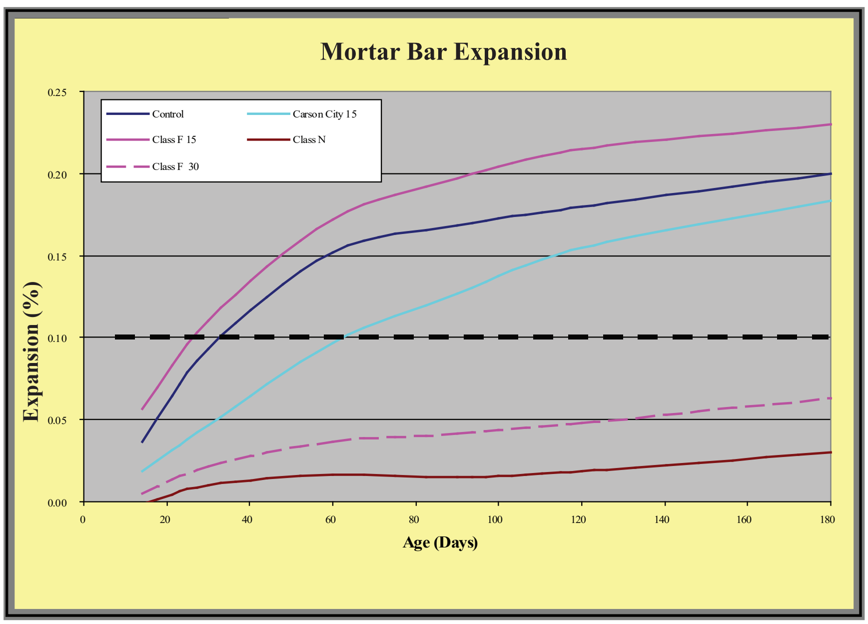

Figure 1-9. Mortar Bar Expansion Test Results.

Caltrans Materials Engineering and Testing Service (METS) currently maintains a source list of innocuous aggregates for concrete. Source testing must pass a combination of California Test 554 and either ASTM C 1260 or ASTM C 1293. For these aggregates the minimum amount of required fly ash is lower. Ground granulated blast furnace slag, with its wider use, is also acceptable as a solution to ASR mitigation.

Delayed Ettringite Formation

Ettringite is the common name for hexacalcium aluminate trisulfate hydrate, $(CaO)_6(AL_2O_3)(CSO_3)_3 \times 32H_2O$. Ettringite typically forms during the hardening phase within all concrete. (In cement chemistry the notation would be $C_6AS_3H_{32}$ where C stands for CaO, A for Al_2O_3, S for SO_3, and H for H_2O.) Since it normally forms during the hardening process there are no disintegrative forces on the concrete. But at high curing temperatures ettringite formation is inhibited. Gradually, after the hardening, the molecules combine to form the ettringite, but when the formation is delayed, the resulting crystal exerts pressure within the concrete shape and causes disintegrative cracking. This phenomenon is called Delayed Ettringite Formation or DEF.

The only agreed upon way to prevent DEF is to avoid high curing temperatures. The temperature limit varies between agencies. At present, Caltrans uses 160ºF as the upper limit.

Abrasion

Naturally occurring abrasive elements include wind, flowing water, floating ice, and debris. Man-made abrasive sources include moving traffic, snow plows, and tire chains. Abrasive forces can wear away the cement and aggregate particles or dislodge individual aggregate particles from the concrete. The water-cement ratio, mixing, placing and finishing techniques, and curing conditions all have a greater influence on resistance to abrasion than does aggregate quality. Soft aggregates require more cement to reduce wear, while hard aggregates require just enough cement to secure the aggregate in place against the abrasive action.

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CHAPTER 2
CONCRETE CONSTRUCTION MATERIALS

Table of Contents

Concrete Construction Materials **2-1**

Cementitious Materials **2-1**
Portland Cement 2-4
Portland Cement Types 2-5
Blended Portland Cements 2-6
Portland Cement Manufacturing Process 2-7
Particle Size and Appearance 2-8
Specific Gravity 2-8
Hot Cement 2-8
Supplementary Cementitious Materials 2-8
Fly Ash 2-11
Metakaolin 2-12
Ground Granulated Blast Furnace Slag 2-13
Silica Fume 2-14
Rice Hull Ash 2-15
Specialty Cementitious Materials 2-16
Type I, Type II or Type III (White) Portland Cement 2-16
Magnesium Phosphate Cements 2-16
Calcium Aluminate Cements 2-18
Aggregate 2-18
Contamination 2-19
Moisture 2-19
Shape and Texture 2-20
Gradation 2-20
Alkali-Silica Reactivity 2-20
Water 2-24
Concrete Admixtures 2-26
Type AE, Air-Entraining Admixtures 2-28
Air-Entrainment Advantages 2-29
Air-Entrainment Disadvantages 2-29
The Air-Entrainment Process 2-30
Dosage 2-31

Water Content 2-32
Aggregate 2-32
Other Admixtures 2-33
Mixing 2-33
Concrete Temperature 2-34
Vibration 2-34
Type A–Water Reducing Admixtures 2-34
Type B – Retarding and Type D – Water Reducing and Retarding Admixtures 2-35
Type C – Accelerating and Type E – Water Reducing and Accelerating Admixtures 2-36
Type F – High Range Water Reducers and Type G – High-Range Water Reducer and Set Retarder 2-37
Type S – Specific Performance Admixtures 2-39

2 CONCRETE CONSTRUCTION MATERIALS

What are the components of concrete? The standard answer, "cement, aggregate, water and possibly admixtures" is the same as it was 100 years ago. However, with today's 21st century concrete, the standard answer is an oversimplification. Portland cement is only one of many cementitious materials. Aggregates are more than inert fillers. Recycled water may contain excessive amounts of alkali chemicals. The various materials that may be encountered in the construction and repair of concrete structural elements are described in this chapter.

Cementitious Materials

Cements are substances that bind fragments of solid matter into a compact whole. For building materials this is taken to mean a fluid or semi-fluid (plastic) material that eventually hardens within an aggregate matrix (stones, pebbles, sands, bricks, and the like) to form structural elements of predetermined shape. Historically, adhesives were derived from a number of naturally available materials. Asphaltic materials were used as glue and sealants in areas where asphalt naturally seeped to the earth's surface and either accumulated in pools or combined with local aggregates and hardened into rock asphalt. Adhesives are still made from boiled animal and fish parts, tree sap, milk protein (casein), and polymers like natural rubber. Early mortars were based on mud and clay.

Calcium-based cements were first used in the mortar that united the stone blocks used for the massive masonry projects built in ancient Egypt. Early calcium-based mortar was made with calcium sulfate (gypsum) and sand. Gypsum is water soluble, but was a suitable masonry binder for construction in the dry Mediterranean climate. Although the technology has been traced to ruins of ancient Mesopotamian and Greek structures, the use of lime cement became widely used when it was later adopted for Roman construction. Lime was produced by heating limestone to high temperatures. The lime powder was subsequently mixed with water to form a mortar that hardened over time. The ancient engineers knew lime-based mortar gained strength over time; but the reason behind the strength was unknown. As civilization developed, cement derived from heating calcareous (calcium containing) rocks became the preferred material for binding an aggregate matrix. Over 3,000 years would pass before scientists determined that lime mortar gained strength as carbon dioxide permeated through the mortar and combined with the mortar to form calcium carbonate in the process known as carbonation.

Limestone, chalk and seashells are composed of calcium carbonate, $CaCO_3$. Historically, materials containing calcium carbonate were heated (calcined) to produce quicklime (calcium oxide, CaO). Carbon dioxide (CO_2) was driven off by the heating process to become atmospheric gas.

$$CaCO_3 + \text{Heat} \Rightarrow CaO + CO_2$$

Adding water to calcium oxide creates slaked lime, or calcium hydroxide.

$$CaO + H_2O \Rightarrow Ca(OH)_2 + \text{Heat}$$

As CO_2 gradually permeates through the hardened mortar, carbonation occurs, where carbon dioxide and calcium hydroxide combine into calcium carbonate (limestone).

$$2Ca(OH)_2 + 2CO_2 \Rightarrow 2CaCO_3 + 2H_2O + \text{Heat}$$

Roman buildings constructed with lime sand mortars owed their durability to thorough mixing and compacting. Romans later discovered that the fine grinding of volcanic sands or tuff from Pozzuoli, a seaport near Naples and Mt. Vesuvius, resulted in a material that was stronger than the lime sand mortar. The Pozzuolian sands contained silica and alumina which reacted with calcium hydroxide from slaked lime to become hydraulic cement that activated and hardened in the presence of water and was insoluble under water. Pozzolanic materials are named after the volcanic ash from the Pozzuoli area. The ancient Roman writer Vitruvius wrote: "there is a species of sand which possesses extraordinary qualities. It is found…in the neighborhood of Mount Vesuvius…mixed with lime and rubble, it hardens as well under water as in ordinary buildings." Roman also know that finely crushing clay bricks and mixing it with lime produced a similar product. Roman pozzolanic concrete was used by Roman architects for structures such as the Pantheon (Figure 2-1). The Pantheon's dome (diameter: 142 feet) is the largest unreinforced concrete dome in the world. Greece produced the same kind of hydraulic cements well into the 20th century by mixing lime with the volcanic tuff from the island of Thera, now called Santorini.

Figure 2-1. Roman Pantheon, 120 AD.

The use of pozzolanic materials and lime is the foundation of modern concrete, because the volcanic ash and clay brick consists of silica and alumina that combine with lime after the addition of water. Two precursors of Portland cement, Hydraulic Lime Cement and Natural Cement, were developed in the late 18th and early19th century. Like Greek and Roman hydraulic cements, the new cements developed strength due to the mixing of lime with silica or alumina compounds. For Natural Cement, septaria nodules, consisting of limestone and clay, were mined in Rosenberg, New York and subsequently calcined. Hydraulic Lime Cements were produced by mixing limestone with clay in a kiln.

Modern Portland cements contain compounds of calcium, silica and alumina that can be precisely blended during the manufacturing process. Supplementary cementitious materials (SCMs), like fly ash or silica fume, that harden through the pozzolanic reaction with free lime produced as a byproduct of Portland cement hydration are also standard components of 21st century concrete. Ground volcanic sand, fly ash, silica fume, slag, and rice hull ash provide reactive silica (SiO_2), for the pozzolanic reaction.

As cement chemistry knowledge grew, the chemical equations became familiar and abbreviations were used to represent complex chemical formulas. Silicates (SiO_2, SiO_4, SiO_5, etc.) were collected with the term "S." Hydrates, or chemical multiples of water (H_2O) were referred to as "H," calcium hydroxide was abbreviated "CH," and calcium-silica-hydrate (e.g., $3CaO{\bullet}2SiO_2{\bullet}8H_2O$) was abbreviated CSH. The abbreviated reaction for pozzolanics composed of silica became:

$$2S + CH + H \Rightarrow CSH + Heat$$

Clay based pozzolans such as metakaolin provide silicates as well as alumina (Al_2O_3) based materials. Alumina is symbolized in the following chemical equations as "A." Metakaolin (Al_2O_3 (SiO_2)), a calcined clay material, would be symbolized as "AS_2." The resulting aluminous cement product calcium aluminate was termed "C_3A."

$$AS_2 + CH + H \Rightarrow CSH + C_3A + Heat$$

Ground granulated blast furnace slag (GGBFS), a byproduct of iron smelting, contains silicates, alumina and quicklime (CaO). Like lime, GGBFS reacts directly with water to produce $CA(OH)_2$.

$$AS + H \Rightarrow CSH + C_3A + Heat$$

Portland Cement

Portland cement is hydraulic cement produced by pulverizing clinker, which consists of calcium silicates formed from limestone ($CaCO_3$), silica (SiO_2), alumina (Al_2O_3), and ferrous oxide (Fe_2O_3). ASTM C150 is the standard specification for Portland cement. Portland cement also may include water, 5% limestone, and processing additives like oxides of magnesium or iron.

The four major calcium-based compounds present in Portland cement and signifi ant characteristics are listed in Table 2-1.

Table 2-1. Portland Cement Calcium Compounds.

Compound and Abbreviation	Chemical Formula	Typical Percent	Characteristics
Dicalcium silicate (C_2S)	$2CaO{\bullet}SiO_2$	10-40%	Low early strength, lower hydration heat (60 cal/g), excellent ultimate strength.
Tricalcium silicate (C_3S)	$3CaO{\bullet}SiO_2$	35-65%	Good early strength, medium hydration heat (120 cal/g), good ultimate strength.
Tricalcium aluminate (C_3A)	$3CaO{\bullet}Al_2O_3$	0-15%	Good early strength, high hydration heat (320 cal/g), good ultimate strength.
Tetracalcium aluminoferrite (C_4AF)	$4CaO{\bullet}Al_2O_3{\bullet}Fe_2O_3$	5-15%	Good early strength, medium hydration heat (100 cal/g), good ultimate strength.
Note: 100 cal/g = 418.5 joule/g			

Depending on proportions of starting compounds (shown in chemical notation), the resulting clinker (shown in cement chemistry notation) would be:

$$CaCO_3 + SiO_2 + Al_2O_3 + Fe_2O_3 + \text{Heat} \Rightarrow C_2S + C_3S + C_3A + C_4AF + CO_2$$

The proportions of the basic compounds control the ratios of the various types of Portland cement in the final product. Heat of hydration is a function of reaction to water; a faster reaction generates more heat. Cement with a high early strength requirement would have larger percentages of C_3S and C_3A. Cement with a requirement for low heat production would have more C_2S. As cement becomes finer, the relative surface area increases; this increases water demand, reaction to water, heat of hydration and early strength. Substituting equal amounts of SCMs like fly ash for Portland cement generally reduces hydration heat 15–30%, however ultrafine SCMs do not significantly reduce the heat of hydration.

Portland Cement Types

Portland cements used on Caltrans projects must comply with ASTM C 150, which identifies five cement types. The type and major characteristics of each type are listed in Table 2-2.

Table 2-2. Portland Cements.

Cement Type	Common Name	Cement Characteristics
I	Ordinary Portland cement	General purpose cement, suitable for all uses when the special properties of the other types are not required.
II	Modified or Moderate sulfate resistance	Greater resistance to sulfate attack, $C_3A < 8\%$. Slower strength development. To moderate hydration heat, $C_3S + C_3A < 58\%$.
III	High early strength	Similar to Type I, but more finely ground, hydrates faster, hardens faster. $C_3A < 15\%$. Strength gains compared to Type I: 180% at 3 days, 120% at 90 days, 100% at 360 days.
IV	Low heat of hydration	Formulated for use in mass concrete where the heat generated must be minimized. Hydration and hardening are at a slower rate than Type I cement. $C_2S > 40\%$, $C_3S < 35\%$, $C_3A < 7\%$.
V	Sulfate resistant	Formulated for use when severe sulfate attack from groundwater or soil is anticipated. $C_3A <$ 5% for higher sulfate resistance. Hydration and hardening are at a slower rate than Type I cement.
Note: ASTM C150 also recognizes entrained air as a specifiable concrete; this is noted by the addition of the suffix "A" to Types I, II, and III cements.		

Blended Portland Cements

Within the five general type classifications, there are a number of blending options; these options are summarized in Table 2-3. Selected materials are added during the manufacturing process to obtain desired properties or characteristics. Blended cements are designated by a code that includes the primary cement type followed by a descriptive term or letter. Binary blended hydraulic cement is a combination of Portland cement and a supplementary cementitious material that is either slag (S) or a pozzolan (P). The labeling convention for blended cements is in the form Type I (P%) or Type I (S%), where the cement type is listed first, the SCM (P for pozzolanic and S for slag) second with the percentage of the SCM, and then special properties are listed. For example, "Type I (P15)-A" designates a cement containing a blend of Type I cement, with 15% pozzolanic material and air-entrainment admixture added. Type I (S50) MH designates a Type I cement with 50% slag that produces a moderate heat of hydration level. Type IIT (S15) (P20) is Type II cement ternary mix with 15% slag and 20% pozzolan.

Table 2-3. Secondary Designations for Blended Cements.

Label	Description
X (%)	Cementitious Materials where X = P, S, or T. (P for pozzolanic, S for Slag, T for Ternary, is a mix with two SCMs, and (%) = percentage of SCM use).
A	Air-entraining admixture added to improve workability and durability. Batch plants typically prefer to maintain control over air entrainment by adding admixtures as part of the batch process rather than using premixed cements.
S	Sulfate Resistance (where MS = moderate sulfate resistance, HS = high sulfate resistance).
H	Heat of hydration (where MH = moderate heat, LH = low heat).
M	Modified with expansive admixture that causes a volume increase in concrete after initial set; this compensates for volume reduction associated with shrinkage.

Portland Cement Manufacturing Process

The cement manufacturing process consists of three following basic steps:

1. Initial crushing and proportion of raw materials.
2. Slow burning the raw materials into clinker.
3. Final mixing and fine grinding clinker into Portland cement powder.

Initial crushing sequentially reduces limestone and other materials from 3 inches to ¾-inch diameter particles. After initial crushing, materials are proportioned, mixed and coarse ground in a rotating mill containing thousands of steel balls that pulverize the materials. Grinding may be performed with dry materials, or water may be added during the process.

The homogeneous fine amalgam is then conveyed into a sloped rotating cylindrical kiln (oven), typically 10-12 feet in diameter and 500 feet long, for calcination. Cement kilns are steel plated, lined with fire brick and rotate at approximately one revolution per minute. Oil, gas or powdered coal is burned at the lower end of the kiln, heating the materials to about 2,700^{o}F. Carbon dioxide is driven off as the materials pass through the kiln. Calcium silicates, aluminates and aluminoferrites are amalgamated into clinkers about ¾-inch diameter during the 4-hour calcination process. After the clinker has cooled, calcium sulfate (gypsum) may be added and the mixture is ground into a fine powder: Portland cement.

Particle Size and Appearance

Finished Portland cement is a white or grayish-white substance resembling flour in texture and appearance. Most cement will pass a sieve with 40,000 openings per inch. Blaine Fineness is a measure of surface area per unit weight; for ordinary Portland cement, the Blaine fineness is typically 300 to 500 square meters per kilogram. Smaller-size particles hydrate faster than larger-sized particles, but require more water to maintain workability.

Specific Gravity

The specific gravity of Portland cement varies depending on the source of the raw materials and the manufacturing processes. Typically the specific gravity will vary from 3.05 to 3.20 with an average of 3.15. Specific gravity does not influence concrete quality and is used to determine concrete unit weights.

Hot Cement

Final grinding is a heat intensive process, cement temperatures can approach 200^{o}F. The normal time interval (usually several weeks) between manufacture and work site delivery allows cement to cool to ambient temperature. However, if cement is shipped too soon after manufacture, it may arrive at the job site hotter than ambient temperature; thus the term "hot cement." Hot cement can cause problems in fresh concrete. The increased temperatures can reduce set time and workability, particularly on warmer days.

Supplementary Cementitious Materials

Supplementary Cementitious Materials (SCMs) are natural or man-made pozzolanic materials that are added to concrete to reduce the demand for Portland cement, improve workability and durability. ASTM C125 defines a pozzolan as either a siliceous or siliceous and aluminous material, which in itself possesses little or no cementitious value. SCMs will react with calcium hydroxide (CH) in the presence of water to form calcium silica hydrates (CSH). The resulting concrete is stronger, less permeable, and more durable but will require more time for strength gain.

Natural pozzolans include some clays and shales, opaline cherts, diatomites, and glassy materials of volcanic origin like tuffs and pumices. Clays, shales and cherts require calcination to become pozzolanic. Metakaolin, one of the only natural pozzolans approved for Caltrans use, is calcined kaolin clay. Diatomites are natural deposing of pozzolanic material; however they are almost always mixed with clay and would need calcination to utilize the pozzolanic effect. Volcanic glasses are inherently pozzolanic, calcination rarely improves their pozzolanic activity.

Man-made pozzolans include fly ash, ultra-fine fly ash, silica fume, and rice hull ash, are byproducts of combustion required for power production. Ground granulated blast furnace slag (GGBFS) is the byproduct of iron smelting. Because of the very small particle size, ultra fine fly ash, rice hull ash, and silica fume are high reactivity pozzolans

SCMs that are approved for use on Caltrans projects are listed on the Caltrans METS Authorized Materials List (AML) (http://www.dot.ca.gov/hq/esc/approved_products_list/) under "Cementitious Materials for Use in Concrete". The AML varies over time because products are added and deleted as warranted. A good practice is checking the AML for a material's current status prior to approving the material for use. SCM technical and specifications references are summarized in Table 2-4. The effects of SCM usage on quality characteristics are summarized in Table 2-5.

Table 2-4. Supplementary Cementitious Materials.

	Color	Specific Gravity	Usage	Specifications
Fly Ash Ultra fine Fly Ash	Grey or Tan	1.90 - 2.80	8 - 25%	ASTM C618 AASHTO M 295
Metakaolin	Off-white - Grey	2.50 - 2.60	5 - 10%	ASTM C618 AASHTO M 295
Ground Granulated Blast Furnace Slag	Off-white	2.85 - 2.95	25 - 60%	ASTM C989 AASHTO M 302
Silica Fume	Light or Dark Grey	2.20 - 2.60	3 - 10%	ASTM C1240 AASHTO M 307
Rice Hull Ash	Grey-Black	2.05 - 2.20	5 - 15%	AASHTO M 321
*Note: 50% where maximum strength is required. Higher percentages (>60%) may be used in mass concrete or for ASR mitigation.				

Table 2-5. Summary of SCM Characteristics.

Characteristic	GGBFS	Fly Ash	Metakaolin	Silica Fume	Rice Hull Ash
Water Demand	▼	▼▼	▲	▲▲	▼
Workability	▲	▲	▼	▼▼	▼
Initial Set	▼	▼▲	◀▶	◀▶	▼
Final Set	◀▶	◀▶	◀▶	◀▶	▼
Heat of Hydration	▼	▼	◀▶	◀▶	◀▶
Dry Shrinkage	Unknown	◀▶	◀▶	▲▲	Unknown
Plastic Shrinkage	◀▶	◀▶	◀▶	▲	◀▶
Early Strength	▼	▼	▲▲	▲▲	▲
Long Term Strength	▲	▲	▲▲	▲▲	▲
Permeability	▼	▼	▼▼	▼▼	▼
Sulfate Resistance	▲▲	▲▲	▲	▲	▲
Chloride Resistance	▲	▲	▲▲	▲▲	▲
ASR Mitigation	▲▲	▲▲	▲	▲	▲

Legend

▲ - Increased / Improved

▲▲ - Significantly Increased / Improved

▼ - Reduced / Decreased

▼▼ - Significantly Reduced / Decreased

▼▲ - Effect Varies

◀▶ - No Change

Fly Ash

Fly ash and ultra fine fly ash are pozzolanic materials and are normally less expensive than Portland cement as they are byproducts of coal combustion for power generation. Ultra fine fly ash is chemically identical to fly ash. The physical difference is in size, ultra fine fly ash median size is about 6 µm, and fly ash is about three times larger at 19 µm.

Fly ash must comply with AASHTO M 295. Class F fly ash, produced from anthracite (bituminous coal) contains a minimum of 70% combined silica and alumina. Class C fly ash contains a minimum of 50% combined silica and alumina. Class F fly ash may have up to 12% CaO. Because Class C fly ash could have up to 30% CaO, it acts as a lime cement, combining with H_2O and CO_2 to form $CaCO_3$ (limestone).

Freshly mixed concretes that use Class F fly ash exhibit the following:

- Longer set time
- Improved workability
- Lower heat of hydration
- Lower water demand
- Reduced bleed water
- Reduced segregation

Hardened concretes using Class F fly ash have:

- Reduced permeability (improving chloride and sulfate corrosion resistance)
- Improved abrasion resistance
- Lower early strengths (< 28 days) with higher long term strength (> 28 days)
- Increased ASR resistance
- No change in early age shrinkage cracking of concrete

Typical chemical composition of fly ash

- Silica: 52%
- Alumina: 23%
- Iron Oxide: 11%
- Calcium Oxide: 5%
- Sulfate: 0.8%
- Sodium Oxide: 1%
- Potassium Oxide: 2%

Fly ash is beneficial in:

- Freeze-thaw areas because of its low permeability (resistance to chloride penetration) and abrasion resistance
- High temperature zones because of its longer setting time
- Marine or sulfate rich soil environments because of its sulfate resistance
- Long life structures (pavements, bridges and buildings)
- Mass concrete pours because of lower heat of hydration
- Concrete where potential for ASR is high

Metakaolin

Metakaolin, a natural pozzolan, is derived from the calcination of high-purity kaolin clay. The product is then ground to between 1 - 2 µm (about 10 times finer than cement), but since it is manufactured and not waste from another process, it is significantly more expensive than other pozzolans. Metakaolin particles fill voids between other concrete materials and result in a denser mix. Metakaolin must comply with AASHTO M 295 as a Class N (natural) pozzolan. Metakaolin contains a minimum of 70% combined silica and alumina.

Freshly mixed concretes using metakaolin have:

- Increased water demand
- Significantly reduced workability
- Decreased heat of hydration
- No change in set time

Hardened concretes using metakaolin have:

- Significantly increased early and long-term strength
- Increased flexural strength
- Reduced permeability (improving chloride and sulfate corrosion resistance)
- Increased abrasion resistance
- Increased ASR resistance
- No change in plastic shrinking cracking

Typical chemical composition of metakaolin:

- Silica: 53%
- Alumina: 43%
- Iron Oxide: 0.5%
- Calcium Oxide: 0.1%

- Sulfate: 0.1%
- Sodium Oxide: 0.05%
- Potassium Oxide: 0.4%

Metakaolin is beneficial in:

- Freeze-thaw areas because of its low permeability (resistance to chloride penetration) and abrasion resistance
- Low temperature zones because of its shorter setting time
- Chemically aggressive environments because of lower permeability
- Marine or sulfate rich soil environments because of its sulfate resistance
- Situation where early strength gain and high strength are required
- Situations where visual appeal because of lighter color is important

Ground Granulated Blast Furnace Slag

Ground Granulated Blast Furnace Slag (GGBFS) or slag cement, is a glassy, granular material, created as a byproduct of iron smelting, that has been ground to suitable fineness to have cementitious properties. GGBFS must comply with AASHTO M 302 grade 100 or 120. The grade is the slag activity index. Concrete mixes that include grade 120 GGBFS usually exceed comparable amounts of Portland cement concrete strength after 7 days. Concrete mixes that include grade 100 GGBFS usually exceed comparable amounts of Portland cement concrete strength after 28 days. Portland cement usually exceeds Grade 80 GGBFS in strength, regardless of age. Grade 80 produces less heat during hydration and may be specified in situations where strength is less important than hydration heat, as in mass concrete. Concrete with GGBFS has an initial green hue that fades to grey after a few days exposure to air.

Freshly mixed concrete using GGBFS may have:

- Longer initial set time
- Lower water demand
- Improved workability
- Lower heat of hydration
- No change in plastic shrinkage cracking

Hardened concretes using GGBFS have:

- Reduced permeability (improving chloride and sulfate corrosion resistance)
- Lower early strengths (1-7 days) with higher long-term strength (7+ days)
- Increased ASR resistance

Typical chemical composition of GGBFS:

- Silica: 35%
- Alumina: 12%
- Iron Oxide: 1%
- Calcium Oxide: 40%
- Sulfate: 9%
- Sodium Oxide: 0.3%
- Potassium Oxide: 0.4%

GGBFS should be used in:

- Freeze-thaw areas because of low permeability
- High ambient temperature zones because of longer setting time
- Marine or sulfate rich soil environments because of sulfate resistance
- Long life structures (pavements, bridges, and buildings) since higher long term strength
- Mass concrete pours because of lower heat of hydration
- With fly ash and silica fume in ternary concrete blends

Silica Fume

Silica Fume, also know as fume silica or micro silica, is an extremely fine pozzolanic material, typically 100 times finer than Portland cement, but is more expensive than Portland cement. Silica fume is a byproduct of the silicon and ferrosilicon industries and produced through the electric arc furnaces. Silica fume must comply with AASHTO M 307.

Freshly mixed concretes using silica fume have:

- Significantly increased water demand
- Significantly decreased workability
- Less bleed water

Hardened concretes using silica fume have:

- Significantly increased early and long-term strengths
- Reduced permeability (improving chloride and sulfate corrosion resistance)
- Significantly increased chloride resistance
- Better sulfate resistance
- Increased ASR resistance

Typical chemical composition of silica fume:

- Silica: 90%
- Alumina: 0.4%
- Iron Oxide: 0.4%
- Calcium Oxide: 1.6%
- Sulfate: 0.4%
- Sodium Oxide: 0.5%
- Potassium Oxide: 2.2%

Silica Fume should be used in:

- Freeze-thaw areas because of its low permeability (resistance to chloride penetration) and abrasion resistance
- Chloride or sulfate rich environments because concrete permeability is reduced
- Long life structures (pavements, bridges and buildings) because of increased strength and durability
- With fly ash or GGBFS in ternary mixes to improve durability

Rice Hull Ash

Rice Hull Ash (RHA) is a pozzolanic material formed as a byproduct of the rice milling process. RHA exhibits several similar properties to that of silica fume as it is comparable in chemical composition. Although RHA is an economical substitute for Portland cement, it is only approved for minor concrete uses.

Freshly mixed concrete using RHA have:

- Longer initial set time
- Comparable final set time
- Lower water requirement
- Decreased workability
- No change in heat of hydration

Hardened concretes using RHA have:

- Increased early and long-term strengths
- Reduced permeability (increased chloride and sulfate resistance)
- Increased abrasion resistance
- Increased ASR resistance

Typical chemical composition of RHA:

- Silica 90%
- Alumina -
- Iron Oxide 0.21%
- Calcium Oxide 0.22%
- Sodium Oxide 0.01%
- Potassium Oxide 0.04%

Specialty Cementitious Materials

Unique application requirements such as architecturally specified color and concrete repairs can be met with specialty cements. White cement can be blended with standard Portland cement for a variety of colors between white and grey. Fast-setting concretes which gain acceptable strength in a matter of hours can be used for structure concrete repair.

Type I, Type II or Type III (White) Portland Cement

White Portland cement is used to produce a white finished concrete primarily used for architectural concrete. Small amounts of white cement may be added to adjust the color of a mortar patch used to repair a section of concrete. The white color is obtained by keeping iron and magnesium oxides to less than 0.5% of the raw materials, and by modifying the manufacturing process. The primary use of iron oxide in regular cement function is as flux in the calcining process. When iron oxide is eliminated, a higher calcination temperature (about 500^{O}F higher) is required. White cement is also ground more finely than Portland cement.

Magnesium Phosphate Cements

Magnesium phosphate cement (MPC) is produced by calcining mined magnesia, forming a substance consisting of magnesium and calcium oxides. Hardening is initiated by adding phosphoric acid, which yields magnesium phosphate and calcium phosphate.

All MPCs are prepackaged products. Two generic types are available commercially:

- Two-component products, which consist of a dry granular material and a liquid solution of phosphoric acid
- Single-component products, which contain phosphoric acid in dry powder form and are activated by adding water

For both types, the recommended mix proportions must be followed carefully to ensure satisfactory results. MPCs may be extended up to 60% with pea gravel aggregate filler.

Surface preparation should receive careful attention. Most manufacturers recommend sandblasting to obtain the clean, sound surface that is necessary for adequate bond. MPC bonds well to dry Portland cement concrete. Epoxy has been used to enhance the bond, but its use is not considered necessary. Provided the hardened surface is dry, MPC will bond to previously placed MPC and thus is well suited to stage construction. MPC will not bond to damp or wet concrete. The moisture intolerance may continue for as long as a week after placement, and has resulted in bond failures where the patch material was placed on a force dried (by heating or other means) surface. Debonding occurred because moisture equilibrium in the substrate, when restored, contaminated the magnesium phosphate material at the bond line interface.

NOTE
Magnesium phosphate will react with aluminum, so aluminum tools or mixing bowls should not be used.

MPCs are self-leveling; consequently, they are not suitable for use on steep slopes or vertical surfaces. Water cannot be added to improve the workability of the two-component products, which may be a disadvantage under some circumstances. For the single-component products, following manufacturer's instructions and use only enough water to make the material workable. Water should not be added to aid surface finishing as this will result in high water content at the surface, reducing strength and lowering abrasion resistance.

Set times are typically 15 to 30 minutes, and full strength is achieved within 45 minutes to 1 hour. The normal set time may be retarded by the addition of borax or other buffers used in accordance with the manufacturer's recommendations. (Tests indicate that 1 ounce of borax per 50-pound bag of prepackaged mix will retard the set time 5 minutes at 72^{O}F.) Curing is not required; curing compound prevents the escape of gases generated by the chemical reaction, which inhibits the hardening process. Lab tests indicate that reinforcing steel does not corrode as fast in magnesium phosphate concrete as in normal Portland cement concrete. Accordingly, MPCs are suitable for bonding dowels as well as patching bridge decks.

Calcium Aluminate Cements

High aluminate cements (HAC) are produced by fusing limestone and bauxite in an electric or blast furnace to produce a clinker, which is then cooled and ground. Like Portland cement, the high alumina cements gain strength by hydration; however, the water-cement ratio is critical, so it is essential to use the mix proportions recommended by the manufacturer.

As the name implies, HACs have a high concentration of aluminates. The calcium to aluminate ratio is typically 40:60 as compared to the approximately 50:50 ratio for normal Portland cements.

Unmodified HAC has a significant strength loss at high temperatures (over about 135^{O}F when moisture is present). Under these conditions, the hardened concrete will undergo a crystalline conversion that changes the internal structure of the concrete and results in a strength loss of up to 70%; calcium sulfate is added to counteract this.

HACs are usually single-component, and water-activated packages; the strength gain occurs rapidly after the initial set period of 15 to 30 minutes, depending on the temperature. Typical compressive strengths exceed 3,000 psi in 3 hours and 5,000 psi in 24 hours. Lab tests indicate that the set time will be retarded by the addition of borax, and accelerated by the addition of lithium carbonate. They may be extended up to 100% with pea gravel filler.

Placing and finishing characteristics are similar to normal Portland cement concrete. HACs are generally self-leveling, but can be modified for low-flow application. Although lab tests indicate that high alumina cements will bond to a damp substrate, bond failures have been observed in the field when placed under wet weather conditions. Accordingly, a dry substrate is recommended.

Initial shrinkage is low. Curing is unnecessary for most applications; however, a curing seal may prevent surface cracking when ambient conditions are hot, dry or windy. After a surface skin has formed, concrete made with modified high alumina cement will not bond to itself. To prevent delamination between lifts, patches must be placed in a single lift. If multiple lifts are unavoidable, abrasive blast cleaning of the surface between lifts will be necessary.

Aggregate

Aggregates typically comprise 60 to 75% of concrete volume. Once thought of as inert filler materials, aggregate affects water-cement ratio and contributes to concrete strength. Porous, fractured, and chemically active aggregates negatively affect concrete durability. Suitable aggregate requires several quality control checks prior to use.

Typical aggregate sources are:

- Natural gravels and sands: excavated, washed, and screened for gradation requirements
- Natural rock: excavated, crushed, washed, and screened to meet aggregate gradation requirements

Other aggregates such as marine dredged aggregate or recycled concrete aggregate are not typically used for structural concrete. Marine aggregates may contain shells and deleterious seawater salts. Recycled concrete aggregate is used primarily for pavement. Good quality aggregate will consist of clean, hard particles having sufficient strength and durability to resist deterioration under normal conditions of exposure and wear. Coarse aggregate must be able to withstand abrasion and surface wear caused by vehicular traffic and environmental factors such as flowing water and freezing weather.

Contamination

Aggregate must be free from contamination, because clay and organic matter reduce the bond between aggregate and paste. As contaminated aggregates have a greater surface area, the ratio of cement paste to aggregate surface area is reduced, resulting in a leaner mix and weaker concrete than anticipated. Contaminants such as clay and organic matter are lighter than aggregates and tend to rise as concrete consolidates, weakening the surface strength and reducing durability.

Moisture

The four moisture conditions are:

- Oven dry
- Air dry
- Saturated Surface Dry (SSD)
- Wet

Of the four moisture conditions, saturated surface dry (SSD) is the standard for mix design moisture assumptions. Oven dry aggregate, has been baked dry; there is no water on particle surfaces or within pore spaces. Air dry aggregate is surface dry, but carries some pore water. The pores of SSD aggregates are filled with water, but no water is adsorbed to the exterior surface. SSD is the condition where aggregate has the least effect on the water-cement ratio. Wet aggregate pores are filled with water, and water is adsorbed on the exterior surface.

Porous aggregate maintains a water balance with the local environment. If surface moisture is present or humidity is high, moisture will be absorbed. When humidity is decreasing, moisture is released to the atmosphere. During severe weather conditions, absorbed moisture may freeze and expand, causing aggregate deterioration and defective concrete.

Shape and Texture

Aggregate shape and texture affect durability, strength, and water demand. Shape is categorized by sphericity and roundness. Sphericity measures compactness of shape (surface area divided by volume), where a sphere or a cube would be more compact than a plate. Roundness refers to angularity at the particle edges, rounded edges signify better workability. The surface texture can range from glassy to rough. Smooth aggregate requires less mixing water, is more workable but has less area for bonding with cement paste.

Gradation

Over the years a great deal of study and research has been devoted to grading theory and much has been learned about the influence of aggregate gradation on the properties of a concrete mixture. Thus far, however, no one has discovered a simple, reliable method of determining the "ideal" grading for a given aggregate; consequently, most grading curves have been developed empirically and are based on experience rather than theory.[1] The blend of aggregate particles in each specified sieve size must meet the specified particle size distribution. When the gradation falls out of tolerance, the variation from specification will have negative effects on workability, ultimate strength, density, and shrinkage characteristics.

Alkali-Silica Reactivity

Alkali-silica reactivity (ASR) is a two-step process with three required ingredients.

- Reactive aggregate is the first required ingredient. Reactive aggregates are typically cherts, like opal or obsidian, made of hydrous or amorphous silica (SiO_2), siliceous limestones and dolomites, intensely strained quartzes and quartzite. The key characteristic is the availability of silica. Aggregate taken from granite, gneiss, schist, sandstone and basalt are more commonly tested and found to be innocuous as the silica is locked into larger crystal formations.
- Alkali-metal ions, Sodium (Na) and Potassium (K), are limited by Caltrans specifications to a maximum of 0.6% of the weight of Portland cement. Other sources of alkali-metal ions can be admixtures or salt contamination (seawater, deicing or wind-blown materials).

[1]The term "ideal" grading means the particle size distribution that will produce maximum workability for the specified cementitious content, provided the particular distribution can be obtained economically and will not produce a concrete mixture that is incompatible with the purpose intended.

- Moisture is the expansive component. If moisture could be excluded, ASR would stop before the expansive phase starts.

In the first part of the reaction, one or more alkali atoms, Sodium (Na) and Potassium (K), in combination with a hydroxide (OH) molecule react with silica (SiO_2) to create alkali-silica gel.

$$Na(K)OH + SiO_2 \Rightarrow Na(K)SiO + H_2O$$

In the second part of the reaction, moisture is absorbed, making an alkali-silica hydrate, which is an expansive gel. (See Figure 2-2).

$$Na(K)SiO + H_2O \Rightarrow Na(K)SiO_2 \cdot H_2O$$

As the alkali-silica gel absorbs water and expands, pop-outs may occur at the surface and in severe cases, extensive map cracking can occur.

(a)
Concrete pore solutions is dominated by Na, K & OH (with minor amounts of Ca).

If the silica in the aggregate is reactive - the OH and then the Na & K will react with the SiO_2.

(b)
The product of the reaction is an alkali-silica gel composed of Na, K, Ca & Si.

The gel forms around and within the aggregate.

(c)
The gel imbibes water from the surrounding cement paste.

The gel expands.

Eventually the swelling pressures may exceed the tensile strength of the surrounding paste allowing expansion and cracking of the concrete.

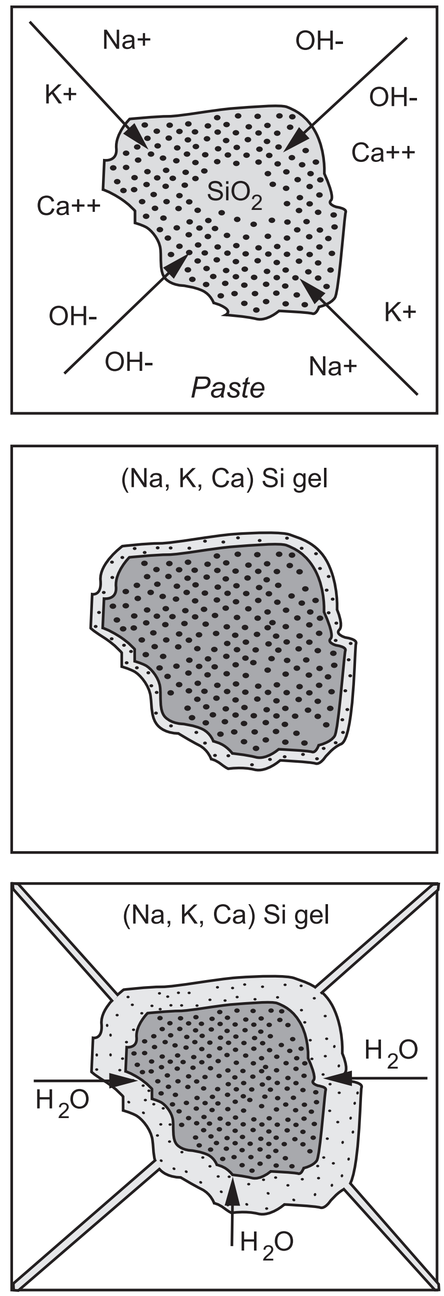

Figure 2-2. Alkali-Silica Reaction[2].

[2] The Use of Lithium to Prevent or Mitigate Alkali-Silica Reaction in Concrete Pavements and Structures, FHWA-HRT-06-133, March 2007

ASR is controlled by:

- Use of innocuous aggregate, removing free silica stops the reaction before it occurs.
- Limiting alkali metal content in cement, similar to aggregate, removing reactants limits the reaction. However, sodium from road de-icers or seawater may still find its way into the aggregate.
- Adding SCMs to the concrete mix reduces the alkali-metal ion content. If say 30% fly ash were used as cementitious, there would then be a 30% reduction in alkali-metal content. Instead of 0.6% maximum limit, the content would be 0.6 – (0.3 x 0.6) or a maximum of 0.42%.
- Adding lithium nitrate as an admixture provides a new alkali-metal to the mix. Lithium is lighter than the other alkali-metals and responds quicker to forces attracting alkali-metals and silica. When lithium reacts with silica the gel forms, but since the gel does not absorb water, it is not expansive; which defeats the ASR reaction.

Aggregate testing to verify non-reactive materials is performed under ASTM C1260 or ASTM C1293. Aggregate that passes the ASTM testing is certified as innocuous. All aggregates currently accepted by Caltrans as innocuous are on the Caltrans Authorized Materials List under Aggregates for Concrete http://www.dot.ca.gov/hq/esc/approved_products_list.

Low-alkali cements are specified under Standard Specifications Section 90. SCMs control ASR by combining with alkali before it combines with aggregates. The SCMs need be in sufficient amounts to remove available alkali from the mix. The amount needed depends on the SCM. Although research has shown silica fume and Metakaolin may be effective at 10%, current specifications set the minimum amount at 12%. Class F fly ash with less than 10% CaO is effective at 25%. Class F fly ash with more than 10% CaO require more than 25% fly ash. An SCM like ground granulated blast furnace slag may require up to 50% slag in the mix. The equations listed in Standard Specifications Section 90 must be met to ensure adequate use of SCMs.

Lithium nitrate may also be combined with 15% Class F fly ash. Lithium is chemically more reactive than Sodium or Potassium in single replacement reactions, so lithium will combine more readily with silica. Individual molecular components vary with availability, the reactants are lithium nitrate and silicon dioxide and the products are lithium silicate and a nitrate. The assumption is that other alkali will combine with the nitrate (sodium nitrate or potassium nitrate).

$$LiNO_3 + SiO_2 \Rightarrow LiSiO_2 + NO_3$$

Water

As a general rule, any water suitable for drinking may be used for concrete production and curing. In the future, if wastewater treatment continues to advance, recycled water may become a common part of a concrete mix. Currently, water added to concrete may come from:

- Batch plants adding water from municipal or private supplies
- Batch plants adding reclaimed water (from municipal and batch plant operations)
- Ice added for additional cooling on hot days
- Transport trucks added on the job site
- Moisture adsorbed by aggregates

The water quality requirements for washing aggregate, mixing, and curing concrete are found in Section 90-1.02D of the Standard Specifications. Although non-potable water may be used, there are limitations on impurities:

- No oil
- No substances that significantly change set time, reduce compressive strength, or cause discoloration
- Restricted presence of alkali ($Na_2O + 0.658\ K_2O$) to < 300 parts per million (ppm)
- Restricted presence of chloride (650 ppm) and sulfate (<1,300 ppm) ions for prestressed concrete. Note that the specification limit applies to the chlorine and sulfate ions, not to compounds containing these elements in combined form

The adverse effects of materials dissolved in water are listed in Table 2-6. If there is any doubt regarding water quality, the best course is to test the water.

Table 2-6. Adverse Effect of Dissolved Chemicals on Concrete.

Chemical	Altered Set	Strength Reduction	Other
Alkali Carbonates/Bicarbonates ($NaCO_3$, $NaHCO_3$, KCO_3, $KHCO_3$)	Yes	Lower 28 day strength	Alkali-silica reaction, may affect air entrainment
Chloride	Yes	No	Increased drying shrinkage, steel corrosion, efflorescence
Sulfate	No	No	Expansive sulfate reaction
Seawater	No	Lower 28 day strength	Potential for steel corrosion, alkali-silica reaction, higher potential for efflorescence, may affect air entrainment
Alkali (Sodium (Na) Potassium (K))	No	Lower 28 day strength	Alkali-silica reaction, may affect air entrainment
Sugar	Yes	Lower 28 day strength	N/A
Silts and clays	No	No	Changes in workability
Oil	No	Lower 28 day strength	N/A
Alkali (Sodium (Na) Potassium (K))	No	Lower 28 day strength	Alkali-silica reaction, changes in workability
Acids	No	Tannic acid reduces strength	High strength acids may alter workability
Inorganic Salts (manganese, zinc, copper, sodium, and lead)	Retarded set	Lower 28 day strength	Changes in workability

Concrete Admixtures

Admixtures are added to the basic concrete mix of cementitious materials, aggregate and water to modify workability and durability characteristics of concrete. Some admixtures affect fresh concrete by increasing fluidity and workability, reducing the water demand, or retarding or accelerating the setting time. Other admixtures affect the properties of hardened concrete by increasing its strength and durability. Table 2-7 lists the various admixture types that are accepted on Caltrans projects. When used properly, admixtures will produce the intended results, such as entrained air or set acceleration, in a manner that could not be achieved economically by any other means. While admixtures are widely used to enhance the desirable characteristics of concrete, no admixture is 100% beneficial, as all admixtures can produce undesirable results under certain circumstances. To ensure that the desired results are obtained, it is essential to anticipate the effect of a given admixture on the characteristics of the plastic concrete mixture as well as on the hardened concrete, before that admixture is used in the work. Admixtures should not be used indiscriminately, and they should not be viewed as substitutes for the best general recommended concrete construction practice and procedure.

Table 2-7. Concrete Admixtures.

Type	Admixture Description
AE	Air Entrainment
A	Water Reducer
B	Set retarder
C	Set accelerator
D	Water reducer and set retarder
E	Water reducer and set accelerator
F	High range water reducer (superplasticizer)
G	High range water reducer and set retarder
S	Specific Performance, such as corrosion inhibition, shrinkage reduction, hydration stabilization, and alkali-silica reaction reduction. Note: New Type S admixtures that have not been placed on the AML would require a contract change order prior to approval of their use in a mix design.

Table 2-8 summarizes the advantages and disadvantages of admixture types commonly used for structure concrete work.

Table 2-8. Admixture Characteristics.

Admixture Type	Advantages	Disadvantages
Water-reducing	Water reduction. At constant water content, increases workability and facilitates consolidation. At constant penetration, reduces water content, which increases strength and reduces permeability. At constant water content, lignosulphonic types reduce bleeding.	At constant water content, the hydroxyated types increase bleeding. (May be an avantage under some conditions.) Mixtures with low to modeate slump may be more difficult to finish due to surface stickiness With high range water reducers, an acceleated rate of slump loss may occur, causing a sudden loss of workability.
Set-retarding	Delays initial set time, thereby extending the time available for concrete placing and finishing.	Increases bleeding. Unless dosage is controlled, the delayed set time may result in an unwanted and excessive delay in hardening
Accelerating	Decreases time to initial set. Increases initial rate of strength gain.	Excessive dosage may severely reduce time to initial set, or may produce flash set Increases drying shrinkage. Accelerators with high chloride content may aggravate corrosion of embedded reinforcing steel.
High Range Water Reducer (HRWR)	Water reduction. Reduced cementitious content. Increases workability. Higher early strength and long-term strength. Reduced permeability.	Additional admixture cost. Higher slump. Must coordinate with air entrainment. Varied response to different cements. Mild discoloration or blemishing and presence of air voids on exposed surfaces.

Section 90 of the Standard Specifications covers the use of admixtures on State highway projects. The specifications list three admixture classifications: chemical, air-entraining, and lithium nitrate. Only admixtures that have been previously tested and approved may be used in the work. Admixtures that have been approved for use on State highway projects are shown on a list of approved chemical admixtures for use in concrete issued by Caltrans METS.

When more than one admixture is used, the admixtures must be compatible. That is, the admixtures must not react with each other, and the admixture combination must not produce any detrimental effect on the concrete. Compatibility should not be a concern if the admixtures are products of the same manufacturer. However, if admixtures from different manufacturers are proposed for use, the combination should be discussed with the Caltrans METS organization and approval obtained before use. If there is any uncertainty, the combination should be tested to verify compatibility.

When an air-entraining admixture is used with any other liquid admixture, the air-entraining admixture must be the first admixture dispensed into the concrete mixture. It is essential that this batching sequence be followed; otherwise the effectiveness of the air-entraining admixture may be severely impaired.

Liquid admixtures are furnished in 55-gallon drums. The drums should remain sealed until used to prevent contamination, and stored in accordance with manufacturer's recommendations, which should be shown on the label affixed to the container. For most liquid admixtures, protection from freezing will be the only special precaution indicated.

Some liquid admixtures have a limited shelf life. This should not present a problem at commercial plants where large quantities of concrete are produced and admixtures are used routinely. It may present a problem when admixtures are used infrequently and only in small amounts.

Type AE, Air-Entraining Admixtures

Air-entrained concrete contains literally billions of closely spaced, microscopic-sized air bubbles uniformly distributed throughout the cement paste. Production and distribution of the air bubbles occurs during the mixing cycle by using air-entraining cement (Type IA, IIA or IIIA) or by adding an air-entraining admixture to an otherwise normal concrete mixture. Unlike entrapped air voids, which are about the size of the smaller sand particles, the air bubbles produced by air-entrainment are extremely small, ranging in size from about 1 to 3 thousandths of an inch in diameter. Because they are so small, as many as 500 billion bubbles may be present in a cubic yard of air-entrained concrete.

Entrained air improves workability by increasing the volume of the cement-water paste, helping the plasticity of the mix. The micro-bubbles of air act as ball bearings to reduce internal friction. Entrained air cushions aggregate particles, allowing them to slide past each other more easily. The improvement in workability will be particularly noticeable in lean mixes and in mixes made with angular and/or poorly graded aggregates. Without air-entrainment, the concrete mix relies on the fluidity of paste for workability; in those mixes about twice as much water would be required for a workable mix as would be needed merely for chemical reaction with the cementitious materials. The water-cement ratio is limited because of the adverse effect excessive water has on the cohesiveness of the fluid mixture and on the strength and durability of hardened concrete.

Air-Entrainment Advantages

On State projects, an air-entraining admixture will be specified wherever the estimated annual number of freeze-thaw cycles exceeds 15 and the concrete will attain high moisture content during the winter months. This includes most of Northern and Central California above the 1,000-1,500 foot elevation mark, and the mountainous regions of Southern California as well. Freeze-thaw resistance is a particularly desirable physical property in concrete that may become saturated with water. As water freezes, it expands, producing pressure within the concrete mass. This internal pressure causes stresses that exceed the tensile strength of concrete, and this in turn leads to the surface scaling and delamination, which is characteristic of freeze-thaw damage.

The ability of concrete to resist deterioration that occurs as a consequence of exposure to repeated cycles of freezing and thawing is significantly improved by the use of entrained air. In air-entrained concrete, the air bubbles produce a system of spherical air voids that serve as reservoirs to accommodate the expansion of free water within the concrete mass. As the water freezes, the expansive pressure is relieved as the excess water volume is taken up by the air voids, thus preventing damage to the concrete. The phenomenon is similar to leaving a gap of air at the top of a water bottle when you freeze it, so that the bottle will not burst.

There are additional benefits, which include improved workability, reduced bleeding and segregation of fresh concrete, and lower permeability of the hardened concrete which leads to improved resistance to damage from deicing salt and sulfates.

Air-Entrainment Disadvantages

Adding entrained air without adjusting mix proportions, will result in a slight strength reduction in most cases. The actual strength loss varies with the air content, and depends on cementitious content as well. Minor concrete, may lose a moderate amount of strength. Richer mixes will show a strength loss, acting inversely to increases in cement and air content. For strength mixes, air-entrainment will reduce the 28-day compressive strength (as compared to a similar mix without entrained air) about 5% for each 1% of air added.

If the mix proportions of a normal concrete mixture are adjusted with proper reduction in the amounts of mixing water and fine aggregate, or if the mix is properly designed for air-entrainment, the resulting strength reduction will be small and more than offset by improved workability and increased durability.

When high strength is required, as will be the case for some prestressed concrete construction, the use of air-entrained concrete may make it more difficult to attain the specified strength. Even though air-entrainment will permit a reduction in the amount of mixing water, obtaining the specified strength will generally require an increase in the cement content, and this in turn will increase the water demand. This increased water demand will offset, to some extent at least, the initial water reduction made possible by air-entrainment. When the specified strength of air-entrained concrete exceeds about 4,000 psi, a water-reducing admixture will usually be required as well.

Both the rate at which bleeding occurs and the amount of bleed water reaching the surface are reduced by air-entrainment. The air bubbles in the paste buoy up the aggregate and cement particles; this action reduces their rate of settlement toward the bottom of the concrete mass after placement. Bleeding is retarded because water in the mixture moves less easily through the barrier created by the billions of microscopic-sized air bubbles. When entrained air is present in a concrete mixture, the rate of settlement is reduced; if reduced enough, the bleed water formed will evaporate as soon as it reaches the surface, and eliminate the appearance of bleeding. Because of reduced bleeding, and with other factors remaining the same, air-entrained concrete can be finished sooner than ordinary concrete. Reduced bleeding might be an adverse finishing factor during periods of hot weather.

The Air-Entrainment Process

All concrete contains voids that are the result of air bubbles produced by mechanical agitation of the concrete as it is mixed. Most of these bubbles, which are about the same size as the smaller grains of sand, either dissipate or escape from the mix before the concrete hardens. Some, however, become trapped between the smaller sand particles and cannot escape or dissipate; hence the term “entrapped” air. In most ordinary concrete, the entrapped air voids constitute about 1 to 1.5% of the total volume of the hardened concrete.

The voids in air-entrained concrete are also the result of air bubbles formed during the mixing process. Mixers of the rotating drum type, such as those used on transit mixers, are particularly effective in their ability to produce the myriad of bubbles that is characteristic of air-entrained concrete. Air-entraining admixtures function in two ways to produce air-entrained concrete. First, the surface tension of water is reduced, which enables the shearing action developed as the mixer drum rotates to create smaller bubbles. Second, the hydrophilic (charged) end of the admixture molecule is attracted to the cement grains while the hydrophobic (water repelling) end forms a coating on the air bubbles, encasing the bubbles in a film of air-entraining agent and cement. The film is cohesive and imparts a high degree of stability to the normally fragile air bubbles, which prevents them from coalescing or dissipating.

Most air-entraining admixtures are organic materials which are classified chemically as surfactants[3] or surface-active agents. The most widely used and the oldest, air-entraining substance is neutralized vinsol resin. Vinsol resin is a by-product of the process that removes various solvents and resins from pine wood pulp. In its natural state, vinsol resin is insoluble in water and must be neutralized with sodium hydroxide to form a soluble soap which is the basis of the commercial formulations. Besides vinsol, other commercial air-entraining agents are made from petroleum distillates, animal and vegetable fats and oils, and synthetic materials.

There are a number of air-entraining admixtures on the market today, and several brands are approved for use on State highway projects. All function in the same way, and based on lab tests, no particular brand is clearly superior to the others. However, under field conditions one admixture may produce more consistent results than another, for reasons that may not be evident at the time. Because of this, changing admixture brands may be helpful in any case where results are erratic for no apparent reason.

Dosage

The obvious factor affecting the air content of a given batch of air-entrained concrete is the amount or dosage of air-entraining admixture used in the batch. All admixture manufacturers furnish recommended dosage rates to achieve specified air content, but these rates should be viewed as no more than starting estimates since the actual dosage for a given set of conditions will depend on a number of other factors. The cementitious content, water-cement ratio, fine-coarse aggregate ratio, use of other admixtures, concrete temperature, transit batching, delivery, and construction methods employed will all affect air content to some degree. The effect of each admixture must be considered to obtain the maximum effectiveness of the air-entraining admixture.

As the cement content of a concrete mixture is increased, the proportional amount of air-entraining agent needed per unit of cement increases as well. To maintain a given air content in a concrete mix of normal consistency (ball penetration of about 1 to 2 inches), the required dosage will increase as cementitious materials are added.

Cement particle size is a factor for consideration, with finer grinds requiring a higher dosage than coarser grinds to obtain the same air content. Type III cement may require as much as twice the amount of the same air-entraining admixture as a Type I or Type II cement to maintain the same air content. The use of fly ash will increase the amount of air-entraining

[3] The term “surface-active” designates a substance that is capable of reducing the surface tension of the liquid in which it is dissolved. Such substances have a linear molecular structure in which one end of the molecule is hydrophobic (repels water) and the other end is hydrophilic (has an affinity for water) and is electrically charged.

agent needed when compared to a similar concrete with no fly ash. Ultra-fine SCMs like silica fume require increased dosages for comparable air-entrainment levels. This is due primarily to the greater fineness of the pozzolanic particles.

The amount of alkali present in cement will affect air content to some degree. Low-alkali cements, such as those specified for State highway work, require a 20 to 40% higher dosage than regular cements.

Water Content

The amount of water in the mix has a significant effect on air content. As water is added to a mix, there is more free water available for the generation of air bubbles; consequently, the addition of water in even small amounts will usually result in an increase in the air content of a given concrete mixture with no increase in admixture dosage. Other factors being equal, mixes with lower water-cement ratios will require larger dosages of air-entraining agent than mixes with higher water-cement ratios. When air-entrained concrete is used for cast-in-place prestressed construction, specified concrete high-strength requirements will dictate a low water-cement ratio mix design, and this may make it difficult to obtain the specified air content at normal admixture dosage rates. In such cases, the use of a water reducing admixure will produce a more fluid mixture and make it easier to obtain the specified air content.

Mixes with very low water-cement ratios, such as those used for bridge deck overlays where the w/c ratio may be as low as 0.32 or less, will require a substantial increase in the amount of air-entraining agent, possibly as much as 10 times the normal dosage, to obtain the specified air content. If a high range water reducer is also used, additional air entrainment may be needed.

Aggregate

Fine aggregate contributes to total air content by retention of air bubbles in the sand grain interstices, so that an increase or decrease in fine aggregate will increase or decrease the air content as well. For example, a 1% increase in the fine aggregate (with a corresponding decrease in coarse aggregate) will cause the total air content to increase about 0.1%. Thus a more heavily sanded mix will require a lower air-entrainment dosage to obtain the same air content. An excessive amount of very fine material (material passing the No. 200 sieve) will reduce the total air content. Therefore, when the fine aggregate has a relatively low sand equivalent value, a larger dosage of admixture will be required to produce a given air content. Coarse aggregate with a low cleanness value will have the same effect.

Other Admixtures

In general, less air-entraining agent will be needed to produce a given air content when a water reducing admixture is also used. For lignosulfonate-based materials, the required dosage of the air-entraining admixture may be reduced by as much as 80% of the amount that would be needed if no water reducer were used. For organic-acid types, the expected reduction will be lower, ranging from about 20 to 40%.

The effect of high-range water reducers on air content is not always predictable. Reductions of up to 50% in the amount of air-entraining agent needed have been achieved in some cases; in other cases the dosage requirement was actually increased. Highly fluid mixtures occasionally experience a loss of air content with time, and it has not been possible in all cases to achieve the specified air content even though additional air-entraining agent is added. Other factors that influence air content, such as cement and aggregate, and the concrete temperature, may have a more pronounced effect when high-range water reducing admixtures are used. Because of the uncertainties associated with the use of high-range water reducers, trial batches are recommended to ensure an adequate concrete mix.

Mixing

Mixing action is the most important factor in the production of air-entrained concrete. Because of differences in mixing time and mixing action and with other factors being equal, the amount of air entrained for a given dosage will vary with the type and condition of the mixer, the amount of concrete being mixed, and the mixing rate.

As a general rule, more air will be entrained as the mixing speed is increased, within the range of mixing speeds recommended by the manufacturer. Also, increased air content may occur if the mixer is loaded to less than its rated capacity; however, decreased air content may result from overloading.

Prolonged mixing will cause a significant reduction in the air content of a given mixture. Loss of air is the result of reduced fluidity that occurs over time. Loss of fluidity decreases the ratio of air formation to air escape, resulting in a net air loss. Furthermore, since air-entrainment improves initial workability, a reduction in the air content decreases fluidity independent of any other cause, and this has a compounding effect that accelerates the air loss. (As fluidity is decreased, air formation decreases as well. The net air loss further decreases fluidity, which increases the rate at which air loss occurs. The effect is exacerbated as fluidity is reduced significantly during the latter part of the mixing period.)

In most cases air content can be restored by adding water to increase the fluidity of the mix. However, additional water increases the water-cement ratio and reduces ultimate strength.

Concrete Temperature

The temperature of the concrete affects the air content, with less air being entrained as concrete temperatures increase. The relationship between temperature and air content is roughly linear, but inversely acting; that is, an increase in concrete temperature will produce a commensurate decrease in air content, while a decrease in concrete temperature will increase the amount of air in the mix, other factors remaining equal.

For a typical strength concrete mix, experience has shown that an internal temperature variation of about 20^{O}F will result in a corresponding change in the air content of about 30% for the same admixture dosage. This means that a 20^{O}F rise in temperature will require a 30% increase in the admixture dosage to maintain the same air content. A falling temperature will require a similar decrease in dosage.

With other factors remaining unchanged, the normal air temperature variation expected during the course of a typical concrete pour can produce a wide fluctuation in the air content. In view of this, the effect of temperature variation should be anticipated and admixture dosages adjusted accordingly.

Vibration

Tests have shown that the amount of vibration needed to adequately consolidate a properly proportioned concrete mixture does not result in any significant loss of entrained air. There is some air loss, but this is primarily in the form of large bubbles of entrapped air; there is little loss of intentionally entrained air during normal vibration, which should not exceed about 15 seconds.

Excessive vibration can have a negative effect on air content. Air loss during vibration is also a function of the consistency of the concrete as measured by the ball penetration test. That is, for a given amount (time) of vibration, the air loss will increase as the penetration increases.

Type A--Water Reducing Admixtures

Water reducing admixtures decrease the amount of mixing water needed to produce concrete of a given consistency by at least 5% and more typically 6-10%, thus allowing the use of a lower water-cement ratio for a given workability or increased workability for a given water-cement ratio and fixed cement content. Water reducers cannot advance set time by more than 1 hour or retard set time by more than 1.5 hours. Water-reducing admixtures increase the fluidity of the cement paste by lubricating and disbursing the cement particles and decreasing the surface tension of the water. Generally, water reducers do not act on supplementary cementitious materials.

Based on their chemical composition, most commonly used water reducing admixtures fall into one of three groups:

- The HC group, which contain salts and various modifications of hydroxylated carboxylic acids.
- The LS group, commonly called "lignins", which contain salts and various modifications of ligno-sulfonic acids.
- The PS group, which are composed of polymer compounds.

For most admixture brands, some set retardation will occur at normal dosages, even with those admixtures (ASTM Type A) that are not marketed as retarders. Typically, set retardation will not exceed 1 hour for Type A admixtures. Overdoses, caused by considering SCM as Portland cement can cause the set time to exceed 1 hour.

When used at the manufacturer's recommended dosages, a water reduction of 6 to 10% is typical for all normal range admixtures. Increased water reduction can be achieved with higher dosages, but this may result in excessive retardation.

Typically, the HC and PS water reducers increase the rate at which bleeding occurs, but because the total water content is reduced, the actual amount of bleed water reaching the surface is about the same as would be the case for a similar concrete without the water-reducing admixture. The lignin-based water reducers reduce the bleed rate, and this may cause problems in hot, dry or windy conditions. At maximum dosages, slump and workability loss will occur at a faster rate than similar concrete without an admixture.

The HC and PS admixtures do not entrain air, but since they produce a more workable concrete, less air-entraining agent is required to produce a given air content when these admixtures are used. The lignin based admixtures (LS) normally add about 1 to 2% mixed air to the mix.

Type B – Retarding and Type D – Water Reducing and Retarding Admixtures

The principal use of set retarders is to delay the initial set for a predetermined period of time to compensate for the effect of accelerated setting that normally occurs during hot weather. Retarders may be used in concrete intended for large volume footings, piers and similar locations where a delay in setting is advisable to eliminate cold joints and facilitate bonding of successive lifts. Many set retarding admixtures also function as water reducers, and these are called water-reducing retarders. Mixing water reductions of 5 to 10% are typical for these retarders.

Commercially available retarders and water-reducing retarders are formulated to delay setting from 1 to 3 hours, and the admixture manufacturers will provide dosage recommendations to achieve the delay desired. However, the actual delay for a given dosage is difficult to predict, because the concrete temperature, cementitious properties, and the water-cement ratio will affect the performance of the admixture. The use of trial batches is recommended in any situation where it is necessary to ascertain the delay time in advance with a reasonable degree of precision.

Once the initial set has occurred, retardation has little effect on early strength development, with normal strength being reached within 1 to 2 days following placement. When a retarder having water reducing properties is used, the 28-day strength is often 10 to 20% higher than comparable non-retarded concrete.

Type C – Accelerating and Type E – Water Reducing and Accelerating Admixtures

An accelerating admixture is a substance which, when added to a concrete mixture, will shorten the time to initial set, or increase the rate of hardening and strength development, or both. Until the concrete sets, accelerators have no direct effect on the properties of plastic concrete. Bleeding is usually reduced, but this is attributed to rapid hardening rather than any change in the plastic concrete itself. The most important consideration is the shorter time available to place, consolidate, and finish the concrete when an accelerator is used. Concrete temperature affects the setting time to some degree, and most accelerators are more effective, i.e., the rate of acceleration is increased, at the lower end of the allowable concrete temperature range.

The most widely used accelerating admixture is calcium chloride, which is a soluble inorganic salt. Calcium chloride shortens the setting time by accelerating the hydration of the tricalcium and dicalcium silicate compounds in the cement. Calcium chloride increases the early strength of concrete as well; typically, about 75% of the expected 28-day strength is reached within 3 to 5 days.

Calcium chloride is available commercially as a liquid solution or in the form of dry flakes. Under the current specifications, neither form may be used as the chloride ion induces reinforcement corrosion. However, from a quality control standpoint, the use of liquid-calcium chloride is preferred since it is added with the mixing water and uniformly dispersed throughout the mix. When added in dry form, there is less assurance that all particles are dissolved prior to discharge, particularly when the mixing period is relatively short. If flakes must be used, consideration may be given to dissolving the flakes in a measured amount of water and adding the resulting solution to the mixing water prior to batching. Because of its tendency to promote corrosion of embedded steel, most authorities no longer recommend the use of calcium chloride for prestressed construction or in any reinforced concrete structure in a moist environment. Note that under current policy, the use of calcium chloride, or any other accelerator containing chlorides, is not permitted in structures on State highway construction projects under any circumstances.

Moderate acceleration of early strength gain may be achieved by using a calcium-nitrite corrosion inhibitor. While not marketed as an accelerator, calcium nitrite will promote both accelerated set and early strength. Typically, the expected 28-day strength will be attained in about 7 to 10 days. If desired, a retarder can be used to offset the adverse effects of rapid setting, with no adverse effect on early strength gain.[4]

Type F – High Range Water Reducers and Type G – High-Range Water Reducer and Set Retarder

The principle difference between the high-range water reducers (HRWR) and high range water reducer and set retarders is the ability of the high-range water reducers to produce a highly fluid, flowing concrete at normal water-cement ratios or a workable concrete at greatly reduced water-cement ratios. When high-range water reducers are used, the admixture is measured and dispensed in accordance with the manufacturer's recommendations. Note that this may include adding all or a portion of the admixture at the job site.

HRWRs are relatively new admixtures, having first appeared on the market in the mid-1970s. Because of their ability to produce flowing concrete, they are often referred to as superplasticizers. Typically these HRWRs will be used for one of the following reasons:

- To create self-leveling, flowing concrete without increasing the water content and without sacrificing strength. When used to produce flowing concrete, a typical HRWR will transform a relatively stiff conventional concrete mixture with a ball penetration reading of 1 inch into a fluid, self-leveling mixture with a ball penetration reading of 4 inches or more.
- To produce high-strength concrete by reducing the water content and thus the water-cement ratio, but without any loss of workability. When used as water reducers, HRWRs permit the reduction of as much as 30% of the water in a conventional mixes with no loss of slump or workability.

[4] Although several products are marketed commercially as corrosion inhibitors, only the calcium nitrite materials are also accelerators. Calcium nitrite inhibitors are furnished in liquid form. Because of the relatively large quantity required, the mixing water volume is reduced by the volume of admixture added.

- To save cement by reducing both the water and the cement content while maintaining the same water-cement ratio and the same degree of workability. When used to save cement, HRWRs will permit a reduction in cement of up to 15% while maintaining the same concrete strength.

HRWRs are made from the salts of organic sulfonates. Depending on their primary purpose, they are divided into two types. The pure water reducers (ASTM Type F) are formulated from formaldehyde condensates of naphthalene or melamine sulfonic acid. The HRWR retarders (ASTM Type G) are made by blending hydroxyl compounds and ligno-sulfonic acid. Both HRWR types may be used to obtain the water reduction required; however, the ASTM Type G admixtures will delay the initial set as well. Depending on dosage, the delay will range from 30 minutes to 2 hours, or more.

HRWRs function by dispersing the cement particles; therefore, they will function most efficiently in concrete that is rich in cement or contains a mineral additive such as fly ash

When an HRWR is added to a normal concrete mix to produce a high-slump, flowing mixture, the concrete often segregates and bleeds. To prevent this, the mix must be redesigned to thicken the sand-cement paste. This is usually accomplished by increasing the fine aggregate by 4 or 5% or by adding a pozzolanic material. Regardless of the fines content, there is a maximum HRWR dosage beyond which segregation is inevitable in a flowing mixture.

Segregation and bleeding will not be increased when an HRWR is used simply to reduce the water content of a mix that remains within the normal consistency and workability range.

HRWRs have an unpredictable effect on air content. In theory, the amount of air-entraining agent needed should be increased when the naphthalene or melamine-based water reducers are used and decreased when the lignosulfonate-based retarders are used. However, this does not always occur in practice, so frequent testing for air content is indicated.

All superplasticized mixes experience a rapid loss of slump after the admixture is added. As the slump loss occurs, the mix gradually returns to its original consistency. Depending on temperature, humidity, cement characteristics, and other factors that affect slump loss, the slump change achieved by the superplasticizers is usually lost within 45-60 minutes or sooner, if the mix temperature is above about 70°F. If slump loss occurs because of delayed concrete placement or some other reason, the intended characteristics of the supeplasticized mix can be restored by one or more additional dosages of the admixture. However, each additional dosage reduces the amount of entrained air by 1 to 1.5%, so that an additional dosage of air-entraining admixture may be required as well.

Other than loss of air content, increasing the dosage to offset slump loss will have no detrimental effect on the concrete if an ASTM Type F admixture is being used. If a water-reducing retarder (ASTM Type G) is being used, however, the set retardation time will be increased proportionally to the amount of admixture added. Accordingly, some caution is advisable when considering an increase in the dosage of any Type G admixture to offset actual or anticipated slump loss.

Because the effect of the HRWRs is lost so quickly, they are usually added at the site rather than at the batch plant. When transit mixers are being used, the admixture should be mixed with a small quantity of water, and then added to the concrete at the front of the mixing drum through a pipe or hose placed inside the drum. Any additional air-entraining agent should be added at the same time. (Note that the mixer drum must be stopped while the admixtures are added.) Adding admixtures to the concrete at the discharge end of a transit mixer drum is not recommended because this procedure does not ensure a uniform dispersal of the admixture throughout the mixture. In view of the many uncertainties associated with the use of HRWRs, the use of one or more trial batches is recommended to determine the amount of admixture needed to achieve the intended results and, if air-entrained concrete is required, to ascertain the effect of the HRWR on air content.

Type S – Specific Performance Admixtures

Performance requirements outside the bounds of water reduction and set modification are classed as Type S admixtures. The dry-cast concrete industry uses a no-slump concrete that is compacted into precast molds by extreme vibration, packing or centrifugal spinning. Specific performance admixtures are added to dry-cast concrete for plasticizing, water repulsion, and efflorescence control. With transportation structures, specific performance admixtures (Table 2-9) are used to produce concretes with additional properties.

Table 2-9. Type S (Specific Performance) Admixture Working Characteristics.

Admixture Purpose	Advantages	Disadvantages
Shrinkage Reducing	Reduces surface tension of water to reduce cracking associated with drying shrinkage during early age curing by reducing water mass loss.	May delay set-time and reduce early strength. Requires optimization with air entrainment. High cost.
Corrosion Inhibitor	Blocks corrosive action of chlorides on reinforcing steel. Decreases permeability. Interrupts creation of galvanic response.	May accelerate initial set. Cracking reduces effectiveness.
Alkali-Silica Reaction Inhibitor	Reduce ASR by forming non-expansive gel with reactive aggregate. Requires SCM use to mitigate excess heat of hydration.	Accelerated hydration, heat generation, and set time. May increase shrinkage and reduce workability. High cost.
Viscosity Modifier	Controls bleeding. Reduces segregation while flowing through formwork. Improves workability. Promotes uniform surface appearance.	Overdosing leads to decreased workability, retarded set, and coarse air bubbles. Underdosing leads to low viscosity, bleeding and segregation. May require HRWR to retain workability.
Rheology Modifier	Improves placement, consolidation and finishing in low slump and slip-form mixes that could result from use of ultra fine SCMs, manufactured aggregate, or reduced water. Reduces concrete pump pressure requirements.	Similar to Viscosity Modifiers

Chapter 3
Review of Concrete Mix Designs

Table of Contents

Introduction **3-1**

Design of Concrete Mixes **3-1**

Definition **3-2**

Supplementary Cementitious Materials **3-3**

Aggregate **3-3**

Grading Limits **3-5**

Maximum Density and Balanced Grading **3-5**
General Information 3-6

Workability and Gradation **3-6**

Particle Shape and Surface Texture **3-6**

Water **3-7**
Admixtures 3-8

Mix Design Procedure (Absolute Volume Method) **3-9**

Review of Concrete Mix Designs **3-13**
Initial Submittal Review 3-14

Checking Submitted Cementitious Material Requirements and Proportioning **3-17**

Checking Submitted Aggregate Gradations **3-20**

Checking Amount of Water and Penetration **3-21**

Checking Submitted Chemical or Air-Entraining Admixtures **3-21**

Minor Concrete Binary Mix **3-48**

References **3-52**

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3 REVIEW OF CONCRETE MIX DESIGNS

Introduction

This chapter involves review of concrete mix designs, an essential factor ensuring contract compliance resulting typically in a product meeting its design objectives. Basic principles of concrete mixture design are initially addressed to give a brief overview as to how different types and amounts of materials affect the properties of concrete.

Design of Concrete Mixes

In a broad sense "designing" a concrete mix means selecting the proportions of fine and coarse aggregate, cementitious materials, admixtures, and water, that when combined will produce concrete having certain desired qualities and properties. Requirements to be met by the mix design are generally selected based on the intended use of the concrete, exposure conditions, dimensions of structural elements, and physical properties of the concrete required. Concrete quality is directly related to the amount and properties of the materials used, and methods and environment in which it is placed, finished, and cured. Concrete mixtures should be kept as simple as possible, as an excessive number of ingredients often make a concrete mixture difficult to control.

As a measure of the overall quality of the hardened concrete, strength should be set at the lowest value necessary to ensure that concrete having the desired quality will be obtained. As a measure of the workability of the plastic mixture, consistency must be compatible with job conditions. While in a plastic state concrete should ideally be of such consistency that it may be readily placed and consolidated without segregation.

A properly designed concrete mixture will possess the desired workability for the fresh concrete and the required durability and strength for the hardened concrete. Typically, the volume of a mix is about 10 to 15% cementitious material, 60 to 75% aggregate, and 15 to 20% water. Entrained air in concrete mixes may also take up to 8%. A concrete mixture that does not have enough paste to fill all the voids between the aggregates will be difficult to place and will produce rough honeycombed surfaces and porous concrete. A mixture with an excess of cementitious paste will be easy to place and will produce a smooth surface; however, the resulting concrete is likely to shrink more and be uneconomical.

Definitions

The following definitions are basic to a complete understanding of concrete mix design principles:

- *Water - Cement Ratio (W/C)* is defined as the ratio of the quantity of water to the quantity of cementitious material (sum of Portland cement and supplementary cementitious material) in the concrete mixture. It is usually expressed in terms of weight. If units are not indicated a ratio by weight is understood; that is, a water - cement ratio of 0.50 is understood to mean a ratio of one-half pound of water for each pound of cementitious material in the mix.

- *Specific Gravity (S.G.)* is the ratio of the mass of a given volume of material to the mass of an equal volume of water.

- *Bulk Specific Gravity (Saturated Surface Dry)* is the ratio of a given volume of material with its permeable voids filled with water to the mass of a volume of water equal to the total volume of the material. The total volume includes the combined volume of solid matter, permeable voids, and impermeable voids.

- *Absolute Volume* of a loose material is defined as the actual volume occupied by the solid particles of the material. The absolute volume of a material may be computed from the known weight and specific gravity, as follows:

$$\frac{\textit{Mass of Loose Material}}{\textit{(S.G.)} \times \textit{Unit Weight of Water}} = \textit{Absolute Volume}$$

- *Density* is defined as the weight of a known absolute volume of a material, usually expressed in pounds per cubic foot (pcf). The density of any material is equal to the product of the specific gravity of the material and the unit weight of water. In terms of density, it may be computed as follows:

$$\frac{\textit{Mass of Loose Material}}{\textit{Absolute Volume}} = \textit{Density}$$

- *Cementitious Factor* is defined as the sum of cement and supplementary cementitious material in a unit volume of concrete. It is usually expressed in lb/yd^3.

- *Free Water* is the total water in the mixture minus the water absorbed by the aggregates in reaching a saturated surface-dry condition.

Supplementary Cementitious Materials

Cementitious materials will exhibit binding properties and characteristics similar to that of Portland cement. Most concrete used on State projects will incorporate Portland cement and any individual or combination of the following materials to meet the contract requirements:

- Ground Granulated Blast Furnace Slag
- Fly Ash
- Raw or Calcined Natural Pozzolans
- Metakaolin
- Silica Fume
- Rice Hull Ash (not approved for structural elements via the 2010 Special Provisions)

Since the cementitious materials along with water combine to form the paste or "binder" that holds the aggregates together, maximizing the paste's quality is prudent. In a properly proportioned concrete mixture the cementitious paste surrounds and separates the individual aggregate particles preventing the physical interlocking of the particles. Absent the strengthening effects of aggregate interlocking, the paste alone must carry the loads imposed on the concrete. As a consequence, the strength of hardened concrete depends almost entirely on the strength of the cement paste. Therefore, the properties of concrete are influenced by the properties of cementitious materials. The type(s) and proportion(s) of cementitious materials affect both the fresh and hardened properties of concrete. An understanding of structure concrete characteristics can provide insight into any issues that may arise with concrete construction.

When choosing the types and amounts of cementitious materials it is advantageous to know what effects/characteristics each has on the concrete properties. Table 2-5 in Chapter 2 provides some information that pertains to the different types of supplementary cementitious materials (SCMs) when considering both freshly mixed and hardened concrete.

Aggregate

The term "aggregate gradation" is defined as the particle size distribution as determined by separation with standard sieves. Sieve analysis, screen analysis, grading analysis and mechanical analysis are terms used synonymously in referring to the process by which aggregate gradation is determined.

A "grading analysis" is made by passing a representative sample through a graded series of sieves and recording the percent passing, retained on, and/or falling between successive sieves in the series. The standard sieves used for grading fine aggregates are Numbers 4, 8, 16, 30, 50, 100 and 200. The screen number is the number of square openings per square inch. So a No. 4 screen would have 4 openings and a No. 50 would have 50 openings per square inch. The standard sieves for grading coarse aggregate are those having square openings of 2, 1 1/2, 1, 3/4 and 3/8 inches, plus the No. 4 and No. 8 sieves from the fine aggregate series. Standard grading charts, which have lines at intervals representing successive standard sieves, are used to plot particle size distribution.

Aggregate gradation is a highly technical subject. Over the years a great deal of study and research has been devoted to grading theory and much has been learned about the influence of aggregate gradation on the properties of a concrete mixture. However, thus far no one has discovered a simple, reliable method of determining the "ideal" grading for a given aggregate. Consequently, most grading curves have been developed empirically and are based on experience rather than theory.

Coarse aggregate consists of natural gravel, crushed gravel, crushed rock, reclaimed aggregate or combinations thereof. Fine aggregate consists of either natural sand or a combination of natural sand and manufactured sand. Coarse aggregate will vary in size depending on the purpose for which the material is being produced. When good coarse aggregate is available, the best concrete is produced by using the greatest percentage of the largest size of aggregate per cubic yard of concrete, which is compatible with job requirements. With reinforced concrete construction the maximum size of coarse aggregate will be limited by wall thickness, space between adjacent reinforcing bars, and/or similar structural features.

Fine aggregate serves two purposes. First, it is an inexpensive filler of most of the voids that exist in even the best-graded coarse aggregate. Second, it improves workability acting as a lubricant to facilitate concrete placement. The minimum quantity of fine aggregate needed to accomplish both purposes is the proper amount to use. Other factors being equal, very fine sands are uneconomical and will increase the cement demand because of the additional surface area to be coated with cement paste. Conversely, very coarse sands will produce harsh, unworkable mixes. In general, uniformly graded aggregates having neither a deficiency nor excess of any one size, which give a smooth grading curve, will produce the most satisfactory results.

Aggregate gradation is an important consideration in the design of a concrete mix because it affects the workability and the consistency of the concrete mixture, and is an influencing factor on both cost and quality of the finished product.

Grading Limits

Grading limits for aggregates are found in the Standard Specifications or the Special Provisions for a particular project. The specifications and/or provisions will include limits for all primary sizes of aggregate (fine and coarse) as well as limits for the combination of sizes that are used in the actual concrete mixture.

The primary size grading limits are intentionally broad to allow for economy through the use of aggregate from various sources. Limits of proposed gradations, often termed "X-values," provide flexibility and allow the contractor independent design judgment (within specified limits) to establish their compliance and operating ranges for the listed sieve sizes. Aggregate from any given source, however, must have a specific grading which may neither vary significantly nor be changed during its use on a project, except under extenuating circumstances as discussed in the following paragraph.

Before beginning work the contractor submits, for approval, a proposed aggregate grading which must be within the broad limits of the contract requirements. When approved, the grading must be maintained within the limits of variation permitted by the specifications. In those few instances where reasonable plant adjustments will not compensate for a major change in gradation, such as a change caused by moving from one pit to another, the contractor may request a change from the previously approved gradation. Note, however, that all requested revisions should be justified before they are approved which may require a resubmittal and requalification of the mix design.

Aggregates of the various primary sizes must be combined in such a way that the grading of the resulting aggregate mixture will lie within the combined grading limits in the specifications.

Maximum Density and Balanced Grading

The term "maximum density" is not a reference to weight when used to describe aggregate proportioning. Rather, it refers to the particle size distribution that will produce a mixture with the least volume of voids; that is, a mixture in which the voids or spaces between the larger particles are filled with smaller particles. Maximum density will be achieved when the particle-size distribution is such that no size greatly predominates and all sizes are uniformly distributed within the grading limits. Uniform particle size distribution is called "balanced" grading.

To ensure maximum density and prevent segregation the specifications require the grading of the 1″ x No. 4 coarse aggregate and the fine aggregate to be balanced. This ensures that "gap" grading does not occur. In some applications, "Gap" grading (omitting a particular size aggregate like 3/8 inch) can be useful but the likelihood of segregation, lower density, harsh mixes, and reduced "workability" are more apt to take place. The requirement is considered in the following examples:

General Information

- Contract provisions require the difference between the No. 16 sieve and the No. 30 sieve to be within 10 to 40%.
- The specification limits for the No. 16 sieve are from 55% to 75% passing.
- The limits for the No. 30 sieve are 34% to 46% passing.

Scenario #1 - An aggregate sample has 75% passing the No. 16 sieve and 34% passing the No. 30 sieve, thus both percentages fall within the acceptable range. However, the difference exceeds 40% thus the design is unbalanced.

Scenario #2 - An aggregate sample has 55% passing the No. 16 sieve and 46% passing the No. 30 sieve, thus both percentages fall within the acceptable range. However, the difference is less than 10% thus the design is unbalanced.

Workability and Gradation

Uniformly graded aggregate will produce the most workable concrete mixes. As a general rule, a grading to achieve maximum density will produce a mix of satisfactory workability as well.

Other factors being equal, the grading of the fine aggregate will have a greater effect on workability than will the grading of the coarse aggregate. Workability is particularly sensitive to the amount of material between the No. 50 and No. 100 sieves. A deficiency in this size may cause excessive bleeding. A grading of sand in which one or two sizes greatly predominate should be avoided. Such sand has a large void content and will require a large amount of cement paste to produce a workable mixture.

Particle Shape and Surface Texture

Both shape and surface texture characteristics of the individual aggregate particles have a pronounced effect on the workability of a concrete mixture.

Rounded aggregates result in a smooth, easily worked mixture whereas angular, elongated particles tend to interlock with each other and behave in a manner similar to a log jam producing what is referred to as a “harsh” mix.

Smooth, rounded particles will have a higher volume per unit of surface area than will angular, rough-textured particles, so that a minimum amount of cement paste will be needed to completely coat all particles. Aggregates having a high percentage of rough, angular particles will require more water to produce workable concrete, and more cement to maintain a given water-cement ratio, than smooth rounded aggregates.

Aggregate particles should be free of excessive amounts of flat, elongated pieces. This is particularly important in thin members where the flat surfaces of the aggregate may have a detrimental effect on concrete strength by creating a “weakened plane” along an axis subject to shear or diagonal tension stresses.

Because of differences in shape characteristics, more spherical (rounded) material may be added to a given quantity of cement paste than either cubical or prismoidal material; therefore, natural gravel is generally more economical than crushed stone since it will permit the use of a leaner mix (and therefore less cement) to obtain the same strength and workability.

Historically California Test Method 515 was used to limit the angularity of the fine aggregates used on state projects which provided insurance that the sand proposed for use would not result in an excessive amount of water to obtain the required workability. However, in addition to the typical historical concrete ingredients of cement, aggregate, and water the concrete mixtures of today have evolved to include the incorporation of chemical admixtures and supplementary cementitious materials that are designed for a given strength and workability. Thus, it was deemed no longer necessary and the CT 515 requirement was eliminated as a fine aggregate specification in 2010

Water

Almost any natural water that is potable and has no pronounced taste or odor may be used as mixing water for concrete. Keep in mind some waters that are not fit for drinking may still be suitable for concrete.

Excessive impurities in mixing water may not only affect setting time and concrete strength, but also may cause efflorescence, staining, corrosion of reinforcement, volume instability, and reduced durability. The Standard Specifications Section 90, sets limits on chlorides, sulfates, alkalis, and solids in mixing water. When using reclaimed water be sure to test water quality for compliance.

When using admixtures it is important to realize that performance and efficiency of chemical admixtures may be influenced by certain compounds in water. For example, the dosage of air-entraining admixture may need to be increased when used with hard waters containing high concentrations of certain compounds or minerals.

Admixtures

Those ingredients other than the basic components of concrete (Portland cement, supplementary cementitious materials, water, and aggregates) that are added to the concrete mixture prior to or during mixing are considered admixtures. Various types of admixtures, discussed in Chapter 2, can be used to help achieve desirable properties for concrete. The effectiveness of admixtures depends on factors such as:

- Type and Brand
- Water content of mix
- Amounts of materials in concrete
- Aggregate shape, gradation, and proportioning
- Temperature of concrete
- Consistency
- Mixing time

When a trial batch is necessary to prequalify the mix, the concrete should be batched in similar placement conditions (temperatures, humidity, etc.) to ensure the dosage(s) of admixture(s) used represent effects on the fluid and hardened properties or the concrete. Typically, the amount of admixture recommended by the manufacturer or the optimum amount determined by laboratory tests should be used. When liquid admixture dosages exceed 1/2 gallon (64 oz.) per cubic yard, the volume shall be included when determining the amount of free water in the mix.

If a water-reducing or water-reducing and retarding admixture is used, the specifications allow an optional 5% reduction by weight of cementitious material required in the mix as long as a minimum cementitious content of 505 pounds per cubic yard is maintained. If the reduction in cementitious material is made the dosage of the admixture shall be no less than the dosage used in determining approval of the admixture measured in fluid ounces per 100 pounds of cement. This qualifying dosage rate will be listed on the Authorized Materials List for the admixture. Also, when referencing the manufacturer's suggested dosage rates, be sure to note and distinguish whether the rate is suggested in fluid ounces per 100 pounds of cement or cementitious materials.

In general, the dosage of chemical admixtures should be used based on the Portland cement content of a mix, and not on the amount of cementitious material (Portland cement + supplementary cementitious), unless so stated in the manufacturer's published recommendations. Water reducing admixtures (Types A and F) increase set time but only affect Portland cement. As an example, Grace WRDA 64 at a dosage rate of 3 fluid ounces per 100 pounds of cement at 72°F retards set time 1.4 hours beyond normal set time. An overdose caused by assuming supplementary cementitious material were comparable to Portland cement would cause a significant increase in set time, which could become another cause for plastic shrinkage should the surface water evaporate quickly.

Mix Design Procedure (Absolute Volume Method)

Concrete, in a plastic state, may be visualized as a mixture of cementitious paste and aggregate with the paste completely surrounding and separating the individual aggregate particles. Thus the volume of concrete produced by a given quantity of paste and aggregate will be the sum of the following:

- Absolute volume of the aggregates
- Absolute volume of cement
- Absolute volume of supplementary cementitious materials
- Volume of water
- Volume of entrained air, if any
- Volume of additional admixtures, if applicable

The yield of batched concrete is an exception to the rule that "the whole is the sum of its parts." For example, a concrete mixture might contain:

- 1.3 cubic yards cementitious material
- 2.9 cubic yards sand
- 3.9 cubic yards gravel
- 1 cubic yard water

The sum of the individual volumes is 9.1 cubic yards, yet when mixed the batch may yield only 7 cubic yards of concrete. The sand fills the voids in the gravel and the cement paste fills the remaining voids. The presence and volume of entrapped or entrained air in the cement paste also needs to be considered.

Example #1

For a normal cementitious mixture the absolute volume method of mix design first calculates the absolute volume of cementitious material and the volume of water, in cubic feet per cubic yard, using the specified cement content and the desired wate -cement ratio.

For example, assume a mix having a water-cement ratio of 0.5 with a total cementitious material content of 675 pounds per cubic yard (pcy). The following specific gravities and proportions are assumed as listed:

- 70% Portland Cement with S.G. = 3.15
- 10% Ground Granulated Blast Furnace Slag with S.G. = 2.90
- 20% Class F Fly Ash with S.G. = 2.30
- Fine and coarse aggregates with Bulk S.G. (SSD) = 2.72

The calculations are:

Absolute Volume of Cementitious Material $\Rightarrow$

$$\left(\frac{0.70\times 675\ lb}{3.15\times 62.4\ lb/ft^3}\right)+\left(\frac{0.10\times 675\ lb}{2.90\times 62.4\ lb/ft^3}\right)+\left(\frac{0.20\times 675\ lb}{2.30\times 62.4\ lb/ft^3}\right)=3.72ft^3$$

Absolute Volume of Water $\Rightarrow$ $\left(\frac{0.50\times 675\ lb}{62.4\ lb/\ ft^3}\right)=5.41\ ft^3$

The absolute volume of the aggregates is determined by subtracting the absolute volume of cement and the volume of water in the batch.

Absolute Volume of Aggregates $\Rightarrow$ $27\ ft^3 - 3.72\ ft^3 - 5.41\ ft^3 = 17.87\ ft^3$

Relative amounts of fine and coarse aggregate are obtained by multiplying the absolute volume of all aggregates by the percentage of each aggregate used in the mix. Percentages of aggregate are dependent on the properties of the aggregate (size, shape, porosity, texture, etc). A mix designer could use tables based on empirical relationships to determine proper percentages. For this example we will assume a 40/60% ratio between fine and coarse aggregate. The calculations are:

Absolute Volume of Fine Aggregate $\Rightarrow$ $0.40 \times 17.87\ ft^3 = 7.15\ ft^3$

Absolute Volume of Coarse Aggregate $\Rightarrow$ $0.60 \times 17.87\ ft^3 = 10.72\ ft^3$

The theoretical batch weights per cubic yard of concrete are as follows:

Fine Aggregate $\Rightarrow$ $7.15\ ft^3 \times 2.72 \times 62.4\ lb/ft^3 = 1{,}214\ lb$

Coarse Aggregate $\Rightarrow$ $10.72\ ft^3 \times 2.72 \times 62.4\ lb/ft^3 = 1{,}820\ lb$

Portland Cement $\Rightarrow$ $0.70 \times 675 lb = 472.5\ lb$

Ground Granulated Blast Furnace Slag $\Rightarrow$ $0.10 \times 675 lb = 67.5\ lb$

Class F Fly Ash $\Rightarrow$ $0.20 \times 675 lb = 135\ lb$

Water $\Rightarrow$ $5.41\ ft^3 \times 62.4\ lb/ft^3 = 338\ lb$

Mix design quantities would be as follows:

Fine Aggregate $\Rightarrow$ *1,214 lb*

Coarse Aggregate $\Rightarrow$ *1,820 lb*

Portland Cement $\Rightarrow$ *473 lb*

Ground Granulated Blast Furnace Slag $\Rightarrow$ *68 lb*

Class F Fly Ash $\Rightarrow$ *135 lb*

$$\text{Free Water} \Rightarrow \frac{338\ lb}{8.34\ lb/gallon} = 40.5 gallons$$

To batch concrete accurately, adjust the theoretical batch weights to account for the weight of free water in the fine aggregate. Free water is measured as a percentage of the weight of the fine aggregate, and is referred to as "moisture content" in mix design terminology.

Assuming moisture content is 6% the batch weights are adjusted as follows:

Weight of Free Water $\Rightarrow$ *0.06 x 1,214 lb = 73 lb*

Adjusted Fine Aggregate $\Rightarrow$ *1,214 lb + 73 lb = 1,287 lb*

Adjusted Weight of Free Water $\Rightarrow$ *338 lb - 73 lb = 265 lb*

Therefore, actual batch proportions could be as follows:

Fine Aggregate $\Rightarrow$ *1,287 lb*

Coarse Aggregate $\Rightarrow$ *1,820 lb*

Portland Cement $\Rightarrow$ *473 lb*

Ground Granulated Blast Furnace Slag $\Rightarrow$ *68 lb*

Class F Fly Ash $\Rightarrow$ *135 lb*

$$\text{Free water} \quad \frac{265\ lb}{8.34\ lb/gallon} = 31.8\ gallons$$

<u>Example #2</u>

When air entrainment is introduced into a concrete mixture there is some reduction in strength if no changes are made in the mix proportions. The reduction occurs because the total volume of concrete produced is increased by an amount equal to the volume of entrained air, thus reducing the cementitious factor.

The procedure usually followed when designing air-entrained mixes is to first design a mix assuming no air, and then adjust the proportions to compensate for the volume of the entrained air. Air-entrained concrete will have greater workability than normal concrete with other factors remaining constant. The reduction in volume is accomplished by reducing the water content and the amount of fine aggregate. Usually, no change will be necessary in the volume of coarse aggregate. Most air-entrained concrete used on State highway projects will require a minimum of 590 pounds of cementitious material per cubic yard with a 6% air content. Although some industry manuals may replace the sand solely with the volume of air content, experience shows that for 6% air in combination with 590 pounds of cementitious material, while maintaining the same degree of workability, the water content may be reduced by about 33% of the volume of the entrained air.

To maintain the same relative yield, the proportions of the ingredients used in an air-entrained concrete mixture must be adjusted to compensate for the increased volume of air. Since an air-entrained concrete also will require less water than normal concrete of similar design, the usual procedure is to design the concrete mix assuming no air and then reduce the volume of fine aggregate by an amount equal (volumetrically) to the difference between the volume of entrained air and the reduction in water.

For example, assume a mix having a water - cement ratio of 0.48 with a total cementitious material content of 590 lb/yd^3. The following specific gravities and proportions are assumed as listed:

- 75% Portland cement with S.G. = 3.15
- 5% Ground Granulated Blast Furnace Slag with S.G. = 2.90
- 20% Class F Fly Ash with S.G. = 2.30
- Fine and coarse aggregates with Bulk S.G. (SSD) = 2.70

Absolute Volume of Cementitious Material $\Rightarrow$

$$\left(\frac{0.75 \times 590lb}{3.15 \times 62.4\ lb/ft^3}\right)+\left(\frac{0.05 \times 590lb}{2.90 \times 62.4\ lb/ft^3}\right)+\left(\frac{0.20 \times 590lb}{2.30 \times 62.4\ lb/ft^3}\right) = 3.24\ ft^3$$

$$\text{Absolute Volume of Water } \left(\frac{0.48 \times 590lb}{62.4\ lb/ft^3}\right) = 4.54\ ft^3$$

Assuming a 40/60% ratio between fine aggregate to coarse aggregate:

$$\text{Absolute Volume of Fine Aggregate} \Rightarrow (27\ ft^3 - 3.24\ ft^3 - 4.54\ ft^3) \times (0.4) = 7.68\ ft^3$$

$$\text{Absolute Volume of Coarse Aggregate} \Rightarrow (27\ ft^3 - 3.24\ ft^3 - 4.54\ ft^3) \times (0.6) = 11.53\ ft^3$$

Reducing the volume of fine aggregate and water to account for the volume of air in the mix gives the following adjusted mix design:

Volume of Air $\Rightarrow$ $0.06 \times 27\ ft^3 = 1.62\ ft^3$

Coarse Aggregate (No adjustment) = $11.53\ ft^3$

Cementitious Material (No adjustment) = $3.24\ ft^3$

Water (adjustment) $\Rightarrow$ $0.33 \times 1.62\ ft^3 = 0.53\ ft^3 \Rightarrow 4.54\ ft^3 - 0.53\ ft^3 = 4.01\ ft^3$

Fine Aggregate (adjustment) $\Rightarrow$ $0.67 \times 1.62\ ft^3 = 1.09\ ft^3 \Rightarrow 7.69\ ft^3 - 1.09\ ft^3 = 6.60\ ft^3$

TOTAL VOLUME = $27.0\ ft^3$

Review of Concrete Mix Designs

Before using any Portland cement based concrete the contractor is required to submit in writing a copy of their mix design(s). An integral part of quality assurance is the review of submitted concrete mix design(s). Attention must be paid to details in the Plans, Special Provisions, and Standard Specifications to ensure a compliant concrete mix is used in the work. On most projects there will be multiple concrete mix designs. Care must be taken to ensure that the proper concrete mixes are designated to be used at the proper locations.

It is the contractor's responsibility to design a mix using ingredients that are in compliance with the contract requirements. If a submitted concrete mix design does not meet the contract requirements this must be brought to the contractor's attention. Except for rare cases, a mix design not in accordance with contract documents should not be used on the project. If the parameters of the mix design specifications are not met, a change order must be written to allow the use of the concrete mix design.

There are various Authorized Materials Lists available to ensure quality products are being incorporated into our structures. Authorized and/or prequalified lists are available for the following concrete ingredients:

- Portland cement
- Supplementary cementitious materials
- Innocuous aggregates
- Air-entraining admixtures
- Chemical admixtures

If products used in the mix design are listed as authorized, they are acceptable for incorporation into concrete mixes. However, it is still required for the contractor to provide manufacturers' information and/or certificates of compliance with their initial submittal.

Aggregate that is not on the "innocuous aggregate" list may still be used in the concrete mix but must be considered "non-innocuous aggregate."

Initial Submittal Review

Upon receipt of a submitted mix design, an initial review must take place to ensure you have received a complete submittal. If information is missing it should be brought to the contractor's attention promptly. Table 3-1 will provide you with a guideline regarding your initial review:

Table 3-1. Concrete Mix Design Submittal Checklist.

A) CEMENT	
□ Yes □ No	On Authorized Materials List?
□ Yes □ No	If type II, III, or V Portland cement does it contain more than 0.60% by mass of alkalies?
□ Yes □ No	Autoclave expansion > 0.50%?
□ Yes □ No	If type II Portland cement does Tricalcium silicate content exceed 65%?
B) BLENDED CEMEMT	
□ Yes □ No (If No, skip to C)	Blended cement used?
□ Yes □ No	Blended cement materials on Authorized Materials List??
C) SUPPLEMENTARY CEMENTITIOUS MATERIALS	
□ Yes □ No	All SCMs on Authorized Materials List?
C-1) Fly Ash	
□ Yes □ No (If no skip to C-2)	Is Fly Ash used?
□ Yes □ No	Meets AASHTO M295, Class F?
□ Yes □ No	Sodium oxide (Na_2O) total and equivalent included?
C-2) ULTRA FINE Fly Ash	
□ Yes □ No (If No, skip to C-3)	Is Ultra Fine Fly Ash used?
□ Yes □ No	Meets AASHTO M295, Class F?
□ Yes □ No	Sulfur trioxide (SO_3) content included?
□ Yes □ No	Loss on ignition percentage included?
□ Yes □ No	Sodium oxide (Na_2O) total and equivalent included?
□ Yes □ No	Particle size distribution included?

Table 3-1. Concrete Mix Design Submittal Checklist (continued).

□ Yes □ No	Strength Activity Index included?
□ Yes □ No	Expansion at 16 days via ASTM C1567 included?
C-3) RAW OR CALCINED NATURAL POZZOLAN	
□ Yes □ No (If no, skip to C-4)	Is Raw or Calcined Natural Pozzolan used?
□ Yes □ No	Meets AASHTO M295, Class N?
□ Yes □ No	Sodium oxide (Na_2O) total and equivalent included?
C-4) METAKAOLIN	
□ Yes □ No (If no, skip to C-5)	Is Metakaolin used?
□ Yes □ No	Meets AASHTO M295, Class N?
□ Yes □ No	Silicon dioxide (SiO2) and Aluminum oxide (Al_2O_3) content included?
□ Yes □ No	Calcium oxide (CaO) content included?
□ Yes □ No	Sulfur trioxide (SO_3) content included?
□ Yes □ No	Loss on ignition included?
□ Yes □ No	Sodium oxide (Na_2O) equivalent included?
□ Yes □ No	Particle size distribution included?
□ Yes □ No	Strength Activity Index included?
C-5) GROUND GRANULATED BLAST FURNACE SLAG (GGBFS)	
□ Yes □ No (If no, skip to C-6)	Is Ground Granulated Blast Furnace Slag used?
□ Yes □ No	AASHTO M302 Grade 100 or 120?
C-6) SILICA FUME	
□ Yes □ No (If no, skip to C-6)	Is Silica Fume used?
□ Yes □ No	Meets AASHTO M307?
□ Yes □ No	Reduction in mortar expansion included?
D) AGGREGATE	
□ Yes □ No	Are proposed gradation(s) included?

Table 3-1. Concrete Mix Design Submittal Checklist (continued).

◻ Yes ◻ No (If yes, reduced "X" allowed in Section B)	Aggregates on Innocuous Aggregates List?
D-1) COARSE AGGREGATE	
◻ Yes ◻ No	Loss via CT 214 included?
◻ Yes ◻ No	Loss in Los Angeles Rattler included (CT 211)?
◻ Yes ◻ No	Cleanness value included (CT 227)?
D-2) FINE AGGREGATE	
◻ Yes ◻ No	Loss via CT 214 included (waived if durability index of fine aggregate is 60 or greater)?
◻ Yes ◻ No	Durability index via CT 229 included (only necessary if CT 214 does not meet qualifications)?
◻ Yes ◻ No	Organic impurities results included (CT 213)?
◻ Yes ◻ No	Sand equivalent included (CT 217)?
E) CHEMICAL ADMIXTURES	
◻ Yes ◻ No (If No, skip to F)	Chemical admixture(s) used?
◻ Yes ◻ No	On Authorized Materials List?
◻ Yes ◻ No	Dosage verified per plans or manufacturer's recommendations
F) AIR - ENTRAINING ADMIXTURE	
◻ Yes ◻ No (If No, skip to F)	Air-entraining admixture(s) specified or used?
◻ Yes ◻ No	On Authorized Materials List?
◻ Yes ◻ No	Dosage verified per plans or manufacturer's recommendations

Upon receipt of a complete submittal you should start with checking the mix design to ensure it meets all contract requirements including:

- Chemical and physical requirements for cement, SCMs, and admixtures.
- Aggregate properties, testing results and gradations.
- Manufacturer's recommended dosage rates for admixtures.
- Shrinkage limits met per AASHTO T160 if applicable.

Once you determine that all material prerequisites previously mentioned have been met you will want to proceed with checking the aggregate gradations and proportioning of materials to ensure they meet the contract requirements.

Checking Submitted Cementitious Material Requirements and Proportioning

First, you will need to determine if the type(s) of cementitious materials chosen to be used are on the Authorized Materials List. In addition, all applicable test requirements listed in the specifications need to be verified to ensure compliance

The Special Provisions and/or Standard Specifications will depict the minimum cementitious content requirement for the type of facility or structure you are building. The summation of submitted cementitious components must meet the minimum/maximum cementitious material content limits. If the submittal includes a blended-cement, the percentage of each component must be provided. Form DS-OS C70 can assist you with checking to ensure compliance with the specifications.

For the upcoming equations the following terms are defined:

UF = Silica fume, metakaolin, or UFFA, including the amount in blended cement, lb/yd^3.

FA = Fly Ash or natural pozzolan conforming to the requirements in AASHTO Designation: M295, Class F or N with a CaO content up to 10%, including the amount in blended cement, lb/yd^3.

FB = Fly Ash or natural pozzolan conforming to the requirements in AASHTO Designation: M295, Class F or N with a CaO content up to 15%, including the amount in blended cement, lb/yd^3.

F = Fly Ash or natural pozzolan complying with AASHTO M295, Class F or N, including the quantity in blended cement, lb/yd^3. F is equivalent to either FA or FB.

SL = GGBFS including the amount in blended cement, lb/yd^3.

MC = Minimum amount of cementitious material specified, lb/yd^3.

MSCM = The minimum sum of SCMs that satisfies Equation (1) for general concrete, lb/yd^3.

PC = The amount of Portland cement, including the amount in blended cement, lb/yd^3.

TC = Total quantity of cementitious material used, lb/yd^3.

Note the precision of the equations in the specification is intentional. If the value is listed to the nearest tenth (0.1) in the specifications then the calculated answer of the equation should likewise be calculated to the nearest tenth (0.1).

For general Portland cement concrete the SCM content shall conform to either Option A or B:

Option A: Any combination of Portland cement and at least one SCM, satisfying Equations 1 and 2:

Equation (1) $$\frac{(25\times UF)+(12\times FA)+(10\times FB)+(6\times SL)}{MC} \geq X \text{ - Ensures minimal use of SCMs}$$

X = 1.8 for innocuous aggregate, 3.0 for all other aggregates.

Equation (2) $MC - MSCM - PC \geq O$ *- Limits amount of Portland cement*

Option B: 15% of Class F Fly Ash with at least 48 ounces of $LiNO_3$ solution added per 100 pounds of Portland cement. CaO content of the Fly Ash shall not exceed 15%.

If the concrete is designated for a freeze-thaw region (without exposure to de-icing chemicals) this additional equation must be met:

$$\frac{(41\times UF)+(19\times F)+(11\times SL)}{TC} \leq 7.0 \text{ - Limits SCM amount in mix}$$

For concrete designated as exposed to de-icing chemicals, Equations 1 through 5 must be satisfied:

Equation (1) $$\frac{(25\times UF)+(12\times FA)+(10\times FB)+(6\times SL)}{TC} \geq X \text{ - Ensures minimal use of SCMs}$$

Equation (2) $$\frac{4\times(FA+FB)}{TC} \leq 1.0 \text{ - Limits Fly Ash to maximum of 25\% of TC}$$

Equation (3) $$\frac{(10\times UF)}{TC} \leq 1.0 \text{ - Limits UF to maximum of 10\% of TC}$$

Equation (4) $$\frac{2\times(UF+FA+FB+SL)}{TC} \leq 1.0 \text{ - Limits SCMs to maximum of 50\% of TC}$$

Equation (5) $$\frac{27\times(TC-MC)}{MC} \leq 5.0 \text{ - Limits TC to 18.5\% maximum increase above MC}$$

Precast concrete is the one situation, only if the aggregates are listed as innocuous, in which the contractor can use 100% Portland cement. Precast concrete shall conform to one of the following options.

Option A: Any combination of Portland cement and SCM (if necessary), satisfying the following equation:

Equation (1) $$\frac{(25\times UF)+(12\times FA)+(10\times FB)+(6\times SL)}{MC} \geq X \text{ Ensures minimal use of SCM}$$

Where:

$X =$ 0.0 if precast members are constructed with Portland cement concrete using aggregate that is “innocuous” in conformance with the provisions.
$X =$ 3.0 for all other aggregates.

Option B: Fifteen percent of Class F Fly Ash with at least 48 ounces of $LiNO_3$ solution added per 100 pounds of Portland cement. CaO content of the Fly Ash shall not exceed 15%.

Option C: Any combination of supplementary cementitious material and Portland cement may be used if the expansion of cementitious material and aggregate does not exceed 0.10% when tested in conformance with the requirements in ASTM C1567.

Concrete designated for a corrosive enviroment must meet the prescriptive requirements of the Standard Specifications.

Due to the increased use of SCMs, the following applies:

- If the specified 28-day compressive strength is greater than 3,600 psi, 42 days will be allowed to meet the specified 28-day strength requirement.
- If the proportions of cementitious material satisfy the following equation, 56 days will be allowed to meet the specified 28-day strength requirement.

$$\frac{(41\times UF)+(19\times F)(11\times SL)}{TC} \geq 7.0$$

Checking Submitted Aggregate Gradations

You will need to determine what primary coarse size aggregate the contractor has chosen to use in their mix design and ensure that it meets the specified requirements of the contract. A contractor is allowed to use multiple stockpiles for their chosen primary coarse aggregate sizes if they deem necessary. If a primary coarse aggregate or fine aggregate is separated into two or more sizes, contractors are required to submit the gradation and proposed proportions of each size, both separately, and combined. The combined gradation of all aggregate must meet the contract requirements. They must show the percentage passing for each required sieve size.

The coarse aggregate(s) used may be any singular or combination of the following:

- Gravel
- Crushed rock
- Crushed gravel
- Reclaimed aggregate (must still meet all aggregate contract requirements)
- Iron blast furnace slag that has been air-cooled then crushed (not allowed in structures work containing reinforcement)

The fine aggregate(s) used may be any singular or combination of the following:

- Natural Sand
- Manufactured Sand (derived from crushing larger aggregates)

In order to provide contractors with some flexibility when designing their mix they are allowed to choose their “Limits of Proposed Gradation” otherwise commonly referred to as “X-values” for certain sieve sizes of their chosen primary aggregate nominal size and fine aggregate. Their submitted “Limits of Proposed Gradation” must lie within the range provided by the contract documents. The allowable window for percentage passing of the applicable certain sieve sizes will adjust based on the contractor’s chosen X-values for primary coarse size and fine aggregate.

The final step will be to verify that the contractor’s submitted gradation for the primary coarse aggregate, fine aggregate, and combined aggregate gradings meet the contract requirements. Form DS-OS C70A is available to assist you with checking of the submitted gradations.

In accordance with the State Contract Act, verify that the material and aggregate source(s) comply with the Surface Mining and Reclamation Act of 1975 (SMARA). Mining operations determined to be in compliance are listed on the AB 3098 SMARA Eligible list. You can obtain this list from the Division of Construction or the Department of Conservation’s web site at:

http://www.consrv.ca.gov/OMR/ab_3098_list/index.htm

Also, see Section 7-103D, "Surface Mining and Reclamation Act" of the Construction Manual to determine if the proposed materials site is exempt from SMARA.

Checking Amount of Water and Penetration

You can determine the allowable penetration or slump pending on what type of facility or structure you are constructing via Section 90 of the Standard Specifications. Keep in mind that if a Type F or G chemical admixtures is used, a significant increase in slump is allowed.

The typical amount of free water allowed (unless adverse conditions are met) must not exceed 310 pounds per cubic yard (pcy) plus an additional 20 pcy for each 100 pounds of cementitious material in excess of 550 pcy. A gallon of water weighs approximately 8.34 pounds at 60°F. Remember that the amount of free water is defined as the total water in the mixture minus the water absorbed by the aggregates in reaching a saturated surface-dry condition.

When calculating the total quantity of free water, liquid admixtures must be accounted for in the mix design as equivalent to free water if the dosage is more than 0.5 gallon (64 fluid ounces) per cubic yard of concrete.

Checking Submitted Chemical or Air-Entraining Admixtures

Admixtures must be used if specified in the contract. Multiple chemical and/or air-entraining admixtures shall be compatible when used together, and the manufacturer's recommendations must include a statement that the admixtures are compatible with the types and quantities of SCMs used. The amounts used shall be in accordance with the manufacturer's recommendations.

Air-entrainment use is optional if not specified by the contract. Requirements regarding allowable or targeted air content are dependent on whether air entrainment is specified or used optionally. If air entrainment is specified, the target air content will be provided in the contract and may vary depending upon the location of the District where the project is being constructed.

Binary Mix Design Check

Example #1 uses the following mix design:

Concrete Mix Design
CM381016F
Specified Compressive Strength: 4,000 psi

Contractor:	Charismatic Construction
Project:	Hwy 52 – Contract 03-256804
Source of Concrete:	Poncherello's Quality Concrete
Construction Type:	Bridge Deck Concrete
Placement:	Pump or Tailgate

Weights per Cubic Yard (Saturated, Surface Dry)

	Quantity	Specific Gravity	Yield (ft^3)
ASTM C-150/Type II Mod. cement, lb	480.0	3.15	2.44
ASTM C-168/Class F Fly Ash, lb	165.0	2.43	1.09
Water, lb	285.0 (34.2 gal)	1.00	4.57
1″ x #4 Red Rock Aggregate, lb	1,804	2.89	10.0
Yosemite Sand, lb	1,425	2.69	8.49
322N – Type A (oz/100 lb Portland cement)			
Total Air,%	1.5 + - 0.5		
			Total = 27.0 ft^3

Limits of Proposed Gradation (X-Values)

Aggregate		Sand	
3/4″	X = 80	No. 16	X = 69
3/8″	X = 17	No. 30	X = 46
		No. 50	X = 26

Aggregate Gradation Analysis Results (Percent Passing)

Aggregate		Sand	
1 1/2″	100%	3/8″	100%
1″	100%	No. 4	100%
3/4″	81%	No. 8	84%
1/2″		No. 16	61%
3/8″	8%	No. 30	37%
No. 4	1%	No. 50	20%
No. 8	0%	No. 100	7%
		No. 200	3%

Figure 3-1. Example Concrete Mix Design - Binary Mix.

<u>**Given:**</u>

Material sources:

Portland cement = Lehigh Southwest, Permanente Plant Type II/V
Class F Fly Ash, Headwaters Resources Inc., Delta Power Plant Class F Fly Ash
Coarse Aggregate = Teichert, Martis Valley Pit
Fine Aggregate = Western, Western Aggregate Mine

<u>**Initial Material(s) Check**</u>

Is Portland cement on Authorized Materials List?	YES
• Tricalcium Silicate Content ≤ 65%?	YES
• ≤ 0.60% by mass of alkalies?	YES
• Autoclave Expansion < = 0.50%?	YES
Is Class F Fly Ash on Authorized Materials List?	YES
• Calcium Oxide Content ≤ 15%?	YES
Is Coarse Aggregate source SMARA listed?	YES
Is Coarse Aggregate on Innocuous Aggregate List?	YES (SMARA 91-58-0001)
• (CT 214) ≤ 10% loss via Soundness test	YES
• (CT 211) Los Angeles Rattler ≤ 45%	YES
• (CT 227) Cleanness ≥ 75	YES
Is Fine Aggregate source SMARA listed?	YES (SMARA 91-29-0004)
Is Fine Aggregate on Innocuous Aggregate List?	YES
• (CT 213) Organic Impurities = "Satisfactory"	YES
• (CT 217) Sand Equivalent ≥ 75	YES
Is Type A Admixture (Polyheed 322N) on List?	YES
Shrinkage Information (AASHTO T 160) submitted/met?	YES

Materials:

Type II Portland cement = 480 lb/yd^3 (S.G. = 3.15)
Fly Ash, Class F (CaO content 14%) = 165 lb/yd^3 (S.G. = 2.35)
Aggregate Type = Innocuous

Calculate the Total Cementitious Material Content (TC)

Minimum cementitious (MC) content via the Standard Specifications for bridge deck concrete = 675 lb/yd^3. If a water reducing admixture is used, a 5% by weight reduction of cementitious material content is allowed via the Standard Specifications if the dosage meets or exceeds the dosage used in determining approval of the admixture. The mix dosage of 5 oz per 100 lb cement is greater than the dosage rate of 4 oz per 100 lb used to qualify the Polyheed 322N given on the Authorized Materials List, thus a 5% reduction is allowed if the contractor elects.

$$MC = 675\ lb/yd^3 \times (100\% - 5\%) = 641\ lb/yd^3$$

Total Cementious Material Content (TC)

$$TC = 480\ lb/yd^3 + 165\ lb/yd^3 = 645\ lb/yd^3$$

$$645\ lb/yd^3 > 641\ lb/yd^3$$

<u>OK</u>

Check Equation #1 for General Concrete:

$$\frac{(25 \times UF) + (12 \times FA) + (10 \times FB) + (6 \times SL)}{MC} \geq X$$

UF = 0 (No Silica Fume used in mix)
FA = 0 (No Fly Ash or Natural Pozzolan used with CaO content ≤ 10% in mix)
FB = 165 lb/yd^3
SL = 0 (No ground granulated blast furnace slag in mix)
MC = 641 lb/yd^3
X = 1.8 for innocuous aggregate

$$\frac{(25 \times 0) + (12 \times 0) + (10 \times 165) + (6 \times 0)}{641} = 2.57$$

$$2.60 \geq 1.8$$

OK

Calculate the Minimum Supplementary Cementitious Material (MSCM) to be used in Equation #2

Note: In order to calculate the MSCM of Equation #1, iterations are necessary. To simplify this calculation start from left to right and enter the SCM values, up to the actual amount in the mix until the left side of the equation is equal to the required X value (X = 1.80 in this case)

Equation #1

$$\frac{(25 \times UF) + (12 \times FA) + (10 \times FB) + (6 \times SL)}{MC} \geq X$$

Enter in the amount of UF up to the actual (in this case 0 lb/yd^3)

$$\frac{(25 \times 0)}{641} = 0$$

Enter in the amount of FA up to the actual (in this case 0 lb/yd^3)

$$\frac{(25 \times 0) + (12 \times 0)}{641} = 0$$

Enter in the amount of FB up to the actual (in this case 165 lb/yd^3)

$$\frac{(25\times0)+(12\times0)+(10\times165)}{641} = 2.57$$

2.57 > X = 1.8 so the Fly Ash (FB) quantity needs to be adjusted so the equation = 1.8. Solve for FB to obtain the MSCM value.

$$10 \text{ x } FB = (1.8\text{x}641) - (25\text{x}0) - (12\text{x}0)$$

$$FB = \frac{(1.8\times641)+(25\times0)(25\times0)}{10}$$

$$FB = 115$$

<u>MSCM = 115 lb/yd^3</u>

Check Equation #2

$$MC - MSCM - PC \geq 0$$

MC = 641 lb/yd^3
MSCM = 115 lb/yd^3
PC (total quantity of Portland cement) = 480 lb/yd^3

641 - 115 - 480 = 46
46 lb/yd^3 > 0
<u>OK</u>

Verify the allowed Strength Development Time

A total of 56 days is allowed to obtain the required compressive strength if the following equation is met:

$$\frac{(41\times UF)+(19\times F)(11\times SL)}{TC} \geq 7.0$$

UF = 0 (No Silica Fume used in mix)
F = (Fly Ash A and Fly Ash B combined) 165 lb/yd^3
SL = 0 (No ground granulated blast furnace slag in mix)
TC (Total quantity of cementitious material in mix) = 645 lb/yd^3

$$\frac{(41\times0)+(19\times165)+(11\times0)}{654} = 4.79$$

4.8 < 7.0

Thus the 56-day total is not allowed to gain the specified strength. The contractor must abide by the specified time allowance of the contract.

Free Water

The allowable amount of free water per the Standard Specifications Section is 310 lb/yd^3 plus 20 lb/yd^3 for each 100 lb of cementitious material in excess of 550 lb/yd^3. Calculate the allowable amount of free water:

$$310 + \frac{(645\text{-}550)}{100} \times 20 = 329$$

Total allowable amount of free water = 329 lb/yd^3
or 329/(8.34 lb/gallon) = 39.4 gallons/yd^3
39.4 gallons/yd^3 > 33.9 gallons/yd^3
OK

Gradation Check

Given:

Aggregate Sizes

- Primary Coarse Aggregate = 1″ x No. 4
- Fine Aggregate = Sand

Check to see if the submitted X-values for the primary aggregate sizes are in compliance with the limits of proposed gradation.

Coarse Aggregate (1″x No. 4)

Sieve Size	Limits of Proposed Gradation	Submitted X-Value	Status
3/4″	52-85	80	OK
3/8″	15-38	17	OK

Fine Aggregate (Sand)

Sieve Size	Limits of Proposed Gradation	Submitted X-Value	Status
No. 16	55-75	69	OK
No. 30	34-46	46	OK
No. 50	16-29	26	OK

Next check the coarse and fine aggregate gradings individually. All values must lie within the operating ranges given in the specifications.

Table 3-2. Coarse Aggregate (1″ x No. 4) Grading Checklist - Binary Mix.

Sieve Size	X-Value	Percentage Passing Based On X-Value	Percentage Passing Operating Range Limits	Contractor	Status
1.5″			100	100	OK
1″			88 - 100	100	OK
3/4″	80	X +/- 15	65 - 95	81	OK
3/8″	17	X +/- 15	2 - 32	8	OK
No. 4			0 - 16	1	OK
No. 8			0 - 6	0	OK

Table 3-3. Fine Aggregate (sand) Grading Checklist - Binary Mix.

Sieve Size	X-Value	Percentage Passing Based On X-Value	Percentage Passing Operating Range Limits	Contractor	Status
3/8″			100	100	OK
No. 4			95 - 100	100	OK
No. 8			65 - 95	84	OK
No. 16	69	X +/- 10	59 - 79	61	OK
No. 30	46	X +/- 9	37 - 55	37	OK
No. 50	26	X +/- 6	20 - 32	20	OK
No. 100			2 - 12	7	OK
No. 200			0 - 8	3	OK

Also, the Specifications state that in addition to the above required grading analysis, the fine aggregate sizes must be distributed such that:

- Difference between the total percentage passing the No. 16 and No. 30 sieves is from 10 to 40 (Check = 61 - 37 = 24) $\Rightarrow$ OK
- Difference between the percentage passing the No. 30 and No. 50 sieves is from 10 to 40 (Check = 37 - 20 = 17) $\Rightarrow$ OK

Next you need to check the combined gradation of the aggregates. You will need the percentage of volume of each primary aggregate size.

The proportions of each aggregate size can be calculated as follows:

Coarse Aggregate = (1,804 lb/2.89) / ((1,804 lb/2.89) + (1,425 lb/2.69)) = 54.1%
Fine Aggregate = (1,425 lb/2.69) / ((1,804 lb/2.89) + (1,425 lb/2.69)) = 45.9%

Table 3-4. Combined Aggregate Grading Checklist - Binary Mix.

Sieve Size	Percentage Passing Coarse Aggregate	Percentage Passing Fine Aggregate	Mix Percentage of Coarse Aggregate	Mix Percentage of Fine Aggregate	Combined Total Percentage Passing	Specifications Limits	Status
1.5″	100	100	54.1	45.9	100	100	OK
1″	100	100	54.1	45.9	100	90-100	OK
3/4″	81	100	54.1	45.9	89.7	55-100	OK
3/8″	8	100	54.1	45.9	50.2	45-75	OK
No. 4	1	100	54.1	45.9	46.4	35-60	OK
No. 8	0	84	54.1	45.9	38.6	27-45	OK
No. 16	0	61	54.1	45.9	28.0	20-35	OK
No. 30	0	37	54.1	45.9	17.0	12-25	OK
No. 50	0	20	54.1	45.9	9.2	5-15	OK
No. 100	0	7	54.1	45.9	3.2	1-8	OK
No. 200	0	3	54.1	45.9	1.4	0-4	OK

Example Calculation (for No. 4 sieve):
(0.541 x *1) + (0.459* x *100) = 46.4%*

The resulting gradation analysis would be plotted out as shown:

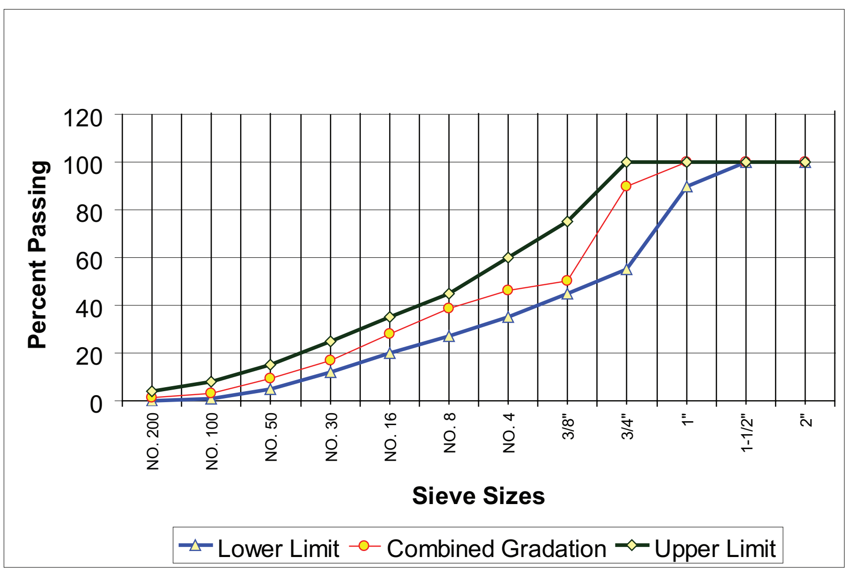

Figure 3-2. Aggregate Gradation Chart - Binary Mix.

Ternary Mix Design Check

Example #2 uses the following mix design for stem and soffit concrete with freeze-thaw exposure.

ARNIE'S REDI-MIX

A DIVISION OF MANPREET COMPANY, INC.

SHEET 1 OF 2

Date 5/23/2010

DESIGN OF CONCRETE MIX

Report To:	Michael Knight Construction	Project	CA. NO. 06-986524
	1564 Kitt Way	Location	Hwy 180, Beaver Dam Bridge
	Big Bend, CA 96532		PM 15.24

MATERIAL AND LABORATORY DATA

AGGREGATE SOURCE: F.A. Aggregrate Products C.A. Aggregrate Products

A.S.T.M.	C-29	C-127	C-40	C-117	C-127
AGGREGRATE	WEIGHT PER CU.FT.	SPECIFIC GRAVITY	ORGANIC	DECANTATION	ABSORPTION
1" x No. 4		2.67			1.7
3/8" x No. 8		2.65			2.3
Sand		2.62			2.5

MECHANICAL ANALYSES PERCENT PASSING U.S. STANDARD SIEVES

	1 1/2"	1"	3/4"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	F.M.
1" x No. 4	100	100	78	8	2						7.12
3/8" x No. 8		100	100	91	17	3					5.89
Sand				100	100	84	60	38	20	8	2.9
Combined	100	100	91	60	46	37	26	16	9	3	5.12

Cementitious Factor(lb/yd^3)	600 lb/yd^3(Cementitious)	Water/Cementitious Ratio	0.505
Minimum 28 Day Strength	4000 psi	Penetration	2" Max
		Admixtures	Darex II
Entrained Air Required	3.0%(Optional)	Cement Type	II - V

	Material	Percentage Parts	Quantities	
Mix No. CT-600-7F Location: Stem/Soffit			lbs	Based upon aggregates in saturated surface dry condition. Correction necessary for free moisture on aggregates
	1" x No. 4	42.0	1174	
	3/8" x No. 8	15.0	416	
	Sand	43.0	1180	
	Flyash		125	
	GGBFS		175	
	Cement		300	
	Total Water		303	
	Admixture	per mfr.		
	Totals	100.0	3673	

This mix will produce concrete meeting the design criteria when produced, sampled, and tested in accordance with ASTM C-94

Respectfully Submitted

By ______________________

Jeremy Jessup, Manager

Figure 3-3. Example Concrete Mix Design - Ternary Mix.

ARNIE'S REDI-MIX

A DIVISION OF MANPREET COMPANY, INC.

SHEET 2 OF 2

Date 5/23/2010

Report To: Michael Knight Construction
1564 Kitt Way
Big Bend, CA 96532

Project CA. NO. 06-986524
Location Hwy 180, Beaver Dam Bridge
PM 15.24

PROPOSED GRADATION OF PRIMARY AGGREGATE NOMINAL SIZES
PORTLAND CEMENT CONCRETE

AGGREGATE SOURCE - Aggregate Products

In Compliance with Section 90-1.02C(4), Aggregate Gradings, of the 2010 Standard Specifications the undersigned proposes to use Portland Cement Concrete Aggregate as follows:

	1.5" x 3/8" (Primary Size)			1" x No. 4" (Primary Size)			Fine Aggregates			Combined Gradation	
Individual Sizes Proposed Percentages	SRM Proposed Gradation	Caltrans Ind. Agg. Limits	Caltrans Operating Range	SRM Proposed Gradation	Caltrans Ind. Agg. Limits	Caltrans Operating Range	SRM Proposed Gradation	Caltrans Ind. Agg. Limits	Caltrans Operating Range	SRM Proposed Gradation	Caltrans Operating Range
Sieve Size											
2"											100
1.5"						100					100
1"				100		88-100				100	90-100
3/4"				X=80	52-85	X+-15				91	55-100
3/8"				X=30	15-38	X+-15			100	60	45-75
No. 4				6		0-16	100		95-100	46	35-60
No. 8				1		0-6	84		65-95	37	27-45
No. 16							X=60	55-75	X+-10	26	20-35
No. 30							X=38	34-46	X+-9	16	12-25
No. 50							X=20	16-29	X+-6	9	5-15
No. 100							8		2-10	3	1-8
No. 200							3.5		0-5	1.3	0-4

Respectfully Submitted

By ______________________________
Jeremy Jessup, Manager

Figure 3-3. Example Concrete Mix Design - Ternary Mix (continued).

Given:

Material Source:

Portland cement = Cemex, Victorville Plant Type II/V
Class F Fly Ash = Headwaters Resources Inc., Bridger Power Plant Class F Fly Ash
GGBFS = LaFarge North America, Seattle Plant, Grade 100
Coarse Aggregate = (1″ x No. 4) = J.F. Shea Aggregate Products
Coarse Aggregate = (3/8″ x No. 8) = J.F. Shea Aggregate Products
Fine Aggregate = J.F. Shea Aggregate Products

Initial Material(s) Check

Is Portland cement on Authorized Materials List?	YES
• Tricalcium Silicate Content ≤ 65%?	YES
• ≤ 0.60% by mass of alkalies?	YES
• Autoclave Expansion ≤ 0.50%?	YES
Is Class F Fly Ash on Authorized Materials List?	YES
• Calcium Oxide Content ≤ 15%?	YES
Is GGBFS on Authorized Materials List?	YES
Are Coarse Aggregate sources SMARA listed?	YES (SMARA 91-45-0019)
Are Coarse Aggregates on Innocuous Aggregate List?	NO
• (CT 214) ≤ 10% loss via Soundness test	YES
• (CT 211) Los Angeles Rattler ≤ 45%	YES
• (CT 227) Cleanness ≥ 75	YES
Is Fine Aggregate source SMARA listed?	YES (SMARA 91-45-0019)
Is Fine Aggregate on Innocuous Aggregate List?	NO
• (CT 213) Organic Impurities = "Satisfactory"	YES
• (CT 217) Sand Equivalent ≥ 75	YES
Is Air-Entraining Admixture (Darex II AEA) on List?	YES

Given:

Type II-V Portland cement = 375 lb/yd^3 (S.G. = 3.15)
GGBF Slag = 175 lb/yd^3 (S.G. = 2.90)
Fly Ash, Class F (CaO content 10%) = 125 lb/yd^3 (S.G. = 2.30)
Aggregate Type =Non-Innocuous

Check the Total Cementitious Material Content (TCMC)

Minimum cementitious (MC) content via the Standard Specifications for stem/soffit concrete = 590 lb/yd^3

$$MC = 590\ lb/yd^3$$
$$TCMC = 375\ lb/yd^3 + 125\ lb/yd^3 + 175\ lb/yd^3 = 675\ lb/yd^3$$
$$675\ lb/yd^3 > 590\ lb/yd^3$$

OK

Check Equation #1 for General Concrete:

$$\frac{(25 \times UF) + (12 \times FA) + (10 \times FB) + (6 \times SL)}{MC} \geq X$$

UF = 0 (No Silica Fume used in mix)
FA = 125 lb/yd^3
FB = 0 (No Fly Ash or Natural Pozzolan used with CaO content > 10% and < 15% in mix)
SL = 175 lb/yd^3
MC = 590 lb/yd^3
X = 3.0 for non-innocuous aggregate

$$\frac{(25 \times 0) + (12 \times 125) + (10 \times 0) + (6 \times 175)}{590} = 4.32$$

$$4.30 \geq 3.00$$

OK

Calculate the Minimum Supplementary Cementitious Material (MSCM) to be used in Equation #2

Note: In order to calculate the MSCM of Equation #1 multiple iterations are necessary. To simplify this calculation start from left to right and enter the SCM values, up to the actual amount in the mix until the left side of the equation is equal to the required X value (X = 3.0 in this case). Equation #1 is intentionally set up with the largest multiplication factor on the left and the remaining factors follow in descending size (25, then 12, then 10 and finally 6). When checking a mix design, the minimum supplementary cementitious material (MSCM) can only be obtained by working sequentially from the left of the equation.

Equation #1

$$\frac{(25 \times UF) + (12 \times FA) + (10 \times FB) + (6 \times SL)}{MC} \geq X$$

Enter in the amount of UF up to the actual (in this case 0 lb/yd^3)

$$\frac{(25 \times 0)}{590} = 0$$

Enter in the amount of FA up to the actual (in this case 125 lb/yd^3)

$$\frac{(25 \times 0) + (12 \times 125)}{590} = 2.54$$

Since 2.54 is less than 3.0 continue moving left to right to satisfy the equation:
Enter in the amount of FB up to the actual (in this case 0 lb/yd^3)

$$\frac{(25\times 0)+(12\times 125)+(10\times 0)}{590} = 2.54$$

Continue moving left to right to satisfy the equation:
Enter in the amount of SL up to the actual (in this case 175 lb/yd^3)

$$\frac{(25\times 0)+(12\times 125)+(10\times 0)+(6\times 175)}{590} = 4.32$$

4.32 > X = 3.0 so the GGBFS quantity needs to be adjusted so the equation = 3.0.
Always solve for SL to obtain the MSCM value.

$$6 \times SL = (3.0 \times 590) - (25 \times 0) - (12 \times 125) - (10 \times 0)$$

$$SL = \frac{(3.0\times 590)-(25\times 0)-(12\times 125)-(10\times 0)}{6}$$

$$SL = 45$$

MSCM = 125 lb/yd^3 + 45 lb/yd^3 = 170 lb/yd^3

Check Equation #2

MC = 590 lb/yd^3
MSCM = 170 lb/yd^3
PC (total quantity of Portland cement) = 375 lb/yd^3

590 - 170 - 375 = 120 lb/yd^3

45 lb/yd^3 > 0

OK

Check Equation #3 for Freeze Thaw Requirement:

$$\frac{(41\times UF)+(19\times F)+(11\times SL)}{TC} \leq 7.0$$

UF = 0 (No Silica Fume used in mix)
F = 125 lb/yd^3
SL = 175 lb/yd^3
TC (total quantity of cementitious material in mix) = 675 lb/yd^3

$$\frac{(41\times0)+(19\times125)+(11\times175)}{675} = 6.37$$

$$6.4 < 7.0$$

OK

Free Water

The allowable amount of free water per the Standard Specifications is 310 lb/yd^3 plus 20 lb/yd^3 for each 100 lb of cementitious material in excess of 550 lb/yd^3. Calculate the allowable amount of free water:

$$310 + \frac{(675\text{-}550)}{100}\times 20 = 335$$

Total allowable amount of free water = *335 lb/yd³*

$$335\ lb/yd^3 > 303\ lb/yd^3$$

OK

Gradation Check

Given:

Aggregate Sizes

- Coarse Aggregate #1 = 1″ x No. 4
- Coarse Aggregate #2 = 3/8″ x No. 8
- Fine Aggregate = Sand

Check to see if the submitted X-values for the primary aggregate sizes are in compliance with the contract limits of proposed gradation.

Coarse Aggregate (1″x No. 4)

Sieve Size	Limits of Proposed Gradation	Submitted X-Value	Status
3/4″	52 - 85	80	OK
3/8″	15 - 38	30	OK

Fine Aggregate (Sand)

Sieve Size	Limits of Proposed Gradation	Submitted X-Value	Status
No. 16	55 - 75	60	OK
No. 30	34 - 46	38	OK
No. 50	16 - 29	20	OK

Next check the coarse aggregate grading. All values must lie within the operating range given in the specifications.

The proportions of each aggregate size can be calculated as follows:

Coarse Aggregate #1 = (1,174 lb/2.67) / ((1,174 lb/2.67) + (416 lb/2.65) + (1,180 lb/2.62)) = 42%
Coarse Aggregate #2 = (416 lb/2.65) / ((1,174 lb/2.67) + (416 lb/2.65) + (1,180 lb/2.62)) = 15%
Fine Aggregate = (1,180 lb/2.62) / ((1,174 lb/2.67) + (416 lb/2.65) + (1,180 lb/2.62)) = 43%

Coarse Aggregates

Calculations for Combined Gradings of Coarse Aggregates

- $1.5'' \text{ Sieve} = \frac{(100\% \times 42\%) + (100\% \times 15\%)}{(42\% + 15\%)} = 100\%$
- $1'' \text{ Sieve} = \frac{(100\% \times 42\%) + (100\% \times 15\%)}{(42\% + 15\%)} = 100\%$
- $3/4'' \text{ Sieve} = \frac{(78\% \times 42\%) + (100\% \times 15\%)}{(42\% + 15\%)} = 83.8\%$
- $3/8'' \text{ Sieve} = \frac{(8\% \times 42\%) + (91\% \times 15\%)}{(42\% + 15\%)} = 29.8\%$
- $\text{No. 4 Sieve} = \frac{(2\% \times 42\%) + (17\% \times 15\%)}{(42\% + 15\%)} = 5.9\%$
- $\text{No. 8 Sieve} = \frac{(0\% \times 42\%) + (3\% \times 15\%)}{(42\% + 15\%)} = 0.8\%$

Table 3-5. Coarse Aggregate Grading Checklist - Ternary Mix.

Sieve Size	X-Value	Percentage Passing Based On X-Value	Percentage Passing Operating Range Limits	Contractor	Status
1.5″			100	100	OK
1″			88 - 100	100	OK
3/4″	80	X +/- 15	65 - 95	83.8	OK
3/8″	30	X +/- 15	15 - 45	29.8	OK
No. 4			0 - 16	5.9	OK
No. 8			0 - 6	0.8	OK

<u>Fine Aggregate (Sand)</u>

Table 3-6. Fine Aggregate Grading Checklist - Ternary Mix.

Sieve Size	X-Value	Percentage Passing Based On X-Value	Percentage Passing Operating Range Limits	Contractor	Status
3/8″			100	100	OK
No. 4			95 - 100	100	OK
No. 8			65 - 95	84	OK
No. 16	60	X +/- 10	50 - 70	60	OK
No. 30	38	X +/- 9	29 - 47	38	OK
No. 50	20	X +/- 6	14 - 26	20	OK
No. 100			2 - 12	8	OK
No. 200			0 - 8	0	OK

Also, the Specifications state that in addition to the above required grading analysis, the fine aggregate sizes must be distributed such that:

- Difference between the total percentage passing the No. 16 and No. 30 Sieves is from 10 to 40 (Check = 60 - 38 = 22) ⇒ OK
- Difference between the percentage passing the No. 30 and No. 50 Sieves is from 10 to 40 (Check = 38 - 20 = 18) ⇒ OK

Next you need to check the combined gradation of the aggregates. You will need the percentage of volume of each primary aggregate size.

Table 3-7. Combined Aggregate Grading Checklist - Ternary Mix.

Sieve Size	Percentage Passing Coarse Aggregate	Percentage Passing Fine Aggregate	Mix Percentage of Coarse Aggregate	Mix Percentage of Fine Aggregate	Combined Total Percentage Passing	Specification Limits	Status
1.5″	100	100	57	43	100.0	100	OK
1″	100	100	57	43	100.0	90 - 100	OK
3/4″	83.8	100	57	43	90.8	55 - 100	OK
3/8″	29.8	100	57	43	60.0	45 - 75	OK
No. 4	5.9	100	57	43	46.4	35 - 60	OK
No. 8	0.8	84	57	43	36.6	27 - 45	OK
No. 16	0	60	57	43	25.8	20 - 35	OK
No. 30	0	38	57	43	16.3	12 - 25	OK
No. 50	0	20	57	43	8.6	5 - 15	OK
No. 100	0	8	57	43	3.4	1 - 8	OK
No. 200	0	3.5	57	43	1.5	0 - 4	OK

Example Calculation:
Combined total for the 3/8″ sieve = (29.8 x 0.57) + (100 x 0.43) = 60%

The resulting combined gradation analysis would be plotted out as shown:

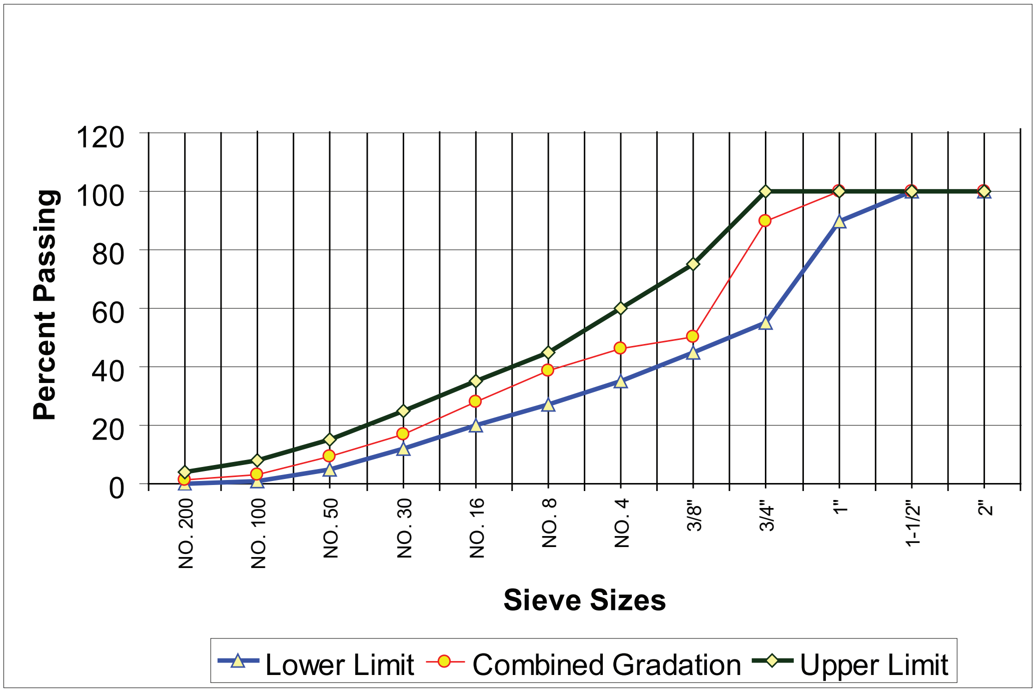

Figure 3-4. Aggregate Gradation Chart - Ternary Mix.

High Strength Binary Mix in Corrosive Enviroment

Example #3 uses the following mix design:

Below are listed current proportions for the mix as produced from the production facility scheduled to service the project referenced. Note that adjustments to proportions may be necessary to maintain contract performance in the case of delivery from another production location or in the event of material changes necessitated by supply issues.

Mix Description : SUPERSTRUCTURE **W/C Ratio :** 0.29

Design Strength : 8000 psi **Slump :** 5.00 in +/-1.00

Usage : Superstructure Frames

	MATERIAL	SSD	VOLUME
CEMENT :	Hanson Permanente Type II/V	690	3.51
FLYASH :	Headwaters Bridger Fly Ash	229	1.71
SAND :	Hanson Sechelt Sand	998	6.03
AGGREGATE :	Hanson 1" X #4 Sechelt	1318	7.76
AGGREGATE :	Hanson 1/2" Sechelt	564	3.36
AIR :	1.50 % +/-1.00		0.41
WATER :	ASTM C1602-04,C1603-04	263	4.21
ADMIXTURE :		**4062**	**27.00**

ASTM C-494 (G-3000) TYPE F 7.0 ozs./cwt +/- 3

ASTM C-494 TYPE B & D 4.0 ozs./cwt +/- 2

Tetraguard 7.0 ozs./cwt

Concrete Aggregates - Gradation Analysis

Percent Passing								
CA & FA >	**CA # 1**	**CA # 2**	**CA # 3**	**CA # 4**	**FA # 1**	**FA # 2**	**FA # 3**	**Combined**
Sieve Size	**45.80 %**	**19.60 %**			**34.70 %**			**100 %**
2"	100	100			100			100 %
1 1/2"	100	100			100			100 %
1"	100	100			100			100 %
3/4"	85	100			100			93 %
1/2"	40	90			100			71 %
3/8"	25	60			100			58 %
#4	5	7			100			38 %
#8	2	2			87			31 %
#16	0	0			68			24 %
#30	0	0			45			16 %
#50	0	0			20			7 %
#100	0	0			5			2 %
#200	0	0			1			0 %
F. M.								
Specific Gravity	2.72	2.69			2.65			

X-VALUES

COARSE AGGREGATE

¾" = 85

3/8" = 25

FINE AGGREGATE

No. 16 = 68

No. 30 = 45

No. 50 = 20

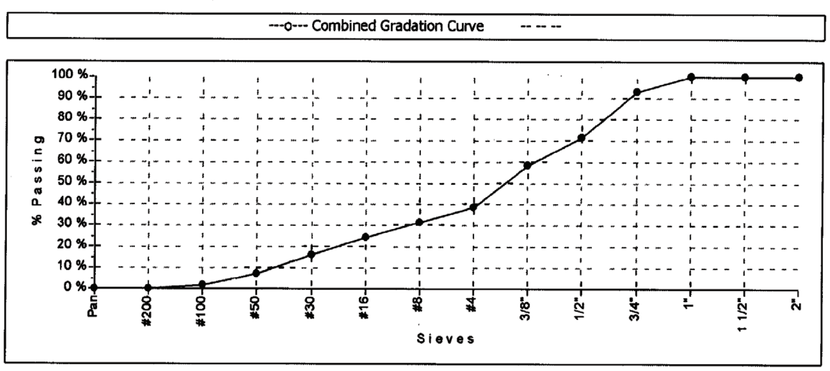

Figure 3-5. Example Concrete Mix Design - High Strength Binary Mix.

<u>Given:</u>

Material Sources:

Portland cement = Hanson, Permanente Plant Type II/V
Class F Fly Ash = Headwaters Resources Inc., Bridger Power Plant Class F Fly Ash
Coarse Aggregates = Hanson, Sechelt (British Columbia, Canada)
Fine Aggregate = Hanson, Sechelt (British Columbia, Canada)

Initial Material(s) Check

Is Portland cement on Authorized Materials List?	YES
• Tricalcium Silicate Content ≤ 65%?	YES
• ≤ 0.60% by mass of alkalies?	YES
• Autoclave Expansion < = 0.50%?	YES
Is Class F Flyash on Authorized Materials List?	YES
• Calcium Oxide Content ≤ 15%?	YES
Is GGBFS on Authorized Materials List?	YES
Are Coarse Aggregate sources SMARA listed?	N/A (IMPORTED FROM BC, CANADA)
Are Coarse Aggregates on Innocuous Aggregate List?	NO
• (CT 214) ≤ 10% loss via soundness test	YES
• (CT 211) Los Angeles Rattler ≤ 45%	YES
• (CT 227) Cleanness ≥ 75	YES
Is Fine Aggregate source SMARA listed?	N/A (IMPORTED FROM BC, CANADA)
Is Fine Aggregate on Innocuous Aggregate List?	NO
• (CT 213) Organic Impurities = "Satisfactory"	YES
• (CT 217) Sand Equivalent ≥ 75	YES
Is Type F Admixture on List?	YES
Is Type B&D Admixture on List?	YES
Is Type S Admixture (Tetraguard) on List?	YES
Shrinkage Information (AASHTO T 160) submitted/met?	YES

<u>Given:</u>

Type II/V Portland cement = 690 lb/yd^3 (S.G. = 3.15)
Class F Fly Ash, (CaO content 6%) = 229 lb/yd^3 (S.G. = 2.15)
Aggregate Type = Innocuous
Type F High Range Water Reducer Used
Type B, C Retarding and Water Reducing Admixture Used
Shrinkage Reducing Admixture Used (Tetraguard)

Notes:

- Shrinkage admixture used to control shrinkage due to high cementitious content of mix
- Note the low water - cementitious ratio of 0.29
- Retarders and water reducers necessary due to high cementitious content and low water - cementitious ratio

Calculate the Total Cementitious Material Content (TC)
Minimum cementitious (MC) content via the Standard Specifications for concrete in corrosive environment = 675 lb/yd^3. In this particular application there was no maximum given for the amount of cementitious material that could be used. The optional reduction in cementitious material (when using a water reducer or water reducing/retarding admixture) is not allowed for concrete in a corrosive enviroment.

$$MC = 675\ lb/yd^3$$

$$TC = 690\ lb/yd^3 + 229\ lb/yd^3 = 919\ lb/yd^3$$
$$919\ lb/yd^3 > 675\ lb/yd^3$$

OK

In a corrosive enviroment cementitious material must be comprised of one of the following via the Standard Specifications:

- Twenty-five percent of either Fly Ash or natural pozzolan with a calcium oxide (CaO) content of up to 10%, and 75% Portland cement
- Twenty percent of either Fly Ash or natural pozzolan with a calcium oxide (CaO) content of up to 10%, 5% silica fume, and 75% Portland cement
- Twelve percent of silica fume, metakaolin, or ultra fine Fly Ash (UFFA); and 88% Portland cement
- Fifty percent ground granulated blast furnace slag (GGBFS) and 50% Portland cement

This mix uses Fly Ash with a CaO content < 10%. There is no silica fume or GGBFS used in this mix thus #2 through #4 do not apply. Check to see if the mix complies with the #1 requirement that the total cementitious material shall consist of 25% Fly Ash by weight.

$$\text{Total Fly Ash \%} = \frac{229}{(229+690)} = 25\%$$

OK

Verify the Allowed Strength Development Time

A total of 56 days is allowed to obtain the required compressive strength if the following equation is met:

$$\frac{(41\times UF)+(19\times F)+(11\times SL)}{TC} \geq 7.0$$

UF = 0 (No Silica Fume used in mix)
F = 229 lb/yd^3
SL = 0 lb/yd^3
TC (total quantity of cementitious material in mix) = 919 lb/yd^3

$$\frac{(41\times 0)+(19\times 229)+(11\times 0)}{919} = 4.7$$

$$4.70 < 7.0$$

NOT OK

Thus the 56-day total is not allowed to gain the specified strength. The contractor must abide by the specified time allowance of the contract.

Free Water
The allowable amount of free water per the Standard Specifications is 310 lb/yd^3 plus 20 lb/yd^3 for each 100 lb of cementitious material in excess of 550 lb/yd^3. Calculate the allowable amount of free water:

$$310 + \left(\frac{(919\text{-}550)}{100}\right)\times\ 20 = 384$$

Total allowable amount of free water = *384 lb/yd^3*
384 lb/yd^3 > 263 lb/yd^3
OK

Gradation Check
Given:
Aggregate Sizes
- Coarse Aggregate #1 = 1″ x No. 4
- Coarse Aggregate #2 = 1/2″ x No. 4
- Fine Aggregate = Sand

Check to see if the contractor submitted X-values for the primary aggregate sizes and if they are in compliance with the contract limits of proposed gradation.

Coarse Aggregate (1″x No. 4)

Sieve Size	Limits of Proposed X-value	Submitted X-value	Status
3/4″	52 - 85	85	OK
3/8″	15 - 38	25	OK

Fine Aggregate (Sand)

Sieve Size	Limits of Proposed X-value	Submitted X-value	Status
No. 16	55 - 75	68	OK
No. 30	34 - 46	45	OK
No. 50	16 - 29	20	OK

Next check the coarse aggregate grading. All values must lie within the operating range given in the specifications in order to approve the mix design.

The proportions of each aggregate size can be calculated as follows:

Coarse Aggregate #1 = (1,318 lb/2.72) / ((1,318 lb/2.72) + (564 lb/2.69) + (998 lb/2.65)) = 45.2%
Coarse Aggregate #2 = (564 lb/2.69) / ((1,318 lb/2.72) + (564 lb/2.69) + (998 lb/2.65)) = 19.6%
Fine Aggregate = (998 lb/2.65) / ((1,318 lb/2.72) + (564 lb/2.69) + (998 lb/2.65)) = 35.2%

Coarse Aggregates

Calculations for Combined Gradings of Coarse Aggregates

- 1.5″ Sieve = $\frac{(100\%\times 45.2\%)+(100\%\times 19.6\%)}{(45.2\%+19.6\%)}$ = 100%
- 1″ Sieve = $\frac{(100\%\times 45.2\%)+(100\%\times 19.6\%)}{(45.2\%+19.6\%)}$ = 100%
- 3/4″ Sieve = $\frac{(85\%\times 45.2\%)+(100\%\times 19.6\%)}{(45.2\%+19.6\%)}$ = 90%
- 3/8″ Sieve = $\frac{(25\%\times 45.2\%)+(60\%\times 19.6\%)}{(45.2\%+19.6\%)}$ = 36%
- No. 4 Sieve = $\frac{(5\%\times 45.2\%)+(7\%\times 19.6\%)}{(45.2\%+19.6\%)}$ = 6%
- No. 8 Sieve = $\frac{(2\%\times 45.2\%)+(2\%\times 19.6\%)}{(45.2\%+19.6\%)}$ = 2%

Table 3-8. Coarse Aggregate Grading Checklist - High Strength Binary Mix.

Sieve Size	X-Value	Percentage Passing Based On X-Value	Percentage Passing Operating Range Limits	Contractor	Status
1.5″			100	100	OK
1″			88 - 100	100	OK
3/4″	85	X +/- 15	70 - 100	90	OK
3/8″	25	X +/- 15	10 - 40	36	OK
No.4			0 - 16	6	OK
No.8			0 - 6	2	OK

<u>**Fine Aggregate (Sand)**</u>

Table 3-9. Fine Aggregate Grading Checklist - High Strength Binary Mix.

Sieve Size	X-Value	Percentage Passing Based On X-Value	Percentage Passing Operating Range Limits	Contractor	Status
3/8″			100	100	OK
No. 4			95 - 100	100	OK
No. 8			65 - 95	87	OK
No. 16	68	X +/- 10	58 - 78	68	OK
No. 30	45	X +/- 9	36 - 54	45	OK
No. 50	20	X +/- 6	14 - 26	20	OK
No. 100			2 - 12	5	OK
No. 200			0 - 8	1	OK

Also, the Specifications state that in addition to the above required grading analysis, the fine aggregate sizes must be distributed such that:

- Difference between the total percentage passing the No. 16 and No. 30 Sieves is from 10 to 40 (Check = 68 - 45 = 23) ⇒ OK
- Difference between the percentage passing the No. 30 and No. 50 Sieves is from 10 to 40 (Check = 45 - 20 = 25) ⇒ OK

Next you need to check the combined gradation of the aggregates. You will need the percentage of volume of each primary aggregate size.

Table 3-10. Combined Aggregate Grading Checklist - High Strength Binary Mix.

Sieve Size	Percentage Passing Coarse Aggregate	Percentage Passing Fine Aggregate	Mix Percetage of Fine Aggregate	Mix Percentage of Coarse Aggregate	Combined Total Percentage Passing	Total Percentage Limits	Specification Status
1.5″	100	100	64.8	35.2	100.0	100	OK
1″	100	100	64.8	35.2	100.0	90-100	OK
3/4″	90	100	64.8	35.2	93	55-100	OK
3/8″	36	100	64.8	35.2	58	45-75	OK
No. 4	6	100	64.8	35.2	39	35-60	OK
No. 8	2	87	64.8	35.2	32	27-45	OK
No. 16	0	68	64.8	35.2	24	20-35	OK
No. 30	0	45	64.8	35.2	16	12-25	OK
No. 50	0	20	64.8	35.2	7	5-15	OK
No. 100	0	5	64.8	35.2	2	1-8	OK
No. 200	0	1	64.8	35.2	0.4	0-4	OK

Example Calculation:
Combined total for the 3/8″ sieve = (36 x 0.648) + (100 x 0.352) = 58%

The resulting combined gradation analysis would be plotted as shown:

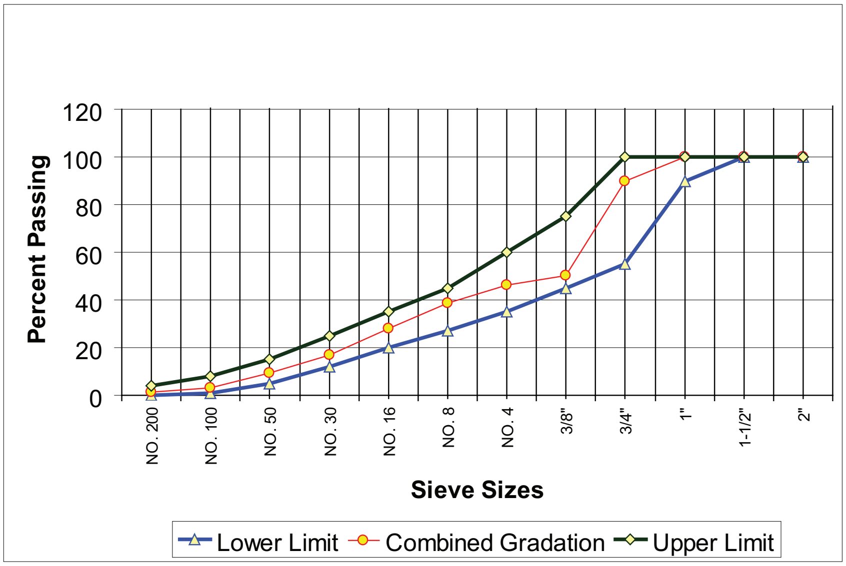

Figure 3-6. Aggregate Gradation Chart - High Strength Binary Mix.

Ternary Mix Design Check (Freeze-Thaw Region with Exposure to De-Icing Chemicals)

This example does not include the following mix design requirements:

- Authorized Materials List Verification and physical/chemical test requirements for the following:
  - Cementitious Materials
  - Innocuous Aggregate
  - Chemical and Air-Entraining Admixtures
- Aggregate Gradation Analysis
- SMARA Verification for Aggregates
- Free Water Check

Note: See Examples #1 through #3 for more information regarding the above topics. This example is solely provided to illustrate the equations used when concrete is designated as being exposed to freeze-thaw condition with exposure to de-icing chemicals.

Given:
Type II Portland cement = 375 lb/yd^3 (S.G. = 3.15)
Silica Fume = 50 lb/yd^3 (S.G. = 2.40)
Fly Ash, Class F (CaO content 14%) = 140 lb/yd^3 (S.G. = 2.30)
Aggregate Type = Innocuous
Air-Entrainment = 6 $\pm$ 1.5%
Water Reducing Agent used (Type A)
28-day Compressive Strength Requirement = 3,625 psi

Calculate the Total Cementitious Material Content (TC)

$$TC = 375 + 50 + 140 = 565\ lb/yd^3$$

Minimum cementitious (MC) content via the Standard Specifications for freeze-thaw regions is 590 lb/yd^3. If a water reducing admixture is used, a 5% by weight reduction of cementitious material content is allowed via the Standard Specifications if the mix dosage meets or exceeds the dosage used in determining approval of the admixture. In this case we will assume the dosage requirement is met, thus a 5% reduction is allowed.

$$MC = 0.95\ (590\ lb/yd^3) = 561\ lb/yd^3$$
$$565\ lb/yd^3 > 561\ lb/yd^3$$

<u>**OK**</u>

For concrete designated as exposed to de-icing chemicals Equations 1 through 5 must be satisfied:

Check Equation #1

$$\frac{(25\times UF)+(12\times FA)+(10\times FB)+(6\times SL)}{MC} \geq \boldsymbol{X}$$

UF = 50 lb/yd^3
FA = 0 (No Fly Ash or Natural Pozzolan used with CaO content $\leq$ 10% in mix)
FB = 140 lb/yd^3
SL = 0 (No ground granulated blast furnace slag in mix)
TC = 565 lb/yd^3
MC = 561 lb/yd^3
X = 1.8 for innocuous aggregate

$$\frac{(25\times 50)+(12\times 0)+(10\times 140)+(6\times 0)}{565} = 4.7 \geq 1.8,\ OK$$

Check Equation #2

$$\frac{4\times(FA+FB)}{TC} \leq 1.0 \Rightarrow \frac{4\times(0+140)}{565} = 1.0 \leq 1.0,\ OK$$

Check Equation #3

$$\frac{(10\times UF)}{TC} \leq 1.0 \Rightarrow \frac{(10\times 50)}{565} = 0.90 \leq 1.0,\ OK$$

Check Equation #4

$$\frac{2\times(UF+FA+FB+SL)}{TC} \leq 1.0 \Rightarrow \frac{2\times(50+0+140+0)}{565} = 0.70 \leq 1.0,\ OK$$

Check Equation #5

$$\frac{27\times(TC\text{-}MC)}{MC} \leq 5.0 \Rightarrow \frac{27\times(565\text{-}561)}{561} = 0.20 \leq 5.0,\ OK$$

Verify the allowed Strength Development Time per SSP 90-1.02A
For concrete in freeze-thaw regions the qualification for a 56-day allowable strength gain time period does not apply. Thus, the contractor must abide by the allotted time allowance of the contract.

Minor Concrete Binary Mix

This example does not include the following mix design requirements:

- Authorized Materials List Verification and physical/chemical test requirements for the following:
  - Cementitious Materials
  - Innocuous Aggregate
  - Chemical and Air-Entraining Admixtures
- Aggregate Gradation Analysis
- SMARA Verification for Aggregate
- Free Water Check

Note: See Examples #1 through #3 for more information regarding the above topics. This example is solely provided to illustrate the use of equation requirements.

<u>**Given:**</u>
Type II Portland cement = 411 lb/yd^3 (S.G. = 3.15)
Rice Hull Ash = 137 lb/yd^3 (S.G. = 2.10)
Aggregate Type = Innocuous

Air Entrainment = 1.5 +/- .5%

Calculate the Total Cementitious Material Content (TC)
Minimum cementitious (MC) content via the Standard Specifications for Minor Concrete = 505 lb/yd³.

$$MC = 505\ lb/yd^3$$

$$TC = 411\ lb/yd^3 + 137\ lb/yd^3 = 548\ lb/yd^3$$
$$548\ lb/yd^3 > 505\ lb/yd^3$$

OK

Check Equation #1

$$\frac{(25 \times UF)+(12 \times FA)+(10 \times FB)+(6 \times SL)}{MC} \geq X$$

UF = 137 lb/yd³ (Specifications state Rice Hull Ash to be considered Type UF if used)
FA = 0 (No Fly Ash or Natural Pozzolan used with CaO content $\leq$ 10% in mix)
FB = 0 (No Fly Ash or Natural Pozzolan used with CaO content $\geq$ 10% and $\leq$ 15% in mix)
SL = 0 (No ground granulated blast furnace slag in mix)
MC = 505 lb/yd³
X = 1.8 for non-innocuous aggregate

$$\frac{(25 \times 137)+(12 \times 0)+(10 \times 0)+(6 \times 0)}{505} = 6.8$$

$$6.80 \geq 1.80$$

OK

Calculate the Minimum Supplementary Cementitious Material (MSCM) to be used in Equation #2
Note: In order to calculate the MSCM of Equation #1 iterations are necessary. To simplify this calculation start from left to right and enter the SCM values, up to the actual amount in the mix until the left side of the equation is equal to the required X value (X = 1.8 in this case)

Equation #1

$$\frac{(25\times UF)+(12\times FA)+(10\times FB)+(6\times SL)}{MC} \geq X$$

Enter in the amount of UF up to the actual (in this case 137 lb/yd^3)

$$\frac{(25\times 137)}{505} = 6.8$$

6.8 > X = 1.8 so the Rice Hull Ash (UF) quantity needs to be adjusted so the equation = 1.8. Solve for UF to obtain the MSCM value.

$$25 \times UF = (1.8 \times 505) - (12 \times 0) - (10 \times 0) - (6 \times 0)$$

$$UF = \frac{(1.80\times 505)-(12\times 0)-(10\times 0)-(6\times 0)}{25}$$

$$UF = 36$$

<u>MSCM = 36 lb/yd^3</u>

Check Equation #2

$$MC - MSCM - PC \geq 0$$

MC = 505 lb/yd^3
MSCM = 36 lb/yd^3
PC (total quantity of Portland cement) = 411 lb/yd^3

$$505 - 36 - 411 = 58$$
$$58 \text{ lb/yd}^3 \geq 0$$

<u>OK</u>

Verify the allowed Strength Development Time

A total of 56 days is allowed to obtain the required compressive strength if the following equation is met:

$$\frac{(41\times UF)+(19\times F)+(11\times SL)}{TC} \geq 7.0$$

UF = 137 lb/yd^3 (Specifications state Rice Hull Ash to be considered Type UF if used)
F = 0 (No Fly Ash used in mix)
SL = 0 (No Ground Granulated Blast Furnace Slag used in mix)
TC (total quantity of cementitious material in mix) = 548 lb/yd^3

$$\frac{(41 \times 137)+(19 \times 0)+(11 \times 0)}{548} = 10.3$$

$$10.3 > 7.0$$

OK

Thus the 56-day total is allowed to gain the specified 28-day minimum specified compressive strength.

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Ricketts, Jonathan T., M. Kent Loftin, and Frederick S. Merritt, Standard Handbook for Civil Engineers, 5th ed., McGraw-Hill, 2004

21st Century Concrete Guidelines, State of California Department of Transportation, 2009

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Kosmatka, Steven H., Beatrix Kerkhoff, and William C. Panarese, Design and Control of Concrete Mixtures, 14th ed., Portland Cement Association, 2002

CHAPTER 4
PROPORTIONING, MIXING AND TRANSPORTING

Table of Contents

Proportioning, Mixing And Transporting **4-1**

Introduction **4-1**

Proportioning **4-1**
Aggregate Storage and Handling 4-2
Cementitious Material Storage and Handling 4-7
Weighing and Measuring Equipment 4-8
Manual and Automatic Batching 4-11
Moisture Meters 4-12
Chemical Admixtures 4-14
Controlling Batch Temperature 4-16
Cooling Concrete 4-16
Heating Concrete 4-19

Mixing and Transporting **4-20**
Ready-Mixed Concrete 4-22
Central-Mixed Concrete 4-23
Transit-Mixed Concrete 4-24
Inspection of Mixing Equipment 4-27
Transporting Mixed Concrete 4-27
Time or Amount of Mixing 4-29
On-Site Mixing 4-30

References **4-31**

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4 PROPORTIONING, MIXING AND TRANSPORTING

Introduction

This chapter provides discussion of the contract provisions governing proportioning, mixing and transporting concrete. The objective of the contract provisions is to obtain concrete that is consistent with the accepted mix design (discussed in previous chapter), uniform, homogeneous, workable and in its cured form exhibits the expected structural and aesthetic properties. Experience has shown that the variability within and across batching processes warrants provision for method and quality assurance inspections of process, equipment and materials to assure the quality specified is obtained. Experience has also resulted in the industry's creation of batching, mixing and transporting practices that result in quality, uniform and economical concrete that satisfies the contract provisions. Figure 4-1 shows a typical batch plant.

This chapter discusses the objectives of the provisions contained in the Standard Specifications. The intention is that more thorough understanding of these objectives will improve the bridge engineer's effectiveness in assuring concrete quality and resolving issues of deficiency.

Proportioning

To ensure a quality product, it is essential that the concrete mixture contain the same relative proportions of each ingredient as intended by the mix design. Accurate proportioning is an important element in the concrete production process. In addition, storage and handling of concrete constituent materials can impact how consistent the batched concrete is with the intended mix design.

Figure 4-1. Basic Batch Plant Shows Aggregate, Cement and Supplemental Cementitious Material Storage, Conveyor Feed System, Chemical Admixture Tanks and Dust Collector.

Aggregate Storage and Handling

The provisions for storage and handling are contained in the Standard Specifications. These provisions are warranted because segregation, breakage and contamination can all occur during normal handling and stockpiling operations. Segregation and breakage can occur to such a degree that workability and concrete quality are significantly impacted. Aggregate that is well screened and graded at the aggregate plant may be contaminated, segregated and/or poorly graded by the time it reaches the mixer.

Contamination could be any foreign material but the most common forms of contamination are from materials of different gradations intermingling and from stockpiles (Figure 4-2) that are in contact with the ground. Physical separation between the ground and the stored aggregate and physical separation of materials with different aggregate sizes is the most common prevention.

Figure 4-2. Aggregate Storage Bunkers.

All handling and stockpiling operations will cause some segregation and breakage but good handling and stockpiling practices minimize the impact. If an issue exists or is anticipated, evaluate whether the following practices are being employed.

Reducing the number of times the aggregate is handled and the height from which aggregate is dropped can decrease breakage. Storage bins should be kept as full as practical to minimize breakage and reduce segregation. See Figure 4-3 (a and b) for typical storage bins.

If aggregates are to be stored in stockpiles, some precautions are necessary. To minimize segregation of sizes, stockpiles should be built up in layers of uniform thickness, and the top of the stockpile should be level, not cone-shaped. When building stockpiles, coarse aggregate should fall in such a way that larger particles are not thrown beyond smaller particles, and particles do not run down the stockpile-side slopes.

Figure 4-3a. Aggregate Storage Bins.

Figure 4-3b. Aggregate Storage Bins.

Segregation can be further managed by keeping the aggregate in a number of stockpiles with narrow-grade ranges. Taken to the extreme, each single aggregate size would be stored in its own stockpile. If all aggregate is the same size then it is not possible for segregation to occur. This extreme measure may never be practical but the advantage of separating aggregate stockpiles into narrower grade ranges can be conceptualized. Each time the grading range is reduced the impact of segregation is reduced. Stored in this manner, the final gradation can be created at the time of batching.

Fine aggregate has fewer tendencies to segregate when it is damp. If the fine aggregate is dry and is dropped from a conveyor belt or bucket, even a light wind may blow out the finest particles.

Aggregates from different sources or from different locations in the same pit or material site, should not be placed in the same stockpile. Figure 4-4 provides illustrations of some of the concepts discussed above.

Where crushers are employed to manufacture smaller aggregate sizes, the crushed material may have harsh, sharp edges. This condition can affect the water demand and workability of the concrete but can also increase the potential of excessive undersizing being created as the sharp edges chip off during handling and mixing operations. Historically, it has been considered advisable to limit the amount of angular and friable aggregate in concrete. California Test 515 was used to control these characteristics in fine aggregates used in concrete produced for State projects. One method of managing this issue is by using tumblers. The crushed aggregate is passed through a revolving cylinder with lifters allowing the chip prone particles to be removed prior to its incorporation into the finished concrete aggregate. Also, rock ladders retard the fall and can be utilized to reduce breakage. A rock ladder is illustrated in Figure 4-4. Also, plasticizers and water reducers can be used to reduce the increased water demand.

Because the impacts of utilizing angular and friable aggregates can be mitigated, the Department changed the Standard Specifications to designate all structure concrete by strength and eliminated the California Test 515 requirement. This change provides additional sand sources for concrete suppliers and the compressive strength acceptance tests provide the State with verification that the sand's characteristics have been successfully mitigated.

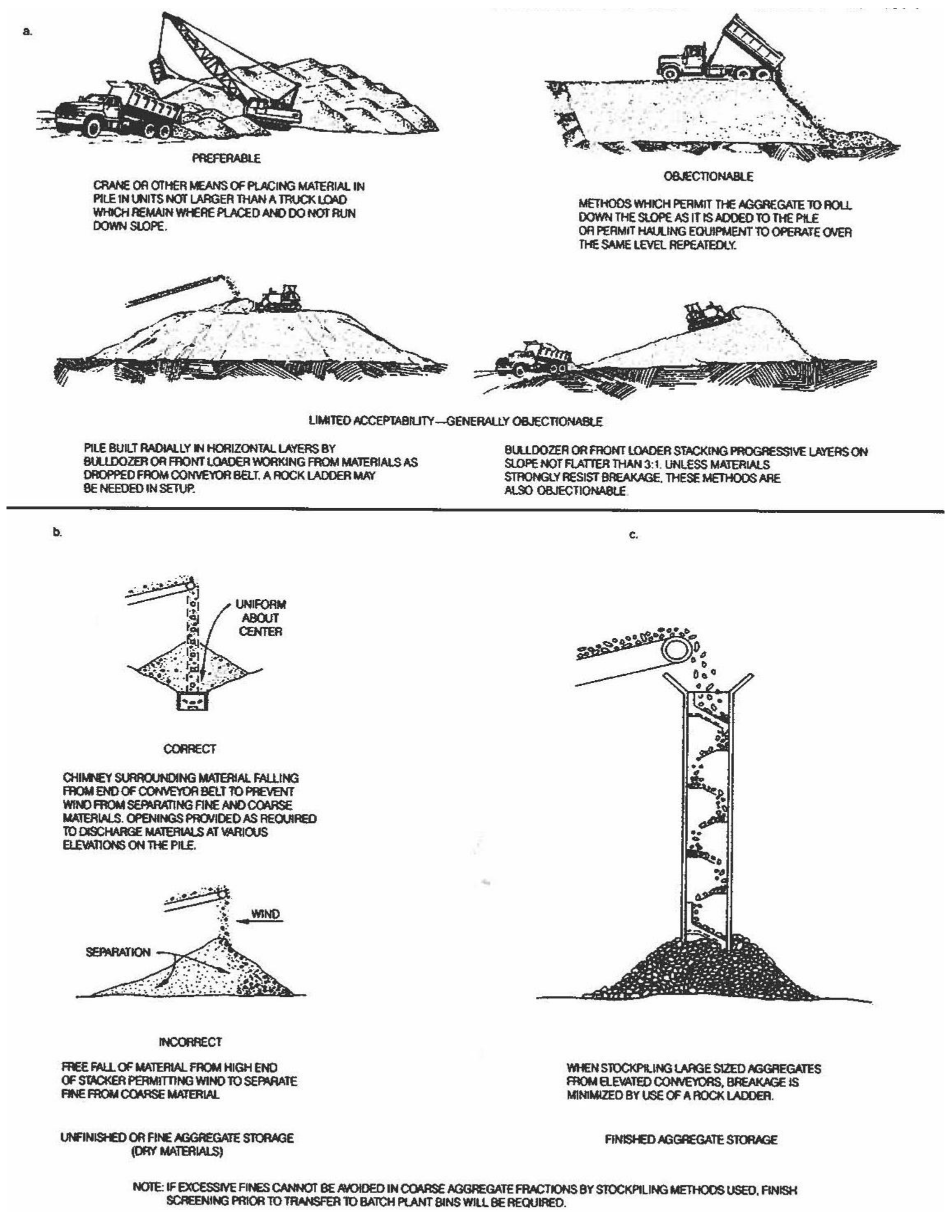

Figure 4-4. Correct and Incorrect Methods of Handling and Storing Aggregates[1].

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[1] ACI Committee 304, Guide for Measuring, Mixing, Transporting and Placing Concrete. *ACI304R-00*

After the initial screening to separate the aggregates into the various primary sizes, in many plants, the aggregate will be handled several times as it is moved from stockpiles to bunkers to the batch plant where it is finally used to produce concrete.

Depending on the amount of handling and the durability of the aggregate particles, significant degradation of the coarse aggregate may take place between the initial screening and the actual batching of the concrete. With a very soft aggregate an excessive amount of fine particles may be generated through handling the coarse aggregate which could result in a combined grading that is out of the grading limits for the No. 100 and No. 200 sieves.

To mitigate the effect of degradation of coarse aggregate, plants can install equipment to remove the undersize particles by rescreening the aggregate immediately prior to batching. This process is known as finish screening.

Preliminary tests like the California Test 211 Abrasion of Coarse Aggregate by Use of the Los Angeles Rattler Machine, California Test 214 Soundness of Aggregates by Use of Sodium Sulfate, and California Test 229 Durability Index, provide assurance that the aggregate is of sufficient quality that, with reasonable care in processing and handling, the aggregate will be in an acceptable condition at the time of incorporation into the batched concrete.

Section 6 of the Construction Manual provides the Department's policy on the size, frequency, and location of sampling and testing of concrete materials. Note that aggregate to be used in structure concrete is sampled at the point of incorporation into the batched concrete. This procedure provides assurance that all the processing and handling of the aggregate has not caused significant segregation, contamination or degradation.

Cementitious Material Storage and Handling

According to ACI Guide for Measuring, Mixing, Transporting, and Placing Concrete, all cement should be stored in weathertight, properly ventilated structures to prevent absorption of moisture. Storage facilities for bulk cement should include separate compartments for each type of cement used. The interior of a cement silo should be smooth, with a minimum bottom slope of 50 degrees from the horizontal for a circular silo and 55 to 60 degrees for a rectangular silo. Silos should be equipped with non-clogging air diffuser flow pads through which small quantities of dry, oil-free, low-pressure air can be introduced intermittently at approximately 3 to 5 psi to loosen cement that has settled tightly in the silos. Storage silos should be drawn down frequently, preferably once per month, to prevent cement caking.

Each bin compartment from which cement is batched should include a separate gate, screw conveyor, air slide, rotary feeder, or other conveyance that effectively allows both constant flow and precise cutoff to obtain accurate batching of cement.

Bags of cement should be stacked on platforms to permit circulation of air. For a storage period of less than 60 days, stack the bags no higher than 14 layers and for longer periods, no higher than 7 layers. As an additional precaution the oldest cement should be used first.

Supplementary cementitious materials should be handled, conveyed, and stored in the same manner as cement. The bins, however, should be completely separate from cement bins without common walls that could allow the material to leak into the cement bin.

Weighing and Measuring Equipment

Concrete governed by the Standard Specification Section 90-1.02F require the aggregates and cementitious material to be proportioned by weight. There are some exceptions to this requirement such as Rapid Strength concrete and polyester concrete, which have other specifications governing their production.

Bulk cement and cementitious material are required to be separate and distinct from aggregate weighing equipment. The quantity of water may be weighed or measured by a water meter.

Proportioning by weight is preferred for its accuracy, flexibility and simplicity. The driving factor for requiring proportioning by weight is a phenomenon known as bulking. Bulking is explained as follows: a volume of moist sand in a loose condition weighs much less than the same volume of dry sand. Sand is typically stockpiled in a wet state with surface moisture in the range of 0 to 5% (Mindess and Young) and free moisture content as high as 6 to 8% can be stable in fine aggregate (Committee 304). The reason for these high moisture values is that in addition to the thin film of moisture on the sand particles, water can be held in the interstices between the particles as the result of formation of menisci. This phenomenon is illustrated in Figure 4-5 below. The formation of these menisci creates thicker films of water between particles, pushing them apart and increasing the apparent volume of the aggregate (Mindess and Young). This can result in significant errors if proportions are measured by volume. To a lesser extent, coarse aggregate is also affected.

Therefore, batching by weight is the preferred method because once the moisture content has been determined, then a ton of aggregate is a definite quantity and by using the aggregate's specific gravity the volume it occupies in the concrete can be calculated.

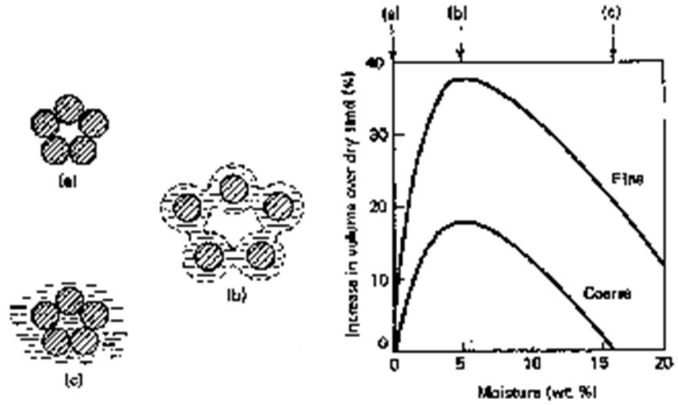

Figure 4-5. Bulking Phenomenon. (Mindess and Young)

Depending on the manner in which the aggregates are weighed, batch plants are of two general types: cumulative weighing and individual weighing. Cumulative weighing plants are equipped with a single weigh hopper and scale for all aggregates, whereas individual weighing plants are equipped with a separate weigh hopper (Figure 4-6) and scales for each size of aggregate.

The weigh hopper pictured is suspended by four hanger eyes, carried by four load cells. The load cells transmit data back to the batch plant control station. One of the load cells is pictured in Figure 4-7.

Figure 4-6. Weigh Hopper Under Aggregate Bins.

Figure 4-7. Load Cell at One Corner of a Weigh Hopper.

All working parts must be in good condition, free from friction, readily accessible for inspection and cleaning, and protected from falling or adhering material. Elements of the weigh hopper must not rub against other elements or the framework of the plant. The weighing container and the gates should be tight against leakage and the weighing system must be firmly supported on an unyielding foundation.

Weigh hoppers should clean themselves thoroughly after each batching cycle; otherwise appreciable amounts of material may become packed in the corners with the result that correct weight will be shown on the scale but something less than full weight will actually reach the mix.

Batch plant scales are relatively rugged and have a high degree of accuracy and reliability. Practically all of the working parts are clearly visible and generally accessible. Required tolerances of accuracy, however, depend on proper maintenance of the scale system. The Standard Specifications Section 9-1.01[2] requires all weighing and measuring devices used to proportion materials be tested and approved in accordance with California Test 109 Method for Testing of Material Production Plants.

[2] 2006 Standard Specifications, or 2010 Standard Specifications, Section 90-1.02B.

Under current Department of Transportation policy, a representative of the District Materials Engineer, on an annual basis, inspects all commercial batch plants furnishing material for State highway projects as prescribed by this test method. The District Materials Testing Branch maintains a file of current California Test 109 test reports for all batch plants within the District. Annual batch plant inspections are scheduled by the District Materials Testing Branch, and the owner of the plant makes arrangements. In most cases, these inspections will not involve field personnel. However, copies of the plant inspection report for a particular batch plant are furnished to projects using materials from that plant.

The testing procedures required by California Test 109 will be performed by a County Sealer of Weights and Measures, a Scale Service Agency, or by a representative of the State Division of Measurement Standards. All test procedures must be performed in the presence of the engineer.

In addition, Standard Specifications Section 9-1 requires the contractor to arrange for testing of proportioning devices as frequently as the engineer may deem necessary to ensure their accuracy. This specification gives job personnel the authority to require a supplemental plant inspection should there be any question as to the accuracy of the weighing and/or measuring equipment. Accordingly, the bridge engineer should request a California Test 109 inspection and report if there is any reason to believe the scales or other measuring or metering devices are not functioning properly. For example, erratic and unexplained variations in mix consistency and/or yield might warrant further inspection and testing of batch plant equipment.

Manual and Automatic Batching

In a manually operated batch plant, the weigh hoppers are equipped with gates that are opened and closed manually, with the accuracy of the weighing operation being dependent on the operator's visual observation of the scale. The gates may be provided with air, hydraulic or electric power assisted devices. Water metering is controlled by hand operated valves.

Most batch plants are equipped with automatic proportioning systems. These automatic systems include a means of entering the batch weights into a control mechanism and gates, which open automatically at the beginning of the weighing cycle for each material and close automatically when the designated weight of that material has been reached. Additionally, the scales are interlocked in such a manner that:

- The charging mechanism cannot be opened until the scale has returned to zero.
- The charging mechanism cannot be opened if the discharge mechanism is open.
- The discharge mechanism cannot be opened if the charging mechanism is open.

- The discharge mechanism cannot be opened until the designated weight has been reached within the allowable tolerance.

Although automatic batching is not required for structure concrete work, all automatic batching systems must meet the requirements of Standard Specifications Section 90-1.02F(4)(c).

Moisture Meters

Automatic proportioning devices are required by the Standard Specifications to be equipped with electrically actuated moisture meters to indicate the moisture content of the fine aggregate as it is batched.

Moisture meters of the type specified measure the amount of electrical current passing between a probe in the sand and the steel wall of the weigh hopper. The current measurement is converted to percent moisture to permit direct reading on the indicator scale. See Figure 4-8.

Figure 4-8. Moisture Meter at the Bottom of a Fine Aggregate Bin.

Moisture meters are not required for structure concrete work unless the concrete is produced at an automatic batch plant. This does not mean that moisture compensation is not important. To demonstrate:

Assume that 1 cubic yard mix requires 1,200 pounds of sand, 2,000 pounds of coarse aggregate and 25 gallons of water. A 1% variation in the moisture content of the sand would equal 12 pounds of water (approximately 1.5 gallons). This changes the water required by 6%. A rule of thumb guide has been that 1 gallon added to 1 cubic yard will increase the slump 1 inch.

The above example illustrates that monitoring and managing moisture content in sand is critical to consistent, quality concrete production. Moisture that is unaccounted for can significantly increase the water to cement ratio, decrease strength, increase the slump, increase shrinkage, increase cracking, and increase permeability.

> In Waddell and Dobrowolski: "Keeping sand moisture variations to a minimum is more of an art than a science and requires constant monitoring of the final product and proper response by the plant operator. This job is made easier if the plant is equipped with a moisture meter in the fine aggregate since there is no way that one can work with the current moisture and be drying samples. The moisture meter is only a tool that one must learn to use and does not alter the need for constant monitoring as the results of any sampling are related to the fact that only a very minute portion of the aggregates is sampled. It is not necessary that the meter be highly accurate as it is that the meter be responsive and has a high degree of repeatability. Thus when the meter shows a change, it's time to make an adjustment.
>
> In certain situations the moisture variations in the supply make it impossible to reach the desired slump even with the most conscientious efforts of the plant operator. In these cases the answer is to reduce the design water content and produce a mix with lower slump and then add only enough water on the job…to increase the slump into the desired range."

Reducing variations in the moisture content of the sand is beneficial in controlling water in the mix. Stockpile management can also play a role in controlling moisture. Allowing delivered sand time to reach a stable and uniform moisture content will reduce variation (Staff of Reasearch and Education Association). The Standard Specifications Section 90-1.02F requires that aggregates be dried and drained to a stable moisture content and that the fine aggregate moisture content be no more than 8% at the time of batching.

Chemical Admixtures

Standard Specifications Section 90-1.01D states that chemical admixtures used in the work must be of a type and brand on the Department's Authorized Materials List (http://www.dot.ca.gov/hq/esc/approved_products_list/pdf/Approved_Chem_Admix_List.doc). In addition to this list, the manufacturer's published information and their sales representative can be a valuable source of information pertaining to the performance of chemical admixtures. Most chemical admixtures are in a liquid form. The Standard Specifications Section 90-1.02F(4)(b) includes several requirements for admixture dispensing equipment and procedures at concrete batch plants.

When the use of a liquid admixture is required or contemplated, these requirements should be discussed with the contractor and/or concrete supplier before work begins to ensure that the procedures are understood and that the plant is equipped with the proper dispensing mechanism. Dispensing equipment must have sufficient capacity to contain and measure the total required admixture dosage for the batch (See Figure 4-9 Liquid admixture storage units). Dispensing equipment must be located so that the graduated measuring cylinder is visible to the batch plant operator.

Figure 4-9. Liquid Admixture Storage Units.

Unless liquid admixtures are added to pre-measured water for the batch, the dispensing equipment must allow the admixture to flow into the stream of batch water (see Figure 4-10 Manifold where liquid admixture enters the stream of batch water). When more than one liquid admixture is used in the mix, a separate measuring and dispensing unit is required for each admixture. The dispensing procedure must be such that the admixtures are added at different times during the batching cycle. When an air-entraining admixture is used with any other liquid admixture, the air-entraining admixture must be the first admixture dispensed into the concrete mixture. It is essential that this batching sequence be followed; otherwise the effectiveness of the air-entraining admixture may be impaired.

Figure 4-10. Manifold Where Liquid Admixture Enters the Stream of Batch Water.

Relative to the other ingredients, liquid admixtures are added to the batch in very small amounts. To ensure accurate proportioning, it is essential that specification requirements governing admixture dispensing procedure and equipment be followed, and that dispensing equipment be well maintained. Evaporation and freezing of the liquid could adversely affect the performance of the admixture. Both conditions should be avoided.

Unless the plant is approved for production of concrete pavement, there is no assurance (and no specification requirement) that the admixture dispensing equipment will be interlocked with the batching control mechanisms. Accordingly, when liquid admixtures are to be used in concrete produced at a batch plant equipped with automatic batching controls, the means by which the admixtures are added should be investigated. If the admixture dispensing equipment is not automated, addition of the admixtures must be carefully controlled to ensure the proper dosage.

Controlling Batch Temperature

Standard Specifications Section 90-1.02G(2) requires that concrete must be 50°F to 90°F immediately prior to placing. Concrete temperatures can be regulated by controlling the temperature of the ingredients. The contribution of each constituent is determined by its temperature, specific heat and weight fraction. This is the basis for Equation (4-01), which can be used to calculate the concrete temperature in either Fahrenheit or Celsius (Mindess and Young):

$$T_{concrete} = H(T_a W_a + T_c W_c) + T_a W_{wa} + T_w W_w \,/\, H(W_a + W_c) + W_{wa} + W_w \qquad (4\text{-}01)$$

Where H = the approximate specific heat of cement and aggregate (925 J/kg°C or 0.22 Btu/lb°F), W_a, W_c, W_{wa}, and W_w are weights (kg or lb) of aggregate, cement, aggregate moisture and mixing water; and T_a, T_c, and T_w are the temperatures of aggregate (including moisture), cement, and mixing water, respectively.

Using the equation, it will become apparent that the high specific heat of water offsets its small fraction of weight so that its contribution to the composite temperature of the concrete is about the same as that of the aggregate. In cooling applications the use of ice makes water even more effective since additional heat is absorbed in the melting of the ice. To capture the effect of the heat of fusion when using ice, the Equation (4-01) is modified as follows

$$T_{concrete} = H(T_a W_a + T_c W_c) + T_a W_{wa} + T_w W_w - F_i W_i \,/\, H(W_a + W_c) + W_{wa} + W_w + W_i \qquad (4\text{-}02)$$

Where W_i = the weight of ice and F_i its latent heat of fusion (335 kJ/kg or 145 Btu/lb).

Cooling Concrete

High concrete temperatures increase the water requirements to maintain a given slump; decrease setting times, and hence the time available for placement, consolidation, and finishing; increase the likelihood of plastic shrinkage; and lower the ultimate strength. Concrete should remain plastic long enough so that each lift can be placed without development of cold joints or discontinuities in the concrete. Air entrainment is also affected. The amount of air-entraining admixture required to produce a given air content increases with temperature.

High concrete temperatures are commonly related to concreting in hot weather. High concrete temperatures may also be an issue in mass concrete and lightweight concrete applications. These issues are discussed in Chapter 7 of this manual.

Many techniques exist for managing high temperatures. They range from simple and inexpensive to more complex and costly. Some simple methods include storing concrete ingredients in the shade, spraying aggregates with water, covering aggregates with reflective covers, and painting water tanks and line white.

More elaborate methods include admixtures, chilling the mixing water (see Figure 4-11 for a picture of a water chiller), using ice as some portion of the mixing water, cooling aggregates with chilled water or air, and vacuum cooling of aggregates. Also, liquid nitrogen at a temperature of –320°F can be used to chill mixing water, aggregates, or concrete (ACI Committee 304). Liquid nitrogen can be used to chill water or injected directly into central or truck mixers to achieve desired temperatures (See Figure 4-12).

Figure 4-11. Water Heater/Chiller.

Figure 4-12. Liquid Nitrogen Being Injected Into a Transit Truck to Cool the Concrete.

On a weight basis the mix water has the greatest effect on the final concrete temperature. This stems from its higher specific heat, which is about five times as high as for the other concrete-making materials. However, be weight, there is usually about ten times more aggregate than water in structure concrete.

Concrete can be cooled to a moderate extent by using chilled mixing water. The quantity of cooled water cannot exceed the mixing water requirement. The maximum reduction in concrete temperature that can be obtained is approximately 10°F. The concrete temperature equations above can be used when all variables are known but in general, if all other variables are held constant, to obtain a 1°F drop in concrete temperature the mix water temperature must be reduced by 3.6°F. For example, a concrete producer would chill the 70°F mix water down to 52°F to lower the concrete temperatures from 80°F to 75°F.

If a water chiller is not available, or the desired reductions in mix temperature involve chilling the water beyond its freezing point (32°F), ice can be used. The amount of cooling is limited by the amount of mixing water available for ice substitution. For most concrete, the maximum temperature reduction is approximately 20°F. For correct proportioning the ice must be weighed. Ice has a two-way cooling effect. First it draws heat from the concrete for melting of the ice, and then the resulting water at 32°F provides continued cooling capacity. The temperature equation to be used when ice is utilized has been provided above but, as a general rule, for every 1°F drop in concrete temperature desired, replace 2% of the total mix water with ice. For example, to cool a concrete mix containing 280 pounds per cubic yard (pcy) of water from 80°F down to 60°F, replace 40% (112 pcy) of the mix water with ice. The ice must be added directly into the concrete as part of the mixing water. With ice crushed from 0.06 inch to 0.1 inch in thickness, melting takes place in about 30 seconds.

When ice is used as a portion of the mixing water, care must be taken not to exceed the total requirements of mixing water and to verify that all the ice has melted at the time of discharge. A situation occurred where ice purchased in unit weight bags actually exceeded the weight shown on the bag and led to unintentionally exceeding the total required water. If ice is used it must be proportioned by weight as it occupies a greater volume than the same mass in liquid form. Additionally, because ice readily sublimes, bagged ice is usually packaged with a greater mass of ice than is reported on the bag and loses mass over time. If used, the weight must be verified to accurately proportion the water.

Admixtures can also be useful in controlling temperature. The moment cement contacts water, hydration begins and heat is liberated. ASTM C 494, Types D and G water reducing and retarding admixtures are beneficial in hot weather concreting due to their ability to slow the chemical reaction between water and cement and decrease the water required for a given workability.

Chilled water, ice and admixtures, used either in combination or separately, are all helpful tools for successful hot weather concreting. For more information on this topic refer to ACI 305, "*Hot Weather Concreting*."

Because of the low specific heat and relatively small mass of cementitious material in a mix its effect on the concrete temperature is minor. It requires approximately a 10°F change in the cementitious material temperature to change the concrete temperature 1°F. (Kosmatka and Panarese). The temperature of cement should not be disregarded though, see Chapter 1 of this manual for the discussion of false set and its relationship to cement temperature.

Heating Concrete

In cold weather where there is a risk of freezing the concrete, slow development of strength, and excessive thermal stresses upon cooling to ambient temperatures.

To deliver concrete within the specified temperature range, it may be necessary to heat one or more of the concrete ingredients. Overheating should be avoided as it may accelerate chemical action, cause excessive loss of slump, and increase the water requirement for a given slump. Also, the warmer the concrete is placed, the greater the drop to ambient temperatures. Correspondingly there will be a greater decrease in volume and increase in thermal stresses.

The Standard Specifications limit the heating of aggregate and water to 150°F.

Mixing and Transporting

Contract requirements governing the mixing and delivery of concrete are found in Standard Specifications Section 90-6.01[3].

All concrete used on State highway projects must be mixed in mechanically operated mixers, except that when permitted by the engineer, batches up to but not exceeding 1/3 cubic yard may be mixed by hand. See Standard Specifications Section 90-6.05[4] for requirements governing the use of hand-mixed concrete.

When mixing concrete, the objective is to obtain a workable mixture in which the various ingredients are uniformly distributed and the aggregate particles are uniformly and completely coated with the cement paste. All commercially manufactured concrete mixers are capable of meeting this objective, provided they are not overloaded, are maintained in good operating condition, and are operated at the mixing speed recommended by the manufacturer.

When charging the mixer, a small amount of water should enter the drum ahead of the solid materials. The remaining water should be added uniformly over the entire charging period.

Care must be taken to avoid loss of material during the charging cycle. Cement in particular must be charged by means that will prevent wind loss or the accumulation of cement particles on the surface of conveyors or hoppers, or any other condition that would vary or reduce the amount of cement in the mixture.

When charging truck mixers, a sock is often used to direct the material into the truck loading chute. The sock, usually made of rubber or canvas, funnels the material into the drum and prevents spillage during the charging operation (see Figures 4-13 and 4-14). If the sock is torn or badly worn, material may escape and be wasted on the ground, with a consequent loss of cement or other materials.

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[3] 2006 Standard Specifications, or 2010 Standard Specifications, Section 90-1.02G.
[4] 2006 Standard Specifications, or 2010 Standard Specifications, Section 90-1.02G(5).

Figure 4-13. Sock Used at a Truck Mixer Charging Chute.

Figure 4-14. Sock Used at a Truck Mixer Charging Chute.

Equipment used to supply water to the mixer must be constructed and arranged so that the amount of water added can be accurately measured. Tanks used to measure water must be designed so that water cannot enter the tank while water is being discharged into the mixer. The amount of water added as indicated by a measuring device must be within 1.5% of the actual amount of water discharged into the mixer. The equipment must provide a means of checking the amount of water delivered by direct discharge into a container of known volume.

If retarding or air-entraining admixtures are being used, they should be added to each batch at the same time in the charging cycle. If this is not done, significant variation (from batch to batch) in the time of initial set or the percentage of entrained air may occur. When an air-entraining admixture is used with any other liquid admixture, the air-entraining admixture is typically added first. It is prudent to acquire the manufacturer's mixing recommendations whenever multiple admixtures are used in a mix design.

Mixers should not be loaded in excess of the manufacturer's rated capacity, nor should they be operated at speeds that exceed the speed designated by the manufacturer as mixing speed. Increasing production by speeding up or overloading the mixer may result in inadequately mixed concrete and should not be permitted.

Concrete mixers must be capable of producing uniform concrete. Uniformity is determined by differences in penetration or slump.

The difference in penetration and slump is found by performing the respective tests on 2 samples of mixed concrete from the same batch or truck mixer load. The maximum allowable variations are provided in Standard Specifications Section 90-1.02G(6). Similarly, the variation in the proportion of coarse aggregate is determined from the results of tests on 2 samples of mixed concrete from the same batch or truck mixer load, and may not exceed 170 pounds per cubic yard computed in accordance with California Test 529. The use of mixing equipment that cannot produce concrete meeting these requirements should not be permitted. (See Standard Specifications Section 90-1.02A)

Ready-Mixed Concrete

Ready-mixed concrete is defined as concrete that is batched, mixed and delivered to the job while in a plastic state.

Modern batching equipment and transit-mix trucks make it possible to deliver accurately proportioned concrete, mixed and ready to deposit into the forms, to virtually any location where concrete construction is contemplated. Virtually all of the concrete used in structure construction on State highway projects is ready-mixed.

Ready-mixed concrete is further classified according to the method by which the concrete is mixed and delivered. These classifications are central-mixed concrete, shrink-mixed concrete, and transit-mixed concrete.

The term "central-mixed" describes ready-mixed concrete that is mixed completely at a stationary plant, and then transported to the point of delivery in truck agitators or, when warranted by job conditions or circumstances, in non-agitating, truck-hauling equipment.

The term "shrink-mixed" describes ready-mixed concrete that is partially mixed in a stationary mixer, and then transferred to a transit-mixer where the mixing is completed. Shrink-mixed concrete is used to increase the truck's load capacity by reducing or "shrinking" the volume of the constituent parts.

The term "transit-mixed" describes ready-mixed concrete that is mixed completely in a truck mixer. This is sometimes referred to as "dry batching" and the previous two methods as "wet batching".

Central-Mixed Concrete

A typical central-mixing plant will include either a revolving blade mixer or a revolving drum mixer. Aggregates are conveyed to storage bins above the weigh hopper. Cement is bulk-stored in a silo and water is metered into a storage tank adjacent to the stationary mixer. Batching is automatic and a 90-second mixing time is typical, although the specifications now permit a mixing time of 50 seconds (minimum) when directed by the engineer. The maximum mixing time allowed is 5 minutes.

Central-mixed concrete may be delivered in non-agitating "dump-crete" trucks. These trucks, which typically range from 4 to 8 cubic yards capacity, have special dump boxes with smooth sides and rounded corners to facilitate dumping of the concrete. Under the current specifications, when non-agitating equipment is used, discharge must be completed within 1 hour after mixing.

Truck agitators are used for longer hauls. As is the case with the dump-crete trucks, consistency of the mixed concrete must be correct because water may not be added unless the equipment is capable of revolving at mixing speed. Central-mixed concrete transported in truck agitators must be discharged within 90 minutes after the agitator is charged. A truck agitator is pictured in Figure 4-15.

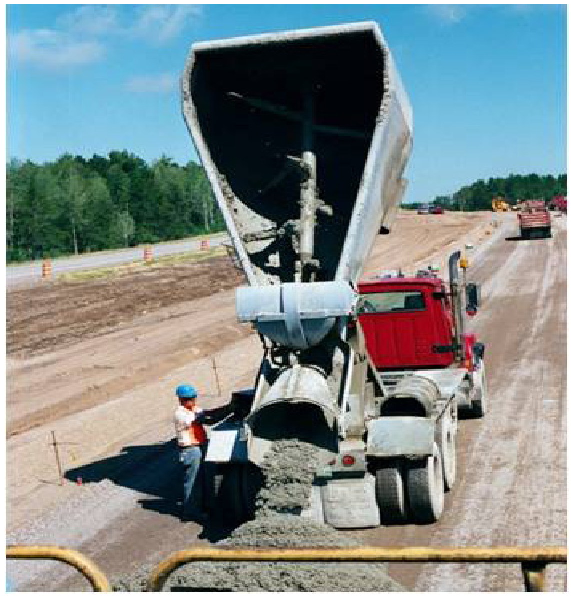


Figure 4-15. Truck Agitator.

Transit-Mixed Concrete

Many commercial concrete plants incorporate stationary mixers and thus by definition are central-mix plants. However, virtually without exception concrete for structure construction on State highway projects will be delivered to the work site in transit-mixing equipment, regardless of whether the technical description of the product is central-mixed, shrink-mixed or transit-mixed concrete. Accordingly, this discussion of transit-mixed concrete and transit-mixing equipment will be applicable to all structure concrete, including central-mixed concrete. Note also that the specifications provide that when concrete is partially mixed at a central plant (i.e., shrink-mixed concrete) all requirements for transit-mixed concrete will apply.

For discussion, a truck mixer (Figure 4-16) may be considered as three separate machines, each with a separate function to perform. First, it is a concrete mixer; second, it is an agitator-conveyor; and finally, it is a machine that can discharge its load either directly into the forms or into an intermediate conveyor such as a crane bucket or concrete pump hopper.

Figure 4-16. Truck Mixer.

The specifications require identification plates to be attached to truck mixers. These plates must show the truck's capacity as a mixer and, if applicable, as an agitator, along with mixing and agitating speeds, all as rated by the manufacturer. Mixing speed is generally not less than 6 revolutions per minute nor greater than 225 peripheral feet per minute; while agitating speed is usually between 2 and 4 revolutions per minute.

Truck mixers must be equipped with electrically or mechanically actuated revolution counters. The truck mixer should be revolving as the materials are charged into the mixing drum. Some truck mixers have a charging speed. If so, the charging speed should be used; otherwise the batch should be received at mixing speed.

Although the specifications require concrete to be mixed at the rate of rotation designated as mixing speed by the manufacturer, there is no minimum revolution requirement. The specification requirement is that the number of revolutions at mixing speed be not less than recommended by the manufacturer, and not less than the number necessary to produce uniform concrete as determined by California Tests 533 and 529 (Standard Specifications Section 90-1.02G(4)).

When water is added after the concrete reaches the job site, the specifications require a minimum of 30 revolutions at mixing speed before the concrete is discharged.

In general, the total number of revolutions is not as important as the number of revolutions at the proper mixing speed. This is the case because manufacturers give a range of mixing speeds, typically from about 6 to 16 revolutions per minute. At a very slow mixing speed, inadequate mixing may occur even though the concrete has been mixed for the minimum number of revolutions recommended by the manufacturer. Experience shows that optimum mixing will occur at a mixing speed that is slightly slower than the recommended maximum speed.

Inadequate mixing can seldom be detected by a casual observation of the mixture. Thus where conditions are such that inadequate mixing may occur, consistency and uniformity tests should be performed. These tests will also reveal inadequate mixing from causes other than an insufficient number of drum revolutions, such as worn mixer blades and other mixer deficiencies.

When transit-mixing equipment is used as an agitator-conveyor, it is generally capable of carrying a greater load. (Keep in mind, however, that when carrying a load based on agitator capacity, the truck may not be used as a mixer.)

When agitating, the mixing drum must revolve continuously. Since agitator speeds are slow, it is not always possible to judge from a distance whether the drum is revolving at the proper speed, or even if it is revolving at all.

An almost universal shortcoming of both truck mixers and truck agitators is their tendency to segregate the mix so that the last portion contains an excess of coarse aggregate. Placement of concrete should be controlled so the latter part of the batch is discharged into a location where the excess coarse aggregate will not cause difficult . Locations immediately adjacent to forms, reinforcing steel, expansion joints and other critical points should be avoided, if possible.

In general, concrete must be discharged from truck mixers within 90 minutes, or before 250 drum revolutions, from the time cement is added to the mixture. Since either elapsed time or drum revolutions may govern the discharge, both must be checked. Under conditions contributing to rapid stiffening of the concrete or when the concrete temperature is ≥ 85°F, the engineer has the authority to reduce the 90-minute period.

For a long haul, such as might be required in a rural area, the time between batching and discharge has been extended by adding the cement, not at the batching plant, but at a point closer to the work. During the haul between the batch plant and the point at which the cement is added, the mixer should not be revolving, as this would subject the aggregate to unnecessary degradation.

Inspection of Mixing Equipment

Mixers should be inspected periodically to determine whether they are in satisfactory operating condition. Any hardened concrete or mortar which has accumulated on the mixer blades should be removed and the blades inspected. Mixer blades wear more quickly in the center and a dished blade indicates excessive wear. Most manufacturers recommend repairing or replacing mixer blades when any part or section is worn more than 2 or 3 cm below the original dimensions.

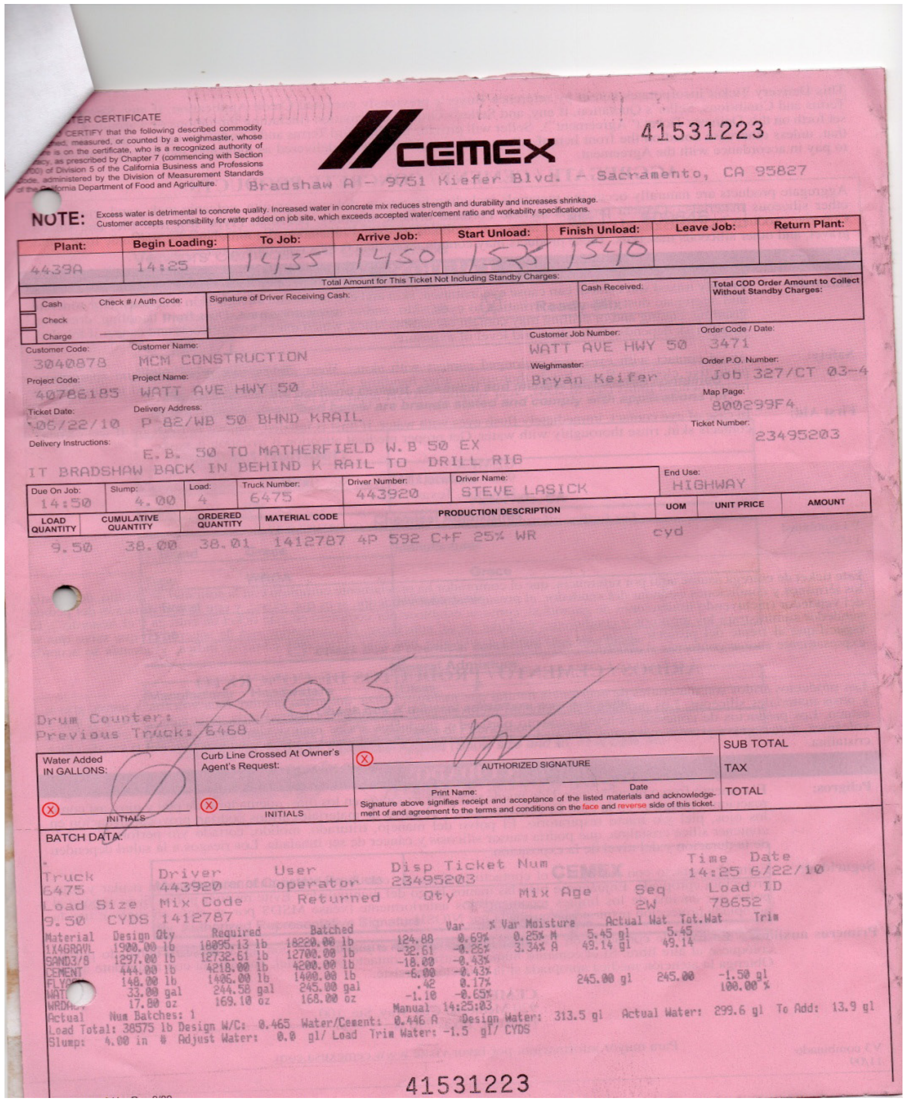
The interior of the mixer drum must be clean. Deposits of hardened concrete or mortar, which sometimes accumulate inside the drum and on the mixing blades, will reduce the effectiveness of the mixer and should be removed.

The water supply system should be inspected periodically for leaking valves and fittings. Truck mixers in particular should be checked to see that there is no leakage from the water tank into the mixing drum. Water metering equipment should be checked for accuracy by discharging into measured containers.

Transit-mixing equipment should be checked periodically to see that the revolution counters are functioning properly and that the required manufacturer's rating plate is affixed to the truck and is clean and clearly visible. The water measuring equipment should be inspected for leaks, and the accuracy of the gauge or meter verified by discharging a measured amount of water into a container of known volume.

Transporting Mixed Concrete

Specifications require that each load of concrete delivered to the job site be accompanied by a weighmaster certificate showing the mix identification number, a non-repeating load number, the date and time the material was batched, the total quantity of water added to the load for transit-mixed concrete, the revolution counter reading at the time the truck mixer is charged with cement, and actual scale weights in pounds for the ingredients batched. Figure 4-17 shows an example of a weighmaster certificate, often called a batch ticket.



...TER CERTIFICATE
...CERTIFY that the following described commodity ...ed, measured, or counted by a weighmaster, whose ...e is on the certificate, who is a recognized authority of ...acy, as prescribed by Chapter 7 (commencing with Section ...00) of Division 5 of the California Business and Professions ...ode, administered by the Division of Measurement Standards of the California Department of Food and Agriculture.

CEMEX

41531223

Bradshaw A - 9751 Kiefer Blvd. - Sacramento, CA 95827

NOTE: Excess water is detrimental to concrete quality. Increased water in concrete mix reduces strength and durability and increases shrinkage. Customer accepts responsibility for water added on job site, which exceeds accepted water/cement ratio and workability specifications.

Plant:	Begin Loading:	To Job:	Arrive Job:	Start Unload:	Finish Unload:	Leave Job:	Return Plant:
4439A	14:25	1435	1450	1525	1540		

Total Amount for This Ticket Not Including Standby Charges:

Cash / Check / Charge — Check # / Auth Code: — Signature of Driver Receiving Cash: — Cash Received: — Total COD Order Amount to Collect Without Standby Charges:

Customer Code: 3040878 — Customer Name: MCM CONSTRUCTION — Customer Job Number: WATT AVE HWY 50 — Order Code / Date: 3471

Weighmaster: Bryan Keifer — Order P.O. Number: Job 327/CT 03-4

Project Code: 40786185 — Project Name: WATT AVE HWY 50 — Map Page: 800299F4

Ticket Date: 05/22/10 — Delivery Address: P 82/WB 50 BHND KRAIL — Ticket Number: 23495203

Delivery Instructions: E.B. 50 TO MATHERFIELD W.B 50 EX IT BRADSHAW BACK IN BEHIND K RAIL TO DRILL RIG

End Use: HIGHWAY

Due On Job:	Slump:	Load:	Truck Number:	Driver Number:	Driver Name:
14:50	4.00	4	6475	443920	STEVE LASICK

LOAD QUANTITY	CUMULATIVE QUANTITY	ORDERED QUANTITY	MATERIAL CODE	PRODUCTION DESCRIPTION	UOM	UNIT PRICE	AMOUNT
9.50	38.00	38.01	1412787	4P 592 C+F 25% WR	cyd		

Drum Counter: 2 0 5
Previous Truck: 6468

Water Added IN GALLONS: 0 — INITIALS

Curb Line Crossed At Owner's Agent's Request: — INITIALS

AUTHORIZED SIGNATURE — Print Name: — Date

Signature above signifies receipt and acceptance of the listed materials and acknowledgement of and agreement to the terms and conditions on the face and reverse side of this ticket.

SUB TOTAL
TAX
TOTAL

BATCH DATA:

Truck 6475 — Driver 443920 — User operator — Disp Ticket Num 23495203 — Time 14:25 — Date 6/22/10
Load Size 9.50 CYDS — Mix Code 1412787 — Returned — Qty — Mix Age — Seq 2W — Load ID 78652

Material	Design Qty	Required	Batched	Var	% Var	Moisture	Actual Wat	Tot.Wat	Trim
1X4GRAVL	1900.00 lb	18095.13 lb	18220.00 lb	124.88	0.69%	0.25% M	5.45 gl	5.45	
SAND3/8	1297.00 lb	12732.61 lb	12700.00 lb	-32.61	-0.26%	3.34% A	49.14 gl	49.14	
CEMENT	444.00 lb	4218.00 lb	4200.00 lb	-18.00	-0.43%				
FLY...	148.00 lb	1406.00 lb	1400.00 lb	-6.00	-0.43%				
WAT...	33.00 gal	244.58 gal	245.00 gal	.42	0.17%		245.00 gl	245.00	-1.50 gl
WRDA...	17.80 oz	169.10 oz	168.00 oz	-1.10	-0.65%				100.00 %

Actual — Num Batches: 1 — Manual 14:25:03
Load Total: 38575 lb Design W/C: 0.465 Water/Cement: 0.446 A Design Water: 313.5 gl Actual Water: 299.6 gl To Add: 13.9 gl
Slump: 4.00 in # Adjust Water: 0.0 gl/ Load Trim Water: -1.5 gl/ CYDS

41531223

Figure 4-17. Example of a Weighmaster Certificate.

The batch ticket is an indispensable tool in the quality assurance effort. Each ticket is to be collected and initialed at the job site. The following information can be assessed: Verify that the mix number on the ticket matches the mix number that was approved for the current concrete placement operation; in this example, Mix no. 1412787. The mix number may not be specific to your contract so verify that your project has been identified on the ticket; in this example, Watt Ave HWY 50. Verify that the date is accurate; in this example, batched

06-22-10. The Standard Specifications require that the concrete be discharged within 90 minutes of being batched. The time of batching is shown on the ticket; in this example, Time Batched 14:25. Weighmaster certificate and weighmaster Bryan Keifer are shown as required by the Standard Specifications. The Truck #6475 is shown on the ticket and this should be verified to be certain that the batch ticket represents the contents contained in the truck. The batch tickets are filed in the job files so notes can be placed on the ticket with the inspector's initials. The number on the drums revolution counter should be recorded on the ticket. In this case 205 revolutions were recorded. Batch weights and moisture content of the aggregates, 3.34% for sand and 0.25% for coarse aggregate, are reported. The batch weights can be compared to the weights required by the mix design and the moisture content can be used to verify the reported water in the batch.

No water in excess of that in the approved mix design can be incorporated into the concrete. If a contractor requests to add water at the job site a review of the ticket is required to evaluate if there is a balance of water that can be added without exceeding the amount contemplated by the approved mix design. If any water is added it should be noted on the ticket and initialed by the inspector. If a sample has been taken from the batch and tests performed on that sample this too can be noted on the ticket.

Time or Amount of Mixing

Although thorough mixing is essential to the production of high quality concrete, prolonged mixing beyond that necessary to produce a uniform mixture should be avoided. If a uniform, workable concrete mixture can be obtained at a given number of drum revolutions, there is no benefit to be gained by additional mixing.

Prolonged mixing, even at agitating speeds, is objectionable for several reasons. First, mixing results in a grinding action that causes degradation of aggregate particles, and this in turn increases the amount of very fine material in the mix.

Second, mixing produces friction, which raises the concrete temperature. Both of these factors reduce plasticity, which increases the amount of water needed to maintain a given consistency.

Tests by the Portland Cement Association indicate that for an average concrete (medium strong aggregate and initial concrete temperature of 70°F) the slump decreased from 13 cm to 6 cm over a 1 hour period. The slump loss was attributed to aggregate degradation, to heat generation due to friction, and to cement hydration during the mixing period. An even greater slump loss would be expected with softer aggregate, higher initial concrete temperature, and/or a longer mixing time.

Finally, for air-entrained mixes, prolonged mixing can cause a significant reduction in air content. (See Chapter 9 of this manual for a discussion of this phenomenon.)

On-Site Mixing

The specifications permit the use of paving and stationary mixers to produce structure concrete at the job site. Such mixers may be of the revolving drum or revolving blade type, may be tilting or non-tilting, and may or may not be equipped with loading skips and movable discharge chutes.

When used for structure concrete, the required mixing time for both paving and stationary mixers is greater than or equal to 90 seconds and less than 5 minutes.

In multiple-drum mixers, the transfer time between drums is counted as part of the required mixing time.

Paving and stationary mixers must be equipped with an automatic timing device, which can be set and locked. The timing device and discharge mechanism must be interlocked so that during normal operation no part of the batch will be discharged until the specified mixing time has elapsed.

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CHAPTER 5
CONCRETE CONSTRUCTION

Table of Contents

Concrete Construction **5-1**

Quality Assurance: Pre-Construction Review **5-2**
Specifications 5-2
Standard Plans 5-2
Other Reference Documents 5-2
Technical Resources 5-3
Specific Pre-Construction Practices 5-3
Bridge Construction Surveying 5-3

Concrete Forms **5-4**
Specifications 5-5
Forming System 5-7
Form Fasteners 5-8
Form Liners 5-10
Miscellaneous Form Types 5-12
Steel Forms 5-12
Fiber Form Tubes 5-13
Metal Decking 5-13
Pre-Fabricated Forms 5-15
Form Workmanship 5-15
Structural Adequacy of Formwork 5-16
Formwork Design 5-16
Formwork Design Review 5-17
Deflection 5-17
Lateral Loading 5-17
Unit Weight of Concrete 5-18
Rate of Placement 5-18
Other Factors 5-19
Admixtures 5-19
Lateral Pressure Values (ACI 347-04) 5-19
Form Release Agent 5-21
Formwork Removal 5-22

Concrete Placement.. 5-23
Prior to Placing Concrete 5-23
Concrete Delivery 5-24
Quality Assurance 5-25
Forms 5-25
Mix Consistency and Uniformity.. 5-25
Concrete Tickets 5-25
Concrete Placement Time.. 5-26
Transit Mixer Drum Revolution 5-26
Concrete Temperature 5-26
Certificates of Compliance 5-26
Weight Limits.. 5-27
Adding Water to Concrete 5-27
Delivery Rate.. 5-28
Conveyance of Concrete 5-29
Chutes.. 5-30
Concrete Buckets.. 5-31
Drop Chutes 5-33
Belt Conveyors 5-34
Concrete Pumps 5-35
Slick Line.. 5-36
Placing Concrete 5-37
Depositing Concrete.. 5-38
Segregation 5-39
Prevention of Concrete Defects.. 5-40
Protecting Concrete.. 5-41

Consolidating Concrete.. 5-42
Vibrating Concrete 5-43
Internal Vibrators.. 5-43
External Vibrators 5-46
Consequences of Improper Vibration.. 5-47
Revibration.. 5-49

Finishing Plastic Concrete 5-49
Bleed Water 5-49
Strike Off.. 5-50
Initial Finishing 5-51
Machine Finishing: Bridge Deck 5-54
Other Concrete Finishing Equipment.. 5-56
Finishing Air-Entrained Concrete 5-57

Curing Concrete **5-58**
Plastic Shrinkage Cracking 5-59
Specified Concrete Curing Methods 5-62
Water Method 5-62
Plastic Sheets 5-63
Burlene® 5-64
Forms-In-Place Method 5-66
Curing Compound Method 5-67
Fogging 5-69
Application 5-70
Quality Assurance 5-72
Safety 5-74
Waterproof Membrane Method 5-75
Curing Structures 5-75
Curing Minor Structures 5-76

Surface Finishes **5-76**
Specified Concrete Finishes 5-76
Ordinary Surface Finish 5-77
Patching Snap-Tie Holes 5-78
Repair of Rock Pockets 5-79
Class 1 Surface Finish 5-79
Whip Blasting 5-80
Engineer's Roles and Responsibilities 5-81
Unacceptable Class 1 Finishes 5-82
Class 2 Surface Finish 5-82
Safety and Environment 5-82
Working Over Water and Environmentally Sensitive Areas 5-83
Working Over Traffic 5-83

Special Concrete Construction Practices **5-83**
Specification Requirements 5-84
Hot Weather Construction 5-84
Control of Concrete Temperature 5-85
Mixing and Delivery 5-86
Placing and Finishing 5-87
Curing 5-88
Use Of Admixtures 5-88
Cold Weather Construction 5-89
Mixing And Delivery 5-89
Placing and Finishing 5-90

Curing 5-90
Measuring Concrete Temperatures 5-91
Placing Concrete Under Water 5-91
Seal Course Concrete 5-91
Concrete Placement 5-92
Water and Slurry Displacement Method 5-93

Bridge Construction Practices 5-94
Foundations 5-95
Concrete Piles 5-95
Footings 5-96
Columns and Walls 5-96
Bent Caps 5-97
Superstructure 5-97
Stem and Soffit-Box Girder 5-98
Stem/Deck Construction Joint 5-99
Bridge Decks 5-100
Barrier Rails 5-102
Emergency Construction Joints 5-102
Weight Limits 5-103

5 CONCRETE CONSTRUCTION

Even though the concrete produced by the batch plant has all the attributes of a quality structure material, as described in the foregoing chapters, and may also be of excellent quality and capable of producing a strong, durable concrete product, the quality of the concrete will be only as good as the handling, placing, finishing, and curing methods employed.

With the delivery of the mixed concrete to the job site, the production phase ends and the construction phase begins. During the construction phase, the primary responsibility of ensuring the quality of the concrete and the resultant concrete structure rest almost exclusively with the field engineering staff. The quality assurance requirements of concrete construction constitute far and away the bulk of the inspection work associated with the predominant bridge construction project in Caltrans – concrete box girder bridges.

This chapter is devoted to the concreting practices during the construction phase, with emphasis on methods and procedures that will ensure a high quality finished product. This chapter is intended to provide field engineering staff with guidance on the various aspects of concrete construction operations on a typical bridge project; starting with the delivery of the concrete to the jobsite and then moving onwards, covering fundamental concrete construction operations like transporting and handling concrete, depositing, consolidating and curing concrete. While best practices are described, the chapter is not intended to serve as a how-to reference.

Formwork, which precedes the actual concrete delivery in a typical bridge construction project, is included in this chapter because not only is it integral in rendering the form, shape, and appearance of the final concrete product, but it also to a large extent, influences the quality of concrete. Additionally, discussion of hot and cold weather concrete construction and underwater concrete placement, etc., is also included.

The content of this chapter is by design narrow in scope to emphasize the predominant work encountered in Caltrans structure construction projects, cast-in-place box girder structures, and retaining walls. A brief introductory overview of precast concrete construction, slip-form concreting, shotcrete, building construction, and segmental bridge construction is included for the purpose of context and to provide direction toward detailed treatment elsewhere, either in the appendix or in a separate manual.

Quality Assurance: Pre-Construction Review

Care and a rigorous quality control process employed by the contractor and a diligent observance of established quality assurance standards by inspectors responsible for the concrete work are key elements in ensuring a quality concrete structure. Diligent preparation on the part of the contractor and on the field engineering staff is also a key factor in achieving a superior concrete structure.

Specifications

To ensure that the concrete inspector is capable of enforcing the contract specifications and implementing the quality assurance standards in concrete placement, the concrete inspector must have a thorough knowledge of the work to be done as well as the proper method or methods by which it may be accomplished. A key component of such preparation is the review of the contract plans and specifications. The inspector should have a working knowledge of the core concrete specifications and a thorough understanding of the contract Special Provisions because, as a supplement to the Standard Specifications, the "Specials" contain job specific requirements that may be unique to particular circumstances on the project.

Standard Plans

In addition to a comprehensive understanding of the contract plans, a thorough review of the Standard Plans, which are contractual supplements to the contract plans, is also essential to make sure that the details and plan notes thereon, which may not be shown or included on the contract plans, are duly incorporated during construction. Due to the density of information on a typical plan sheet, careful review is necessary to ensure that the details and requirements in the plan notes are not overlooked. For example, the "wall offset value" for Type I walls, reinforcement of openings in concrete, and detailing on prestressing, among others are details found in the standard plans, which could easily be missed without the extra effort and care in reviewing the appropriate plan sheet.

Other Reference Documents

Additional concrete construction guidance can be found in the Bridge Construction Memo (BCM) found in the Bridge Construction Records and Procedures Manual. Inspectors should take the time to review applicable memos and, if appropriate in the context of the contract administration procedures, integrate their guidance with the contract plans and specifi ations. The Caltrans Construction Manual provides construction administration procedures and defines quality control test requirements. The local Structure Materials Representative from

Caltrans Materials Engineering and Testing Services can provide documentation when required for both California Test Methods (CTM) and ASTM tests.

Technical Resources

While every effort was made to produce a comprehensive set of guidance on concrete placement in this manual, it is recognized that other technical resources are available, including those online that may be useful in the furtherance of an inspector's knowledge and expertise in concrete and concrete placement such as but not limited to:

- The Portland Cement Association's publication, "Concrete Construction Practices"
- American Concrete Institute
- APA – The Engineered Wood Association

However it should be noted that if the information found in these external sources contravenes Caltrans specifications, policies, or standing practices and procedures, the Caltrans standards prevail, especially in contract administration. As an example, the Department uses proprietary test procedures, the CTM, in lieu of the widely available ASTM test methods and as such the ASTM test methods or other externally sourced procedures should not be used when a CTM has been specified.

Specific Pre-Construction Practices

Before work begins, an inspector should review the contractor's proposed concrete construction methods. For example, while it is not required, it is advisable to have a "pre-pour" meeting to discuss the issues that may delay placing the concrete, such as not having all reinforcement in place and secure with sufficient clearance. It is best to work closely with the contractor and to advise them as soon as possible of conditions that would prevent the timely start and completion of the concrete placement.

Bridge Construction Surveying

The location, position, and grades of a bridge, wall, or structure, are controlled by survey stakes provided by the District survey crew. In addition, Structure Construction has made it a practice to provide line and grade sufficiently close to the working area to enable the contractor to layout the work using tools which are normally associated with the work of a bridge construction crew, such as string or wire lines, plumb bobs, carpenter's levels, and tapes. At this time, the practice of providing line and grade relieves the contractor of the need to use survey instruments to layout bridge location.

As such, in addition to the pre-construction review of concrete material quality assurance requirements, field engineers also review the surveying requirements of the job. The field engineer has dual surveying duties. The first duty is to verify that the staking provided by the District survey crew is correct. And the second, is to provide “Structure Construction Control Surveying,” which is the staking performed by the bridge inspection staff to set the bridge lines and grades sufficiently close to the working area, typically using the control stakes, called the “Original Structure Stakeout” that was provided by the District survey crew.

For State-administered bridge construction projects, the District survey crew will typically provide “control stakes” for the bridge foundation. These control stakes serve as the fundamental control for location, position and elevation of the bridge being constructed. As such, it is incumbent upon the field engineer to ensure that these stakes are correct by exercising prudent, mostly basic, coordinate calculations and checking the stakes in the field, a task that has been made easier with the introduction of Total Stations and surveying software such as COGO. Technical support is available from Structure Construction.

In addition to verifying the survey stakes, the engineer must also review the bridge deck contour sheet (also called “four scale”) for accuracy. Deck contour sheets are scaled topographic plans showing the elevation of the bridge deck. The deck contour sheet defines the upper limit of the bridge elevations and is used to determine the grades of various components of the bridge. Given its importance in controlling the vertical geometry of the bridge, it is vital that field engineers ensure that the grades generated are accurate.

One of the best practices to ensure that the bridge deck contour sheet is accurate is to match it with the road contours. Plotting and matching the lines and grades not only of the bridges but also of walls and other structures against the roadway plans, general topography, as-built jobsite condition, etc., will not only ensure a structure cohesive with its surroundings but could also forestall possible conflicts early in the project to minimize possible impacts on the project. One example is to plot the top of retaining walls and match the plot to the topography, smoothing out any unwanted dips or bumps, ensuring an aesthetically pleasing structure. It is also considered a good practice to plot elements of walls, such as weep holes to ensure that they will be above grade per the plans and in a nice alignment.

Concrete Forms

Concrete is a unique building material in that it may be cast into any desired shape while in the plastic state, and it will then retain that shape after hardening. Concrete is given its intended shape through the use of forms, which must be built in such a manner that the resulting concrete member will be correct as to size and shape. Forms also serve to control alignment, elevation, and position of the concrete members within the completed structure.

The term formwork can be defined in the broadest sense to include the total system of support for freshly placed concrete – from sheathing to all supporting members, hardware and necessary bracing. Concrete forms are more than just mere molds that give the hardened concrete its final shape, dimension and position. It is also the system that renders texture and surface characteristics to the formed concrete surface. As such, the quality of workmanship taken in building the form often determines the amount of subsequent work needed to obtain the required finished surface.

Specifications

Aside from the basic requirement in the specifications that all concrete forms shall be mortar tight, true to the dimensions, lines, and grades of the structure, and of sufficient strength to prevent appreciable deflection during the placing of concrete, the specifications place particular emphasis on the quality and workmanship of forms that are used for exposed concrete surface.

Fundamentally, forms for concrete surfaces exposed to view shall produce a smooth surface of uniform color and texture. Due to its importance in achieving a uniform concrete surface, the specifications require that forms for exposed surfaces shall be faced with form panels and that each element shall be formed with the same forming material or with materials that produce similar concrete surface textures, color, and appearance. The specifications define form panels as a continuous section of form facing material, unbroken by joint marks. The form panels for exposed surface shall be in good condition, free of defects, such as scars, dents, or delaminated areas.

The specifications explicitly require that form panels for exposed surfaces shall be plywood conforming to or exceeding the requirements of U.S. Product Standard PS 1 for Exterior B-B (Concrete Form) Class I Plywood, or any material which will produce a smooth uniform concrete surface substantially equal to that which would result from the use of that plywood. Figure 5-1 is a typical grade stamp.

Figure 5-1. Typical PS-1 Grade Stamp.

Form panels for exposed concrete surfaces shall be arranged in symmetrical patterns conforming to the general lines of the structure with uniform widths of not less than 3 feet and in uniform lengths of not less than 6 feet, except at the end of continuously formed surfaces where the final panel length is less than 3 feet[1] or where the width of the member formed is less than 3 feet. Furthermore, it is required that the forms for exposed surfaces shall be constructed with chamfers at all sharp edges of the concrete to prevent mortar runs.

For exposed vertical surfaces, the specifications also require that panels shall be placed with the long dimension horizontal and with its joints level and continuous. There are also specific requirements for form panels for curved surfaces of columns and walls with sloping footings.

[1] 2006 Standard Specification , Section 51-1.05, or 6 feet in 2010 Standard Specifications, Section 51-1.03C(2)(a).

Forming System

For most structure concrete work, the forming system will consist of a form surface material or sheathing, studs, wales, and kickers; all adequately braced to retain the concrete within the required line, grade and dimensions. In addition to the basic form components of sheathing, studs and wales, a complete forming system also includes forming accessories such as ties, anchors and spreaders.

Sheathing is the forming material that is in direct contact with the concrete and is therefore the element that renders the texture and surface characteristics of the formed concrete. Plywood is by far the most common sheathing material but, depending on the type of construction and surface finish required, form sheathing can be surfaced lumber, steel and in some cases synthetic materials, such as form liners. The APA's publication "Concrete Forming Design and Construction Guide" is an excellent source of technical information on plywood sheathing material and form design.

Other sheathing material may be used in forms for exposed concrete surfaces as long as it produces a smooth uniform concrete surface substantially equal to that which would result from the use of specified plywood, such as metal forms. Increasingly form liners are being specified as a sheathing material for exposed concrete surface as a means to render architectural treatment to the formed concrete surface.

The form sheathing is reinforced by a system of studs and wales in order to allow it to withstand the load imposed by the concrete during placement. For jobsite-built plywood forms common in bridge construction for such custom-built bridge elements as abutments, retaining walls and box girder superstructure, studs are vertical form elements, usually made of dimensional lumber, to which the sheathing is nailed. Essentially, studs serve as sheathing reinforcing that allows the sheathing to confine the freshly placed concrete. The studs are spaced close enough to limit the deflection of the sheathing between the studs. Wales are horizontal form components that support the vertical studs. Wales are usually installed in pairs into which concrete ties are inserted to keep the form from separating. The actual stud and wale spacing is governed by deflection criteria, lateral concrete load, and allowable stresses. Figure 5-2 is a typical wall-forming system.

Figure 5-2. Typical Concrete Wall Forms.

Vertical concrete forms must be braced to resist lateral loads such as wind and construction loads. Kickers are usually installed diagonally between the form and the ground or stable platform to keep vertical forms plumb and provide a measure of stability to the whole system. Kickers are important because if the forms are not properly braced they may deflect, in which case remedial work may be necessary to obtain the required lines and grades.

Chamfers are wooden triangular shaped fillets used in bridge construction to prevent mortar runs and bevel the corners of various bridge elements. Forms for exposed surfaces shall be constructed with triangular fillets not less than 3/4″× 3/4″ attached to produce smooth straight chamfers at all sharp edges of the concrete. They are also used as a "grade strip" to control the grades of various bridge components.

Form Fasteners

Form fasteners are form bolts, clamps or other devices used as necessary to prevent spreading of the forms during concrete placement. Form fasteners and anchors shall be of those types that can be removed as required for form bolts per ordinary surface finish specifications without chipping, spalling, heating, or otherwise damaging the concrete surface. No metal shall be left closer than 1-1/2 inches to the surface of concrete.

Note: Using twisted wire loops to hold forms in position is not permitted.

Concrete form ties, also known as snap ties, he-bolts, she-bolts, taper ties, tie rods or form clamps are tensile units used to prevent the spreading of the forms during concrete placement. The most common ties used in bridge construction consist of an internal tension unit and an external holding device. Ties can have built-in spacers or spreaders to keep the forms apart at a set distance. Some ties are designed so that they can be removed, or have portions of them removed, at a certain depth from the surface of the concrete. There are two basic types of the internal tension unit: continuous single member and internal disconnecting member.

Continuous single member ties consist of a single piece tensile unit and a specially designed holding device that is used to hold the tensile unit tight against the exterior of the form. Some of these ties have an integral form-spreading feature that is built into the tie. The common concrete snap-tie used in bridge construction is snapped at a predetermined depth where the rod section has been weakened to facilitate “snapping.” Some single member ties, such as she-bolts and taper ties may be pulled completely from the concrete.

The internal disconnecting member in which the tensile unit has an inner part with threaded connections to removable external members (bolts), completes the tensile unit, and have varied devices for holding them against the outside of the form. The internal member generally remains in the concrete. This type of tie is available for light or medium loads but finds its greatest application under heavier construction loads.

Tie wedges are metal plates used in conjunction with concrete ties to distribute the load from the concrete tie to a bigger area on the formwork element, usually the form wales. Wedge shaped in profile, the tie wedge slips over the head of a snap tie and is used to tighten the snap tie against the form by driving the wedge until there is no slack in the concrete tie. After the tie is tight, the tie wedge is nailed to the form to ensure that it will not move and loosen during concrete placement.

Spreaders are devices that maintain the correct spacing between the opposite faces of a form, such as those of walls or bridge box girder stems. They may be integral or fabricated with the concrete ties or custom made, usually from dimensional (typically 2″ × 4″) lumber, cut in the field to the exact width of the formed element. Contractors may sometimes employ both a concrete tie with an integral spacer with a custom made wood spacer to ensure that the correct width is maintained, especially as the ties are tightened. Wood spreaders are never embedded in the concrete.

Form anchors are devices that are cast into the concrete for later use in supporting forms or for lifting pre-cast members. There are two basic parts: the embedded anchoring device, whose design varies with the load to be carried and the strength of concrete in the structure; and the bolt or other external fastener which is removed after use, leaving a set-back hole that must be patched. Driven type of anchorages, such as powder-actuated nails driven by “nail guns”, shall not be used for fastening forms or form supports to concrete, except as provided for in the specifications.

Form Liners

The term "form liner" refers to any sheet, plate, or layer of material attached directly to the inside face of forms to improve or alter the surface texture and quality of the finished concrete. While plywood, hardboard, and steel, have application as form liners, the term is commonly associated with form liners made out of synthetic or plastic materials. The principal types of plastic liners in current use are: elastomeric, rigid, and fiberglass reinforced.

Elastomeric liners are made of rubber-like plastic formulations that are flexible enough to be peeled away from cast concrete surfaces with slightly undercut areas. They require good support and usually are adhered to form sheathing. Tough, wear resistant; 100 to 200 uses are reported possible with reasonable care. Peeling capability is rarely lost if the liner remains attached to a rigid backing. A typical form liner panel is shown in Figure 5-3.

Rigid type form liner formulations, including ABS and poly-vinyl chloride sheets, are stiff enough to be considered self-supporting. They are attached to sheathing by nails, staples, or screws and are available in standard sheets up to 10 feet long, or on special order up to 30 feet lengths. Some manufacturers provide interlocking joints at the edges of the sheets to maintain continuity of pattern. The panels are particularly suited to ribbed or fluted wall surfaces.

Similar in appearance and function to other rigid plastics, but much stronger, fiberglass reinforced form liners have longer potential service life. Better quality glass fiber reinforced plastic liners have an extra gel coat of the plastic resin at the contact surface to keep glass fibers from blooming through the resin skin.

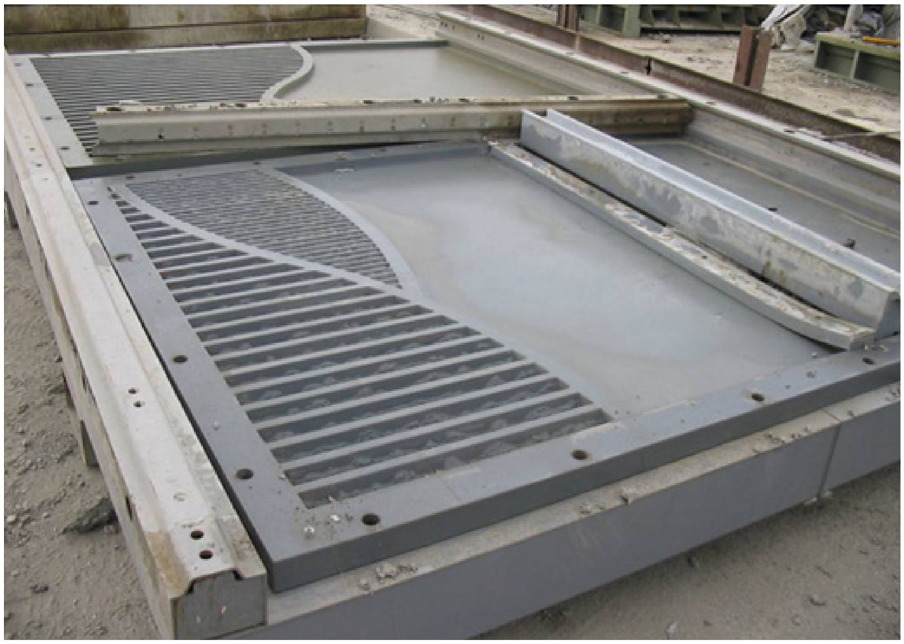

Figure 5-3. Form Liner Panel.

Figure 5-4. ABS Panel Liner and Resulting Wall.

Miscellaneous Form Types

While plywood is the predominant material used for jobsite-built forms for such custom-built bridge elements as abutments, retaining walls, and box girder superstructure, other form systems using different materials are usually used for bridge members with standardized shape and size, such as columns and barrier rails.

Steel Forms

The most common steel forms used in bridge construction are for columns (Figure 5-5) and barrier rails. These elements have a fairly standardized shape and size, which lends itself to the use of prefabricated forms. The durable nature of steel forms makes them economical for repeated use. Similar to wooden forms, steel forms consist of the form surface, studs, wales and kickers, all made out of steel. Steel forms are heavy and rigid, which while less prone to failure, are also limited in ability to follow certain dimensional requirements. For example, the steel forms for barrier rails come in straight 10-foot sections and cannot be placed to form barrier rails of tight radii. Steel forms are also commonly used in pre-casting yards where repeated use of a standard form is the norm.

Figure 5-5. Steel Column Forms.

Fiber Form Tubes

Fiber tube forms are typically used in bridge construction to form the exposed portion of "pile-extension" columns in slab bridges. Fiber tube forms are complete units with no extra fastenings and require only a minimum of external bracing to keep them plumb. This tube form, which consists of laminated fiber plies that are spirally constructed and are available with wax-impregnated inner and outer surface for weather and moisture protection, are considered single use items. Where the columns are to be exposed, the inner surface is coated with polyethylene. Fiber forms can be cut by saw to the exact length desired, and cut sections can be adapted to forming half-round, quarter-round, and round columns. They are also used to form the voids in voided-slab bridges. Sonotube is a proprietary trademark for the most widely available cylindrical fiber form.

Figure 5-6. Fiber Forms Used for Forming Voided Slab Structure.

Metal Decking

Metal decking (Figure 5-7) is a type of form that is left permanently in place and may become an integral part of the completed structure. When allowed by the contract, at the contractor's option and at no cost to the State, galvanized metal decking, which are ribbed or corrugated steel sheets, may be used in lieu of the typical forming system. If allowed, the specification requires that the steel deck forms conform to the requirements in ASTM Designation: A653/A653M (Designation SS, Grades 33 through 80) having a coating designation G165.

The specifications require that detailed shop drawings shall be submitted to the engineer for authorization. Metal decking is assumed not to provide additional support or reinforcement; hence, the deck reinforcement shown on the plans shall not be altered. The deck thickness shall not be reduced and must be measured from the surface of the bridge deck to the top of the metal decking ribs or flutes. Metal decking shall be galvanized and shall not be used in freeze-thaw areas[2].

Figure 5-7. Metal Deck Forms.

[2] Memo To Designers 8-7 and Standard Special Provisions 51-1.03.

Pre-Fabricated Forms

Specifications now allow the use of pre-fabricated plywood panels for the bridge superstructure soffit forms provided that when pre-fabricated panels are used, the form filler panels joining the pre-fabricated panels shall have a uniform minimum width of 1 foot and shall produce a smooth uniform surface with consistent longitudinal joint lines between the pre-fabricated panels. See BCM 125-3.0 “Prefabricated soffit forming panels” for further guidance.

Form Workmanship

Size, shape, and alignment of abutments, columns, superstructure, and other concrete structural elements depend on accurate construction of forms. The forms must be built to the correct dimensions, must be sufficiently rigid under the construction loads to maintain the designed shape of the concrete, must be stable and strong enough to maintain large members in alignment, and must be substantially constructed so they can withstand handling and reuse without losing their dimensional integrity.

The appearance of a finished concrete structure depends to a great extent on formwork quality, particularly the adequacy of the formwork to withstand, without appreciable settlement or deflection, the loads imposed by the concrete while it is in a plastic state. In turn, the behavior of the formwork under load depends on several factors, including the character of the foundation material, the support system used, and the quality of its construction, the quality of the form materials and workmanship, the forming method employed, and the rate and method of concrete placement. The formwork must remain in place until the concrete has hardened enough to maintain its cast shape, or the finished structure may be damaged.

The quality of surface finish of the concrete is affected by the material of the form. For example, if a patterned or textured finish is to be secured by use of a textured liner, the liner must be properly supported so that it will not deflect and cause indentations in the concrete surface. A correct combination of form material and releasing agent can contribute materially to eliminating air holes or other surface imperfections in the cast concrete.

With regard to compliance with the specification, keep in mind that the intent of the specification is to assure good workmanship in form construction. Compliance should be as important to the contractor as to the State, since the degree of care taken by the contractor when building forms will determine the amount of subsequent work needed to obtain the required finished product. For example, if panel joints are not tight, the resulting grout leakage will increase the surface finishing required. Dirty form surfaces and form panels having worn or ragged edges will also increase surface finishing costs. Improperly braced forms will deflect, in which case remedial work may be necessary to obtain the required line and grade.

Wherever the concrete surface is to be visible and appearance is important, the proper type of form tie or hanger that will not leave exposed metal at the concrete surface is essential. Specifications often require that no metal be left closer than 1 inch to the surface of the concrete. If any moisture gets to the tie end, rust stains will gradually appear on the surface of the concrete. Greater depth of break-back or threaded ends of internally disconnected tie units allow a better chance of bonding the patch which covers the tie, and also a greater factor of safety in case of spalling of the patch. Although the patch remains in place, it may shrink and leave fine cracks through which moisture and rust gradually seep. Because it is non-corroding, a glass fiber-reinforced plastic tie, commonly referred to as a fiberglass tie, is cut off flush with the concrete surface, leaving no hole to patch.

Structural Adequacy of Formwork

Concrete forms are more than just a mold that shape concrete into its desired size and shape. The forms are also temporary structures that maintain the shape while resisting fluid pressure loads imposed by the freshly placed concrete, plus those due to construction loading during its placement, including the effects of vibrating the concrete.

There are two distinct loading conditions of a complete formwork system, if broadly defined as a total system to include its supporting elements, that must be considered in evaluating the adequacy of the formwork:

- Lateral loading on the vertical surfaces of the concrete form due to the hydrostatic pressure of freshly placed concrete.
- Vertical loading on the formwork supporting elements due to, until it becomes self-supporting, the weight of the concrete. In Caltrans bridge construction practice, the system that supports the elevated vertical load is considered falsework and is subject to the stringent requirements in the falsework specifications.

Another design parameter required in the specifications is the limit on the allowable deflection of formwork and its various components, individually or as a system.

Formwork Design

For typical bridge construction, the formwork generally will be constructed in accordance with standard industry practice for concrete work. In most cases, a formwork design submittal is not required and inspection of bridge formwork is generally a routine procedure.

It should be noted, however, that temporary support systems supporting an elevated vertical load, mostly concreting loading, such as those for box girder superstructures, slab bridges, bent caps, etc., are considered falsework. The specification requires the contractor to submit falsework plans for authorization by the engineer.

Formwork Design Review

While formwork plans are not generally required, the specification, however, grants the engineer the option of requiring the contractor to submit formwork design plans, including supporting calculations and documentation, when deemed necessary. This may be required if the best general practices of the construction industry standards are absent. For example, a design submittal and a review by the engineer may be warranted for unique situations, unusual, possibly unsafe installations, or where a non-typical forming system will be used.

Form design and formwork plans, if requested by the engineer, are reviewed pursuant to the provisions in Section 4-1.01[3], "Intent of Plans and Specifications", Section 5-1.02, "Plans and Working Drawings," and Section 51-1.05[4], "Forms," of the Standard Specifications. Review and authorization protocols, while not specified, are generally similar to those established for falsework review.

Deflection

The specificat ons require that forms for exposed concrete surfaces shall be designed and constructed so that the formed surface of the concrete does not undulate more than either 3/32″ or 1/270 of the center to center distance between supports - studs, joists, form stiffeners, form fasteners or wales. This deflection criteria can serve as a parameter in the design of concrete forms.

Lateral Loading

The vertical surfaces of concrete forms are designed to resist the fluid pressure of concrete plus additional pressure generated by vibrating the concrete. While concrete is not a perfect fluid, if poured quickly, it will develop significant hydrostatic forces on the vertical surfaces of the forms that support the new concrete.

[3] 2006 Standard Specifications, or 2010 Standard Specifications, Section 4-1.02,"Intent".
[4] 2006 Standard Specifications, or 2010 Standard Specifications, Section 51-1.03C(2).

Since concrete is typically deposited in a purposeful manner, it will not act as a true fluid, determining the appropriate lateral loading to be used in form design becomes a subjective exercise. This lack of precision is due to the fact that the basic component of lateral loading, the fluid pressure of fresh concrete, is not only governed by the unit weight of concrete but is also affected by a number of variables, such as the type of cement, concrete temperature, concrete penetration (slump) and rate of concrete placement.

Unit Weight of Concrete

The basic component of the lateral loading on the forms is the unit weight of concrete. Freshly placed concrete behaves temporarily like a fluid imparting a hydrostatic pressure that acts laterally on the vertical surfaces of the forms. As a fluid, the concrete's hydrostatic pressure at any point in the fluid is created by the weight of the superimposed fluid. Consequently, the pressure on the form during concrete placement, if concrete is considered in a state of idealized fluid, follows the standard fluid pressure formula

$$P = wh$$

Where "w" is the unit weight of concrete in pounds per cubic foot (pcf) and "h" the height in feet of the superimposed concrete over a given point. Although it also includes the weight of the reinforcing bars, which is not a factor in lateral loading, 150 pcf is the commonly assumed unit weight of concrete for sake of convenience. It is considered conservative for most form design applications.

However, fresh concrete is a mixture of cementitious materials, solids, (aggregates) and water whose behavior only approximates that of a true liquid for a limited time. The effective lateral pressure used in form design is a modified hydrostatic pressure, where its basic component, the unit weight of concrete, is adjusted by concrete placement factors such as temperature of the concrete mix, rate of placement, the admixtures and cement blends used, and effect of vibration or other consolidation methods.

Rate of Placement

One of the significant factors that affect lateral pressure on concrete forms is the average rate of concrete placement, known colloquially, as the "pour rate." During concrete placement, the lateral pressure on the vertical surfaces of the concrete form at any given point is a product of the height of the concrete above it and the unit weight of concrete in its plastic state. In its plastic state, the lateral pressure at a given point increases as concrete depth above it increases.

However, as soon as concrete becomes less fluid it starts to lose its capacity to impose lateral pressure on the vertical surfaces of the form. Moreover, as concrete starts to set it will also lose its capability to translate the weight of subsequent concrete layers as additional lateral pressure on the forms. As concrete stiffens inside the form, it not only will support its own weight and no longer exert lateral pressure but will be able to carry the weight of the subsequent layer of concrete. This reduces the hydrostatic head only to those layers where concrete is still fluid thereby decreasing the effective fluid pressure on the form. With slower rates of placing, concrete at the bottom of the form begins to harden and the lateral pressure is reduced to less than full fluid pressure by the time concreting is completed in the upper parts of the form.

Other Factors

In addition to the pour rate, other factors such as temperature of the concrete, admixtures, have an effect on the lateral pressure imparted by the concrete on the forms.

Temperature of the concrete during its placement also influences effective pressure on concrete forms because it affects the setting time of concrete. At low temperatures, the concrete takes longer to stiffen and therefore a greater depth can be placed before the lower portion becomes firm enough to be self-supporting. The greater liquid head thus developed results in higher lateral pressures. It is particularly important to keep this in mind when designing forms for concrete to be placed not only during cold weather but also if fly ash or retarding admixtures are used in any weather.

The method of consolidating concrete inside the forms is also a factor that affects the magnitude of the effective fluid pressure. Consolidating concrete using internal vibrators results in temporary lateral pressures, because it causes concrete to behave as a fluid for the full depth of vibration (locally to the area of vibration), generating up to at least 10 - 20% greater pressure. If external vibrators are used, the loads it exerts on the forms must also be taken into consideration in form design as it essentially hammers the form against the concrete.

Admixtures

Chemical admixtures and supplementary cementitious materials have significant effect on lateral pressure, which must not be overlooked. The chemistry coefficient introduced in ACI 347-01 and continued in ACI 347-04 provides a way to quantify the effect of a number of these variables on lateral pressure.

Lateral Pressure Values (ACI 347-04)

The continuing advancement in concrete placement methods and, specifically the rapid progress in admixture technology resulted in significant revisions to American Concrete

Institute's recommended concrete fluid formulas used in form design. As admixtures and cement replacements proliferated, the American Concrete Institute (ACI) recognized that, given current construction practices, the previous form pressure recommendation might have had too small a margin of safety, which could potentially result in form failures. In ACI 347-04, ACI used the accumulating data on lateral pressures to revise the concrete pressure formulas, introducing two new coefficients to cover a variety of concrete mixes:

Cc : Chemistry coefficient
Cw : Unit weight coefficient

For normal Portland cement concrete weighing 150 pcf , having about 2-inch penetration reading, and with normal internal vibration, the ACI recommends use of the following formulas to determine the form design pressure:

For Columns: With a minimum of $600C_w$ psf; but less than $150h$

$$P = C_w C_c [150 + 9000(R/T)]$$

For Walls: With a minimum of $600C_w$ psf; but less than $150h$

If rate of concrete placement is 7 feet per hour or less:

Where: $h \leq 14$ feet:

$$P = C_w C_c [150 + 9000(R/T)]$$

Where: $h \geq 15$ feet:

$$P = C_w C_c [150 + 43{,}400/T + 2800(R/T)]$$

If rate (R) of concrete placement is 7 feet to 15 feet per hour and $h \geq 15$ feet

$$P = C_w C_c [150 + 43{,}400/T + 2800(R/T)]$$

Where: C_c is the chemistry coefficient
C_w is the unit weight coefficient
P is the design pressure, in psf
R is the rate of concrete placement, in feet per hour
T is the concrete temperature, in degrees Fahrenheit
h is the height of plastic concrete above the point under consideration
W is the unit weight of concrete

Table 5-1. Coefficient to be Used in Pressure Equations.

Chemistry Coefficients, C_c	
Types I, II, and III cement without a retarder*	1.0
Types I, II, and III cement with a retarder*	1.2
Other types or blends without retarders*, containing less than 70% slag or less than 40% fly ash	1.2
Other types or blends with retarders*, containing less than 70% slag or less than 40% fly ash	1.4
Blends containing more than 70% slag or 40% fly ash	1.4
Unit Weight Coefficient, C_w	
Concrete weighing less than 140 pcf: C_w=0.5(1+W/145) but not less than 0.80	
Concrete weighing 140 to 150 pcf C_w=1.0	
Concrete weighing more than 150 pcf C_w= W/145	
* Retarders include any admixtures such as a retarder, retarding water reducer, retarding mid-range water reducing admixtures, or retarding high-range water reducing admixture (superplasticizer) that delays setting of concrete.	

Form Release Agent

The specifications require that forms that will later be removed shall be thoroughly coated with a releasing agent prior to use. Release agents, colloquially known as form oil, are applied to form sheathings to prevent concrete from bonding to the form, permit its ready release, and keep the formwork clean. Although not specified, with the increasing use of non-lumber sheathing materials, such as form liners, application-specific form release agents warrant consideration, as different types of release agents affect the resulting formed concrete surface, including discoloration or staining. The specifications stipulate that the form release agent shall not discolor the concrete.

While the specifications only require a commercial quality form release agent that will permit the ready release of the forms, there are products that are specifically formulated as form release agents. In general, there are two broad categories of form release agents: barrier type and chemically active. Barrier types are water-insoluble materials that include neat oils, paraffin wax and silicone oil. The Environmental Protection Agency prohibits the use of uncut or straight diesel oil as a release agent. Chemically active agents are those that have fatty acids that chemically react with the basic materials in concrete and, essentially, produce soap. The formation of the soap film from the ingredients in the cement paste and the chemically active release agent prevents the concrete from bonding to the form surface. ACI 303R-04 is an excellent source of technical information on form release agents.

When used, form release agent should be applied uniformly over all form surfaces in the manner recommended by the manufacturer. Without "form release agents", the forms will invariably adhere to the surface of the concrete when the forms are stripped creating, in some cases, rough, irregular, spalled surfaces. The resulting effect necessitates costly, countless hours to refinish the exposed, concrete surface. A well-prepared contractor will take pains to ensure that the forms are saturated with form oil. During application, the specifications require that the form release agent cannot come in contact with the rebar.

There has been increasing concern on the use of form release agents in applications over bodies of water; check with the Storm Water Prevention Plan or regulations of the local Regional Water Quality Control Boards. There are also local and federal regulations on the volatile organic compounds (VOCs) that have to be considered.

Formwork Removal

Although from a casting standpoint forms may be removed as soon as the concrete has hardened, formwork must remain in place long enough to make sure the concrete is self-supporting and stiff enough to carry its own weight and construction loads without undue deflection or damage. Forms should be removed without damage to the concrete.

The specifications require that all forms be removed. The removal of forms that do not support the dead load of concrete members, other than railings and barriers, shall not begin until at least 24 hours after the concrete for the member has been placed. Note, however, that the specifications also require forms to remain in place until the concrete has gained sufficient strength to prevent surface damage. During periods of cold weather, this provision may require extending the 24-hour period. And in any case, the contractor should not be permitted to begin form removal until preparations have been made to begin curing as soon as the forms are removed.

To achieve the necessary strength, either the forms will be left in place for a specified period of time, or the time of removal will be determined by strength of test specimens. Improper stripping and shoring of bridge members may cause sagging of partially hardened concrete. Falsework, on the other hand, must remain in place until the supported concrete members have attained sufficient strength to support themselves.

Forms for railings or barriers may be removed at the convenience of the contractor after the concrete has hardened. The concrete surfaces exposed by removing forms shall be protected from damage. When the contractor elects to cure railings and barriers by a method other than the forms-in-place method, the forms must remain in place for a minimum of 12 hours. Forms for other concrete members, exclusive of forms supporting the dead load of a member, must remain in place for at least 24 hours after concrete has been placed for that member.

However, the specifications allow the forms to remain in place where the forms are inaccessible or where no permanent access is available. This may occur where there is no permanent access after construction, like, the forms for deck slabs of cast-in-place box girders, the forms for the interior voids of precast members, and the forms in hollow abutments or piers. When formwork will remain after construction, the inside of the cells or voids shall be cleared of all loose material prior to placing concrete.

Concrete Placement

After the concrete has been mixed and delivered to the jobsite, it must be conveyed to the proper location, placed, consolidated, and finished, all within a relatively short period of time. Even though the concrete as delivered is of excellent quality and capable of producing a strong, durable structure, the actual quality of the finished concrete will only be as good as the handling, placing, finishing, and curing methods employed.

Of the many operations involved in concrete construction, handling and placing concrete are the most critical. While proper placing procedure will not in itself ensure a quality concrete product, improper procedures will almost certainly guarantee a poor one. Therefore, the importance of knowing and following correct concrete placement procedures cannot be overemphasized. The basic requirement in the handling of concrete is that the concrete quality and uniformity, such as the water-cement ratio, concrete penetration, homogeneity, and air content have to be maintained throughout the concrete placement process if optimum concrete qualities are to be attained.

Prior to Placing Concrete

The specifications require that concrete shall not be deposited in forms until the work connected with constructing the forms has been completed, all materials required to be embedded in the concrete have been placed, and the engineer has inspected the forms and materials. Work to prepare for concrete placement includes the removal of all dirt, chips, sawdust, water, and other foreign materials from the forms.

Before work begins, the contractor's proposed placing procedure as well as applicable specification requirements should be reviewed. If there is doubt or uncertainty as to whether the contractor's proposed placing methods and procedures will achieve the desired results, the contractor should not be permitted to start the concrete pour. It is considered good practice that, not only all equipment intended to be used during the placement of the concrete must be clean and in good working condition, standby equipment should be available in the event of a breakdown.

The inspector must also ensure that all the requirements per the Storm Water Pollution Prevention Program (SWPPP) and any other requirements by environmental and regulatory agencies that are pertinent to concrete construction, such as concrete wash outs, etc., are adhered to faithfully and rigorously by the contractor.

Advance planning is essential when placing concrete because of the short time concrete remains in the plastic state. Once work is underway, there is no time for experimentation, and little or no time to correct mistakes. Once started, concrete placement should continue uninterrupted until the application of concrete cure. The specifications clearly stipulate that the concrete in each integral part of the structure shall be placed continuously and the contractor will not be allowed to commence work on any integral part unless sufficient material for the concrete is on hand and the contractor's forces and equipment are sufficient to complete the part without interruption in placing the concrete.

Concrete Delivery

Concrete is usually delivered to the job site via truck transit-mixers. However, the Standard Specifications, in effect, allow the contractor to transport concrete by any means of conveyance, providing the consistency and workability of the mixed concrete upon discharge at the delivery point is suitable for adequate placement and consolidation, and providing the mixed concrete, after hauling, conforms to the requirements of Section 90-6.01[5] "General" of the Standard Specifications. The other modes of transporting concrete, such as truck agitators, open top vehicles, barges, etc., are seldom used in a typical Caltrans box girder bridge construction project. However, some have been employed in special cases, and their use should be considered individually in view of the peculiar demands of the project.

[5] 2006 Standard Specifications, or 2010 Standard Specifications, Section 9-1.02G, "Mixing and Transporting".

Quality Assurance

When concrete is delivered to the jobsite, it is the inspector's responsibility to ensure that the concrete being delivered complies with the contract specifications. As the concrete is delivered, the construction phase of concrete production begins. Being in the construction phase, the quality assurance becomes the primary responsibility of the inspectors. Therefore, field engineers responsible for concrete inspection must have a complete understanding of the work to be done as well as the proper method or methods by which it may be accomplished.

Forms

Immediately before placing the concrete, the inside of the surface of the forms and subgrade should be thoroughly moistened with water.

Mix Consistency and Uniformity

The Standard Specifications require that all concrete be homogeneous and thoroughly mixed, and there shall be no lumps or evidence of undispersed cement. Field engineers in charge of concrete inspection should monitor the concrete stream as it is discharged from transit mixers or deposited into the work, not only for the aforementioned specified properties of fresh concrete, but also for evidence of deleterious materials, such as debris, broken bricks, or recycled concrete that may be unknowingly or unscrupulously incorporated into the concrete mix.

Variations in consistency of the mix should be avoided. Changes in penetration, grading, etc., have a cumulative effect on the ease of finishing, and are reflected in the finished surface. Variations in the consistency of the concrete may also be an indication of improper mixing, use of deleterious materials or unacceptable concrete mixing practices and may warrant rejection of the material and/or further investigation of the entire concrete production operation.

Concrete Tickets

The Standard Specifications require that each load of ready-mixed concrete delivered at the jobsite be accompanied by a weighmaster certifi ate, commonly known as "concrete tickets." Concrete tickets should be checked for conformance with specification requirements.

This certificate shall be provided in printed form and must show the mix identification number, non-repeating load number, date and time at which the materials were batched, the total amount of water added to the load, the reading of the revolution counter at the time the truck mixer is charged with cement, and the actual scale weights (pounds) for the ingredients batched. It should be pointed out that the specifi ation expressly prohibits the use of theoretical or target batch weights as a substitute for actual scale weights. BCM 100-3.0 "Transit Mixed Concrete" outlines the procedure to be used for checking concrete tickets.

Concrete Placement Time

Concrete begins to harden as soon as the cementitious materials and water are mixed, but the degree of stiffening that occurs in the first few minutes is not usually a problem. The specification requires that the concrete be placed within 90 minutes or before 250 revolutions of the drum, whichever occurs first, after the introduction of the cement to the aggregates. However, under conditions contributing to quick stiffening of the concrete, or when the temperature of the concrete is 85°F or above, the time allowed may be less than 1.5 hours.

Transit Mixer Drum Revolution

Concrete shall be placed before 250 revolutions of the drum or within 90 minutes, after the introduction of the cement to the aggregates. The minimum number of drum revolutions for transit mixers shall not be less than 70[6], at the mixing speed recommended by the transit mixer manufacturer – or not less than that recommended by the manufacturer of the equipment.

Transit mixers shall be equipped with electrically or mechanically actuated revolution counters by which the number of revolutions of the drum or blades may readily be verified. Mixed concrete may be transported to the delivery point in truck mixers operating at the speed designated by the manufacturer of the equipment as agitating speed, provided the consistency and workability of the mixed concrete upon discharge at the delivery point is suitable for adequate placement and consolidation in place.

Concrete Temperature

The inspector must ensure that the temperature of mixed concrete, immediately before placing, shall be not less than 50°F or more than 90°F. Aggregates and water shall be heated or cooled as necessary to produce concrete within these temperature limits. Neither aggregates nor mixing water shall be heated to exceed 150°F. If ice is used to cool the concrete, discharge of the mixer will not be permitted until all ice is melted.

Certificates of Compliance

The specifications require that the contractor furnish a Certificate of Compliance for each lot of material delivered to the work. For each batch concrete delivered to the jobsite, the

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[6] National Ready Mixed Concrete Association (NRMCA). Ready Mixed Concrete, "How is it produced?"

certificate of compliance shall have the concrete batch being certified clearly identified in the certificate, shall state that the materials involved comply in all respects with the requirements of the specifications, and shall be signed by the concrete supplier. The specifications state that the form of the Certificate of Compliance and its disposition shall be as directed by the engineer.

Materials used on the basis of a Certificate of Compliance may be sampled and tested at any time. The fact that material is used on the basis of a Certificate of Compliance shall not relieve the contractor of responsibility for incorporating material in the work which conforms to the requirements of the plans and specifications, and any material not conforming to the requirements will be subject to rejection whether in place or not. The Department reserves the right to refuse to permit the use of material on the basis of a Certificate of Compliance.

Weight Limits

Concrete trucks traveling on the highway with full loads generally need to use booster axles to meet the axle weight requirements of the California Vehicle Code. When discharging concrete, the booster wheels need to be raised, which increases the loads on the remaining axles resulting in axle loads that exceed the legal load allowed by the Permit Policy. Standard Specifications Section 7-1.02[7], "Load Limitations," allow trucks over legal (exceeding CVC weight limitations) limit on bridges with up to 28,000 pounds for single axles and 48,000 pounds for the tandem axles. This limits most trucks to hauling a maximum 7 1/2 to 8 cubic yards. These trucks should be weighed to confirm allowable specification loading

Adding Water to Concrete

CPD 10-5, "Adding Water to Concrete in the Field" allows water to be added to remix the concrete when the truck arrives on the jobsite and the concrete penetration is less than specified providing the following conditions are met:

- The maximum allowable water-cement ratio is not exceeded as calculated including surface water on aggregates as well as batch water and water added on site.
- Maximum allowable slump is not exceeded.
- Maximum allowable mixing and agitating time (or drum revolutions) are not exceeded.

After the additional water is introduced, the concrete must be remixed at mixing speed for a minimum of 30 revolutions or until the uniformity of the concrete is within the limits described in ASTM C94 (AASHTO M 157).

[7] 2006 Standard Specifications, or 2010 Standard Specifications, Section 5-1.37B, "Load Limits".

Water should be added before the truck discharges the first 1/4 cubic yard. After discharge is started, the contractor usually doesn't know the quantity of concrete with sufficient accuracy to add water with the assurance that the maximum water content will not be exceeded. Water should not be added to a partial load.

Indiscriminate addition of water to make concrete more fluid should not be allowed because it lowers the quality of concrete. The specification requires that concrete be placed while fresh and before it has taken an initial set. Retempering any partially hardened concrete with additional water is expressly prohibited. The later addition of water and remixing to re-temper the mixture can result in marked strength reduction. If early setting becomes a persistent problem, a retarder may be used to control early hydration, especially in high-cement-content mixes.

Delivery Rate

Although not specified, the contractor's expected concrete delivery rate warrants attention. An often overlooked consideration during concrete placement, the concrete delivery rate can have an impact, not only to the productivity of concrete placement, but potentially to the quality of the concrete, if, for example, delays result in cold joints or unplanned construction joints. The specifications are explicit in requiring that concrete in each integral part of the structure must be placed continuously, and the contractor should not be allowed to begin work unless there is sufficient inspected and approved material on hand, and his forces and equipment are sufficient, to complete the contemplated pour without interruption.

The secret of a successful pour is getting a good start and maintaining a constant delivery rate. If the concrete placement is too slow relative to the delivery rate, or if there is a breakdown in placing the concrete, the transit mixers arriving at the jobsite will begin to stack up, causing a delay in discharging their load and, often times, cause the concrete to exceed the specified 90 minutes mixing time. Also, if the delivery rate is too slow to assure a continuous supply of fresh concrete, the resulting interruptions in delivery will exacerbate any concrete placement problem that may develop during the pour, such as cold joints, and will extend the placement period as well, all with undesirable consequences. As a best practice, the interval between concrete delivery should not exceed 20 minutes and shall be adequate to prevent formation of cold joints.

Planning should eliminate or minimize any variables that would allow the concrete to stiffen to the extent that full consolidation is not achieved and finishing becomes difficult. Less time is available during conditions that hasten the stiffening process, such as hot and dry weather or use of accelerators.

Conveyance of Concrete

The method of concrete placement depends on such factors as the size, location, and accessibility of the concrete pour, and to a certain extent on the type of equipment available to do the work. The methods of delivering and handling concrete should be such that it will facilitate concrete placement with a minimum of handling or re-handling, and without damage to the structure or the concrete. The pumping system shown in Figure 5-8 was required to lift concrete almost 200 vertical feet for placement in the Benicia-Martinez Bridge.

There has been rapid improvement in the ability of concrete placing equipment, and particularly pumping systems, to deliver concrete to any desired location in the structure, at a uniform rate and without segregation. Keep in mind, however, that the use of state-of-the-art equipment will not necessarily ensure satisfactory results. A workable mixture of proper consistency along with care and attention to detail on the part of the placing crew are still the essential ingredients of satisfactory concrete placement.

The intent of this section is to give an inspector an insight into the advantages and disadvantages of each type of the commonly used concrete placement equipment in order to ensure that either the right equipment is used by the contractor or the inherent shortcoming of each system does not in any way become detrimental to the concrete being placed on structures.

Figure 5-8. Benicia-Martinez Concrete Pumping.

Chutes

Chutes are trough-shaped pieces of equipment set in an inclined position between the concrete delivery point and point of concrete placement. Chutes should be lined with smooth watertight materials and, to facilitate the flow of concrete, chutes should be rounded on the bottom and large enough to prevent overflow. When in use, the concrete is conveyed along the chute by the action of gravity.

When chutes are used, they must be placed at the inclination required to permit concrete of the specified consistency to flow easily down the chute. When the elevation of the forms is below the delivery point, chutes provide a cost-effective means of concrete placement.

Transit-mixers are equipped with delivery chutes (Figure 5-9) that could be conveniently used to place concrete directly into footings, abutments, or other locations where the truck can be positioned above the placement location.

The greatest objection to the use of chutes is the tendency for the concrete to segregate. To prevent segregation, chutes are often provided with baffle boards or a reversed section at the discharge end. The use of a small hopper at the discharge end is also effective in reducing segregation.

If delivery chutes are used as the sole means of conveying concrete, the inspector must ensure that the concrete is not deposited at just the location nearest to the transit mixers and then moved using vibrators, as this is also a cause for segregation. Also to prevent segregation, ensure that concrete does not fall more than 8 feet or as specified.

When using chutes, adding water or using vibrators to promote the free flow of concrete must not be allowed.

Figure 5-9. Concrete Transit Mixer Truck with Built-In Delivery Chute.

Concrete Buckets

Concrete buckets are still used in bridge construction where the pour volume is small or at remote locations where concrete pumps are not readily available. Although in a typical bridge construction project concrete buckets are usually lifted into position using cranes, hence the term "crane and bucket method", they can also be used with cableways and even with a helicopter to convey concrete directly from a central discharge point to where the concrete will be deposited or to a secondary discharge point.

A typical crane-and-bucket operation (Figure 5-10) will consist of one or more cranes, with each crane handling one or two concrete buckets. The placing rate will vary depending on the location of the crane with respect to the point of concrete placement and the type of work involved. A placing rate of 20 to 30 cubic yards per hour per crane is typical for most operations, although higher rates of about 50 cubic yards per hour, or more are possible with heavier equipment under ideal conditions.

Figure 5-10. Crane and Bucket Method.

Concrete buckets are available in various sizes. Large buckets with capacities of 8 cubic yards and higher have rectangular cross-sections (Figure 5-11), but most buckets in general use on construction work are circular. The lower part of the bucket has sides that slope toward a small gate at the center. Concrete is released by opening this gate. For bridge construction, bucket capacity usually varies between 1/2 and 2 cubic yards, with 3/4 and 1 cubic yard buckets being the most common.

The crane-and-bucket method is still used in bridge construction, even on a limited basis, because a crane is usually on site already for other phases of work, therefore, concrete placements do not require special equipment and setup. The crane has a high degree of mobility, which allows concrete placement under difficult conditions. Clean up of the concrete bucket is minimal. An advantage in the crane-and-bucket method is that, due to a minimal amount of concrete movement during its handling thereby reducing the risk of segregation, a homogeneous concrete mix is assured in most cases.

Figure 5-11. Concrete Bucket Used on the Benicia-Martinez Bridge Project.

The inspector must be aware of some disadvantages of the crane-and-bucket method. When the crane-and-bucket method is used, care should be taken so that while the bucket is being filled with concrete it is positioned on a sheet of plywood in order to catch the spills and to keep the bottom of the bucket frame and boot out of the dirt. Also the crane's radii must encompass the pour front and often there are areas that are inaccessible. High pour rates require the use of additional cranes, which leads to a safety problem with swinging booms. Overhead wires are a serious hazard. Impact due to concrete dropping from a high bucket can cause form failure.

Drop Chutes

Drop chutes are lengths of pipe or tubing that are used to facilitate placing of concrete in walls and columns, typically as a means of ensuring that the concrete does not fall beyond the specified drop limits. Some drop chutes are one-piece tubes made of flexible rubberized canvas or rubber; others are assembled from individual segments of metal cylinders that are fastened together in a manner that permits the lower segments to be removed as concrete is placed, also known as elephant trunks.

Drop chutes direct concrete inside forms and deliver the concrete to the bottom of the forms as a means of avoiding segregating the concrete. Using drop chutes greatly minimizes the contact between concrete and the bar reinforcing steel during concrete placement, which is important in avoiding segregation of the concrete.

Figure 5-12. Depositing Concrete Using a Drop Chute.

Drop chutes should be used in conjunction with a funnel-like device into which concrete can be introduced without spillage. The size of the drop chute should match the size of the form opening to ensure that it can be inserted without interfering with reinforcing steel. Drop chutes should be of sufficient length to ensure that concrete does not fall more than the specified drop limit of 8 feet or make concrete strike the reinforcing steel inside the forms.

Belt Conveyors

As the name implies, a conveyor belt system used in concrete placement consists of a series of portable, motor-driven, continuous belts that carry the concrete horizontally from the jobsite delivery point to the point where the concrete is to be placed, which may be a few hundred yards distant. The belts are supported by light steel framing which is set on the forms.

Belt conveyors have been used on concrete placement operations where there is a need for a continuous concrete placement but it is impractical or impossible to use concrete pumps. Conveyor belt systems have a relatively high capacity as compared to other systems in general use and delivery of concrete may be quite rapid. Conveyor belt systems offer the advantage of rapid delivery of concrete to relatively inaccessible locations. However, the use of conveyor belts on bridge construction is limited by economic considerations to situations where a large volume of concrete (several hundred cubic yards or more) is to be placed, or where the placement location cannot be reached by other types of placing equipment, such as pumps or cranes. Belts are utilized in areas that have impaired vertical clearances, traffic restrictions, and obstructions. Belts can produce pour rates of 65 cubic yards per hour.

There are truck mixers that are equipped with belt conveyors, which can be advantageous because the concrete mixer arrives complete with concrete conveying equipment. These mixer-mounted belt conveyors have adjustable reach and variable speeds. Similar to stand-alone belt conveyor systems, end discharge arrangements are necessary to prevent segregation and leave no mortar on the return belt.

The concrete inspector should be aware that a major disadvantage of belt conveyors is a tendency of the concrete to segregate at intermediate transfer points and at the discharge point. If segregation occurs, it may be necessary to install hoppers or drop-chutes at transfer points and some type of baffle or hopper to recombine the concrete at the discharge point before it is deposited into the forms.

In addition, concrete belt conveyors often require special supports or must be located along girder stems. Cleanup due to spillage is often a problem and care must be taken to place rugs or plastic sheathing at terminal points. Safety of the workers in the area of the terminal section requires special consideration. The chance of segregation is always present. Uneven pour fronts may result from removing support rail sections as the pour progresses, which could later make some areas inaccessible. Often unbalanced falsework loadings are encountered. Since the concrete while it is being conveyed on the belt is uncovered, it is exposed to the risk of rain on one hand, and the drying conditions such as high temperature and high winds on the other.

Because conveyor belts deliver concrete rapidly, it is important that there are enough people and sufficient equipment available to handle the concrete as it is delivered. Failure to correlate the capacity of a conveyor belt system with the size of the concrete placing and finishing crew will result in a reduced placement rate, and much of the inherent advantage of the conveyor belt will be lost. Also, since the concrete must be placed on both sides of the conveyor rail before it is moved back and the rail removed, the operator may fill one side completely before moving to the other. Rate of placement and placing sequence requires careful monitoring to assure proper vibration. Some belts must complete an entire 10-foot section before the finishing machine can move forward.

Concrete Pumps

Currently, concrete pumps are the most popular method for placing concrete. Truck-mounted pumps are more versatile and have higher pour rates than any other previously used method of conveyance. Present day pumps are expected to deliver up to 100 cubic yards per hour without any major breakdowns or malfunctions. More favorable consideration is given to pumps due to this greatly increased reliability. In the past, pumps could be expected to malfunction at least once during a pour. This increased reliability and higher pour rate can be attributed to improvements in pump design and the increased use of admixtures.

The typical mobile pumping system in use today consists of a pump mounted on a truck equipped with an adjustable boom. The concrete, which is deposited directly from the transit mixer into the pump hopper, is pumped through a system of pipes and hoses mounted on the boom to a 5-inch diameter delivery hose, also attached to the boom, which permits the concrete to be deposited at the exact location desired. Pumping rates of 100 cubic yards per hour and vertical lifts of 190 feet or more are easily attainable with today's mobile pumping equipment (the Benicia-Martinez Bridge Project pumped concrete at these rates. The pump truck used a four-segment reticulated arm to deliver the concrete).

Concrete pumps are very mobile and can change locations very quickly unlike cranes with buckets and other previously used methods of conveyance which are generally limited to a single or perhaps two locations for receiving ready-mix concrete. This is very important in keeping a fresh pour front-for-deck concrete placement. In areas where overhead airspace is congested with utility lines, etc., pumping is more advantageous because pumps normally require less headroom. Pumps also offer a less disruptive, ominous presence and are consequently less hazardous since the absence of swinging buckets or belts eliminates evasive maneuvering by the crew.

Aluminum pipe is not allowed in a concrete pumping system. There have been reported instances where pumping concrete through an aluminum pipe resulted in a significant reduction in concrete strength. The strength reduction has been attributed to voids in the cement paste, which are thought to be the result of hydrogen gas produced by a chemical reaction between abraded aluminum particles and certain constituents in Portland cement.

Slick Line

Slick line is a term used in the concrete pumping industry to describe a pumping system consisting of a portable concrete pump and a rigid conduit delivery system that may be used where concrete must be moved horizontally over a long distance, or where access limitations or other considerations preclude the use of mobile pumping equipment.

Figure 5-13. Slick Line Used to Deliver Concrete.

In a typical installation, the concrete is pumped through sections of 6-inch diameter steel pipe joined together to make a single conduit. A flexible delivery hose is connected to the outlet end of the conduit. Portable pumps are capable of moving concrete up to 1500 feet horizontally and 200 feet vertically, at pumping rates in excess of 100 cubic yards per hour. However, according to the American Concrete Pumping Association, the term “slick-line” refers to a pump primer as well as a concrete admixture to allow ease of flow of concrete through the pump and piping. At this time Caltrans Materials Engineering and Testing Services (METS) has not approved the slick line admixture.

Placing Concrete

Proper concrete placing techniques will prevent segregation, eliminate voids, and provide adequate bond strength between successive layers as the concrete is placed, and thereby achieve the "dense homogeneous concrete" intended by the specifications. It is important to note, however, that while proper placing procedures will not necessarily ensure a satisfactory result, improper procedures will almost certainly guarantee an unsatisfactory one. Therefore, the importance of knowing and following correct placing procedures cannot be overemphasized.

Depositing Concrete

Concrete should be deposited continuously as near as possible to its final position without objectionable segregation. The specification requires that the concrete should not be deposited in large piles and moved horizontally into final position. Neither should concrete be dumped into separate piles and then worked together into its final location because such practices result in segregation as mortar tends to flow ahead of the coarser material. In most bridge applications, such as the bridge deck or footings, concrete should be placed starting along the perimeter at one end of the work with each batch discharged against previously placed concrete.

For concrete to be placed in retaining walls, abutments, footings, columns or bents, or thick slabs it is considered good practice to place the concrete in horizontal layers of uniform thickness with each layer thoroughly consolidated before the next is placed. The rate of placement should be rapid enough so that previously placed concrete has not yet set when the next layer of concrete is placed upon it. Timely placement and adequate consolidation will prevent flow lines, seams, and planes of weakness (cold joints) that result from placing freshly mixed concrete on concrete past initial set. Layers should be about 6 to 20 inches thick for reinforced members and 15 to 20 inches thick for “mass concrete” work; the thickness will depend on the width between forms and the amount of reinforcement.

To avoid segregation, concrete should not be moved horizontally over too long a distance as it is being placed in forms or slabs. In some work, such as placing concrete in sloping wingwalls or beneath window openings in walls, it is necessary to move the concrete horizontally within the forms, but this should be kept to a minimum.

Where standing water is present, such as at the bottom of footings or drilled piles, as much of the water should be removed as possible, typically using sump pumps. If water is expected at the bottom of the excavation, it has been considered best practice to slope the bottom of the excavation to a corner where the pump can be installed. For the minimal amount of water left in the excavation, the concrete should be placed in a manner that will not cause it to segregate. In general, concrete should be placed in a manner that displaces the water ahead of the concrete but does not allow water to be mixed in with the concrete; to do so will reduce the quality of the concrete. In all cases, water should be prevented from collecting at the ends, in corners, and along faces of forms.

For spread footings or toes and keys of retaining walls without piles, care should be taken to avoid disturbing subgrade soils so they maintain sufficient bearing capacity to support structural loads. During concrete placement, reinforcing steel clearances should be continuously checked, displaced reinforcing steel should be repositioned, blocked and tied, broken dobies should be replaced and the position of waterstops, deck drains, conduit, and prestressing hardware and appurtenances should be checked and repositioned if displaced.

When concrete is placed in tall forms at a fairly rapid rate, some bleed water may collect on the top surface, especially with non air-entrained concrete. Bleeding can be reduced by placing more slowly and by using concrete of a stiffer consistency, particularly in the lower portion of the form.

When practical, concrete should be placed to a level about a foot below the top of tall forms and 1 hour or so allowed for the concrete to partially set. Placing should resume before the surface hardens to avoid formation of a cold joint. If practical to work around vertical reinforcing steel, it is good practice to overfill the form by 1 inch or so and cut off the excess concrete after it has stiffened and bleeding has ceased.

Fresh concrete should be placed against previously placed concrete rather than away from it. When it is necessary to place concrete on a slope, placement should begin at the lower end of the slope and progress upward. These practices allow vibration to follow immediately behind placement, thereby minimizing segregation. For bridge decks, the placing rate and location should be controlled to facilitate timely vibration. Uniform consistency of concrete and a uniform pour front parallel to the finishing machine should be maintained.

Concrete is sometimes placed through openings, called windows, in the sides of tall, narrow forms. When a chute discharges directly through the opening without controlling concrete flow at the end of the chute there is danger of segregation. A collecting hopper should be used outside the opening to permit the concrete to flow more smoothly through the opening; this will decrease the tendency to segregate.

Segregation

The specifications require that concrete be placed and consolidated by methods that will not cause segregation of the aggregates.

Segregation is the term used to describe non-uniform separation of the coarse-aggregate particles from the sand-cement components of the concrete mixture. Concrete is not a naturally homogeneous material; in its fluid state it may be sensitive to external forces that tend to separate the heavier coarse aggregate particles from the sand-cement mortar. When segregation occurs the concrete mixture becomes unbalanced, portions of the concrete will not have an excess of coarse aggregate and the resulting concrete will have air voids. The portion of segregated concrete with less coarse aggregate tends to, because it has relatively higher water and mortar content, shrink and crack more and have poorer resistance to abrasion. On the other hand, the portion of the segregated concrete with more coarse aggregate tends to be, because of the lack of cement paste, relatively speaking, less susceptible to consolidation, harder to finish, and prone to having concrete defects such as honeycombing.

Preventing segregation is the major consideration in handling and placing of concrete. The methods and equipment used in transporting, conveying, depositing, and consolidating the concrete must not result in the segregation of the concrete components. The two most important factors in preventing segregation are placing the concrete as closely as possible to its intended final location and keeping the drop of the concrete in a vertical direction to a minimum. Accordingly, once placed, concrete should not be moved laterally in the forms by vibrators, and concrete should not be placed by pushing or pulling a drop chute at an angle to the vertical.

The free fall distance is also a segregation factor. The free fall distance is limited by the specifications to 8 feet, but this should be viewed as the absolute maximum. Ideally, the fall-distance should be decreased to the extent possible to reduce the segregation that occurs when concrete strikes reinforcing steel or the sides of the forms above the placement level.

Prevention of Concrete Defects

Although concrete defects are occasionally attributable to an improper batching or mixing procedure, experience reveals that most concrete defects are the direct result of a failure or breakdown in the concrete placing operation. In view of this, it is evident that proper placing methods and procedures along with continuous inspection as the work progresses are the keys to obtaining a defect-free concrete structure. Accordingly, field personnel who have concrete inspection responsibilities will be expected to take affirmative action as necessary to ensure that recommended construction practice is followed at all times.

The most frequently occurring defect in concrete construction is the rock pocket. The term "rock pocket" is used informally to describe anything from a slight surface defect to a hole large enough for a man to crawl through; however, as used herein a "rock pocket" is understood to mean either (1) a portion of the hardened concrete in which the materials have segregated, thus resulting in the formation of a cluster of coarse aggregate particles that are unbonded or only slightly bonded to the surrounding matrix of fine aggregate and cement, or (2) a void within a larger concrete mass in which no concrete is present.

Most rock pockets are the result of improper placing procedures, usually insufficient vibration. When compared to the typical commercial product, concrete used on State highway work has a much lower slump; hence it is stiffer and more difficult to place and consolidate. Thorough vibration is a must if rock pockets are to be avoided.

When deposited into the forms, stiff concrete has a pronounced tendency to "hang up" on the reinforcing steel. This tendency is particularly troublesome in wall forms, where placing too much concrete in a single lift often results in rock pockets at the bottom of the lift. To avoid such rock pockets, lifts should be shallow and the vibrator should penetrate through the fresh concrete into the concrete in the previously placed lift. The more reinforcing steel there is in the forms, and the narrower the forms, the more care is required to ensure adequate vibration and thus avoid the formation of rock pockets.

Voids occasionally occur beneath hinges and expansion joint armor. These voids are caused by entrapped air, and are usually the result of following an improper placing sequence that prevents adequate and/or timely vibration.

Rock pockets may occur when mortar is lost through open joints in improperly constructed forms. Forms that are not mortar tight permit the fines to leak out, causing a rock pocket. This type of pocket is usually quite shallow and of the "popcorn" type. The only cure is mortar tight forms. Occasionally a rock pocket will be formed simply because there is an excessive amount of rock in the mix, but such occurrences are usually attributable to a batching error rather than to an improperly designed mix.

Cold joints are often accompanied by rock pockets. When fresh concrete is placed on the old layer, the vibrator head should penetrate the older layer sufficiently to ensure adequate blending of the new and old concrete.

Sand streaking is the exposure of sand on a concrete surface. It is usually caused by wet mixes that result in excessive bleeding. The use of a stiffer mix and/or air-entrained concrete will greatly reduce bleeding and thereby reduce sand streaking as well. Sand streaking is not usually associated with the type of concrete mixes used for structure construction in California, which is fortunate since it is a defect that is virtually impossible to repair.

Laitance is a soupy mixture of cement, fine sand and water that accumulates on a horizontal concrete surface. Any laitance on a construction joint will be removed when the joint is cleaned in accordance with specification requirements. Laitance occurs on finished surfaces, such as the top of a retaining wall, and will produce a soft surface that is vulnerable to deterioration from the effects of weathering. Laitance is caused by bleed water accumulating on the surface of an excessively wet concrete mix. It is less likely to occur in air-entrained mixes since air entrainment reduces bleeding significantly.

Soft or powdery surfaces may result from several causes, but inadequate curing is the major cause of this type of defect. Once the concrete dries out, the hardening process stops. Proper curing is essential to ensure a sound, powder-free surface.

Protecting Concrete

Unless precautions are taken, adverse weather conditions during and immediately after concrete placement can contribute to undesirable properties in the finished product. For example, while light rain is not always harmful, concrete should not be placed when it is exposed to heavy rain. Rain will dilute the mortar at the concrete surface, and if an appreciable amount of rain falls, it may increase the water-cement ratio sufficiently to decrease both strength and durability at the surface.

The Standard Specifications provide that under rainy conditions, placing of concrete shall be stopped before the quantity of surface water is sufficient to damage surface mortar or cause a flow or wash of the concrete surface, unless the contractor provides adequate protection against damage. Admittedly, administering this particular requirement will require the use of subjective judgment, but the point to keep in mind is that if more than a light rainfall occurs while a pour is in progress, consideration should be given to ordering a suspension until the rain stops, unless the contractor is willing to shelter the work area or otherwise mitigate the potentially adverse effect of moisture contamination.

Prior to the start of the concrete placement operation, plans should be in place that will be followed in the event of rain during the concrete placing operation, especially for concreting operation involving significant areas of concrete that could be exposed to the rain, such as bridge deck. Protective coverings such as polyethylene sheets or tarpaulins should be available and onsite at all times.

The specifications also require that concrete shall not be placed on frozen or ice-coated ground or subgrade or on ice-coated forms, reinforcing steel, structural steel, conduits, precast members, or construction joints. Concrete that has been frozen or damaged by other causes, as determined by the engineer, shall be removed and replaced by the contractor at the contractor's expense. Structure concrete and shotcrete used as structure concrete shall be maintained at a temperature of not less than 45°F for 72 hours after placing and at not less than 40°F for an additional 4 days. When required by the engineer, the contractor shall submit a written plan of the proposed methods for protecting the concrete.

Consolidating Concrete

Consolidation is the process of compacting fresh concrete, to mold it within the forms and the reinforcement and embedded items, and to eliminate rock pockets, honeycomb, and entrapped air. Consolidation can be accomplished by hand or by mechanical methods. The Standard Specifications require that concrete be placed and consolidated by methods that will not cause segregation of aggregates and will result in a dense homogeneous concrete that is free of voids and rock pockets.

Vibrating Concrete

Vibrating the concrete, using either internal or external vibrators, is the most common method of consolidating concrete. The purpose of vibration is to consolidate the concrete into a dense uniform mass free of voids and entrapped air. When done correctly, vibration ensures maximum consolidation of the concrete without causing segregation, and without resulting in an excessive flow of water and fine particles to the surface. In air-entrained concrete, it should not remove significant amounts of entrained air in the concrete mix.

Prior to vibration concrete presents a dry, irregular surface, while vibrated concrete presents a distinctively different appearance. When concrete is vibrated, the internal friction between the aggregate particles is temporarily destroyed and the concrete behaves like a liquid; it settles in the forms under the action of gravity and the large entrapped air voids rise more easily to the surface. A vibrated concrete takes on a moist appearance as the fines move to the top and the large aggregates settle. Internal friction is re-established as soon as vibration stops.

The high frequency vibrators required for concrete consolidation by the Standard Specifications vibrate in an approximate range of 80 to 250 cycles per second. Cycles per second are also referred to as Hertz (Hz). Vibrators may be classified by vibration rate or vibration amplitude, which are inversely related. If the vibration rate appears inadequate, the rate can be measured using a vibrating reed tachometer.

Internal Vibrators

The specifications require all structure concrete to be consolidated with high frequency internal vibrators within 15 minutes after it is placed in the form, with the exception of concrete for certain minor structures and concrete placed under water.

The vibrator type required by the specifications is known in the construction industry as an immersion or internal vibrator, since in use it is “immersed” into the concrete. Flexible-shaft vibrators consist of a vibrating head connected to a driving motor by a flexible shaft. Inside the head, an unbalanced weight connected to the shaft rotates at high speed, causing the head to revolve in a circular orbit. The motor can be powered by electricity, gasoline, or air. The vibrating head is usually cylindrical with a diameter ranging from 3/4 to 7 inches. The smaller diameter heads have the highest vibration rates, possibly exceeding 250 Hz and the smallest amplitude, less than 0.02 inches. Vibration rate and head diameter are also inversely related, vibration rate decreases as diameter increases. As head diameter increases, vibration amplitude also increases. Larger diameter vibrators consolidate larger areas of concrete; the consolidation of a 2-inch vibrator might be 6 inches in radius while a 3-inch vibrator could have an effective radius of 14 inches.

Some vibrators have an electric motor built right into the head, which is generally at least 2 inches in diameter. The dimensions of the vibrator head as well as its frequency and amplitude in conjunction with the workability of the mixture affect the performance of a vibrator.

Proper use of an internal vibrator is important in order to obtain the optimum benefit of concrete consolidation.

Where possible, the vibrator (Figure 5-14) should be inserted vertically and allowed to descend through the concrete by the action of gravity. The points of vibration should be evenly spaced about 24 inches apart; however, the actual spacing should be adjusted so there is some overlapping of the vibrated areas. It should penetrate the layer being placed and at least 6 inches into any previously placed layer to ensure a thorough combining of the two lifts and to prevent “cold joints”. The distance between insertions should be about one to one and one-half times the radius of action so that the area visibly affected by the vibrator overlaps the adjacent previously vibrated area by a few inches.

Figure 5-14. Vibrating Box Girder Stem Concrete.

Once the vibrator has been inserted, it should be held steady as consolidation occurs and then withdrawn slowly. Vibrators consolidate concrete by pushing the coarse aggregate down and away from the point of vibration. This action induces the accumulation of cement paste around the vibrator, usually within 5 to 15 seconds. When the paste first appears near the top of the vibrator head, the vibrator should be withdrawn vertically at about the same rate that it descended.

Skilled operators using vibrators will know the depth of the recently poured layer of concrete and will mark a point along the vibrator hose a distance of the depth of the recent pour plus 1 foot. By dipping the hose into the fresh concrete up to the marked point, the operator of the vibrator will minimize the existence of "cold joints". Experience has shown that concrete is not adversely affected when the lower lifts are revibrated, or by vibration transmitted by embedded reinforcing steel, provided the concrete is still plastic or again becomes plastic under revibration. The height of each layer or lift should be about the length of the vibrator head or generally a maximum of 20 inches in regular formwork.

In thin slabs, the vibrator should be inserted at an angle or horizontally in order to keep the vibrator head completely immersed. In bridge decks and similar thin sections where vertical insertion is not feasible, the concrete may be consolidated with the vibrator in a sloping position.

For a given consistency of concrete there is an optimum amount of vibration that will result in maximum consolidation without appreciable segregation. The length of time a vibrator should be left in the concrete is a function of slump. An insertion time of 5 to 15 seconds will usually provide adequate consolidation. The concrete should move to fill the hole left by the vibrator on withdrawal. If the hole does not refill, reinsertion of the vibrator at a nearby point should solve the problem. To obtain the same degree of consolidation, low-slump concrete requires more vibration than concrete having a higher slump.

Although it appears easy to follow the vibrator manufacturer's use instructions, proper operation requires experience. There are visible changes that occur in concrete as it undergoes consolidation. Larger aggregate settles toward the bottom surface and the upper surface becomes smoother as mortar rises and entrapped air is released. Excessive vibration may reduce the benefits of air entrainment. An experienced vibrator will recognize the changes that occur as consolidation progresses and move the vibrator to the next consolidation location as needed.

The technique of the operator should vary with the depths and complexity of the section. In deep sections where it is possible to get full penetration of the vibrator, it is imperative that the person operating the vibrator hit the concrete approximately every 2 feet and the head of the vibrator enter almost vertically. In thin deck sections the 2 feet separations must also be observed but it is not as important to enter the concrete vertically.

Vibrators should not be used to move concrete horizontally since this will cause segregation. Allowing a vibrator to remain immersed in concrete after paste accumulates over the head is bad practice and can result in non-uniformity. The length of time that a vibrator should be left in the concrete will depend on the workability of the concrete, power of the vibrator, and the nature of the section being consolidated. The vibrator should not be dragged horizontally over the top of the concrete surface. Neither should the vibrator be allowed to run continuously while the operator is occupied with other things. Special care must be taken in vibrating areas where there is a high concentration of reinforcing steel. The vibrator should not be dragged around randomly in the slab. For slabs on grade the vibrator should not make contact with the subgrade.

It is considered best practice that a standby vibrator and generator should be on hand at all times in the event of mechanical breakdown.

Where epoxy-coated rebar or epoxy-coated prestressing steel are used, rubber tipped or resilient-coated vibrator heads should be employed to prevent damage to the epoxy coating on the rebar.

External Vibrators

At locations where the concrete placement configuration precludes the use of internal vibrators, the specification requires the use of external vibrators. When the use of an external vibrator is necessary, the specification requires that the vibrator be attached to the form. As such, the forms must be sufficiently rigid to resist movement and to withstand the forces induced by the external vibrators. Form vibrators, designed to be securely attached to the outside of the forms, are especially useful for consolidating concrete in members that are very thin or congested with reinforcement, to supplement internal vibration, and for stiff mixes where internal vibrators may not be effective. Form vibrators can be either electrically or pneumatically operated.

External vibrators should be positioned about 18 inches below the top surface of the concrete, and they must be moved as necessary to maintain this relative position. They should be spaced to distribute the intensity of vibration uniformly over the form; optimum spacing is best found by experimentation. They are ineffective when operating on empty forms or when positioned more than about 2 feet below the concrete surface. Form vibrators should not be applied within the top yard of vertical forms. Vibration of the top of the form, particularly if the form is thin or inadequately stiffened, causes an in-and-out movement that can create a gap between the concrete and the form. Internal vibrators are recommended for use in this area of vertical forms.

Sometimes it may be necessary to operate some of the form vibrators at a different frequency for better results; therefore, it is recommended that form vibrators be equipped with controls to regulate their frequency and amplitude. Duration of external vibration is considerably longer than for internal vibration, generally between 1 and 2 minutes.

External vibration is not always effective. Best results are attained when the vibrators are securely fastened to the exterior surface at points where form bracing will transmit the vibrations to the nearby concrete.

In heavily reinforced sections where an internal vibrator cannot be inserted, it is sometimes helpful to vibrate the reinforcing bars by attaching a form vibrator to the exposed portions of rebars. This practice eliminates air and water trapped under the reinforcing bars and increases the bond between the bars and surrounding concrete. It is recommended that this technique be used only if the concrete is still workable under the action of vibration. Internal vibrators should not be attached to reinforcing bars for this purpose because the vibrators may be damaged.

Consequences of Improper Vibration

Insufficient vibration may result in voids and rock pockets remaining in the concrete, whereas excessive vibration will cause segregation, increase the amount of surface water, and leave a layer of mortar at the surface. However, for the concrete specified for structure work on State projects, unsatisfactory results are much more likely to occur as the result of too little vibration than from too much. This should be kept in mind when vibrating heavily reinforced sections where special care is required to assure proper consolidation.

Under vibrating concrete causes serious concrete defects that could be detrimental to the quality of concrete and to the integrity of the concrete structural member. Some of the worst defects caused by poorly consolidated concrete include honeycombs, excessive amounts of entrapped air voids sand streaks, cold joints, placement lines, and subsidence cracking.

Honeycomb results when the spaces between coarse aggregate particles do not become filled with mortar. Faulty equipment, improper placement procedures, a concrete mix containing too much coarse aggregate, or congested reinforcement can cause honeycomb.

Excessive entrapped air voids, often called bug holes, are similar to, but not as severe as honeycomb. Bug holes are small or irregular cavities found on the surface of hardened concrete, usually less than 0.6 inch in diameter, that are air voids formed by the entrapment of air bubbles against the forms, especially impervious forms such as steel or plastic form liners. Vibratory equipment and operating procedures are the primary causes of excessive entrapped air voids, but the other causes of honeycomb apply too. If they become a problem, the amount of bug holes can be mitigated by assiduous utilization of proper vibrating techniques. Revibration may be employed to reduce the size and intensity of bug holes. Other mitigation measures such as higher slump, using high range water reducers, and using smaller aggregates to improve workability have often been successful.

Sand streaks result when heavy bleeding washes mortar out from along the form. A wet, harsh mixture that lacks workability because of an insufficient amount of mortar or fine aggregate may cause sand streaking. Segregation from striking reinforcing steel without adequate vibration may also contribute to streaking.

Cold joints are a discontinuity resulting from a delay in placement that allowed one layer to harden before the adjacent concrete was placed. The discontinuity can reduce the structural integrity of a concrete member if the successive lifts did not properly bond together. The concrete can be kept alive by revibration every 15 minutes or less depending on job conditions. However, once the time of initial setting approaches, vibration should be discontinued and the surface should be suitably prepared for the additional concrete.

Placement lines or "pour" lines are dark lines between adjacent placements of concrete batches. They may occur if, while vibrating the overlying layer, the vibrator did not penetrate the underlying layer enough to knit the layers together.

Subsidence cracking may occur at or near the initial setting time as concrete settles over reinforcing steel in relatively deep elements that have not been adequately vibrated. Revibration at the latest time that the vibrator will sink into the concrete under its own weight may eliminate these cracks.

On the other hand, overvibration can also cause concrete defects such as segregation as vibration and gravity causes heavier aggregates to settle while lighter aggregates rise, sand streaks, loss of entrained air in air-entrained concrete, excessive form deflections or form damage and form failure caused by excessive pressure from vibrating the same location too long, and/or placing concrete more quickly than the designed rate of pour.

For a given consistency of concrete there is an optimum amount of vibration that will result in maximum consolidation without appreciable segregation. The length of time a vibrator should be left in the concrete is a function of slump. To obtain the same degree of consolidation, low slump concrete requires more vibration than concrete having a higher slump. In general, with everything being equal, it is considered that under vibrating the concrete is more often a problem than over vibrating the concrete. This should be kept in mind when vibrating heavily reinforced sections where special care is required to assure proper consolidation.

Revibration

The specifications require that after placing, consolidating and initial screeding of concrete for structure footings, more than 2-1/2 feet in vertical dimension and with a top layer of reinforcement, the concrete shall be reconsolidated by the use of internal vibrators for a depth of 1 foot from the top of the footing and then finished. Revibration shall be accomplished as late as the concrete will again respond to vibration, but not less than 15 minutes after the initial screeding has been completed.

Revibration of previously compacted concrete can be done to both fresh concrete as well as any underlying layer that has partially hardened. Revibration can be used to improve bond between concrete and reinforcing steel, release water trapped under horizontal reinforcing bars, and remove additional entrapped air voids. In general, if concrete becomes workable under revibration the practice is not harmful and may be beneficial.

Finishing Plastic Concrete

Unless otherwise specified, after concrete has been consolidated but before application of the curing medium, surfaces of bridge concrete that are not in contact with the forms receive an initial concrete finish, which consists of striking off the top of the concrete to the planned level, grade, or slope and the surface is then finished by floating to seal the concrete surface. All concrete finishing work at this stage shall be performed while the concrete is still in a workable stage.

Bleed Water

Even after concrete has been vibrated as specified, additional consolidation takes place as the heavier materials slowly settle through the mixture. In this process, which is commonly called subsidence or settlement, free water rises to the surface as it is displaced by the heavier particles. Free water appearing on the surface of the concrete is called "bleed water". Ideally, the initial finishing should be completed before bleed water begins to collect on the surface. The concrete surface should then remain undisturbed until the bleed water has evaporated and the surface takes on a dull appearance.

Finishing while bleed water is on the surface is one of the principal causes of defects on concrete surfaces. If bleed water is worked into the surface, the water-cement ratio is significantly increased which reduces strength, entrained-air content, and water tightness of the concrete surface. Also any finishing operation performed on the concrete surfaces while water is present can cause crazing, dusting, or scaling. Floating and troweling the concrete before the bleeding process is completed may also trap bleed water under the finished surface producing a weakened zone or void under the finished surface, occasionally resulting in delamination. The use of low-penetration concrete, provided it has sufficient cement content and a properly graded fine aggregate, will minimize bleeding. Air entrainment also reduces bleeding.

Strike Off

After concrete is vibrated, the exposed concrete surface must be brought to the final level. Strike off, also called screeding, is the process of cutting excess concrete to bring the top surface of the concrete to proper grade. The most common device used in manually striking off concrete is the use of a straight edge, which in most instances in bridge construction, aside from bridge deck, is normally accomplished by using a piece of lumber, usually a 2×4 cut to a convenient length (Figure 5-15), to bring the concrete surface to the proper level as indicated by grade strips, screed rails or by top of forms.

In the manual method of striking off the concrete, the concrete is brought to the proper level by moving a straight edge across the concrete with a sawing motion while advancing forward a short distance with each straightedge to fill in low areas as the straightedge advances. The 2×4 straight edge is moved across the surface with a sawing motion at right angles to the direction of travel. If screed rails are used, any interior supports are removed after the grade is established and any voids left by the removal of the support then filled with concrete.

Figure 5-15. Striking Off Bridge Deck Concrete.

Initial Finishing

In bridge construction, initial finishing is usually accomplished using hand floats, except on bridge decks where mechanical finishing machines are the norm. The floating action, performed by moving the slightly angled float back and forth across the surface of the concrete pushes aggregate down to surface level as the float is extended and pulls cream (water and fine concrete materials) to the surface as the float is pulled back. Floating removes high points and fills low points, while leaving the surface unsealed. An unsealed surface allows bleed water to rise through the concrete pores.

Floats can be made from wood, metal or fiberglass (Figure 5-16). Concrete without air entrainment would tend to have more water in the concrete and a wood float would leave the roughest smooth finish allowing the most bleed water to rise to the surface. Fiberglass also produces a rough surface, but resists the abrasive forces of aggregate and sand that cause wooden floats to become rougher. Air entrained concrete typically arrives with less water in the mix design and produces less bleed water than concrete without air entrainment. A metal float (Figure 5-17) would be more durable than either a wooden or fiberglass float and produce a slightly smoother finish, that would still be rough enough to allow bleed water to escape.

The floating action, which is performed with a scrubbing motion, slightly depresses the larger aggregate particles, leaving a thin layer of surface mortar suitable for final finishing.

When floating concrete, the hand float should be held flat on the concrete surface and moved with a slight sawing motion in a sweeping arc to fill in holes, cut off lumps, and smooth ridges. When finishing large slabs, such as building slabs, power floats can be used to reduce finishing time. Floating produces a relatively even (but not smooth) texture that has good slip resistance and is often used as a final finish for most bridge concrete surfaces. Where a float finish is the desired final finish, it may be necessary to float the surface a second time after it has hardened a little more.

Floating must be completed before bleed water accumulates on the surface. Care must be taken not to overwork the concrete as this could result in a less durable surface. The preceding operations should level, shape, and smooth the surface and work up a slight amount of cement paste. Although sometimes no further finishing is required on most bridge construction finishing, in floor slabs for buildings floating is followed by one or more of the following finishing operations: edging, jointing, floating, troweling, and brooming. A slight hardening of the concrete is necessary before the start of any of these follow-up finishing operations. When the bleed-water sheen has evaporated and the concrete will sustain foot pressure with only about 4-inch indentation, the surface is ready for continued finishing operations.

Figure 5-18 is a photo taken in 1960 showing work on the San Francisco–Oakland Bay Bridge. Concrete was delivered to the pour location in a buggy, placed by shovel and consolidated with a jitterbug. The jitterbug was once a commonly used tool for initial finishing. Jitterbugs are no longer approved for use; internal and external vibration techniques have superseded the jitterbugs usefulness.

Figure 5-16. Hand Floating a Bridge Deck.

Figure 5-17. Metal Bull Float.

Figure 5-18. Jitterbugs are Not Permitted.

To facilitate finish ng some concrete finishers are accustomed to compacting concrete surface with a hand tamp with steel mesh stretched over a frame commonly called a "jitter-bug". Jitterbugging wet concrete forces the coarse aggregate particles down, and brings excessive cement and fine aggregate to the surface, which results in soft surface that leads to crazing and cracking and, for exposed surfaces, dusting and scaling. Jitterbugging is not considered good practice for concrete work, and it should never be used in Caltrans bridge construction projects.

Machine Finishing: Bridge Deck

For bridge decks, the concrete surface is typically struck and finished using power driven finishing equipment, commonly called Bidwell Concrete Finisher (Figure 5-19) in honor of its inventor, Tex Bidwell. When configured for bridge deck finishing, these versatile machines spread, compact, and finish bridge deck concrete one pass at a time.

The basic bridge deck finishing machine consists of a carriage, fitted with concrete finishing equipment, that is suspended from and travels on carrier rails attached under an open welded-steel truss frame. The frame is supported by adjustable legs and wheeled bogies that ride on screed rails. Drive wheels on pipe screed rails move the finishing machine along the length of the bridge. The carriage is equipped with concrete finishing equipment consisting of augers, finishing drums, float pans, and texturing devices that are arranged sequentially to spread, finish, and texture the concrete surface with each pass. The truss assembly can be adjusted to provide the designed bridge grade, profile, and cross section.

Figure 5-19. Bidwell Concrete Finisher.

During bridge deck construction, the augers strike off the concrete to a rough grade and move excess concrete forward. Next, the drums trim and finish the concrete surface to the final grade. While the finishing drums do compact the concrete, they are not solely relied upon to consolidate the deck concrete. Separate vibrators are usually used to properly consolidate the bridge deck concrete. After the drum, a float pan or a series of float pans fills in small voids still remaining on the surface of the concrete. A texturing device, such as burlap or synthetic turf, is typically attached behind the float pan to texture the finished surface

Although one or more vibrators can be attached to the carriage, consolidation of bridge deck concrete is usually done by hand ahead of the finish ng machine. The deck concrete fin shing machine advances from as little as 4 to 6 inches to as much as 18 inches per carriage pass. A work bridge may also be used behind the finishing machine, riding on the same screed rails, to provide access to the freshly placed concrete to allow supplemental work such as additional finishing, fogging, or application of curing compound. For additional information, see the Bridge Deck Construction Manual.

Other Concrete Finishing Equipment

The specifications do not prescribe a particular means and method of finishing the bridge deck or similar riding surface. The "end product specification" only requires that the smoothness of completed roadway surfaces of structures, approach slabs and the adjacent 50 feet of roadway and approach paving, will be tested by the engineer with a bridge profilograph, in conformance with the requirements in California Test 547.

For approach slabs and bridge surfaces of similar size, the concrete surface is sometimes finished using power finishing equipment other than a Bidwell Concrete Finisher, such as the vibrating screed, also known as the Texas screed, shown in Figure 5-20.

Figure 5-20. Texas Screed.

On some segmental bridge projects, such as the Benicia-Martinez Bridge, where the specifications either require the bridge deck to have an overlay or grinding the deck surface, other concrete finishing has been used, such as the rolling screed shown in Figure 5-21.

Figure 5-21. Rolling Screed.

Finishing Air-Entrained Concrete

Air entrainment gives concrete a somewhat altered consistency that permits the start of surface finishing at an earlier stage than is the case with normal (no entrained air) concrete.

As previously noted, air-entrained concrete contains microscopic air bubbles that tend to hold all the materials in the concrete, including water, in suspension. Since air-entrained concrete contains less water and the water is held in suspension for a longer time, little or no bleed water reaches the surface. With reduced bleeding, finishing need not be delayed while waiting for free water to evaporate from the surface.

If floating and finishing is done by hand, the use of an aluminum or magnesium float is essential. A wood float drags and greatly increases the amount of work necessary to accomplish the same result. For deck finishing with a finishing machine, there will be no difference between finishing procedures for air-entrained and non air-entrained concrete, except that the finishing operation may be started sooner on the air-entrained concrete.

Curing Concrete

Curing concrete is the process of keeping an optimum level of moisture and temperature within the concrete for a period of time immediately following its placement, as the concrete hardens, either by preventing or substantially reducing the rate of evaporation of the water from within the concrete mass. Proper curing of concrete is a key element in concrete construction because the effectiveness of the curing process is a significant determinant in achieving the desired superior concrete properties. With proper curing, concrete becomes stronger, more impermeable, and more resistant to stress, abrasion, and freezing and thawing.

For a given concrete, both strength and durability will continue to improve with the passage of time as long as conditions remain favorable for the continued hydration of the cement. It is the first few days following concrete placement that are the most critical, as it is during this initial period that rapid improvement in strength and durability is possible under favorable conditions. If curing conditions are unfavorable or even marginal, improvement in concrete strength will be slow and the intended properties may never be attained.

To ensure proper curing of concrete, two conditions are essential and must be maintained. First, the concrete must be kept moist to prevent the evaporation of water from within the concrete mass, and second, the temperature of the newly placed concrete must be kept within an optimum range of about 50 to 90°F. If these stresses develop before the concrete has attained adequate tensile strength, surface cracking can result. Since the rate of hydration is directly proportional to temperature of the curing concrete, it follows that the ideal curing method will also prevent significant loss of either moisture or temperature during the curing period.

While there is no ideal curing method, there are a number of relatively effective methods by which concrete can be kept in a moist condition, and, depending on exposure, at a favorable temperature as well, such as those specified in the Standard Specifications

For cast-in-place concrete construction, curing methods can be divided into two categories depending on the manner in which moisture loss is prevented:

- Methods that supply additional moisture to the surface of the concrete, such as the continuous application of water or the use of a moisture retaining fabric or blanket, and
- Methods that prevent moisture loss by sealing exposed surfaces of the concrete, as for example, the application of a membrane curing compound.

In cases of curing in hot or cold weather, special care, such as employing precautionary measures to counter the effects of extreme temperatures, may be necessary. Hydration proceeds at a much slower rate when the concrete temperature is low. For example, temperatures below 50°F are unfavorable for the development of early strength; below 40°F the development of early strength is greatly retarded; and at or below freezing temperatures, down to 14°F, little or no strength develops.

Plastic Shrinkage Cracking

The term "Plastic shrinkage cracking" is used to describe the formation of cracks in a horizontal surface of fresh concrete after it has been placed and finished but while it is still in the plastic state. Plastic shrinkage cracking is caused by rapid evaporation of moisture from the concrete surface, and occurs when the rate of evaporation exceeds the rate at which bleed water rises to the surface.

Although plastic shrinkage cracking is frequently associated with hot weather conditions, it can occur at any time when ambient conditions are conducive to a rapid evaporation rate. Loss of water will also cause the concrete to shrink, thus creating tensile stresses within the concrete. The rate of evaporation is a function of four interrelated factors: concrete temperature, air temperature, wind velocity, and humidity. Evaporation rates are illustrated in a nomograph shown in Figure 5-22 which provides a graphic method of estimating the loss of surface moisture for various weather conditions. To use the chart, follow the four steps shown on the chart. If the rate of evaporation approaches 0.2 lb/ft^2/hr (1.0 kg/m^2/hr), precautions against plastic shrinkage are necessary. For the example shown by dotted line, air temperature at 65°F, relative humidity at 35%, concrete temperature at 60°F and wind velocity at 20 mph: the evaporation rate is approximately 0.12 lb/ft^2/hr.

While it is not possible to determine the exact evaporation rate at which cracking will occur under all circumstances, cracking may occur when the rate is as low as 0.1 lb per square foot per hour. When the evaporation rate reaches about 0.2 lb per square foot per hour, cracking is likely unless mitigating measures are employed.

Note: With increased use of supplementary cementitious materials and liquid admixtures, particularly high range water reducers, it is possible to produce a concrete with an extended set time. Additional precautions, such as fogging may be necessary to retard the development of plastic shrinkage cracks.

Preventing plastic shrinkage cracking is more a matter of common sense than concrete technology, since the most important factor is keeping the concrete surface moist until curing begins. This becomes increasingly more important as the predicted rate of evaporation increases due to adverse ambient conditions.

There are two methods by which a concrete surface may be kept in a moist condition. The first is reducing the rate of evaporation and the second is applying moisture to the surface to offset the evaporation loss. While applying moisture to the surface is an obvious solution, it must be done carefully to avoid dilution of the cement paste at the surface, with a consequent lessening of the concrete quality. A better approach is to employ measures that will reduce the evaporation rate directly. Since little can be done to improve ambient conditions, aside from misting and lowering temperature, reducing the evaporation rate can best be accomplished by reducing the temperature of the concrete mixture.

Factors affecting concrete temperature, and methods by which the temperature may be reduced, are discussed in the Hot Weather Construction section later in this chapter.

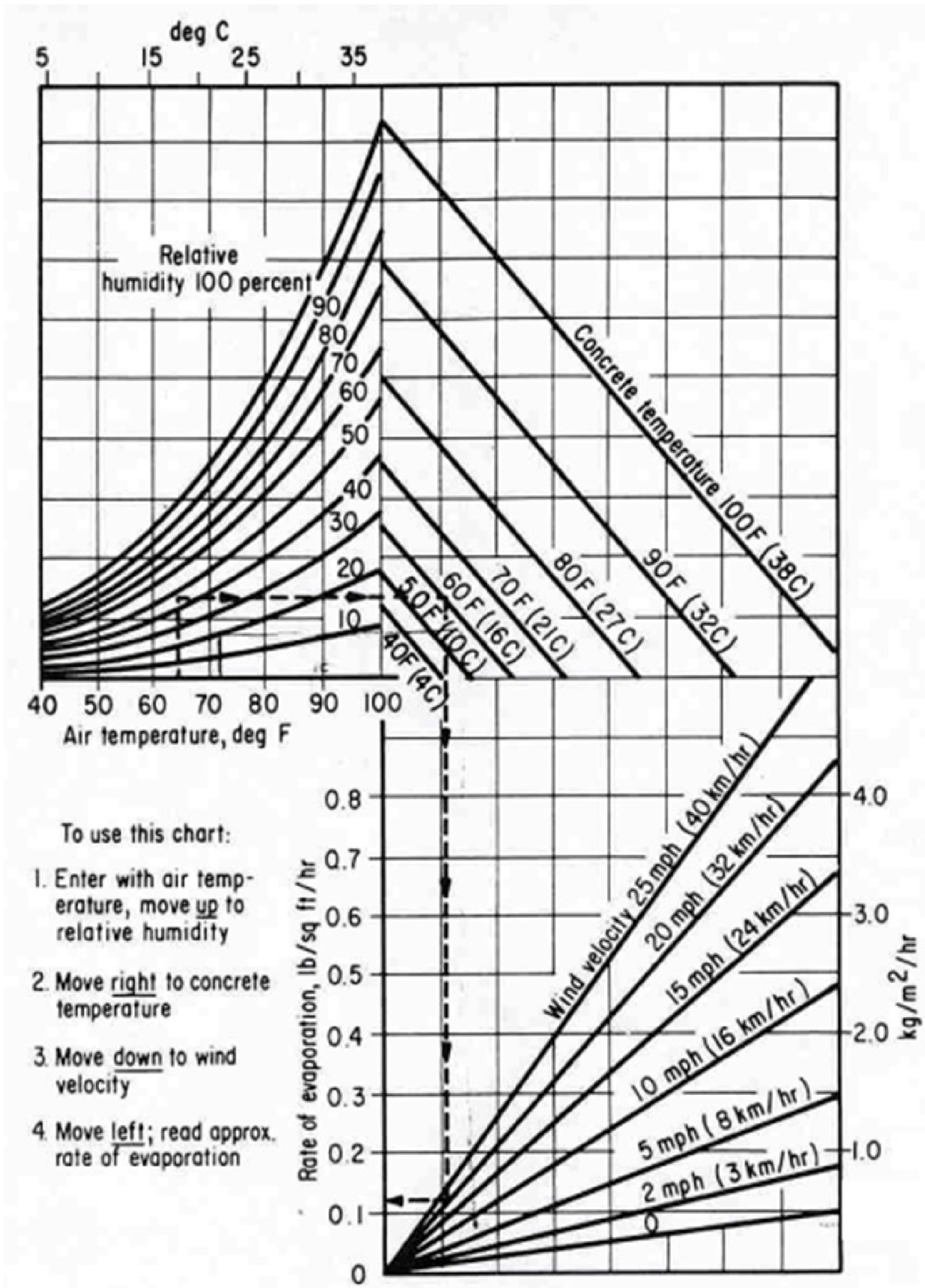

Figure 5-22. Evaporation Rate Nomograph (ACI 308, Standard Practice for Curing Concrete).

Specified Concrete Curing Methods

The specifications require all newly placed concrete for cast-in-place structures, other than highway bridge decks, to be cured by the water method, the forms-in-place method, or when specifically permitted, by the curing compound method. The top surface of highway bridge decks are cured by both the curing compound method and the water method. Note that regardless of the method used, curing begins while the concrete is still plastic and continues for the specified curing period.

Water Method

Under this curing method, the specification requires that concrete surfaces are kept continuously wet by the application of water for a period of 7 days after the concrete has been placed. There are several means by which the intent of the specifications can be met, including continuously spraying the surface, ponding water on the surface, or covering the surface with an absorbent material such as sand, burlap, rugs, or straw, and then keeping the moisture retaining medium saturated.

Water should be applied on exposed surfaces of newly placed concrete as soon as the concrete has hardened sufficiently to prevent any washing away of the cement or damage to the finish. Water should be applied to formed surfaces immediately after the forms are removed, and the exposed surfaces should be kept continuously wet for the remainder of the curing period, or until some other curing medium is applied. When the water method is used, the most important point to keep in mind is that, once the curing period begins, the surface of the concrete must remain moist for the duration of the curing period.

When a curing medium consisting of cotton mats, rugs, carpets, or earth or sand blankets is to be used to retain the moisture, the entire surface of the concrete shall be kept damp by applying water with a nozzle that so atomizes the flow that a mist and not a spray is formed, until the surface of the concrete is covered with the curing medium. The moisture from the nozzle shall not be applied under pressure directly upon the concrete and shall not be allowed to accumulate on the concrete in a quantity sufficient to cause a flow or wash the surface. At the expiration of the curing period, the concrete surfaces shall be cleared of all curing mediums.

As a rule, earth or sand blankets are less effective than other curing mediums because of the tendency of the curing water to wash or spread the material (thus developing thin spots) and because sand, in particular, may be too coarse to retain enough moisture to ensure that the surface remains damp.

When concrete bridge decks and flat slabs are to be cured without the use of a curing medium, the entire surface of the bridge deck or slab shall be kept damp by the application of water with an atomizing nozzle as specified in the preceding paragraph, until the concrete has set, after which the entire surface of the concrete shall be sprinkled continuously with water for a period of not less than 7 days.

Plastic Sheets

The specifications allow the contractor the option of using a curing medium consisting of polyethylene sheeting to cure concrete columns. The specifications limit the use of this curing method solely to columns. Polyethylene film is a lightweight moisture-retarding medium and it is an effective curing method if used with the application of water and the concrete is not allowed to dry out. The sheeting merely retards the evaporation of water from the concrete and a periodic application of water is necessary in order to maintain the optimum level of moisture during the curing process. As an allowed alternative under the water method of curing concrete, the surface of the concrete shall remain moist throughout the curing period.

Specifications require that polyethylene sheeting be 1.0 mm (10 mils) or thicker. When using polyethylene sheeting (Visqueen®) curing medium for concrete columns (Figure 5-23), the sheeting must be new or in near new condition, without tears or holes. The minimum thickness of 10 mils shall be achieved in one layer of materials. The polyethylene must be adequately secured at the top, bottom, discontinuous edges, and at mid-height or no more than 20 feet on the center for columns over 40 feet in height. Joints shall be folded and secured by tape, clamps, or stitching as necessary to ensure a moisture-proof seal. The sheeting shall be fastened such that it will remain within 3 inches of the concrete surface at all times. The use of polyethylene sheeting shall be in conjunction with the application of water. Cure water is to be applied between the sheeting and the concrete surface of the column by means of a soaker hose or a comparable device that provides an even water distribution completely around the perimeter of the column and shall be installed for the entire length of the cure period. Cure water should be applied at least twice daily and as required to keep the column surface moist at all times.

In regions or periods of extreme high temperatures, if the surface temperature of the column concrete under the sheeting cannot be maintained below 140°F, the use of Visqueen shall be discontinued and one of the other specified curing methods shall be used.

White polyethylene sheeting should be used for curing exterior concrete during hot weather to reflect the sun s rays. Black film may be used during cool weathe .

Figure 5-23. Curing Concrete Columns Using Polyethylene Plastic Sheets.

Burlene®

The specifications allow, at the option of the contractor, a curing medium consisting of white opaque polyethylene sheeting extruded onto burlap to cure concrete structures. The polyethylene sheeting shall have a minimum thickness of 4 mils and shall be extruded onto 10-ounce burlap. Furthermore, the medium and any joints therein shall be secured as necessary to provide moisture retention and shall be within 3 inches of the concrete at all points along the surface being cured.

The most common curing fabric allowed under this specification is a commercial curing fabric consisting of polyethylene film bonded to burlap, and trademarked Burlene®. There are no restrictions on which structure elements can be water cured with the use of Burlene, including concrete columns and retaining walls, provided that moisture is maintained throughout the curing period.

Because the burlap water-retaining medium is bonded to a polyethylene layer that averts rapid evaporation, the use of this curing medium will eliminate the need for continuous watering of the covering. However, Burlene will not totally prevent evaporation of water and the fabric should be periodically rewetted-under the plastic-before it dries out. It is important that moisture is maintained throughout the curing period, because alternate cycles of wetting and drying during the early curing period may cause crazing of the surface.

BCM 105-5.0 outlines the requirement on the use of a commercial curing fabric, such as Burlene. The burlap side of the material is to be placed next to the concrete. On decks, the fabric shall be secured by weighing it down or by other methods that ensure a proper seal and protection against the wind (Figure 5-24). On columns and retaining walls, the Burlene shall be secured at the top, bottom, discontinuous edges, and loosely secured at mid-height or at no more than 20 feet on center for columns over 40 feet in height. In addition, on flared or unusually shaped columns or walls the material shall be secured in such a fashion that the Burlene is within 3 inches of the surface of the concrete at all points along the surface being cured. Joints shall be folded and secured by ties, staples, or stitching as necessary to ensure a tight seal when curing columns and walls. Weighted lap splices are acceptable on decks and other similar surfaces.

Figure 5-24. Curing with Burlene Curing Blankets.

If Burlene is used as a curing medium, the cure water is to be applied under the Burlene (between the sheeting and the concrete). The burlap side of the material should be moist at all times. On columns and retaining walls a continuous application of cure water may be required. A soaker hose, or comparable device, that will provide an even water distribution completely around the perimeter of the surface being cured shall be permanently installed for the entire length of the cure period. Water shall be applied as necessary to keep the Burlene and concrete surface moist at all times.

The temperature of the concrete should be monitored during hot weather applications of Burlene curing. Concrete surface temperatures of 140°F or greater must be prevented. If the temperature of the concrete cannot be maintained below 140°F, this method of curing shall be discontinued, and one of the other curing methods allowed for the concrete shall be used.

Burlap must be free of any substance that is harmful to concrete or causes discoloration. New burlap should be thoroughly rinsed in water to remove soluble substances and to make the burlap more absorbent.

Wet, moisture-retaining fabric coverings should be placed as soon as the concrete has hardened sufficiently to prevent surface damage. During the waiting period other curing methods are used, such as fogging or the use of membrane forming finishing aids. Care should be taken to cover the entire surface with wet fabric, including the edges of slabs. The coverings should be kept continuously moist so that a film of water remains on the concrete surface throughout the curing period.

Forms-In-Place Method

Although forms are usually removed as soon as possible to permit their re-use, occasionally a contractor will choose to leave them in place for all or a significant portion of the curing period. The specifications allow that the formed surfaces of concrete may be cured by retaining the forms in place (Figure 5-25). Leaving the forms in place is an effective curing method, provided the forms do not dry out and exposed concrete is kept wet. The forms shall remain in place for a minimum period of 7 days after the concrete has been placed, except that for members over 20 inches in least dimension the forms shall remain in place for a minimum period of 5 days.

Figure 5-25. Forms-In-Place Curing Method.

The specifications require joints between form panels and areas between forms and concrete surfaces be kept moisture tight during the curing period. Additionally, any cracks in the forms or between the forms and the concrete that develop during the curing period must be sealed by methods approved by the engineer. The forms should be inspected periodically during the curing period, and this specification rigidly enforced if necessary; otherwise, the forms-in-place method will not provide an effective cure.

Curing Compound Method

The curing compound method consists of applying one of the membrane-forming curing compounds listed in the specifications to the surface of freshly placed concrete. Liquid membrane-forming curing compounds consist of waxes, resins, chlorinated rubber, and other materials that are designed to retard or reduce evaporation of moisture from concrete. Curing compounds, which are furnished and applied in liquid form, contain volatile substances that evaporate within a short time after application, leaving a thin waterproof membrane that seals the concrete surface. For maximum effectiveness, curing compounds should be applied following completion of surface finishing, but not before the bleed water has evaporated. At the optimum application time, the surface will have a slight moisture sheen and will be damp to the touch.

Curing compounds are the most practical and most widely used method for curing not only freshly placed concrete but also for extending curing of concrete after removal of forms or after initial moist curing. While the most effective methods of curing concrete are wet coverings or water spraying that keeps the concrete continually damp, curing compounds should be able to maintain the relative humidity of the concrete surface above 80% for 7 days to sustain the hydration of the concrete.

The Standard Specifications allow two types of curing compounds, pigmented or nonpigmented. Pigmented curing compound is typically white, which makes it easy to determine if curing compound has been adequately sprayed over newly placed concrete. Nonpigmented curing compounds are clear when applied and remain clear when dry. An option with clear curing compound is inclusion of a fugitive dye that gives a red tint to newly applied curing compound but fades to clear as the compound dries. The fugitive dye allows the inspector to verify complete curing compound coverage and could be specified for concrete floor slabs in buildings.

The specifications require that if curing compounds are used on structures, only pigmented curing compounds shall be used, except for barrier rails. For structures, the curing compound method may not be used on any surface that requires a Class 1 Surface Finish, or on any other surface that is visible from a public traveled way. On hot, sunny days, white-pigmented compounds reduce solar-heat gain, thus reducing the concrete temperature.

Per the specifications, concrete barrier rails are cured using a non-pigmented curing compound. The specifications also allow the contractor the option of forms-in-place method for curing barrier rail. If the contractor elects the forms-in-place method, the forms must remain in place a minimum of 12 hours, and no further curing is required after the forms are removed.

Unlike the water and forms-in-place curing methods, which are easy to understand and use, the curing compound method is a sophisticated procedure. The specification requirements are detailed and explicit, and unless they are followed precisely, the curing compounds will not perform as intended. To ensure a satisfactory cure, field engineers who are responsible for inspection of structure concrete construction should review the applicable Standard Specifications, and any pertinent special provision requirements as well, and then discuss the requirements with appropriate contractor personnel before any work that will involve the use of curing compounds is started.

When the curing-compound method is specified or allowed, the specifications require that the curing compound be applied uniformly to the surface of the concrete. It is imperative that the curing compound be applied uniformly, without skips, sags, or holidays, because the intent in the use of the curing compound is to form a moisture-retaining membrane on the surface of the concrete. Failure to form this moisture-retaining membrane on the surface of the concrete will result in the rapid evaporation of water from the concrete, which could adversely affect the concrete strength, durability, and other desired concrete properties. Complete coverage of the surface must be attained because even small pinholes in the membrane will increase the evaporation of moisture from the concrete.

The curing compound shall be applied to the concrete immediately after the surface finishing operation, immediately before the moisture sheen disappears from the surface, but before any drying shrinkage or craze cracks begin to appear. The concrete surface should be damp when the curing compound is applied. On dry, windy days, or during periods when adverse weather conditions could result in plastic shrinkage cracking, application of a curing compound immediately after final finishing and before all free water on the surface has evaporated will help prevent the formation of cracks.

Fogging

In the event of any drying or cracking of the concrete surface, the specifications require the application of water using an atomizing nozzle - fogging - be started immediately. Fogging the concrete is applying water with a nozzle (Figure 5-26) that so atomizes the flow that a mist not a spray is formed. The moisture from the nozzle shall not be applied under pressure directly upon the concrete and shall not be allowed to accumulate on the concrete in quantity sufficient to cause a flow or wash the surface of the newly placed concrete. Fogging shall be maintained until the specified curing medium is applied. Curing compound shall not be applied over any freestanding water.

Figure 5-26. Fogging Bridge Deck to Minimize Plastic Cracking.

Application

Curing compound shall be applied at a nominal rate of 1 gallon per 150 square feet, unless otherwise specified. At any point, the application rate shall be within ±50 square feet per gallon of the nominal rate specified, and the average application rate shall be within ±25 square feet per gallon of the nominal rate specified when tested in conformance with the requirements in California Test 535. Runs, sags, thin areas, skips, or holidays in the applied curing compound shall be evidence that the application is not satisfactory. Normally only one smooth, even coat is applied at the recommended application rate. However, if two coats are necessary to ensure complete coverage, it is recommended that the second coat should be applied at right angles to the first, to ensure effective coverage.

The specifications require that curing compounds be applied using power-operated spray equipment. The power-driven spray equipment (Figure 5-27) must have the capability to apply the curing compound as a uniform membrane on the surface of the concrete, similar in appearance to uniform application found in painted surfaces. The power-operated spraying equipment shall be equipped with an operational pressure gauge and a means of controlling the pressure. Spray nozzles and windshields on such equipment should be arranged to prevent wind-blown loss of the curing compound.

Figure 5-27. Applying Curing Compound Using Power-Operated Spray Equipment.

While hand spraying of small and irregular areas that are not reasonably accessible to mechanical spraying equipment could be permitted by the engineer, the use of hand-operated sprayers (i.e., Hunt's can) should be extremely rare and only in applications that are truly unachievable with power sprayers. It should not be considered a routine alternative to the specified power sprayers. If used, spraying with Hunt's cans should achieve a uniform coverage of the concrete to form a moisture-retaining medium. Since hand operated sprayers do not disperse the curing compound in a fine spray that is conducive to a uniform application of the cure and are prone to clogging, the amount of curing compound needed to achieve the required uniform coverage is considerably higher. If properly enforced, the contractor will realize that it would be economical to use power sprayers as specified.

When applied, curing compounds should not sag, run off peaks, or collect in grooves. They should form a tough film to withstand early construction traffic without damage, be non-yellowing, and have good moisture-retention properties.

The specifications stipulate that should the film of compound be damaged from any cause before the expiration of 7 days after the concrete is placed in the case of structures and 72 hours in the case of pavement, the damaged portion shall be repaired immediately with additional compound.

If left in place, curing compound acts as a bond breaker between successive layers of concrete, such a situation could occur when concrete for the deck is placed on previously cast concrete girders without properly removing the curing compound from the exposed girder surfaces. The Standard Specifications require complete removal of curing compound prior to placement of additional concrete.

Quality Assurance

While curing compounds are an effective concrete curing medium, its efficacy could only be maintained if the material itself remains true to its intended composition throughout its use. These membrane-forming curing compounds are made up of volatile chemical compounds that need attention during their use and requires vigilance on the inspector's part in order to ensure that we get the maximum benefit out of these curing products.

There are different types of curing compounds, each designed for a specific application, allowed in the specifications. Refer to the Standard Specifications for specifics. Ensure that the correct curing compound is used for each specific application.

For quality assurance purposes, Materials Engineering and Testing Services (METS) policy requires that a curing compound must be tested by the manufacturer and the manufacturer must provide the test results and supporting documentation along with a Certificate of Compliance and Form TL-28 with the shipment of curing compound to the jobsite. In addition, a sample from each batch manufactured for Caltrans is sent to the Chemical Testing Branch for quality assurance testing.

METS policy further requires that field staff ensure that the curing compound is sampled and tested per the Construction Manual (Chapter 6, Section 1). Curing compounds shall not be used until the required evidence or certificate of inspection has been received. Upon final inspection, the curing compound may be released at the jobsite using Form CEM-4102, "Material Inspected and Released on Job". Acceptance of this material does not relieve the vendor and contractor from incorporating materials meeting the specific project documents. Curing compounds may be re-sampled and retested to confirm that the material delivered to the jobsite meets the specification and has been mixed properly prior to application.

The specifications require that at the time of use, curing compounds containing pigments shall be in a thoroughly mixed condition with the pigment uniformly dispersed throughout the vehicle. Pigmented compounds should be kept agitated in the container to prevent pigment from settling out. Agitation shall not introduce air or other foreign substance into the curing compound. A paddle shall be used to loosen all settled pigment from the bottom of the container, and a power driven agitator shall be used to disperse the pigment uniformly throughout the vehicle.

The manufacturer shall include in the curing compound the necessary additives for control of sagging, pigment settling, leveling, de-emulsifying, or other requisite qualities of a satisfactory working material. Pigmented curing compounds shall be manufactured so that the pigment does not settle badly, that is, the pigment does not cake or thicken in the container, and does not become granular or curdled. Settled pigment shall be a thoroughly wetted, soft, mushy mass permitting the complete and easy vertical penetration of a paddle. Settled pigment shall be easily re-dispersed, with minimum resistance to the sideways manual motion of the paddle across the bottom of the container, to form a smooth uniform product of the proper consistency.

Settling or separation of solids in containers, except tanks, must be completely re-dispersed with low speed mixing prior to use, in conformance with these specifications and the manufacturer's recommendations. Mixing shall be accomplished either manually by use of a paddle or by use of a mixing blade driven by a drill motor, at low speed. Mixing blades shall be the type used for mixing paint. On-site storage tanks shall be kept clean and free of contaminants. Each tank shall have a permanent system designed to completely re-disperse settled material without introducing air or other foreign substances.

Curing compounds shall remain sprayable at temperatures above 40°F and shall not be diluted or altered after manufacture. Curing compound shall be formulated so as to maintain the specified properties for a minimum of 1 year. The engineer may require additional testing before use to determine compliance with these specifications if the compound has not been used within 1 year or whenever the engineer has reason to believe the compound is no longer satisfactory.

The curing compound shall be packaged in clean 274 gallon totes, 55 gallon barrels, 5 gallon pails, or shall be supplied from a suitable storage tank located at the jobsite. The containers shall comply with "Title 49, Code of Federal Regulations, Hazardous Materials Regulations." The 274 gallon totes, and the 55 gallon barrels shall have removable lids and airtight fasteners. The 5 gallon pails shall be round and have standard full open head and bail. Lids with bungholes shall not be permitted. Steel containers and lids shall be lined with a coating that will prevent destructive action by the compound or chemical agents in the air space above the compound. The coating shall not come off the container or lid as skins. Containers shall be filled in a manner that will prevent skinning. Plastic containers shall not react with the compound.

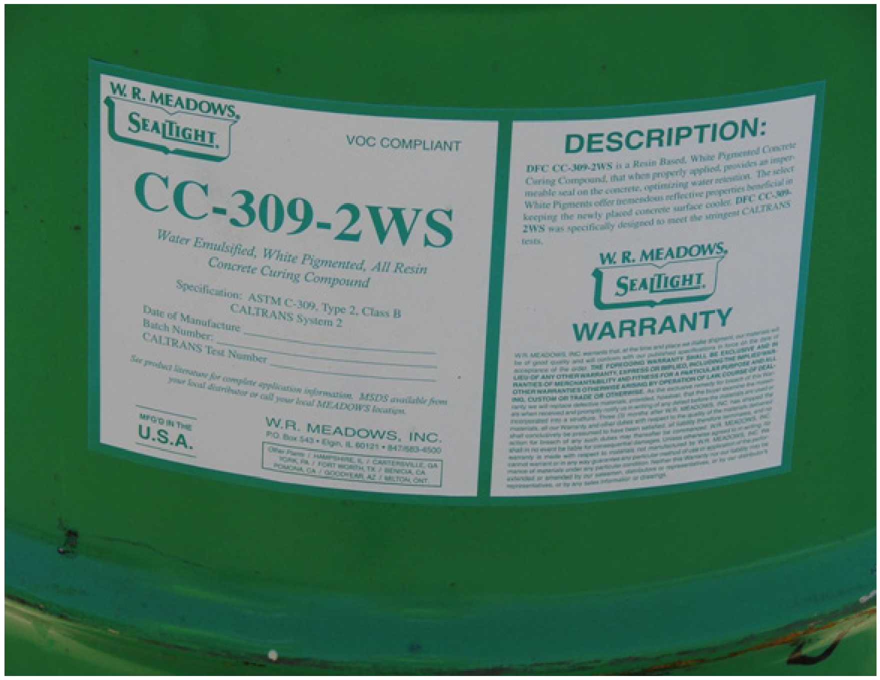

Figure 5-28. Typical Pigmented Curing Compound Label.

Safety

Caution is necessary when using curing compounds containing solvents of high volatility, especially in confined spaces or near sensitive occupied spaces such as hospitals, because evaporating volatiles may cause respiratory problems. Applicable local environmental laws concerning volatile organic compound (VOC) emissions should be followed.

The specifications require that each curing compound container be labeled with the manufacturer's name, kind of curing compound, batch number, volume, date of manufacture, and volatile organic compound (VOC) content. Containers of curing compound shall be labeled to indicate that the contents fully comply with the rules and regulations concerning air pollution control in the State of California. The label shall also warn that the curing compound containing pigment shall be well stirred before use.

Precautions concerning the handling and the application of curing compound shall be shown on the label of the curing compound containers in conformance with the Construction Safety Orders and General Industry Safety Orders of the State of California. When the curing compound is shipped in tanks or tank trucks, a shipping invoice shall accompany each load. The invoice shall contain the same information as that required herein for container labels.

Waterproof Membrane Method

In this curing method, which is used for PCC pavement, the specification requires that the curing membrane remain in place for not less than 72 hours. (For structures, specifications require the Water Method of curing.)

The curing membrane specified as a sheeting material for curing concrete shall conform to the requirements in AASHTO Designation: M-171 for white reflective material. The sheeting material shall be fabricated into sheets of such width as to provide a complete cover for the entire concrete surface. Joints in the sheets shall be securely cemented together in such a manner as to provide waterproof joints with a minimum 4-inch overlap and the sheets shall be weighted down by placing a bank of earth or by other means satisfactory to the engineer. Before the curing membrane is placed, the exposed finished surfaces of concrete shall be sprayed with water, using a nozzle that so atomizes the flow that a mist and not a spray is formed.

Curing Structures

Newly placed concrete for cast-in-place structures, other than highway bridge decks, shall be cured by the water method, the forms-in-place method, or, as permitted by specifications, by the curing compound method.

Pigmented curing compounds cannot be used for portions of structure concrete that is exposed to the view of the public and is specified to receive a Class 1 Concrete Finish. The curing compound method using a pigmented curing compound may be used on concrete surfaces of construction joints, surfaces that are to be buried underground, and surfaces where only ordinary surface finish is to be applied and on which a uniform color is not required and that will not be visible from a public traveled way. If used on construction joints, curing compounds shall be removed, by abrasive methods, from concrete surfaces receiving another layer of concrete. In general, curing compounds are considered to be bond breakers and are detrimental to proper bond between layers of concrete.

The specifications require that the top surface of highway bridge decks shall be cured by both the curing compound method and the water method. For bridge decks, the curing compound shall be curing compound designated in specifications as Pigmented Curing Compound A[8]. If the Contractor elects to use the curing compound method on the bottom slab of box girder spans, the curing compound shall also be Curing Compound A[8].

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[8] 2006 Standard Specifications, Section 90-7.01B, or Curing Compound No.1, 2010 Standard Specifications Section 90-1.03B(3)b.

When deemed necessary by the engineer during periods of hot weather, water shall be applied to concrete surfaces being cured by the curing compound method or by the forms-in-place method, until the engineer determines that a cooling effect is no longer required.

Curing Minor Structures

Concrete surfaces of minor structures, as defined in Section 51-1.02[9], "Minor Structures," shall be cured by the water method, the forms-in-place method, or the curing compound method.

Surface Finishes

Although bridge concrete may possess the requisite strength, durability, and other desired qualities as a structure material, a poor state of its completed appearance may convey an impression of poor workmanship, shoddy construction, low quality, or even unsafe conditions. That is why there is as much emphasis in the specifications on the finish of concrete surfaces as there is on the strength and material quality of the concrete. The specifications place particular emphasis on portions of the structure that are visible from the traveled way, which require a surface finish that is uniform in appearance. This chapter encompasses the contract requirements that pertain to the quality and appearance of surfaces of the hardened concrete of completed structure.

This section covers the concrete finishing the formed surfaces of the hardened concrete, as specified in Section 51-1.18[10], “Surface Finishes,” of the Standard Specifications.

Specified Concrete Finishes

Requirements for final finishing of concrete surfaces are found in Section 51-1.03F of the Standard Specifications. This section describes the work involved and the results required for three classes of surface finish:

- Ordinary Surface Finish
- Class 1 Surface Finish
- Class 2 Surface Finish

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[9] 2006 Standard Specifications, or 2010 Standard Specifications, Section 51-7.

[10] 2006 Standard Specifications, or 2010 Standard Specifications, Section 51-1.03F "Finshing Concrete".

In a broad sense, the specified finishing requirements have two objectives. The first is to remove surface discoloration and repair surface defects, while the second is to obtain a smooth surface having a uniform texture and appearance at locations where these characteristics are desired.

In general, other factors being equal, the amount of surface finishing that will be necessary to meet the contract requirements is a function of the quality of the formwork. The specifications note that the degree of care in building forms and character of materials used in form work will be a contributing factor in the amount of additional finishing required to produce smooth, even surfaces of uniform texture and appearance, free of unsightly bulges, depressions, and other imperfections. If good quality form materials are used and the forms are constructed in a workmanlike manner to the required lines and grades, the need for surface finishing will be much less than if the formwork is constructed otherwise.

Ordinary Surface Finish

Ordinary surface finish consists of filling holes and surface depressions, patching rock pockets, removing fins, and, on surfaces that are visible from a traveled way, removing stains or discolorations. Snap ties, bolts, or other steel form appurtenances must be removed to the specified depth. Deleterious materials on the surface of the concrete, such as nails, tie wires, debris, etc., must be removed to sound concrete and the voids patched properly. After the forms are removed, the ordinary surface finishing should be completed at the earliest possible opportunity.

Ordinary surface finish is specified for all structure concrete surfaces, either as a final finish or preparatory to the application of a higher class finish. Even though ordinary surface finish is required to be applied to all concrete surfaces, this does not mean that the same finishing techniques are appropriate for all surfaces. For example, when plugging form bolt holes or filling voids where form ties were removed, the only consideration for enclosed or buried surfaces is to obtain a sound patch. However, if the surface is visible from a traveled way, appearance is also a factor, so the color of the patch must match the surrounding concrete, which may be achieved by adding a small amount of white cement to the mortar patching material.

Ordinary surface finish, unless otherwise specified, shall be considered as a final finish on the following surfaces:

- The undersurfaces of slab spans, box girders, filled-spandrel arch spans, and floor slabs between girders of superstructures.
- The inside vertical surface of T-girders of superstructures.
- Surfaces that are to be buried underground or covered with embankment and surfaces above finished ground of culverts which are not visible from traveled ways. On

surfaces which are to be buried underground or surfaces which are enclosed, such as the cells of box girders, the removal of fins will not be required.

Patching Snap-Tie Holes

The specifications require that all form bolts and any metal placed for the convenience of the contractor, such as snap ties, shall be removed, to a depth of at least 1 inch below the surface of the concrete and the resulting holes or depressions shall be cleaned and filled with mortar. A sound patch for snap tie holes that is well bonded to the surrounding concrete is essential in attaining the long term surface quality of concrete because if any moisture gets to the tie, corrosion will occur, resulting in expansive rust which would cause surface spalls and rust staining.

Removable portions of ties should be removed unless the contract documents specifically permit otherwise. The specifications allow form bolts projecting into the cells of box girders to be left in place unless deck forms are removed from the cells, in which case the bolts shall be removed flush with the surface of the concrete. Because it is non-corroding, glass-fiber reinforced plastic ties, commonly referred to as fiberglass ties, may be cut off flush with the concrete surface, leaving no hole to patch.

Snap-tie holes, bolt holes and other cavities on the surface of the concrete that are small in area but relatively deep are typically patched with dry-pack mortar. Dry-pack mortar is a stiff mix of cement and sand (usually, one part cement to two and one half parts No. 16 sand) mixed with just enough water to produce a mortar that can be formed into a ball when squeezed gently by hand. Before patching, the cavity must be clean and should be moistened. Dry-pack mortar is typically applied by forcibly ramming it into place. For bigger cavities, it is recommended that the mortar be packed into place in layers of about 1/2 inch thick, with each layer given a scratch finish to improve bond with subsequent layers of mortar. The vigorous packing of the dry-pack mortar is the chief means of ensuring a good bond with the surrounding concrete and minimum patch shrinkage. It is also important that the color of dry-pack patch must match the surrounding concrete, which may be achieved by adding a small amount of white cement to the mortar patching material. Metal tools should not be used in dry packing as they tend to discolor the mortar.

Special admixtures may be used in the patching mix to improve moisture resistance and adhesion of the patch. Non-shrink grouts are also viable options. BCM 105-1.0 states that mortar additives (shrinkage reducers, water reducers, bonding agents, etc.) are acceptable provided the additive must be acrylic-based and it does not have polyvinyl acetate as an active ingredient. However, since the performance of specific patching materials or additives are not generally well founded, it is recommended consulting with your Structural Materials Representative or concrete subject matter experts at the Caltrans Transportation Laboratory (Translab) prior to allowing their use.

Repair of Rock Pockets

The specifications require that all rock pockets and other unsound concrete be removed. The appropriate repair procedure for rock pockets depends on the depth and extent of the voids. For example, if the honeycombed area is shallow (i.e., there is a deficiency in the sand-cement paste that has left the coarse aggregate exposed, but the concrete substrate is sound), a simple dry-pack mortar patch may be suitable. For deeper rock pockets the exposed aggregate may have to be removed and replaced with a bonded patch.

When the rock pockets are widespread and/or a significant amount of reinforcing steel is exposed, a complete structural repair may be necessary. When rock pockets are present, selecting an appropriate repair method will involve subjective judgment. Concrete repairs are covered in Chapter 6 of this manual. It should be noted, however, that the specifications require that if the rock pockets, in the opinion of the engineer, are of such an extent or character as to affect the strength of the structure materially or to endanger the life of the steel reinforcement, the engineer may declare the concrete defective and require the removal and replacement of the portions of the structure affected.

Patches usually appear darker than the surrounding concrete. The specifications require that for exposed surfaces, white cement shall be added to the mortar or concrete in an amount sufficient to result in a patch which, when dry, matches the surrounding concrete. To ensure a proper match, it is considered best practice to make sample patches, using different proportions of white cement, in inconspicuous locations.

The specifications require the mortar or concrete used for patching or repair to be cured in conformance with the provisions in Section 90-7.03[11], "Curing Structures."

Class 1 Surface Finish

Class 1 surface finish consists of finishing the surfaces of the structure as necessary to produce smooth, even surfaces of uniform texture and appearance, free of unsightly bulges, depressions, and other imperfections.

Class 1 surface finish is applied to certain specified surfaces which do not exhibit a smooth, even surface of uniform texture and appearance after the ordinary surface finish is applied. In other words, Class 1 surface finish consists of performing only the "additional" finishing necessary to obtain the "smooth even surfaces of uniform texture and appearance" required by the specifications. From this it should be apparent that if the forms are carefully constructed

[11] 2006 Standard Specifications, or 2010 Standard Specifications, Section 90-1.03B, "Curing Concrete".

and ordinary surface finish diligently applied, little or no Class 1 surface finishing will be necessary to fulfill the intent of the specifications

Class 1 surface finish shall be the final finish for the following surfaces, unless otherwise specified in the special provisions:

- Except for those surfaces listed in ordinary surface finish, the surfaces of bridge superstructures, including the undersurfaces of deck overhangs.
- Surfaces of bridge piers, piles, columns, and abutments, and retaining walls above finished ground and to at least 1 foot below finished ground
- Surfaces of open spandrel arch rings, spandrel columns, and abutment towers.
- Surfaces of pedestrian undercrossings, except floors and surfaces to be covered with earth.
- Surfaces of culvert headwalls above finished ground and endwalls visible from a traveled way.
- Interior surfaces of culvert barrels having a height of 4 feet or more for a distance equal to the culvert height where visible from a traveled way.
- Interior surfaces of pump house motor and control rooms and engine-generator rooms.
- Surfaces of railings.

The specifications require Class 1 surface finish to be sanded, with power sanders or other approved (by the engineer) abrasive means, until the required surface appearance is obtained. The specifications require the use of power carborundum stones or disks to remove bulges and other imperfections. As a best practice, power sanding should be done with a flexible disk abrasive wheel as opposed to a rigid abrasive disk, which, more often than not, produces a burned surface if used improperly. The skill of the person performing the power sanding is a significant factor in achieving the desired Class 1 surface finish. Furthermore, it is recommended that to obtain best results power sanding should be postponed as long as possible.

Whip blasting

Per BCM 105-1.0, whip blasting, also known as light sand blasting or light abrasive blasting, may be used to produce a Class 1 Surface Finish. The abrasive medium used for whip blasting may be silica sand, aluminum carbide, black slag particles or even walnut shells. The type and grading of the abrasive material used in whip blasting significantly affects the surface finish and should, therefore, remain the same throughout.

The skill of the person performing the work is a critical factor that affects the outcome of the desired uniform appearance of the Class 1 surface fin sh. Surface defects due to concrete forming and placement, such as bug holes, leakage lines and cold joints between concrete lifts, tends to be accentuated by light blasting. Hardness of concrete surface, which is a function of concrete mix, time, and forming material, is also a major factor that must be considered when considering the appropriate means and method of whip blasting. Whip blasting generally gives best results when applied to concrete that has a denser than normal surface, such as concrete surfaces that have been formed with steel, fiberglass, or coated high-density plywood forms.

The following precautions should be taken when using abrasive blasting:

- In order to obtain a good appearance, it is necessary to do some experimenting to determine proper grain size, air pressure, and distance of sand blast nozzle from concrete surface, and angle of application of the sand.
- To aid the field engineer and workers in maintaining a uniform whip blast operation, they may apply a light lumber crayon mark approximately 1 foot in length to the surface of the work at random locations. The force of whip blasting operation should be of sufficient force to just remove the mark.
- Caution should prevail as an over aggressive application could result in a damaged surface that would be difficult to repair.
- The signs of an unacceptable result would be excessive pinhole, whip marks, and exposed aggregate.
- Abrasive blasting must conform to Regional Air Quality Management District regulations and permitting requirements.

Once these factors have been determined, the engineer must inspect the work periodically to be assured that workers are following those procedures which have been determined will give satisfactory results.

Engineer's Roles and Responsibilities

As provided in the Standard Specifications, the engineer is the sole judge who determines the amount of additional finishing, beyond the ordinary surface finish, necessary to produce the smooth even surface of uniform texture and appearance specified for Class 1 Surface Finish.

This is a heavy burden for the engineer, since the finishing requirements are among the most subjective in the contract. When determining the results of the contractor's finishing effort against contract requirements, the engineer should keep in mind that finished concrete should look like concrete and not some other material; that surface finishing is as much art as science, so that obtaining the intended result is more a matter of the practitioner's skill than of the particular method or procedure employed; and that compliance with the intent of the contract will be more apparent if the work is viewed from afar, which is how the public views it, than from up close. All factors considered, proper administration of the surface finishing specifications not only requires engineering judgment and experience, it also requires common sense.

Unacceptable Class 1 Finishes

The Standard Specifications require that concrete finish on exposed concrete surface (Class 1 Concrete Finish) shall be achieved solely by abrasive methods. Consequently, per BCM 105-1.0, any sprayed-on cement finish or applications similar to surface painting, where cement or grout (which is cement mixed with water) is applied to the concrete surface as the surface finish, will NOT be acceptable as a Class 1 Surface Finish, unless it is otherwise explicitly provided for in the contract.

Class 2 Surface Finish

Class 2 Surface Finish, or gun finish, is not a common concrete surface finish, and when it is required, it will be shown on the plans or specified in the special provisions. This finish consists of the pneumatic application of a mortar coating, similar to shotcrete, after the ordinary surface finish is completed.

When Class 2 surface finish is specified, ordinary surface finish shall first be completed. The concrete surfaces shall then be abrasive blasted to a rough texture and thoroughly washed down with water. While the washed surfaces are damp, but not wet, a finish coating of machine applied mortar, approximately 1/4 inch thick, shall be applied in not less than two passes. The coating shall be pneumatically applied and shall consist of sand, Portland cement and water mechanically mixed previous to its introduction to the nozzle, or premixed sand and Portland cement to which water is added previous to its expulsion from the nozzle. The proportion of cement to sand shall be not less than one to four, unless otherwise directed by the engineer. Sand shall be of a grading suitable for the purpose intended. The machines shall be operated and the coating shall be applied in conformance with standard practice. The coating shall be firmly bonded to the concrete surfaces on which it is applied.

Safety and Environment

Over the years there has been increasing emphasis placed on mitigating the impact of concrete construction practices on the environment. A lot of these requirements are required in the Storm Water Pollution Prevention Plan (SWPPP) embedded in the contract documents or in permits from regulatory agencies. Similarly, there have been improvements in construction safety practices. A few of those that affect concrete finishing practices are discussed below.

Working Over Water and Environmentally Sensitive Areas

When it is necessary that the contractor perform finishing operations over water ways or environmentally sensitive areas (ESA), all necessary precautions should be taken to prevent cement dust, sand, rinse water, or any other related finishing materials from entering these areas.

Working Over Traffic

When it is necessary that the contractor perform finishing operations over a traveled way, lane closures should be made beneath the finishing operation if there is any possibility that the scaffolding will be lower than the bridge soffit, or if there is any possibility that objects will fall onto the traveled way.

Special Concrete Construction Practices

This section covers special concrete construction practices employed for situations that are not commonly encountered on most Caltrans bridge construction projects, such as hot and cold construction and placing concrete underwater, but presents technical challenges requiring extensive discussion to ensure that the concrete used or placed in these special conditions meets the strength, durability, and other quality standards.

When the ambient temperature is higher or lower than an optimum range of about 50°F to about 90°F, special procedures and precautions are necessary to avoid the detrimental effects on concrete quality that can result from either hot or cold weather conditions.

High temperatures will increase the water demand, decrease workability, reduce strength, and increase shrinkage. Low temperatures will retard strength gain, and severe damage will occur if the water in the cement paste freezes while the concrete is plastic or during the first few hours of the curing period.

The detrimental effects of adverse weather conditions, and the measures that may be employed to mitigate those effects, are explained in the following sections. Engineers on projects involving concrete construction under adverse weather conditions should review contract requirements applicable to such construction, along with the recommended mixing, placing, finishing, and curing procedures that may be employed to ensure that the desired strength and durability properties of the concrete will be obtained despite the adverse weather.

Specification Requirements

When placing concrete under adverse weather conditions, the most effective means of assuring satisfactory results is keeping the concrete temperature within an acceptable range of 50°F and 90°F while it is in the plastic state. However, in cold weather construction, neither aggregates nor mixing water can be heated beyond 150°F.

Hot Weather Construction

When concrete is placed under hot weather conditions, both the placing of the mixture and the characteristics of the hardened concrete are adversely affected by several factors associated with high temperature. For example, the water demand increases as temperatures rise, and this increases the water-cement ratio unless compensating measures are taken. For a given concrete, the amount of water needed to maintain the same consistency will increase as much as 15% as the temperature of the fresh concrete increases from 50°F to 100°F.

At high temperatures the time during which concrete remains plastic is decreased, and this in turn decreases the window within which the concrete must be mixed, delivered to the site, placed, and finished. Rapid stiffening encourages undesirable re-tempering of the mixture by adding water to keep the mix in a plastic state. Reduced workability makes placing and consolidation more difficult, and this may result in the formation of rock pockets and cold joints at locations where fresh concrete is placed against partially hardened concrete.

High temperatures will accelerate setting and early strength gain; however, high temperatures appear to adversely affect gel formation during the hydration process, thus resulting in lower ultimate strength. As previously noted, the increase in water demand associated with higher temperatures will increase the water-cement ratio, resulting in a further strength reduction.

High temperatures increase the tendency for cracks to develop, both before and after the concrete sets. Rapid evaporation of bleed water may cause plastic shrinkage cracking before the surface hardens. Cracks in the hardened concrete may form as a result of drying shrinkage stresses which may be more severe due to the increased mixing water demand or because of volume changes due to cooling of the concrete mass from its elevated initial temperature.

As temperatures increase, preventing loss of water from within the concrete mass becomes more difficult, so curing becomes a more critical activity. Also, controlling the air content in air-entrained concrete is more difficult. For a given amount of air-entraining agent, less air will be entrained as temperatures increase.

Control of Concrete Temperature

The temperature of a concrete mixture varies directly with the temperature of the various ingredients at the time of batching. During hot weather, keeping the temperature of the concrete mixture below the specified maximum of 90°F (80°F for bridge decks) may require cooling one or more of the ingredients before batching.

The effect of each ingredient on the temperature of the mix is a function of the quantity of the ingredient used, its specific heat, and the temperature of the ingredient at the time of batching. Since the aggregates comprise about 70 to 80% of the combined mix, aggregate temperature has the greatest influence on mix temperature. In hot weather, aggregate stockpiles can reach temperatures up to 120°F. When compared to aggregate at 70°F , and with other factors being equal, the use of the warmer aggregate will increase the temperature of the concrete mixture about 30°F.

Several methods may be used to lower aggregate temperature. Shading stock piles and storage bins from the direct rays of the sun will provide some benefit, although this may not be feasible at many plant locations. Sprinkling coarse aggregate stockpiles with water is very effective, and can lower the temperature to a normal range of 60°F to 80°F under optimum ambient conditions. Spraying the coarse aggregate immediately before use will produce some cooling, but this method is not as effective as continuous sprinkling.

Sand in stockpiles is more difficult to cool, but sand (because of its normally higher moisture content) is less likely to experience a significant increase in temperature unless the stockpiles remain unused for a long period of time.

Other factors being equal, lowering the temperature of the mixing water is the easiest and most effective way of lowering the mix temperature. This is the case because of the high specific heat of water, which is four to five times that of most aggregates.

Mixing water should be taken from the coldest available source. If above ground storage tanks are used, they should be shielded from the sun or, if this is not possible, painted white.

Some concrete plants are equipped with refrigeration coils in the water storage tank, which can lower the mixing water to about 40°F. If this is not cool enough to produce the desired concrete temperature, and if cooling aggregate is not feasible, it will be necessary to replace some of the mixing water with chipped or shaved ice. If ice is used to cool the concrete, discharge of the mixer will not be permitted until all ice is melted.

Using ice in the mixing water is highly effective in reducing the concrete temperature because melting ice removes heat at the rate of 144 BTU per pound. For example, if 50% of the water in a typical six-sack mix was replaced by ice, the melting ice would lower the concrete temperature about 20°F and the resulting water at a temperature of 32°F would produce further cooling of about 8°F.

The amount of ice and water used may not exceed the specified maximum water content for the mix. If ice is used, it must be completely melted before discharged from the mixer.

Cooling cement is not generally feasible; but this is not of serious consequence because of the low specific heat of cement and the relatively small amount used in the mix.

The use of liquid nitrogen to cool aggregates is a viable procedure, and may be worth considering under extreme conditions.

Note: There are many published construction guides that a contractor may use as a guide for placing concrete in hot weather that include a version of the ACI nomograph shown in Figure 5-22 that may be used to estimate water evaporation rates while the concrete is plastic. While estimated variables (temperature, humidity and velocity) may be used during the planning stage, it is important to note the variables used during planning must be checked with actual readings during the concrete placement.

Mixing and Delivery

When transit-mixing equipment is used, the trucks should be dispatched and the work should be planned and organized to avoid any delay in placing the concrete after it is delivered to the site. The heat generated by prolonged mixing, even at agitating speeds, will cause a noticeable increase in the concrete temperature, particularly when the trucks are exposed to the direct rays of the sun. This is an important and frequently overlooked consideration.

If an unavoidable delay should occur, the heat generated by mixing can be minimized by stopping the mixer and then agitating intermittently, but this is strictly an emergency procedure. The important point is that truck mixers must be unloaded as soon as possible after they arrive at the site, and the work should be planned accordingly.

If truck mixers are painted white, they will absorb less heat than when painted other colors. For example, after an hour of exposure to the sun, surfaces painted gray may be 5°F to 10°F warmer than similar surfaces painted white and black painted surfaces may be as much as 30°F warmer; and the inside of the mixer will have been heated proportionally. Some suppliers have installed spray bars to apply water to the outside of the drum to create an evaporative cooling effect.

Under hot weather conditions, the Standard Specifications place a restriction on the 90-minute time normally allowed between batching and discharge of concrete when truck mixers are used. The specific provision is that a time less than 90 minutes may be required under conditions contributing to quick stiffening of the concrete, or whenever the ambient temperature is 85°F or above. The specifications give the engineer authority to limit the time available for delivery and standby at the site by reducing the overall time allowed, and the engineer is expected to use this authority in any situation where, in his judgment, a reduction is warranted to ensure a satisfactory end product.

Placing and Finishing

It is important to estimate the probable placing rate, giving due consideration to crew size and equipment available, before work begins. The delivery of concrete to the work site must be controlled so it does not exceed the estimated placing rate.

Prior to placing concrete, the forms and reinforcing steel should be wet down with cold water. Wetting the area around the work is also beneficial since it cools the surrounding air and increases its humidity, thus reducing both the rate of evaporation and temperature rise during the placing operation.

Concrete dries more rapidly as temperatures increase, so extra care is needed to avoid cold joints. For column and wall pours it may be necessary to reduce the thickness of each lift as it is placed to ensure bonding with the previously placed lift.

Controlling the quantity, as well as the rate, of concrete placed may be an important consideration for bridge decks where exacting surface fin shing requirements make it imperative that finishing follow closely behind placing to minimize moisture loss from evaporation. Preventing excessive loss of surface moisture is important for two reasons. First, surface moisture is necessary for satisfactory deck finishing. Second, and more important from a structural standpoint, loss of surface moisture is the primary cause of plastic shrinkage cracking.

Keep in mind that plastic shrinkage cracking is not caused by high temperature alone, since such cracking can occur whenever ambient conditions are favorable for the rapid evaporation of surface moisture. However, the conditions that contribute to plastic shrinkage cracking are exacerbated by hot weather. In view of this fact, special care is necessary to minimize the rate of evaporation of surface moisture when high temperatures prevail. Plastic cracking and bridge deck fogging have been covered previously in this chapter.

During extremely hot periods when temperature conditions are critical, consideration should be given to placing concrete during the early evening or at night when lower temperatures will reduce the rate of evaporation.

Curing

During periods of hot weather, a special effort may be required to ensure that the purpose of curing (i.e., keeping the concrete in a moist condition and within an optimum temperature range) is achieved, as these conditions become increasingly more difficult to obtain as ambient temperatures rise.

All curing equipment and facilities should be at the site of the pour and available so that curing may begin without delay as soon as the concrete is placed and finished. Particular care must be taken during hot weather to ensure that all exposed surfaces are kept continuously wet to prevent moisture loss until the permanent curing medium is applied.

In the case of bridge decks, it may be necessary to apply water to the deck surface (by means of a mist or fog spray) before all finishing is completed to maintain a moist surface. After finishing, the surface must be kept moist until the curing compound is applied. In either case, the amount of water applied should be carefully controlled, and should not exceed the minimum amount required to prevent surface drying.

During hot weather, it is important to keep in mind that for optimum curing, concrete must be kept cool as well as moist during the curing period. Cooling can be achieved by the application of water, beyond the amount needed for moisture retention. Formed surfaces, as well as exposed surfaces, may require cooling.

The specifications provide that during periods of hot weather and when directed by the engineer, water shall be applied to concrete being cured by the curing compound method or the forms-in-place method, until the engineer determines that a cooling effect is no longer required. (Standard Specifications, Section 90-7.03[12].) Application of water pursuant to this specification is paid for as extra work at force account.

When applying water to cool exposed surfaces, avoid the use of water that is excessively cooler than the concrete. Extremely cold water may cause cracking as a result of thermal stresses due to temperature change at the surface.

Use Of Admixtures

Even though admixtures may not be required, their use should be considered during hot weather as a means of mitigating some of the undesirable side effects of hot weather construction.

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[12] 2006 Standard Specifications, or 2010 Standard Specifications, Section 51-1.03H, "Curing Concrete Structures".

A set retarding admixture may be used to delay the initial set and thereby compensate for the accelerating effect of heat. A water reducing admixture will mitigate the loss of workability resulting from moisture loss during mixing and placing under hot weather conditions. An admixture having both set retarding and water reducing properties will be doubly beneficial.

For a given concrete, increased workability may be obtained by using an air-entraining admixture; however, some caution may be advisable for bridge deck construction since air-entrainment slows the rate at which bleed water reaches the surface, and this may have a detrimental effect on surface finishing. If an air-entraining admixture is not specified, the average air content of three successive tests may not exceed 4%, and no single test value may exceed 5-1/2%.

Cold Weather Construction

Provided certain precautions are taken, concrete construction can continue throughout protracted periods of cold weather. To ensure satisfactory results, the temperature of the concrete mixture, when batched, must be high enough that the mixing water will not freeze while the concrete is in a plastic state, the newly placed concrete must not freeze while it is setting, and the hardened concrete must be protected from the cold (and heated if necessary) to maintain an internal temperature that is high enough to assure the required strength gain during the curing period.

Mixing And Delivery

To prevent freezing while the concrete is still plastic, the specifications require a minimum concrete temperature at time of placement of 50°F . Therefore, when the ambient temperature falls below 50°F, it will be necessary to heat the water or the aggregates, or both, to keep the concrete mixture above the required minimum temperature.

When aggregate temperatures are above freezing, in most cases it will be necessary to heat only the water to obtain the required concrete temperature. When ambient temperatures drop below the freezing point, the moisture in the aggregates will freeze unless the stockpiles and batch bins are protected and/or the aggregate heated. Frozen aggregate must be thawed before use; otherwise, thawing in the mixer will result in excessively high water content.

Taking measures to prevent aggregate from freezing is easier than thawing aggregate after it has frozen. To this end, aggregate stockpiles should be protected by covering them with tar paulins and applying heat. Heat may be applied by space heaters or by circulating hot water or steam through pipes at the bottom of the stockpile.

When using hot water some adjustment of the batching cycle will be necessary to prevent the water from mixing directly with the cement, as direct contact may induce premature setting (flash set) or the formation of cement balls. The usual procedure is to add the water and most of the aggregate before adding the cement.

Under the current specifications, neither the water nor the aggregate may be heated above 150°F. This is a reasonable limitation, since 150°F is well-above the temperature needed to produce a concrete mixture within the specified temperature range, even under the most severe conditions likely to be encountered in California.

Placing and Finishing

The temperature of all surfaces that the fresh concrete will contact must be above freezing before concrete is placed. Specifications prohibit placing concrete on frozen or ice-coated ground, subgrade, forms, or any other contact surface. To achieve this condition it may be necessary to heat the forms and reinforcing steel with space heaters ahead of the concrete as it is placed. If the ground is solidly frozen, however, concrete placement should be suspended until the ground thaws and will not freeze again during the curing period.

It is essential that after placement, concrete be protected during the hardening period and that a temperature favorable for hydration be maintained. The specifications require concrete, to be maintained at a temperature of not less than 45°F for 72 hours and not less than 40°F for 96 hours more. Additionally, when requested by the engineer, the contractor must submit his proposed method of complying with this requirement. These are important and necessary provisions because the rate at which hydration occurs slows as the temperature drops, and from a practical standpoint, strength gain stops entirely when the temperature is below about 35°F.

Many methods are available to maintain concrete temperature. When the work area can be enclosed or covered, space heaters are an effective heating method. The use of steam released within an enclosure provides both heat and moisture for proper curing, but a steam generator is less economical than space heaters, so this method is not widely used. Bridge decks are usually covered with a straw blanket, which is an effective means of retaining the natural heat of hydration. Thermal blankets are expensive, both from a first cost and operational standpoint, but they are effective and may be economical under severe weather conditions.

Curing

Concrete generates heat rapidly during the first few days of the curing period. If the forms are well insulated and the exposed surfaces of the concrete are covered properly, it may not be necessary to apply any heat from outside sources to maintain the required minimum temperature. In any case, it is essential to conserve the heat generated, thereby reducing to a minimum the amount of external heat needed. For maximum effectiveness, insulation used to conserve the heat must be in close contact with the concrete surface.

Rapid cooling of the concrete at the end of the curing period must be avoided to prevent surface cracking. As a guide, the maximum temperature drop during the first 24 hours following the end of a curing period when external heat has been applied should not exceed about 40°F.

Measuring Concrete Temperatures

Thermometers provided by the State should be used to measure concrete temperatures. The temperature of hardened concrete should be measured by inserting the stem of the thermometer full depth into a previously formed water-filled hole in the concrete. (A drinking straw makes a good form for this purpose.) When no longer needed, the hole should be plugged at the surface to keep out debris. When measuring the concrete temperature, sufficient time must be allowed for the thermometer to stabilize before the temperature is recorded. Corners and edges are particularly vulnerable, and a special effort should be made to check temperatures at these locations.

Placing Concrete Under Water

Bridge foundations in streams and rivers, and bridge and building construction below the ground water elevation at other locations, may require concrete placement below the water surface. Typical situations include concrete placed to seal cofferdams, concrete placed below groundwater in CIDH piles and caissons, and concrete placed under and around precast sections of subaqueous tunnels and subways.

While a detailed discussion of underwater construction is beyond the scope of this manual, the following discussion of seal course concrete placement and concrete placement using water and slurry displacement procedures has been included as an introduction to the subject.

Contract requirements for seal course concrete are found in Section 51 of the Standard Specifications. Requirements for water displacement or slurry displacement procedures will be found in the specifica ions for the project where that particular construction method is specified.

Seal Course Concrete

By contract definition, a seal course is a layer of concrete that is placed within a watertight cofferdam by means of a tremie or concrete pump. The seal course must be of sufficient strength and thickness to resist the hydrostatic pressure developed at the bottom of the cofferdam when the cofferdam is dewatered.

The specifications permit seal course concrete placement using either tremie methods or by means of a concrete pump. In the past, tremie methods were used exclusively, and the terms "tremie seal" and "tremie pour" are still used informally in connection with seal course concrete placement. Today, most seal course concrete will be placed with a concrete pump, however, tremie placement remains a viable construction procedure and will be the method of choice at any location where the use of pumping equipment is economically infeasible.

Note that a "tremie" is a watertight tube not less than 10 inches in diameter with a hopper at one end. The hopper is supported by a working platform above the water surface, and the tremie tube must be long enough to reach from the platform to the lowest point of deposit. The lower end of both tremie tubes and pump discharge tubes will be equipped with a valve that may be closed to prevent water from entering the tube. This makes it possible to fill the discharge tube with concrete without removing it from the cofferdam.

Seal course concrete must be workable and cohesive with good flowability. This requires a fairly fluid mixture; consequently, the specifications require a nominal slump of 6 to 8 inches when concrete is placed under water. The proportions of fine and coarse aggregates may be adjusted to produce the desired workability with a somewhat higher proportion of fine aggregate than would be used for normal conditions. Seal course cementitious content is controlled by specification.

Concrete Placement

From a construction standpoint, the principal difference between tremie placement and pump placement is that tremie concrete is placed by gravity flow alone, whereas when a pump is used, placement is aided by pump pressure.

To begin placement, the valve at the lower end of the discharge tube is closed, and the tube is lowered into the cofferdam and filled with concrete. The valve is opened to begin placement.

As placement continues, the lower end of the discharge tube should be kept as deeply submerged in the previously placed concrete as conditions will permit. For tremie placement, the depth will depend largely on the head of concrete that can be maintained in the tremie tube. For either placement method, the tube must be lifted slowly to permit the concrete to flow out, care being taken not to lose the seal at the bottom. If the seal is lost it is necessary to raise the tube, close the discharge valve, refill the tube with concrete and then lower it into the concrete before placement may resume.

Every precaution must be taken to minimize segregation. As work progresses, the previously placed concrete should be disturbed as little as possible, and the top surface of the concrete should be kept as near level as possible. The discharge tube should not be moved laterally through previously deposited concrete. When it becomes necessary to move the discharge tube to a new position, the valve should be closed and the tube removed from the concrete before it is moved.

The specifications require a nominal 5-day cure period for seal course concrete before dewatering the cofferdam. However, when water temperatures are below 45°F, a longer cure time may be required to offset the slower strength gain at colder temperatures. Periods of time during which the water temperature is below 38°F are not considered as curing time.

Water and Slurry Displacement Method

The water and slurry displacement methods of underwater concrete placement are similar in that structure concrete is placed without dewatering the excavation in the traditional manner. Displacement methods provide an acceptable alternative construction procedure for concrete placement in CIDH piles and mined shafts where subsurface conditions make dewatering economically infeasible.

Generally, slurry displacement is specified. Water displacement is only acceptable when allowed by the special provisions. In both methods concrete is pumped into place through a discharge pipe or hose, which initially is set at the bottom of the drilled hole or excavated shaft. As concrete placement continues, the discharge end of the pipe or hose remains at the bottom, so that the heavier concrete, as it rises in the hole or shaft, displaces the slurry; hence the name "displacement" method.

Slurry displacement is most often used to facilitate installation of CIDH piles where all or a portion of the pile is below groundwater and the sides of the pile are not self-supporting. In such cases the slurry, which is commonly synthetic drilling slurry or a commercial quality mineral drilling mud, supports the sides of the pile until concrete is placed.[13]

Water displacement may be used to place concrete in CIDH piles constructed in fully cased holes and in mined shafts in rock formations where the excavation is below the ground-water surface and the amount of water present cannot be controlled using conventional de-watering procedures.

[13] Information on these products may be obtained from OSC Headquarters in Sacramento.

The concrete is placed in a continuous operation. High pressure pumping equipment is used to deliver the concrete through a discharge system to the bottom of the drilled hole or excavated shaft. To reduce the required pumping pressure, the discharge tube is raised when the height of the concrete in the pile or shaft reaches about 10 feet. This head is maintained as the remainder of the concrete is pumped into place.

Because the first concrete placed eventually reaches the top of the pile or shaft, it is essential that the concrete remain in a fluid state throughout the placement period. To achieve the desired fluidity, the mix design will include a high-range water-reducing admixture. Additionally, a "test" batch is required to demonstrate that the proposed concrete mix meets contract requirements.

Note: Where concrete is placed under slurry, the current standard specifications establish criteria for the minimum slump.

- For piles where the time required for concrete placement is 2 hours or less, the concrete mix design should achieve a slump of at least 7 inches for twice the concrete placement time.
- For piles where the time required for concrete placement is more than 2 hours, the current Standard Specifications, require that the concrete mix design achieve a slump of at least 7 inches at the end of concrete placement plus 2 hours.

When either water displacement or slurry displacement concrete placing methods are to be used, the specifications will include concrete materials and production specifications, placing procedures, testing requirements, and when a slurry is used, slurry material and production requirements as well.

Bridge Construction Practices

This section is intended to highlight the recommended concrete construction practices that are applicable to each of the various bridge elements, within the context of State construction contracts. Particular emphasis is given to concrete box girder bridges, which is by far the most common type bridge constructed by the Department.

In general, the concrete in each integral part of the structure must be placed continuously and the contractor should not be allowed to begin unless the forms are ready, there is sufficient material on hand, and the contractor's forces and equipment to complete the contemplated work without interruption.

Foundations

Bridge foundations are not only the structural support of a bridge, but they are also in essence the template that serves to fix the location, position and alignment of a bridge. As such, it is incumbent upon the inspector to ensure that the bridge foundations are located, positioned and aligned according to plans. The primary means of establishing the lines and grades for bridges are through the "control stakes" provided by Caltrans' surveyors for the bridge foundations. Since bridge foundations control the bridge lines and grades, the inspectors should invest time and effort in ascertaining that the survey stakes for the bridge foundations are correct before they are constructed.

Concrete Piles

There are two types of concrete piles that are in common use in Caltrans bridge construction projects: pre-cast concrete piles and cast-in-place concrete piles.

Cast-in-place piles are further classified by whether the concrete is placed in a dry hole or deposited under slurry, also known as wet pile. Refer to the Foundation Manual for detailed descriptions of cast-in-place piles.

For piles cast in dry holes, the concrete placing, consolidation and finishing requirements are similar to that of any other bridge structure concrete, with some special requirements unique to this type of piles. In general, piles having a diameter of 30 inches or more are vibrated from the bottom of the reinforcing cage to the top of the pile. Unless otherwise specified, piles with diameters of less than 30 inches, vibration is required only in the top 15 feet of the pile, regardless of the depth of the reinforcing cage. There are no specified limitations on the fall distance for concrete placed in piles, provided that the falling concrete falls to the center of the pile without hitting rebar on the way to the bottom.

For dry piles, the concrete, in some instances, may be placed directly from transit-mix trucks, using the truck-mounted chute. The use of a small hopper at the top of the pile shaft will reduce segregation, since it will recombine the concrete as it leaves the chute and provide a vertical fall through the reinforcing cage. This may be particularly important if a more fluid mixture is being used. Care must be taken to minimize amount of concrete falling directly on the reinforcing steel cage or striking the sides of the pile by using drop chute, for example. To minimize the formation of voids and rock pockets in the lower, unvibrated length of the pile, consideration may be given to increasing the water content to secure a more fluid mixture. In no case however, should the water content exceed the specified maximum amount.

For wet piles, extra care and precaution in placing the concrete is necessary because of the high likelihood of concrete defects. Because of the high risk associated with concrete placed under water, standard special provisions were developed for this challenging concreting process. Additional information can be found in the section entitled "Placing Concrete Under Water" earlier in this chapter of this manual and in the Foundation Manual.

Footings

Immediately before placing concrete, the ground surface on which the footing concrete will be deposited should be thoroughly dampened with water. During concrete placement, the ground should be saturated, but there should not be any puddles of standing water.

Footing concrete is usually deposited directly from transit-mix trucks using truck-mounted chutes. In cases where the distance between the truck and the footing is too great to be reached with truck-mounted equipment, special chutes may be employed. Water should not be added merely to facilitate the use of chutes. Where the bottom of the footing is on a slope, the concrete should be placed first at the low end and then proceed up the slope. In all cases concrete should be deposited near its final location, and use of vibrators for extensive shifting of the concrete should not be permitted.

Revibration is required for footings that are more than 2-1/2 feet thick and have a top layer of reinforcing steel. The purpose of this revibration is to minimize the presence of voids below the top layer of reinforcing bars. Such voids, which may occur as a result of bleed water collecting below the bars, will reduce the bond between the concrete and the reinforcing bars. The specification requirement is that revibration is to be performed as late as the concrete will respond to further vibration, but no sooner than 15 minutes after initial screeding has occurred. To hasten final finishing, most contractors would prefer to perform the required revibration sooner rather than later. When determining the optimum delay period, the engineer should keep in mind that for maximum effectiveness, revibration should be delayed as long as possible.

Following the usual striking-off of concrete to the planned grade, and after revibrating the concrete, the concrete should be hand-floated to obtain an even-textured surface free of voids, water and air pockets. As the concrete sets, the top of the footing should be re-floated to seal the surface.

Columns and Walls

Concrete for columns and walls are usually placed by means of concrete pump or by crane and bucket. When the form height exceeds 8 feet, the specifications require the use of a drop chute below the hopper to prevent segregation and to direct the concrete so that it does not strike the bar reinforcing steel or the sides of the forms above the level of placement. Adjustable-length metal or plastic tubes are commonly used for this purpose. When pumps are used, the pump discharge hose may be used to satisfy this requirement, in which case a hopper will not be needed.

Concrete should be placed in successive lifts, and placement should be planned so that the horizontal level of the concrete rises at a uniform rate over the entire area being placed. In general, lifts should not exceed about 2 feet in height in narrow walls or 1 foot for wider walls and columns; otherwise the effectiveness of vibration will be severely reduced. To ensure bond between lifts, vibrators should penetrate through the lift being placed into the previously placed lift. In any case, no more concrete should be deposited into the forms than can be vibrated conveniently and effectively.

Finally, the placing rate must be controlled so as to avoid excessive form pressure. The pressure exerted against the side form is directly related to the height of the plastic concrete in the form. If the placement rate is greater than assumed in the form design, the resulting higher pressure may produce excessive deflection, or even failure. Additionally, because vibration makes concrete more fluid, form pressures are greater in areas being vibrated, an otherwise stable pour could experience form failure while undergoing vibration.

Bent Caps

Occasionally, a particular sequence of construction will require that concrete be placed in the columns and bent cap in the same pour. In such cases, it is essential that placing of the cap concrete be delayed until the concrete in the columns has taken its initial set. By following this procedure, settlement and shrinkage of the column concrete will occur before the cap concrete is placed, thus substantially reducing shrinkage stresses at the joint between the column and cap.

Superstructure

The current specifications require concrete for girder spans (box girder and cast-in-place T-beams) to be placed in not less than two operations, with the last operation being the top deck slab. Unless otherwise permitted by the engineer, at least 5 days must elapse between each operation. The 5-day minimum time period is included in the specifications to allow the girder stem concrete to gain sufficient strength to resist stresses caused by additional falsework deflection and settlement under the added load of the concrete deck.

Stem and Soffit-Box Girder

In a conventional box girder construction, the concrete for the bottom slab (soffit) is placed with the concrete for the girder stems in one concrete placement operation. Although there is no specified construction sequence, when the soffit and girder stems are placed in a single operation, most contractors place concrete in the bottom slab first, and follow with the girder stems. For this placing sequence, a decision must be made as to how far ahead (of the stems) the soffit concrete should be placed before placing concrete in the stems. This decision is influenced by several factors, including the length and width of the super-structure, the expected rate of concrete placement, and probably most important, the size and experience of the construction crew. Ideally, the bottom slab concrete should be allowed to set until it is stiff enough to retain concrete placed in the stem, but not so long that it cannot be reworked.

From a construction standpoint, no harm will result if stem concrete is placed before the slab begins to stiffen, except that additional work will be required in placing the concrete. However, if too much time elapses, a cold joint may occur between the fresh concrete in the stem and the older concrete in the slab, thus impairing the bond at the cold joint interface. The major drawback to placing the bottom slab first is the need for additional finishing after the stems are poured. In the typical operation, the bottom slab will have been struck-off and initial hand floating completed before concrete is placed in the adjacent girder stems. Vibrating the girder stems will force some concrete out of the stem below the bottom of the stem form, which will require additional clean up and floating of the slab.

Placing the girder stem concrete first has the apparent advantage of reducing the slab fin shing effort, but this saving is seen to be illusory when all factors are considered. When the girder stems are placed first, vibrating of the soffit slab adjacent to the stem tends to draw concrete from the lower part of the stem into the soffit slab area. When this occurs, voids will be created around the longitudinal reinforcing at the bottom of the stem and, for cast-in-placed prestressed construction, below the pre-stress ducts near mid-span. Unless the stem concrete is thoroughly revibrated after the bottom slab is placed and vibrated, rock pockets are a certainty. For longer spans having relatively deep girders, the bottom slab (soffit) may be placed first, as a separate operation, to reduce the size of the combined concrete pou .

Regardless of the placing sequence followed, as the bottom slab is placed, the top surface should be struck off to grade and hand-floated with a wood float. As the concrete sets, the slab should be re-floated as necessary to seal the surface. Girder stem concrete should be placed in lifts, and each lift must be thoroughly vibrated. To ensure bond between successive lifts, the vibrator must penetrate through the fresh concrete and into the previously placed lift. And as previously noted, if the stems are placed first, thorough revibration to the bottom of the stem after the soffit slab is poured is necessary to prevent voids and rock pockets near the bottom of the stem.

Stem/Deck Construction Joint

Since the girder stems and bridge decks are not placed in one single operation, as per specification, the construction joint between these two bridge elements is critical in making the box girder function as a monolithic structural member intended by its design. The horizontal shear capacity across the stem-to-deck joint increases significantly when the construction joint is intentionally roughened to a minimum amplitude of 1/4 inch. In theory, roughening the surface at the construction joint interface allows for the joint surfaces to ride up on each other as they attempt to slip. This action places the reinforcing steel across the joint (stirrups) in tension and provides a strong clamping force. Design codes allow for significantly more horizontal shear capacity when the construction joint is intentionally roughened to a minimum of 1/4 inch amplitude.

Per BCM 105-7.0, the top of girder stems is required to be intentionally roughened to a minimum of 1/4 inch amplitude. With this new method of roughening stem concrete, abrasive blasting will no longer be required to intentionally expose the aggregate. However, abrasive blasting is still required to clean the top surface of the girder stem of materials that are detrimental to a proper concrete bond prior to placing the deck concrete.

Figure 5-29. Stem Concrete with 1/4 Inch Roughening.

Roughening the top of the stems can be obtained with many different methods. The intent is to obtain a rough concrete surface that is not floated or troweled and provides an uneven surface with a minimum of 1/4 inch amplitude across the entire top surface of the girder stem, partially exposing, but not loosening, the coarse aggregate.

During the roughening operation, care should be exercised to avoid excessive dislodging of coarse aggregates when using the roughening tool, floating/troweling of the top surface of the stem, and over vibrating the concrete. Floating forces coarse aggregate into the paste and makes the surface smooth and overvibration causes the cement paste to rise and cover coarse aggregates, also making the surface smooth.

In addition to roughening the stem concrete, it is also extremely important that the surface of the construction joint be cleaned by abrasive means prior to placement of deck concrete to remove all laitance, curing compound, and loose concrete.

Bridge Decks

Of all the elements in a reinforced concrete structure, the bridge deck is the element that is subject to the most severe conditions of exposure and wear. Bridge decks are subjected to high live-load stresses and alternate cycles of wetting and drying, and may be exposed to extreme temperature changes as well. Additionally, in many parts of California, bridge decks are exposed to freezing and thawing and the deliberate application of de-icing salt.

Bridge decks contain a congestion of reinforcing steel, making the use of a workable concrete essential. Because of bleeding and surface finishing requirements, the worst quality concrete in the deck is at the surface. Clearly, to ensure the adequacy and integrity of the completed work, bridge deck construction should receive the engineer's personal attention, with special emphasis on production, placing, finishing, and curing of the concrete.

For recommended practices, inspection procedures, policy, and other information of interest to field engineers who are involved with bridge deck construction, refer to the Bridge Deck Construction Manual. It is noted, however, that the deck construction manual is concerned primarily with engineering and inspection of deck construction as a whole; it does not (and is not intended to) stress the procedures that are necessary to obtain quality concrete. Accordingly, and in addition to the instructions and information in the deck construction manual, the following points are emphasized:

- First and foremost, obtaining a workable concrete with the lowest possible water-cement ratio should be the engineer's goal. Since the deck is the last major element constructed, there should be ample time during previous concrete pours to refine production methods and construction procedures to achieve this goal.

- Emphasis should be placed on reducing the water demand. In particular, the use of a water-reducing admixture will be beneficial under almost all circumstances.

- When pumps are used to place concrete mixtures having a high percentage of crushed aggregate, the increased water demand may be reduced by the use of a workability admixture. While there are several products, which are marketed as pumping aids or lubricants, and are available commercially to facilitate pumping of concrete, only products that are on the METS approved admixture list may be used.

- The deck finishing operation should be planned and carried out so as to minimize the need to apply water to the deck surface. An excessive amount of water applied to the surface, either while finishing is in progress or after finishing is completed but before application of the curing compound, will increase the water-cement ratio at the surface, and this in turn will increase scaling and accelerate deterioration of the wearing surface.

- Water should not be applied merely to facilitate deck finishing. If additional moisture is needed to properly seal the surface, it can be obtained by applying a thin layer of grout by the hand brush method. (Any grout used for this purpose should conform to applicable provisions in Standard Specifications 50-1.09[14].)

- Any water applied should be in the form of a fog spray, and the amount applied should not exceed the minimum amount needed to replenish moisture lost by evaporation.

- Air-entraining admixtures provide increased workability; however, they also reduce the rate at which bleeding occurs, and this may adversely affect deck finishing operations.

- For hot weather conditions, particular emphasis should be placed on keeping the temperature of the concrete mixture as low as practicable, and minimizing the indiscriminate use of water to increase workability and/or to facilitate finishing.

- The specifications require grinding of the completed bridge deck if grinding is necessary to obtain the profile and/or texture requirements specified. While grinding is a contract requirement to improve profile and surface characteristics, it is also detrimental to the structural integrity of the bridge deck, for two reasons. First, grinding reduces the cover over the embedded deck reinforcing

[14] 2006 Standard Specifications, or 2010 Standard Specifications, Sections 50-1.02C and 50-1.03B(2)(d)

steel and second, it abrades the finished surface, leaving exposed and fractured aggregate particles. Grinding is particularly detrimental in areas subject to repeated freeze-thaw cycles and the application of de-icing salt. In view of this, the engineer should keep in mind that deck grinding is not a panacea to solve surface problems that should have been solved during the deck finishing operation.

Barrier Rails

The specifications require that, when ordered by the engineer, the height of the concrete railings shall be adjusted to compensate for the camber and dead load deflection of the superstructure. When concrete barriers are to be constructed on existing pavement or existing structures, the height of the barriers shall be adjusted to compensate for irregularities in the existing grade. The amount of adjustment will be determined by the engineer and will be ordered before the concrete is placed.

Emergency Construction Joints

There is always the possibility that unanticipated circumstances will make it necessary to stop the work and form a construction joint at a location where none was intended. Examples of such situations include form failures, usually caused by form tie failure; equipment breakdowns, either at the site of the work or at the concrete plant; and adverse weather, such as a heavy rainstorm after work begins.

The specifications require that when a construction joint is necessary, it shall be constructed as directed by the engineer. If proper care is taken, an emergency construction joint can be installed at virtually any location in a reinforced concrete structure. In girder stems, the joint should be formed with step keys and additional reinforcing steel added in the manner shown on the Standard Plans for that particular girder type. If the joint is located within the center third of the span, additional stirrup reinforcing should be added as well. In slabs, the joint may be installed at any convenient location.

When installing an emergency joint, it may be necessary to remove some concrete that is already in place. This can be accomplished by cutting a hole in the forms and washing out the concrete if SWPPP concerns can be met, by removing the concrete by hand, or if the concrete has started to set, by removing it with jackhammers or similar equipment. Reinforcing steel should continue through the emergency bulkhead, but if this is not possible for some reason, the means by which the cut bar will be spliced later should be considered, and the bar cut with splicing in mind. Emergency construction joints may be made with any available material, keeping in mind that they must be removed later and the joint prepared for the following pour. The use of expanded metal lath, which may be left in place if the joint surface is sound, is an alternative material for thin slabs.

While the location of the joint in a girder stem will be dictated by the location of the concrete front when the emergency arises, its location with respect to the supporting falsework should be reviewed before work continues. If the joint is located near the center of a falsework span, additional beam deflection occurring when work resumes will remove the support under the previously placed stem. To prevent loss of beam support, it may be advisable to install a supplemental post at the joint location.

During a concrete pour, the need for an emergency construction joint may arise at any time. The engineer should be prepared to take whatever action is required, and should at all times know the best location for an emergency joint, the configuration required, and whether keys and/or additional reinforcing steel will be needed. Until all concrete is in place, the possibility of an emergency occurring should be anticipated, and the placing rate and location controlled with this in mind.

Weight Limits

Weight limitations are addressed in the Standard Specifications. The contractor may redesign the structure if an increased load carrying capacity is needed for construction purposes.

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CHAPTER 6
STRUCTURE CONCRETE REPAIR AND REHABILITATION

Table of Contents

Structure Concrete Repair and Rehabilitation **6-1**

Concrete Repair Process **6-3**
Assess the Defect and Determine the Cause 6-3
Surface Imperfections 6-4
Cracking 6-5
Scaling 6-7
Spalling 6-8
Delamination 6-11
Surface Wear 6-12
Determine Limits of Defective Concrete 6-12
Determine If Repairs are Necessary 6-13
Choose the Repair Strategy 6-14
Cementitious Materials 6-14
Polymeric Materials 6-15
Thermoplastic/Thermoset 6-16
High-Molecular-Weight 6-17
Free Radical Polymerization 6-17
Key Variables 6-17
Methacrylate 6-19
Polyester Concrete 6-20
Urethane 6-21
Epoxy 6-22
Epoxy Mortar 6-23
Remove Defective Concrete 6-25
Place Repair Materials 6-27
Cure The Repair 6-27

Repairing Minor Construction Defects 6-27
Dry-Packing 6-28
Conventional Concrete Patching 6-28
Shotcrete 6-29
Commercial Patching Materials 6-29
Epoxy Mortar Patches 6-30
Repair Testing 6-30

Large Defect Repair - Concrete Replacement 6-30
Preparation 6-31
Placing 6-31

Repairing Defects In New Bridge Decks 6-32
Filling Shallow Voids 6-32
Filling Deep Voids 6-32
Bonding Agents 6-33
Small Area Repair (< Two Inches Deep) 6-34
Large Area Repair (> Two Inches Deep) 6-34
Filling Cracks by Epoxy Injection 6-35

Bridge Deck Rehabilitation 6-36
Surface Preparation 6-38
Low-Slump Portland Concrete Overlays 6-40
Latex-Modified Concrete Overlays 6-41
Conventional Portland Cement Concrete Overlays 6-43
Methacrylate Overlays 6-44
Polyester Concrete Overlays 6-47

6 Structure Concrete Repair and Rehabilitation

Concrete that has been properly manufactured, placed, and cured should last for an indefinite amount of time as evidenced by structures that are still in service past their 50-year design life. Some of this may be attributable to the quality of the materials and workmanship of the original construction or the absence of severe environmental, natural or user impacts. Another factor that may attribute to the structures long-life is that each structure is inspected biannually as part of the Department's maintenance protocol and needed repair and rehabilitation is typically scoped and scheduled into construction contracts.

Remedies for surface finish defects like rock pockets and plastic cracking that have always occurred in concrete construction are found in the Standard Specifications. Once a structure is placed into service, there are many ways that immediate and severe damage may occur that prohibits normal use until repaired. The most prominent causes are vehicular traffic accidents which cause damage by impact, fire, explosion, or chemical leakage, and natural events like floods and earthquakes. Damage can also occur during routine service. The use of tire chains on icy roadways causes surface rutting. Overloaded vehicles may cause cracking or failure of structural elements. Concrete materials may fail due to durability issues also. Freeze-thaw damage gradually erodes the surface away. Alkali-silica reactivity creates an expansive gel within concrete literally forces concrete to fall apart.

Structure Representatives may be called upon to repair damage such as that shown in Figure 6-1. While the details of structural damage repair are developed on a case-by-case basis, the general process is outlined in the following steps:

- Make the existing structure safe
- Isolate the damaged area for safe removal
- Remove defective materials while preserving structural materials that will remain
- Replace defective elements like columns, girders and affected deck areas

Project duties that would normally be covered by the design project manager like developing lane closure charts, detours and storm water pollution prevention plans often are covered by the structure representative as part of the emergency repair.

Figure 6-1. Initial Damage Assessment.

Note: If you are involved in the initial damage assessment in an emergency situation, such as after an earthquake, refer to the OSC emergency operations documents, the Emergency Response Handbook and the Emergency Operations Plan for initial response procedures.

The repair and rehabilitation efforts covered in this chapter are the "bread-and-butter" types that occur regularly during construction and periodically during a structure's life span. Active construction inspection and periodic maintenance inspections after construction are needed to minimize the impacts of defects as they appear by repairing or rehabilitating structures before significant deterioration can occur. The practices described in this chapter are those that experience has shown to be the most effective in obtaining an acceptable final product, but they should not be followed blindly. If there are doubts as to the procedure to be followed, consult the construction engineer before proceeding.

In general, the size and location of the defect will determine the method of repair. For instance, large voids may require removal of unsound concrete. Extensive removal may warrant the installation of formwork prior to replacing concrete. In most cases, however, the defect will be such that it can be repaired satisfactorily either by placing a mortar patch or by "dry-packing" the void. Satisfactory results should not be left to chance. Repair work must receive the same attention to detail by the contractor and the inspecting engineer as the original construction. Achieving satisfactory repair results starts with knowledge of the procedures and materials used for the repair work.

The procedural steps required to repair a defect are listed below and described in this chapter:

- Assess the defect and determine the cause
- Determine the limits of defective concrete
- Determine if repairs are required
- Determine which repair method will be used
- Remove defective concrete as needed for the repair
- Place the repair
- Cure the repair

Additional material in this chapter covers specific repairs that occur regularly as part of the construction or structure rehabilitation:

- Construction defects that often occur such as rock pockets, bulges, and finish irregularities (these defects are relatively minor and correction is often made without removal of placed concrete)
- Bridge deck cracks caused by plastic shrinkage
- Structural repairs that require replacing portions of structural elements
- Bridge deck overlays

Concrete Repair Process

Assess the Defect and Determine the Cause

Typically, defective concrete is found by a visual inspection of the concrete surface. However, the area surrounding the visible defect should be probed with a geologist's or prospector's hammer or similar tool to determine the extent of the defective or unsound concrete. When using a hammer to explore damage, remember the hammer may also damage sound concrete. A thorough investigation of the defect will result in a more effective repair solution. Repairs that ignore environmental influences or usage patterns cannot be expected to endure. Also, there may be multiple causes for damage, such as, structural deficiency, low strength concrete and improper placing techniques that may have been used during construction. Chemically caused deterioration such as chloride or sulfate attack may appear as cracked or broken surfaces; repairing the surface alone does not remedy the underlying cause. Distorted or irregularly shaped structural elements indicate movement that could be caused by chemical attack, excessive loading overloads, or seismic loading. Notable defects fall into several categories:

- Surface imperfections like soft surfaces or surface voids
- Cracking
- Scaling
- Spalling
- Delamination
- Abrasion or surface wear
- CIDH pile anomalies (Refer to the OSC Construction Records and Procedures Manual and the Foundation Manual for pile mitigation and acceptance procedures)

Surface Imperfections

Voided surfaces, like those shown in Figure 6-2, caused by rock pockets, honeycombing, and cold joints are caused by any of the following:

- Improper mix design
- Poor placing
- Improper consolidation
- Leaky forms
- Excessive form deflection
- Or any combination of the above conditions

Soft surfaces are usually caused by improper treatment of wooden forms prior to concrete placement. New plywood may be affected by sunlight exposure. Tops of wall forms may deflect slightly and pull a thin layer of mortar away from parent concrete. Excessive bleed water will carry fine materials to the surface, resulting in laitance or sand streaking. (Refer to Chapter 5 for additional plywood, forming and placing information.)

Figure 6-2. Rock Pocket Damage.

Cracking

Because of concrete's low tensile strength and the volume changes that accompany changes in temperature and humidity, cracking is a concrete characteristic that can occur at any time during the life of a concrete structure. In new construction, cracks occasionally occur as a consequence of problems encountered as concrete is placed, such as excessive falsework settlement or form deflection. Most cracking associated with new construction, however, is plastic shrinkage cracking.

Plastic shrinkage cracks appear in the surface of fresh concrete after it is placed and finished, and while it is still in the plastic state; hence the name "plastic" shrinkage cracking. Plastic shrinkage cracks are a consequence of too rapid evaporation of moisture from the concrete surface.

When curing ceases, the free water within a concrete mass begins to evaporate. As the concrete dries, microscopic changes in concrete structure caused by water loss result in volume reductions of the concrete mass. If the concrete is restrained in any way, say for instance reinforcing steel, cracks will develop. Such cracks are called "drying" shrinkage cracks. Unlike plastic shrinkage cracks which occur on horizontal surfaces, drying shrinkage cracks can occur on any surface of any element in the structure, or within the concrete mass itself.

Cracking of sufficient severity to warrant repair is a rare occurrence in new construction. Severe cracking, when it does occur, occurs most often on the bridge deck and is usually the result of plastic shrinkage, although construction problems such as excessive falsework settlement or deflection can cause a cracking problem as well. The contract remedy for bridge deck cracking is Methacrylate treatment. Epoxy injection is the appropriate repair method for vertical cracks in other elements of a new structure where crack repairing is warranted.

From a structural standpoint, cracks in concrete usually are not a cause for alarm because cracking, in most cases, will not impair the load carrying capacity of a concrete member. There are two exceptions to this general observation: structural cracks and cracks that are indicative of internal stress development.

Structural cracks (i.e., cracks caused by load overstress) can be troublesome, particularly if there is significant crack movement under load. When warranted by safety or maintenance considerations, structural cracking can be repaired, and the integrity of the member fully restored, by filling the cracks (by pressure injection) with an epoxy adhesive.

Corrosion of embedded bar reinforcing steel is a major crack inducing factor in bridge decks in freeze-thaw areas where the deck is subjected to the application of deicing salts. Corrosion of reinforcing steel also occurs in structures located in a marine environment. Corrosion induced cracking, while not of serious consequence from a structural standpoint, is detrimental because it facilitates the ingress of moisture, oxygen and chloride ions, and thereby exacerbates the corrosion process.

Map cracking (sometimes called pattern or craze cracking) may occur several years after construction in a concrete structure made with reactive aggregate. In this case cracking is the manifestation of a more serious problem, which is the formation of expansive gels (alkali-silica reactions) within the concrete mass. This causes disruptive expansive forces which lead, eventually, to complete disintegration of the concrete. Typical map cracking is shown in Figure 6-3.

Figure 6-3. Map Cracking.

Scaling

Scaling is the flaking of surface mortar, often accompanied by the loosening of surface aggregates. Scaling occurs as the result of repeated cycles of freezing and thawing. If concrete cools below the freezing point, free water in the concrete freezes and forms ice crystals. The expansive pressure thus developed causes cracking and mechanical fracturing in the paste. Cracking and fracturing continues with each repeated freeze cycle. Eventually the paste disintegrates to the point where the aggregate particles are no longer bonded together and are easily eroded away. In cases of severe scaling, the mortar fraction of the concrete is completely broken down and loose aggregate can be scooped out by hand.

When a concrete structural element is exposed to deicing salt, air-entrainment may not give complete protection because the salt increases the amount of water absorbed by the paste. This increase occurs because the low vapor pressure of the salt solution allows many more of the entrained air voids to fill with water, and thus they are not available as reservoirs. As the saturation point is reached, air-entrained concrete becomes susceptible to frost damage because of its relatively high porosity and the large amounts of freezable water it contains. Air-entrainment, however, will greatly improve resistance to scaling in all cases.

Resistance to frost damage can also be improved by using a low water-cementitious ratio concrete. As the water-cement ratio decreases, there is a corresponding decrease in the porosity of the cement paste, which reduces the amount of freezable water in the concrete.

Figure 6-4. Severe Bridge Deck Scaling, Small Pothole.

Spalling

A spall is a chip or fragment of concrete that has broken away from a larger concrete mass, examples are shown in Figure 6-5. A pothole or pop out is a roughly circular or oval depression in the deck surface. Potholes are caused by the separation and removal of a portion of the surface concrete, revealing a horizontal or slightly inclined fracture. The formation of a pothole is preceded by the development of a roughly horizontal delamination slightly below the surface.

Historically, spalling has been defined as the chipping or breaking of a slab at a joint or other edge location. However, as deck deterioration attributable to deicing salt became increasingly widespread over the past two decades, the meaning of the term spalling was gradually broadened, and it now includes potholing as well. Today, the two terms are used synonymously to describe surface fracturing in a bridge deck, and spalling in the traditional sense is now referred to as edge spalling.

Figure 6-5. Spalling

Spalling is the consequence of corrosion of the embedded deck reinforcing steel, which occurs in structures where the deck is subject to repeated applications of deicing salt. Corrosion induced spalling may involve substantial thicknesses of concrete. Once initiated, the corrosion process cannot be reversed, and a permanent structural repair (short of complete replacement of the salt-contaminated concrete) is nearly impossible to achieve. Spalls may also occur as a result of vehicle collision, as shown in Figure 6-6.

Figure 6-6. Surface Spalling Caused by Vehicle Impact.

The cycle of events that leads to concrete spalling can begin when deicing salt is applied to inhibit the formation of ice on concrete. In the presence of water, the salt goes into solution with sodium and chloride ions. The chloride ions eventually permeate through the concrete with moisture until they reach the reinforcing steel. Penetration may proceed slowly by permeating the concrete, or more rapidly through any cracks that may be present. Cracks, of various causes, often develop over the uppermost reinforcing steel bars, usually those that run transverse to the highway alignment. Such cracks permit rapid chloride penetration. Once the chloride ion concentration reaches the threshold value, corrosion begins.

As iron oxide deposits (which are the end product of the corrosion process) form on the reinforcing steel in the anodic areas, the deposits occupy a much larger volume than the original metal. The increase in volume is generally considered to be about 13 times the volume of the original metal, and pressures exceeding 4,500 psi have been measured under laboratory conditions. However, much smaller pressures are suffic ent to produce the undersurface fractures that lead to most of the corrosion induced damage in salt-contaminated decks.

The expansive pressure developed as these deposits form acts outward (radially) in all directions from the bar, but because there is little resistance above the bar, the upward acting forces eventually lift the concrete from the bar. This causes a vertical crack to form over the bar (or enlarges the crack if one already exists at this location) and causes a horizontal crack, or delamination, extending outward from the bar.

As corrosion continues, the expansive pressure increases, enlarging the vertical cracking and extending the horizontal delamination. Eventually, concrete above the bar may break loose, forming a conical spall, while the delaminated area enlarges until it reaches a similar delamination produced by an adjacent corroding bar.

Delamination

Delamination of the concrete at or near the level of the top mat of reinforcing steel is characteristic of a chloride-contaminated concrete element where corrosion has occurred. Areas of delamination (the undersurface fracture plane) may cover a large area before there is any visible evidence of a problem. (Note, however, that undersurface fractures may be easily detected by the hollow sound they emit when the concrete is struck with a hammer or the surface is sounded with a chain drag.)

As the delaminated area enlarges, the vertical width of the fracture also increases. Deterioration of the concrete above the fracture is hastened by additional pressure developed by ice forming in the fracture and by vehicle live load forces. Eventually the concrete over the delamination breaks out, forming a depressed area, or spall, in the deck surface. Spalling, because it results in uneven depressions on a bridge deck, has a pronounced adverse effect on traffic, and even moderate spalling will seriously impair the riding quality of a bridge deck.

From a structural standpoint, spalling during its early stages is not a serious defect, for two reasons. First, concrete above the top mat of deck reinforcing does not contribute significantly to the load carrying capacity of the structure. Second, spalling (as visible evidence that corrosion has occurred) is usually well-advanced before there is a significant loss of reinforcing steel. This is true because very little expansive pressure is necessary to crack the concrete. As corrosion progresses, however, severe pitting occurs, resulting in a significant loss of section. Loss of section is accelerated when the reinforcing steel is exposed to atmospheric conditions and pollutants, as would occur when an open spall leaves reinforcing bars uncovered. In such cases the rapid section loss is the result of a relatively small anode (the exposed steel) leading to corrosion of a much larger cathodic area consisting of the adjacent embedded steel.

Traditionally, unsound concrete was found by dragging chain over the bridge deck and marking the affected area. Other sounding devices worked by percussion, like use of a geologist hammer or machines that produce a hammer effect with wheels that resemble sprockets; the individual projections on the wheel strike concrete like a small hammer blow. More recently, technologies like ground penetrating radar, seismic impact sound analysis and infrared thermography have been used to map delaminated areas.

Surface Wear

Deck deterioration caused by abrasive wear is often found in mountainous regions where winter driving conditions dictate the use of studded tires and/or tire chains. Differential wear in the wheel lines, which occurs where chains and studded tires are used, will produce depressions in the deck surface. Where the deck is relatively flat, depressions can result in ponded water and accelerated scaling.

Determine Limits of Defective Concrete

As part of the assessment, include an estimate of how fast damage is occurring. Cracking caused by alkali-silica damage may appear similar to freeze-thaw damage, but there may well be a significant difference in the rate subsequent damage occurs. The use of rock hammer or surface chaining are often used techniques for determining unsound concrete. Nondestructive test methods like ultrasound or x-ray imaging may also be used to efficiently assess internal damage. A Schmidt rebound hammer, a device that measures impact recoil or coring samples will assist the assessment of concrete strength. The damaged area may be large enough that it warrants development of a repair map to help determine quantities and repair strategies. In the example shown in Figure 6-7 the repair layout could be translated to a map of the deck repairs with each area keyed to a spreadsheet line where the repair quantities are tabulated and summed.

Figure 6-7. Delaminated Deck Surfaces Identified for Replacement.

Determine if Repairs are Necessary

Concrete surface cracks in bridge decks may not exceed the treatment thresholds given in the Standard Specifications and may be subsequently ignored. Minor damage may deteriorate so slowly, that the service life of the structure will never be affected. If surface grinding does not meet the required friction coefficient, bridge deck concrete must be replaced or overlaid. Similarly, if a profile correction is needed, the deck concrete must be overlaid or replaced.

Choose the Repair Strategy

After assessing the nature and extent of a defect, selecting the repair method and materials which will restore the structure to its intended purpose and durability at the most economical price is the next step. Repair options are doing nothing, partial replacement with drypacked mortar, conventional concrete, shotcrete, an overlay, surface treatment with a sealant and total replacement.

In the situation where the amount of significant cracking on a newly constructed bridge deck exceeds a specified threshold, methacrylate treatment is the specified treatment. In some circumstances, specifications may require that other coatings such as paint or sealants be used to seal the surface. Defective concrete may be patched, or removed and replaced with fresh concrete. Damaged bridge decks may receive methacrylate treatment or an overlay. In severe instances, the top 2 to 3 inches of concrete may be removed and a complete new riding surface placed. Cracks in walls may be sealed and injected with epoxy to bond the pieces together.

Many of the prepackaged products described herein in generic terms are available commercially under one or more brand names. When using rapid set proprietary products, the field engineer should verify its acceptability by checking the manufacturer's lot number with the Materials Engineering and Testing Services (METS) Structure Materials Representative. Rapid set patching materials may be classified chemically as either cementitious or polymeric substances. The cementitious materials include Portland cements, magnesium phosphate, alumina, and sulfoaluminate cements. The more commonly used polymeric materials include epoxies, methacrylate, polyester-styrene, and urethane.

Rapid set patching materials share many common characteristics. Some harden within minutes, and most within 1 to 2 hours. Most are self-leveling and self-curing. They are available commercially as prepackaged products, or can be manufactured at the job site using readily available component materials. Considering the intended purpose of a rapid setting material, most are relatively inexpensive, and all are easy to use.

Cementitious Materials

Although Portland cement is not a natural rapid setting product, sulfoaluminate cement can be combined with Portland cement to produce rapid setting early strength concrete. Grinding Portland cement to a finer screen accelerates the set time and produces higher early strength. Specifications may permit chloride-free accelerators, Type C accelerator and Type E water reducing/accelerating admixtures.

Portland cement patching materials have an inherent advantage in that the patch and the concrete substrate will have nearly identical thermal and physical properties. These are important considerations because thermal compatibility reduces the stress transmitted to the bond line and physical compatibility results in a relatively homogeneous structural section. Portland cement mortar or concrete is the least expensive patching material for permanent reconstruction of deteriorated concrete, and it (or a cement-based product) is generally used unless there is a compelling reason to use another material or product. Disadvantages include high drying shrinkage, the need to cure the patch, and relatively poor bond between the patch and the substrate. Because of poor bonding characteristics, an epoxy bonding agent is normally used with cement-based patching materials.

The water activated magnesium phosphate/sulfoaluminate cements are a standard repair option. They are also rapid setting, self-leveling cements with good surface bonding qualities. Magnesium phosphate reacts with aluminum, so aluminum finishing tools should not be used. Cementitious materials are described in Chapters 1 and 2.

Polymeric Materials

Polymers are a large class of materials that includes natural and synthetic molecules. Although generally not used in construction, polymers such as epoxy, polyester-styrene and methacrylate are commonly used in bridge repair/rehabilitation work. The typical polymer will consist of a single monomer molecule repetitively assembled into a molecular chain or a mixture of monomers, usually linked together by covalent bonds. Most polymers have high initial shrinkage and a high thermal coefficient of expansion. Intimate contact between the resin and the bonding surface is necessary to ensure adequate bond at the polymer/substrate interface. Polymer patching materials are self-curing, and most will reach about 80% of their ultimate strength in a few hours. The aggregate content must be as high as feasible, considering workability constraints, to increase the thermal compatibility with the concrete substrate. Rounded aggregate will improve workability and dry nonabsorptive aggregate will decrease the total resin demand.

Polymers formed by linking monomers together, without losing material are called addition or chain-growth polymers. Addition reactions are easier to process as volatile byproducts are not produced; this process is used with epoxies and urethanes. Because polymer molecules are large, they generally pack together in a non-uniform fashion, with a mixture of crystalline-like and amorphous material. Crystalline structure is influenced by chain length, chain branching and interchain bonding. Increased crystallinity is associated with an increase in rigidity, tensile strength and opacity (due to light scattering). Amorphous polymers are usually softer and transparent. In some cases the entire solid may be amorphous, composed entirely of coiled and tangled macromolecular chains.

CAUTION
Appropriate safety precautions must be observed when working with polymers. The applicable manufacturer's Material Safety Data Sheet should be furnished with the polymer material reviewed before the material is used. While polymer patching materials are not toxic when used in accordance with current practice for deck rehabilitation work, many have strong fumes which are irritating to some people and inhalation protection may be required. Also, as a general rule, skin contact should be avoided when using polymer materials.

Thermoplastic/Thermoset

Synthetic polymers can be classified as thermoplastic or thermosetting, according to the effect of heat on their properties. Thermoplastics grow softer as temperature increases, eventually melting and as temperature decreases, thermoplastics harden. Repeated heating and cooling cycles do not affect the properties of thermoplastics. Typical thermoplastics are polyesters such as Mylar, polyvinyl chloride (PVC) and acrilonitrile butadiene styrene (ABS). Thermoplastics can be reinforced like fiberglass with short, chopped fibers such as glass. Nylon, also a thermoplastic, is made from repeating units of two monomers, diamine and dicarboxylic acid.

Thermosetting polymers (thermosets) are a one-way reaction formed when resin and initiator are mixed. Thermosets maintain standard properties when heated, as long as temperatures do not exceed the Glass Transition Temperature (T_g), which varies with the particular resin system and quality of preparation methods. Above the T_g, the molecular structure of a thermoset material changes from a rigid crystalline polymer to a more flexible, amorphous polymer where properties such as stiffness, compressive and shear strength, water resistance and color stability drop. When material temperatures drop below T_g, the original material properties return. Polyethylene, a thermosetting plastic is based on repeating units of ethylene.

High-Molecular-Weight

Molecular weight is calculated by summing the individual atomic weights in a molecule. Water's molecular weight (H_2O =18 grams/mole) is determined by combining the atomic weight of oxygen (O = 16, 1 x 16 = 16) with that of hydrogen (H = 1, 2 x 1 = 2). If another oxygen atom were added (H_2O_2), the molecular weight would be 34 grams/mole, but the molecule would be hydrogen peroxide, not water. Polymers are varying length monomer sequences, but the properties of the molecule remain stable regardless of the sequence length. Simple compounds like hydrogen peroxide have low molecular weights. Polymers, because of their chaining nature, have high molecular weights. Polyethylene molecular weights range from 3×10^6 to 6×10^6 grams/mole. When high-molecular-weight is used to describe materials, the intention is to denote the polymeric reaction that links several small molecules into a larger molecule of repetitive elements.

Free Radical Polymerization

Materials like methacrylate and polyester concrete that are currently used on Caltrans projects use free radical polymerization to create polymer chains. Free radicals characteristically have an unpaired electron and readily attack other molecules in efforts to attain electron balance. Introductory chemistry covers electron shell configurations; the first electron shell has space for two electrons and the second shell has room for eight electrons. Hydrogen has one outer electron but needs one more to fill the first shell. The inner shell of a carbon atom is full, but the second shell only has four electrons; four more electrons are needed to fill the second shell. Oxygen also has a full inner shell but 6 electrons in the outer shell, so oxygen will look for 2 additional electrons, like in the formation of water (H_2O).

Polymeric chaining begins with introduction of an initiator, which provides the free radicals, and possibly a promoter or accelerator. The free radical takes an electron from a monomer, making the monomer into a free radical. The chaining process starts when an adjacent monomer links at the free radical site and then becomes a free radical itself. The process continues until the lack of resin leads the chain to create a termination end.

Key Variables

Methacrylate and polyester polymers are initiated by mixing with a free radical and possible promoters and accelerators and create polymers that grow by chain growth; the reaction rate is controlled by temperature and the amount of initiator. Epoxy and urethane polymers consist of resins and hardeners that cross link to form solid substances; reaction rates are controlled by selection of resins and hardeners and temperature. The required application temperature range is typically 50 to 80°F, because temperature affects the speed of the polymerization. The polymerization reaction is extremely sensitive to temperature. If the temperature is even a few degrees outside the specified temperature range there is a real risk that the polymerization reaction will occur too fast at higher temperatures or not occur at all for lower temperatures.

The contractor must prepare a test patch before the full-scale placement of polymer-based-concrete. A test patch demonstrates that the contractor's polymer resin formulation will harden in an appropriate time period; given that the conditions (like substrate, temperature and mixing proportions) match those under which the polymer concrete will actually be placed. If conditions change, a new test patch should be constructed.

An excessive initiator dosage results in an accelerated reaction and lower average weight polymers. Inadequate initiator or an excess of resin will result in an under-cured, softer material that may never solidify.

A key component of polymer overlay specifications, including methacrylate bridge deck treatments is moisture content; there must be dry conditions and low humidity. The ambient humidity must be below 80% and the aggregate moisture content must be less than 0.5% moisture. Polyester concrete uses synthetic resins rather than mineral-based cementitious materials to bind aggregates. Unlike cementitious materials, the chemical reaction that hardens polymer concrete is not based on the introduction of water to initiate a chemical reaction. As a matter of fact, the exact opposite is true in polymer concrete: the presence of water, even minute quantities, will cause the chemical reaction to fail and the polymer will not form. Thus, the aggregate condition of saturated-surface-dry criteria does not apply to polymer overlays.

Since these polymer concretes are typically applied on a previously hardened concrete, the concrete surface receiving the polyester concrete must be clean. Cleaning is usually specified by mechanical means, typically shotblasting, in order to remove any extraneous surface materials that may be deleterious to a solid bond between the underlying concrete substrate and the polymer concrete.

With a binding agent that is totally different from those used in a typical Portland cement concrete (PCC) bridge construction, a different mindset is needed to appreciate the rigorous and exacting polymer concrete specifications. Successful use depends on the contractor's expertise and quality control practices and rigorous implementation by the inspector of the specification and quality assurance standards, based on a sound understanding of the technical basis of the technology.

Methacrylate

Methacrylate can be used to bind aggregates together to form a polymer concrete, but it is generally used to fill and rebond cracks in concrete elements such as bridge decks and as surface sealants. Because of the rapid strength gain and very low viscosity, methacrylate seeps into cracks by gravity flow and returns cracked concrete to near original strength. Current specifications allow a maximum of 5 minutes from mixing to application, 10 additional minutes for squeegee/brooming materials from grooves, and 10 additional minutes before application of sand; the minimum working time by specification is 25 minutes. Current specifications limit the concrete surface temperature to a range of 50 to 100°F. Depending on the manufacturer, typical working times range from 30 to 60 minutes and application temperatures range from 40 to 130°F. Exercise caution when applying methacrylate as the fumes can present a health hazard and can also be ignited by a spark or open flame.

The earliest methacrylates were based on methyl methacrylate resin monomers that were sequentially mixed with promoters and accelerators and then an initiator (cumene hydroperoxide) to create polymethyl methacrylate polymer chains. The initiator broke the double carbon bond on the methyl methacrylate monomer, turning the monomer into a free radical which then chained to another monomer. In the process of chaining, the newly linked monomer turned into a free radical and the polymer chain continued growing until the supply of monomers was depleted. The idealized reaction is shown below in Figure 6-8.

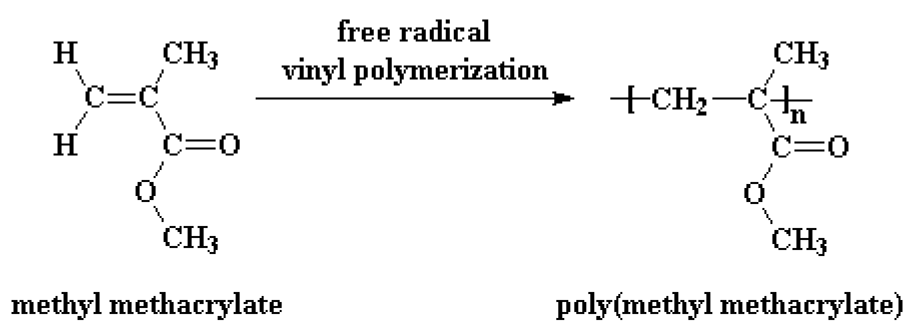

Figure 6-8. Methyl Methacrylate Polymer.

Methyl methacrylate is very viscous. Water has a standard viscosity of 1 centipoise, methyl methacrylates have viscosities as low as 0.6 centipoise. Unfortunately, methyl methacrylate is also highly volatile and burns readily. Urban legends have chronicled several bridge decks that were completely enveloped in fire. Sonny Fereira, Bridge Construction Engineer, promoted the legendary status of methacrylate component volatility as an instructor for the

OSC 1986 Concrete Technology Winter Training Seminar, when he mixed small portions of promoter and initiator directly together in a shielded container with impressive results. Several manufacturers replaced the methyl groups (Figure 6-8, CH_3 atomic weight of 15); with proprietary molecules that had atomic weights approaching 300. The resulting High Molecular Weight Methacrylate, although less viscous than Methyl Methacrylate, was able to meet Caltrans viscosity specification of 25 centipoise. The high weight molecules also reduced volatility and virtually eliminated the possibility of a bridge deck fire.

Polyester Concrete

Polyester-styrene concrete is a versatile construction material used extensively for bridge deck overlays and as an excellent patching material. Physical properties are similar to that of methacrylate except it does not gain strength as rapidly.

Polyester concrete is a solid concrete matrix produced by mixing resin with coarse and fine aggregate to bind the aggregate into a solid concrete mass. Polyester resin hardens into the binding agent through a chemical reaction called polymerization, which is set off by mixing chemical catalysts, called initiators and hardeners, into the resin.

Polyester concrete produces a concrete that is durable, impermeable, and most importantly, achieves high strength in a matter of a few hours. Because of these highly desirable qualities, and also due to their relative high cost in high-quantity applications, the most common application of polyester concrete in Caltrans are thin (3/4" thick) overlays on concrete bridge decks. They have also been used in similar applications such as expansion dam headers.

The raw materials for polyester concrete are dry aggregates, a liquid mixture of polyester resin and styrene monomers and an initiator and promoter (cobalt naphthenate and methyl ethyl ketone peroxide (MEKP)). During the polymerization process the cobalt initiator, causes MEKP to release oxygen molecules which cause the styrene monomers to crosslink with the polyester polymers within a paving mix to form a polymer matrix around the aggregate. The polyester resins molecules shown in Figure 6-9[1] are the first component of the polymerization reaction. Within the polyester resin, carbon atoms are bound together with double covalent bonds, shown in Figure 6-10. The in reactive sites on the polyester molecule shown in Figure 6-9 are identified as “B” in Figure 6-10. The MEKP free radical breaks the double bonds at the reactive sites and forms polymer free radicals. The final component of the polymerization process, the styrene monomer, acts as a coupling agent and forms a bond with the unpaired carbon bond of the polyester resin. As successive bonds form between polymer resins and the styrene monomers, a three-dimensional polymer matrix is formed as shown in Figure 6-11.

1. Systems for Use in Fibre-Reinforced Composite Materials
http://www.azom.com/details.asp?articleid=986 (visited June 9, 2010)

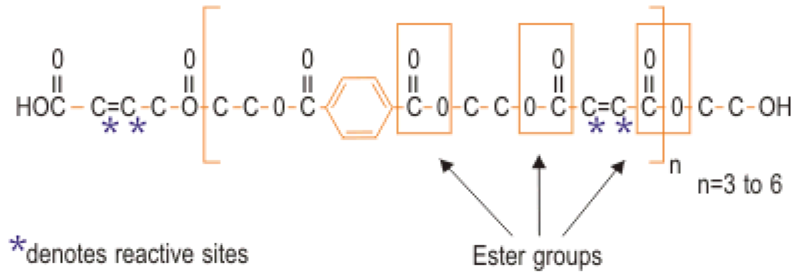

Figure 6-9. Idealized Polyester Resin

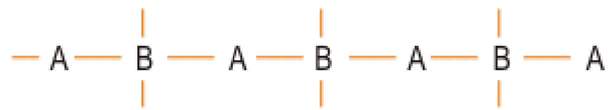

Figure 6-10. Schematic of Polyester Resin (B = Carbon bond).

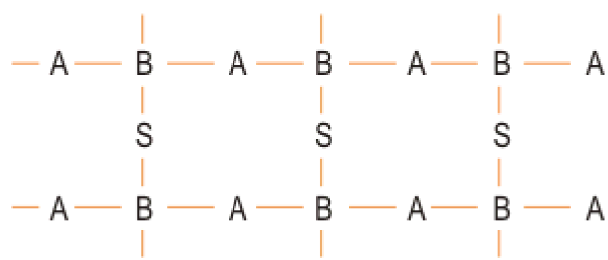

Figure 6-11. Schematic of Cured Polyester Styrene Thermoset.

Urethane

Urethane resins were used in deck rehabilitation work. The deck overlays were performed by alternatively spreading urethane and aggregate until the required section was established. With the development of polyester concrete in the 1990s, urethanes have been replaced with polyester-styrene concrete which is less expensive and otherwise equally suited for the purpose.

As a patching material, urethane resins are easy to work with and bond well to both Portland cement concrete and asphalt concrete. Standard surface preparation is required. Urethane resins can be obtained with a wide range of set times to accommodate usage and temperature requirements, but the resin components must be mixed according to instructions. The amount of initiator cannot be modified as a means of varying the set time. The only variable that can affect set time is surface temperature; a warmer temperature will accelerate set time.

Rapid setting urethanes may set before conventional aggregate mixing is complete. In those instances, a two-step placing procedure was followed. First, coarse aggregate was placed to grade in the void to be filled. Second, after the aggregate is placed, the urethane resin was poured over the aggregate, filling the voids between the aggregate particles and forming a solid mass. Only coarse aggregates are used because urethane resins expand appreciably when mixed with fine materials, such as graded concrete sand.

Epoxy

Epoxies are used extensively in Caltrans as adhesives (for traffic markers, rebar dowels, concrete), crack sealers (vertical wall repairs), and as binders of aggregates to form polymer concrete. Epoxy is one of the oldest and most vesatile, of the polymer patching materials. Epoxies are viscous resins that also harden into a solid mass through the use of chemical catalysts in a polymerization reaction. Many epoxy resins have high viscosity and a putty-like mortar consistency. High viscosity may be an advantage or a disadvantage depending on the location of the patch. However, with epoxy compounds, there are many reaction mechanisms that may occur; consequently, different epoxy compounds may have widely differing properties and characteristics. Epoxy resins provide:

- High strength
- Low shrinkage characteristics
- Good adhesion properties
- Resistance to water, chlorides and sulfates

Epoxies for construction use are two-component systems that are mixed at the site in either a 1:1 or 2:1 ratio. Epoxy is not tolerant to deviation from the specified component ratio, and even minor variations will result in significant strength reduction. Lab tests show a 100 percent decrease in strength resulting from only a 10% variation in the required mixing ratio.

In the past, epoxy was occasionally used for bridge deck overlays. These epoxy overlays were constructed by repeatedly alternating layers of epoxy with layers of aggregate - broadcasted on the previous layer of epoxy - gradually building up the overlay to the specified thickness. Experiments were performed with premixed systems, but a workable system was not developed.

In the 1990s, epoxy injection was specified to seal cracks on newly completed bridge decks; since then methacrylate, which seals smaller cracks than epoxy has become the specified material. Epoxy injection is still used to repair cracked concrete walls. Injection epoxies are formulated specifically for use in pressure injection work. Compared to other epoxy resins, they have good flow characteristics and relatively low viscosity at normal working temperatures. Many can be used in either dry or wet conditions.

Before injecting, the cracks are sealed at the surface with a quick setting epoxy paste which can be applied to the concrete with a small masonry trowel or similar tool. Injection ports, which are usually small sections of plastic tubing 1 to 2 inches in length with a plastic flange at one end, are spaced along the crack at 1 to 2 foot intervals. The flange is set on the concrete and anchored in place with the epoxy paste.

Typical injection equipment consists of a positive displacement pump and gear driven mixing mechanism that can be calibrated to mix the 2 epoxy components at the exact ratio required, which is not necessarily 1:1 or 2:1 as is the case for other epoxies. For convenient operation and easy movement, pump, mixer, and epoxy component storage tanks are usually mounted on the same platform.

After mixing, the epoxy is pumped through a flexible plastic tube to the injection ports. For a normal operation, injection begins at one port and continues until no more epoxy can be injected or until epoxy is observed coming out of one or more adjacent ports. The first port is plugged, and injection resumes at an adjacent port. The operation continues until all ports are injected.

Pumping pressures average about 25 to 30 psi for normal injection work, but the pressure is somewhat dependant on crack width, with smaller cracks requiring higher injection pressures. Pressures of 60 psi or more may be required for very small cracks or cracks that have become contaminated with dirt or other foreign matter.

Filling cracks in concrete by pressure injection of epoxy adhesive is a proven procedure, with many obvious benefits and no disadvantages. When properly performed, epoxy injection will completely restore the structural integrity of a cracked concrete member.

Epoxy Mortar

Since even thin sections of epoxy materials have high strength, epoxy mortar is especially well-suited for use at tapers or at locations requiring a feather edge. Epoxy mortar is made from a mixture of 1 part epoxy to about 4 parts of concrete sand, by volume. Ideally, for a workable mix the mortar should contain slightly more epoxy than is necessary to coat the aggregate particles and fill the voids; however, the amount of epoxy needed for workability varies inversely with the temperature, i.e., as the temperature decreases, more epoxy will be required for the same degree of workability. Experimentation may be necessary to determine the optimum epoxy content. As a rule, it is better to have too much, rather than too little, epoxy in the mix, since lean mixes are difficult to place and the resulting patches may be porous. For filling larger voids, epoxy mortar may be extended by the addition of pea gravel to make an epoxy concrete. As a general guideline, approximately equal portions of sand and pea gravel will produce a workable mixture, provided the amount of epoxy is adjusted to maintain a 1:4 (approximate) ratio of epoxy to total aggregate content. All aggregate used in epoxy mixes must be thoroughly dry; epoxy will not tolerate the presence of even small amounts of moisture in the mix. When mixing the ingredients, the epoxy resin components should be blended and thoroughly mixed to obtain a uniform color before the aggregates are added.

To ensure adequate bond between the epoxy patch and the parent concrete substrate, the concrete surface must be clean and dry when the patch is placed. Sandblasting is the specified cleaning method. For best results, the surface should be sandblasted and then air-blown to remove all sandblast residue and loose aggregate particles from the substrate on the same day that the patches are placed. Epoxy polymers are intolerant of moisture, so the substrate must be thoroughly dry and the exposed surface of the epoxy patch must be protected from moisture contamination until polymerization is complete.

Prior to placing the patch material, the surface to be patched should be primed with a bond coat using the same epoxy that is being used in the mortar or concrete mixture. Typically, epoxy patches are placed by working the material around reinforcing steel and into corners by hand, and then consolidating the material by tamping. For patch thicknesses of more than about 2 inches, the patch should be placed and compacted in 2-inch lifts. Epoxy mortar/concrete patches should be finished with steel, rather than wood, finishing tools. Steel is more effective in sealing the sticky surface that is characteristic of epoxy mixes.

The rate at which epoxy hardens to a solid substance (the cure time) is a function of temperature; the higher the temperature, the shorter the cure time. For most epoxies, the cure time will decrease rapidly as temperatures rise above about 80°F. Conversely, many epoxies will not cure at all at temperatures below about 50°F. Ideally, the best results will be obtained when the parent concrete and the epoxy patch material and atmospheric temperatures are all within a range of 60 to 80°F.

The State-specified epoxy used as a binder in epoxy mortar, ASTM C 881, Type V, Grade 2, Class C, is acceptable for temperatures greater than 60°F. For temperatures between 40°F and 60°F ASTM C 881, Type V, Grade 2, Class B is specified. The Class B epoxy is formulated for cold weather conditions and is a rapid setting epoxy with a very short (10 minutes or less) working window. Mortar made with Class B epoxy may be difficult to mix and place successfully except in small quantities. Class A epoxy for temperatures below 40°F is not permitted under current specifications.

For cold weather placement, it is theoretically possible to accelerate the epoxy cure time by applying heat to the parent concrete and/or to the mix ingredients. This procedure was tried but results were unsatisfactory because of the difficulty in determining in advance the amount of heat necessary to bring the cure time within the ideal range. In most cases too much heat was applied, so that cure times were significantly reduced rather than merely accelerated as was intended. In view of past experience, this practice is not recommended.

Remove Defective Concrete

Before repair work may be started it is necessary to remove the defective material. In a rock pocket area, sufficient material must be removed to expose sound concrete. Depending on the extent and nature of the defect, sandblasting, hand chipping or the use of smaller air-operated chipping hammers will be appropriate. If a jackhammer or similar piece of equipment is used, some chipping with smaller tools may be necessary to clean up the surface because the impact of heavy chipping tools may damage reinforcement and often produces small fractures in the underlying sound concrete as the defective material is removed.

Although visual inspection will reveal most defects, in some cases it will be necessary to check for delamination which is not apparent from an examination of the surface. The most efficient method of locating such defects is to "sound" the concrete by striking the surface with a hammer or by dragging the surface with a noise-producing object such as a chain. The use of a chain is particularly suitable when it is necessary to check a large surface such as a bridge deck; whereas a hammer is useful when checking small areas and vertical surfaces. Delaminated concrete emits a hollow sound when a chain is dragged over its surface or when it is struck with a metal tool. Unsound concrete also includes concrete that encases corroded bar reinforcing steel.

Edge treatment requires special consideration. For voids to be filled with a patch material, either as a permanent repair or as an initial step prior to the construction of a deck overlay, the edge should be saw cut approximately 1 inch deep to avoid a weak feather edge. The best practice is to work with the contractor, agreeing that the saw cut will be performed as a last step. The contractor would remove the initially marked area, the inspector resounds the concrete to verify delaminations have been removed. At that point the final saw cuts are laid out and final concrete removal is completed. To do otherwise could lead to a situation where the contractor applies a square cut, but the delaminated area extends beyond the saw cut and the contractor is required to enlarge the removal area and duplicate cutting efforts.

Chip to the depth necessary to remove all defective concrete; however, the minimum depth of removal depends on the repair procedure to be employed. For “dry-pack” mortar repairs, the minimum depth should be at least 1/2 inch. One-half inch minimum depth is also satisfactory for shotcrete applied without adding aggregate. For other repair procedures, including the use of any material containing aggregate of pea gravel size or larger, a 1-inch minimum depth is recommended.

Current practice prohibits the use of tools or equipment that will remove excessive amounts of sound concrete along with the unsound concrete, or that will damage concrete to remain in place. To meet this restriction on equipment use, it has been common practice to limit the size of equipment to 65-pound jackhammers for removal of large masses of concrete above the level of the reinforcing steel, and to 30-pound jackhammers, and sometimes 15-pound chipping tools, for removal of concrete below the reinforcing.

A clean, sound substrate is required for any concrete repair, so that the minimum amount of concrete to be removed will include all concrete that is structurally unsound, which will be all the concrete above a delamination. Some judgment is required when determining whether to remove additional concrete, beyond the areas of delamination, that is encasing corroded reinforcing steel, but which is otherwise sound. In general, concrete should be removed to expose reinforcing in any case where a crack is visible in the concrete directly over a bar, where the bar is heavily rusted, or where there are heavy rust deposits in the concrete next to the bar. However, the temptation to engage in “rust chasing” along lightly corroded bars should be avoided, as this is not cost effective.

To ensure an adequate repair, all defective material must be removed and the exposed concrete surface must be sound (free of any underlying fractures) and clean. In this context, clean means free of any extraneous material or substance that would impair the bond between the patching material and the concrete substrate.

After removing all unsound concrete, the exposed concrete surface must be cleaned of contaminating substances adhering to a concrete surface. This is normally done by sandblasting, followed by blowing with compressed air to remove sand and dust particles. Sandblasting has the added benefit of removing any fractured aggregate particles remaining in a chipped surface. Wire brushes may be used if the area to be cleaned is small and readily accessible, but wire brushing will not remove fractured particles. Other methods, such as sweeping, washing or air-blowing alone, are less effective and should not be used as the primary cleaning method. Following this operation, the surface should be inspected, and any loose or fractured aggregate particles removed with small air tools or hand chipping to ensure a completely sound substrate prior to placing the replacement material.

A concrete surface that has been chipped, sandblasted, and then air-blown to remove all loose material and surface dust will be sound and sufficiently clean, provided the repair work is done within a reasonable time. If contaminating substances (dust, dirty water, form oil, etc.) are allowed to accumulate on the exposed surface, additional cleaning will be necessary before the repair is made. It is sometimes assumed that a freshly sawn surface is clean since it has been washed by the saw-blade cooling water. Such is not the case, however, since the water actually leaves a residue of fine material on the surface. To ensure bond between a sawed surface and new concrete, a sawed surface should be cleaned by sandblasting.

Regardless of the cleaning method employed, all freshly-cleaned surfaces must be protected until the repair work is performed to prevent contamination by any substance that would reduce or prevent bond; otherwise, additional cleaning will be required. The importance of surface preparation cannot be overemphasized. Remedial work is virtually certain to fail when the patch or other repair is placed upon an unsound or unclean surface.

Place Repair Materials

Removing unsound concrete is the first step in a two-step repair process. The second step is filling the voids that result from the concrete removal with a suitable replacement material. The procedure depends on the extent and depth of concrete removal as discussed in the following sections.

Cure The Repair

Hydraulic cements require moisture for strength development. The chemical transformation that occurs as cement hydrates cannot occur without water. When pozzolanics like silica fume are also included in the repair mix design, curing becomes even more critical as the pozzolanic reaction requires products of the hydration reaction. Curing the repair is the standard method for maintaining moisture while the patch is hardening.

Repairing Minor Construction Defects

Construction defects are usually the result of improper placing methods or techniques. Concrete placement practices are discussed in Chapter 5. When the area to be repaired is relatively large, judgment is necessary to determine whether the resulting void may be patched or whether a structural repair is indicated. As a guideline, a patch will be appropriate when the repair is "cosmetic" only; that is, when the replacement material is merely filling a void in the concrete mass and is not necessary to maintain or ensure the strength and structural integrity of the concrete element. On the other hand, structural repair is necessary when the designed intent of an element is compromised.

Dry-Packing

Dry-packing consists of filling a void with a relatively dry (not fluid) sand/cement mortar and pressing the mortar in place. Dry packing is particularly suited to the repair of rock pockets, since rock pockets usually occur in vertical surfaces or on the bottom of a horizontal member where other repair methods would be difficult to accomplish. When performed properly, dry-packing results in a reconstruction that is equivalent, in strength and durability, to the original concrete.

Surface preparation is as important for dry-packing as for other repair procedures. All defective material must be removed and the exposed surface must be sound and thoroughly clean. Either neat cement or an epoxy adhesive may be used as the bond coat, depending on the size of the void. The proportions for mortar used for dry-packing are 1 part cement to 2 parts sand, measured by volume, with just enough water to bind the materials together and permit placing and compacting.

Shallow voids up to about 1 inch in depth may be filled in a single operation by tamping the material in place with a hardwood stick by hand or by tapping with a hammer. Wooden sticks eliminate the dark, slightly polished surface appearance which often results from the use of metal tools.

Deeper voids should be filled in 1/2 to 1 inch thick layers. The surface of underlying layers should be scratched before the next layer is placed. Successive layers may follow immediately unless plasticity develops. Should this occur, stop work until the mortar sets. The final layer should be slightly over-filled to ensure adequate surface compaction, and then trimmed to the lines of the parent concrete. If the location is visible to the public, the color of the cured patch must match the parent concrete. White cement may be added to trial batches until the correct color is obtained.

For exposed surfaces, at least 3 days of continuous water cure is recommended for dry-pack patches. If the patch is not visible to the public, a membrane cure may be used.

Conventional Concrete Patching

Patching with a sand/cement mortar with aggregate filler may be an appropriate repair procedure, particularly when the void to be filled is relatively deep and covers an extensive area. In most cases pea gravel will be used as the aggregate fille . Such mixes should be proportioned to provide a 1:1 ratio of sand to pea gravel, and should contain enough cement to make a workable mixture. Avoid using too much cement, however, as this will increase shrinkage. The use of a conventional sand/cement mortar without the addition of aggregate is not recommended for repairing concrete because of the undesirable shrinkage characteristics associated with such mixtures. For those locations where a conventional mortar patch would otherwise appear feasible, dry-packing is the preferred method.

Concrete patches in surfaces that require a Class 1 surface finish should be water cured. For other surfaces, either water or the curing compound curing method may be used.

Shotcrete

The term "shotcrete" is given to Portland cement concrete or mortar that is applied pneumatically onto a surface. Shotcrete may be a viable means of reconstructing a vertical or overhead surface when the defective concrete covers an extensive area and the depth of repair is relatively shallow.

Shotcrete for new construction is covered in Section 53 of the Standard Specifications. All specification provisions governing materials, placing and finishing, will apply to shotcrete used to fill voids created by the removal of defective concrete. For repair work, the following requirements will also apply:

- All defective concrete shall be removed as previously described, except that the edges should be tapered inward on a 1:1 slope, rather than cut perpendicularly, to avoid inclusion of rebound material. If pea gravel will be used in shotcrete applied by the wet-mix process, the minimum depth of removal shall be 1 inch.

- The exposed concrete surface shall be cleaned by sandblasting followed by blowing with compressed air.

- The surface to receive the shotcrete should be visibly damp, but significant amounts of free water should not be present. A bond coat is not required.

- Shotcrete should be applied slightly full and the finish surface trimmed with an appropriate float or straight edge to the lines of the parent concrete.

Shotcrete surfaces are cured and protected in accordance with applicable specification requirements for new construction.

Commercial Patching Materials

There are a number of proprietary products marketed commercially as concrete patching materials. However, many of these products have one or more undesirable characteristics (such as low bond strength, excessive rigidity, high chloride or sulfate content, or high shrinkage) and thus are not suitable for patching rock pockets. Some generic materials, such as magnesium-phosphate concretes and high alumina cement products, are marketed under various brand names. These products have a long history of satisfactory use, and may provide a satisfactory repair under certain conditions. However, these products are self-leveling, and therefore will not be appropriate for vertical or overhead surfaces in most cases.

Rapid set patching materials are used in the bridge deck rehabilitation section. While these products are useful when repairing deteriorated concrete, they are not recommended for rock pocket repair in new construction, except under very unusual circumstances.

Epoxy Mortar Patches

Because it is relatively expensive, difficult to work with, and darker in appearance than concrete, epoxy mortar is not widely used as a concrete patching material in new construction. However, the use of an epoxy mortar or concrete may be appropriate at locations where a highly viscous rapid setting material is required. Epoxy may be particularly suitable for patching small voids in vertical and overhead surfaces where appearance is not a factor.

Repair Testing

Following the cure period, patches should be visually inspected and, if deemed necessary by the engineer, tested for structural adequacy. An impact hammer (Schmidt hammer) may be used to determine the approximate strength of patches made by dry packing or with a conventional Portland cement concrete patching material. Tapping the patch with a rock hammer will reveal areas that have failed to bond properly (unbonded areas will produce a hollow sound) but this procedure does not give any strength indication. Because epoxy is a resilient material, epoxy mortar patches cannot be tested with impact tools. Epoxy patches should be examined visually for evidence of obvious defects, such as porosity and/or lack of sufficient resin at the surface.

Large Defect Repair - Concrete Replacement

When the area of defective concrete will result in large voids in primary load carrying members such as slabs and girder stems, it may be necessary to recast the member using replacement concrete of the same mix design as the original concrete to ensure integrity of the structural element. In general, a structural repair (rather than a patch) should be required when the integrity of the member is threatened by the defect. This will include voids that extend entirely through a member, voids where the main reinforcing steel is exposed to the extent that bond is jeopardized, or any void which, if unfilled, would reduce or impair the load carrying ability of the member in any way. Obviously, this is a subjective determination, but if there is any doubt, the engineer should require a structural repair.

Preparation

Detection and removal of defective concrete, surface preparation and application of a bond coat as discussed in preceding sections are applicable to structural repair. When a structural repair is indicated, it will be necessary to remove the defective concrete to a definite line, and this may require removal of concrete that is otherwise satisfactory. In general, removal limits should be straight and square with the structural element being repaired. If, after identifying the limits of the defective concrete, less than about 18 inches of sound concrete remains between the area to be repaired and the top or end of a wall or girder, this sound concrete should be removed as well.

Main bar reinforcing steel should not remain partially embedded in concrete; enough concrete should be removed to provide at least 1 inch clear around all main bars.

Care must be taken to ensure that no void area remains between the top of any concrete placed as a structural repair and the undisturbed sound concrete above. Methods to prevent inclusion of voids listed below:

- Chipping the top surface of the void area on a 3:1 upward slope toward the concrete placing side.
- Form a “chimney” chute above the void to be filled with replacement concrete. Place concrete through the chute to provide a head on the fluid concrete.

Placing

In most cases the same concrete mix will be used for the replacement concrete as used for the original concrete. However, to facilitate placing and further ensure that the entire void area is filled, consideration may be given to the use of a water reducing admixture to increase the fluidity and workability of the replacement concrete, even though a water reducer was not used in the original mix.

All specification requirements for furnishing, placing, curing and protecting new structure concrete will also apply to replacement concrete used in a structural repair.

If it is necessary to make a structural repair in concrete that is to be prestressed, keep in mind that the replacement concrete must attain the specified compressive strength before prestressing forces are applied, and this could delay the stressing operation.

Occasionally, concrete must be removed for aesthetic reasons, such as might be the case after a form failure. The procedures previously discussed for repair of rock pockets will also apply to removal and replacement of aesthetically-defective concrete as well.

Repairing Defects In New Bridge Decks

Defects occurring in new bridge decks fall into two general categories:

- Those which impair the riding quality of the finished deck surface, but which have no structural significance.
- Those which, if not repaired, could affect the structural integrity of the bridge itself.

Defects in the first category include deck areas which for some reason were improperly finished (too high, too low, or too rough) where the condition is serious enough to require remedial work other than grinding. Repair of such defects may require the removal of a substantial portion of the finished deck surface.

The second category includes severe cracking and surface defects like rock pockets. (Cracking that exceeds the crack intensity threshold as defined in the specifications is repaired with methacrylate treatment.) In general, repair procedures for new bridge deck repairs are similar to those described earlier in this chapter of the Concrete Repair Process section. An additional requirement for bridge decks is that the surface must be finished in accordance with current bridge deck finishing specifications

Filling Shallow Voids

When the voids in the deck are scattered, or are shallow and do not extend below the level of the reinforcing steel, current practice calls for filling of the voids with overlay material, regardless of the type of overlay to be constructed. This includes polymer overlay materials. In such cases, mixing and placing of the replacement material will be done concurrently with the overlay, following the methods and procedures specified for overlay construction. The edge should be chipped inward on a 1:1 slope to prevent the formation of pockets of entrapped air at the edge as the overlay is placed.

Filling Deep Voids

Where concrete has been removed below the level of the deck reinforcing over a large area of the deck to expose substantial amounts of reinforcing steel, the voids will be filled with concrete of the same quality as the original construction. If the rehabilitation strategy includes a Portland cement concrete overlay, the voids may be filled when the overlay is placed, using the overlay material. For a polymer concrete overlay, the deck will be reconstructed with Portland cement concrete as a separate operation before the overlay is placed.

When a concrete overlay is to be constructed, determining whether to reconstruct the deck with normal Portland cement concrete or with the concrete overlay material is usually a question of economics, since either method is satisfactory from a structural standpoint. Portland cement concrete is less expensive, and may be easier to handle, than the overlay material, and therefore will be the most economical material to use. Some of the saving is illusory, however, because of the higher labor costs associated with a two-step operation. When the deck section is restored prior to placing an overlay, the usual practice is to use a concrete mix design conforming to applicable specifications for structure concrete. Even though the deck will be covered with an overlay, air-entrainment is advised.

When normal Portland cement concrete is used, an epoxy bond coat will be specified. The epoxy adhesive may be applied by brush or by spray equipment. Application should be uniform over the concrete surface, but it is not necessary to completely coat the exposed bar reinforcing steel.

When the deck section is restored with a concrete overlay material, bonding requirements for the replacement material will be the same as specified for the overlay itself.

Bonding Agents

Of the many available bonding agents, only 2 are acceptable for repairing concrete structures. They are neat cement paste and 2-component epoxy adhesives specifically formulated for bonding new concrete to old concrete. Single-component repair products, even though they may be marketed as epoxy products, are not effective and are not to be used as bonding agents for structure concrete repair work.

Neat cement paste (a mixture of Portland cement and water) may be used to bond patches up to about 1/2 square foot in area and 3 inches in depth, provided no reinforcing steel is exposed. When neat cement is used as a bonding agent, the concrete to be repaired should be kept wet for several hours before patching, but the surface should be dry when the neat cement paste is applied; this is similar to keeping aggregate in the saturated surface dry condition.

Larger patches (or any patch where reinforcing steel is exposed) require a two-component epoxy-bonding agent. The complete removal of all foreign materials from the surface to be patched is particularly important when an epoxy adhesive is used as the bonding agent. Even a thin layer of dust will prevent bonding.

The composition, physical characteristics and directions for use of State specification epoxy adhesives for bonding new and old concrete are found in Section 95 of the Standard Specifications. Epoxy adhesives should be furnished and applied in conformance with the manufacturer's recommendations and, at the engineer's discretion, applicable provisions in Section 95 of the Standard Specifications as well.

To ensure satisfactory results, the concrete surface should be thoroughly dry when the epoxy adhesive is applied, and ambient and surface temperatures should be within the range recommended by the manufacturer. For best results, the contact surface of the concrete should be completely covered with a thin coat of the epoxy adhesive, and the epoxy should be worked onto the surface with stiff brushes. If spray equipment is used, care must be taken to prevent the epoxy from ponding in depressed areas of the concrete. The patching material must be placed before the epoxy adhesive begins to set and within the time limit specified by the manufacturer for the type of epoxy used.

Small Area Repair (< Two Inches Deep)

These areas may be filled with a proprietary concrete patching material that is suitable for deck rehabilitation work, or with epoxy mortar.

For an epoxy mortar repair, the area to be patched should be chipped down square around the edges to a minimum depth of 1/2 inch, and then air-blown clean. If the epoxy patch is placed promptly, no further cleaning will be required. However, if patching is delayed and the exposed surface becomes contaminated, sand blasting will be necessary to ensure an adequate bond. The epoxy patch should be placed in accordance with applicable specifications.

Certain proprietary patching materials, that would be unacceptable for repairing rock pockets in new concrete, may give satisfactory results when used to fill very small voids in bridge decks. This is the case because appearance is not a factor and, for the small volume of material used, shrinkage is not a problem. In general, any proprietary product that is satisfactory for use in bridge deck rehabilitation work can be used for filling small voids in new decks as well.

Large Area Repair (> Two Inches Deep)

When the finished deck surface is too high or too low, the concrete in the affected area must be removed to a minimum depth of approximately 3/4 inch to ensure adequate thickness for a structural patch. These areas are to be restored with a combination of Portland cement mortar and aggregate filler or with Portland cement concrete, as appropriate for the size and thickness of the defect.

Concrete removal is usually accomplished with hand held air-operated impact tools; however, if the defective area covers a large portion of the deck surface, the use of power scarifying equipment may be appropriate. Concrete removal using any type of impact tool or equipment will leave fractured concrete fragments that are still adhering to the surface. To ensure a sound, fracture-free substrate, the area to be patched must be thoroughly sandblasted. After sandblasting, the surface should be air-blown to remove dust and loose particles remaining from the chipping and sandblasting operations. Replacement concrete must be bonded with an epoxy adhesive.

For thin sections (i.e., patches up to 2 inches in thickness) the concrete proportions shall be 658 pounds per cubic yard, 45 to 50% pea gravel aggregate, concrete sand, and just enough water to produce a workable mixture. The pea gravel aggregate should pass a 1/2 inch sieve with 95% being retained on the No. 16 sieve. Sand should meet the specification requirements for fine aggregate. For sections over about 2 inches in thickness, a regular bridge concrete mix may be used. With the exception of the special mix design for thin sections, all contract requirements applicable to original construction will also apply to the concrete used to restore the bridge deck surface.

If air-entrained concrete was specified for the original concrete, an air-entraining admixture must be used in the replacement concrete. Other admixtures may be used if they are considered desirable.

Immediately after depositing, the concrete should be consolidated, and then struck off to the required grade and textured to match the adjoining deck surface. Curing shall conform to applicable contract provisions. Patched surfaces must meet profile, texture and cracking requirements included in the bridge deck finishing specification

Filling Cracks by Epoxy Injection

The 1999 Standard Specifications were the last specifications that included epoxy injection in the bridge deck finishing specification (Section 51). The specification provides that when the “surface crack intensity” exceeds the specified threshold value, all cracks within the affected area “...shall be filled with pressure injected epoxy.” The intent of this specification is to restore the structural integrity of new bridge decks in those cases where excessive cracking has occurred during construction. Pressure injection with an epoxy adhesive was specified because it was the only commercially feasible repair procedure that rebonded cracked concrete. The current specified treatment is methacrylate.

Bridge Deck Rehabilitation

Caltrans structural elements such as concrete bridge decks exist in severe use conditions under potentially harsh environmental conditions. Our structures are subject to continuous surface wear and high live-load stresses, including both impact and fatigue. For the most part, they are exposed to alternate wetting and drying and, in many locations, alternate cycles of freezing and thawing and severe temperature differentials as well. Bridge decks contain a congestion of reinforcing steel, making the use of highly workable concrete essential. Because of finishing requirements and bleeding, the worst concrete in the deck is at the surface. In view of these influencing factors, it is not surprising that concrete bridge decks, in general, experience a relatively more rapid deterioration rate, and require a greater maintenance effort, than other elements of civil engineering structures.

Depending on such variable factors as traffic, degree of exposure to adverse environmental conditions, and quality of initial construction, bridge decks on structures completed during the early years of the freeway building era in California (1950s and 1960s) are beginning to deteriorate, and many will soon be nearing the end of their useful service life. As deck deterioration accelerates, the need to maintain a smooth riding surface and/or the required load-carrying capacity will become an increasingly important highway planning consideration.

Additionally, in recent years in many parts of the country, including California, problems associated with deck deterioration have been exacerbated by corrosion of embedded bar reinforcing steel occurring as the consequence of the deliberate application of deicing salt. Corrosion of embedded reinforcing steel causes undersurface delamination at the level of the top mat of steel, and eventually the deck itself. Although the use of epoxy coated bar reinforcing steel will prevent corrosion damage, California has many hundreds of structures in freeze-thaw areas that were built before the advent of epoxy coated reinforcing steel. All of these structures have chloride-contaminated decks, and all have some degree of corrosion damage. Most have experienced at least some deterioration.

To mitigate the adverse effects of corrosion damage, California began a deck rehabilitation program in the early 1970s. Today, deck deterioration remains a problem, however, because many of the repairs done during the 1970s and early 1980s were intended only to preserve the deck riding surface; they were not a “permanent” repair.

Accelerating bridge deck deterioration statewide, together with the need to restore the structural integrity of the many remaining salt-contaminated bridge decks in freeze-thaw areas, will require an on-going deck rehabilitation effort for the foreseeable future. Accordingly, bridge deck rehabilitation will remain an important part of the Department’s construction program for many years to come.

The factors contributing to deck deterioration have been previously discussed. In summary, they are:

- Cracking
- Scaling
- Spalling
- Delamination
- Abrasion or surface wear

As a rehabilitation strategy, the objective of a deck overlay is to extend the service life of an existing bridge deck at a significantly lower cost than total deck replacement. Thus the ideal overlay will completely restore the bridge deck riding surface with a strong, durable material, provide sufficient additional protection to the existing deck reinforcing steel so that further corrosion will not occur, and be constructed of materials that are relatively easy to produce and install.

Over the years, many overlay materials and construction techniques have been employed in an effort to extend the service life of a bridge deck. These include asphalt concrete in combination with a protective deck membrane; conventional Portland cement concrete; low-slump concrete; polymer-modified concrete; superplasticized concrete; and polymer products currently in favor like methacrylate and polyester concrete. Asphalt concrete overlays have been more or less successful in restoring the deck-riding surface, although the overlays have a relatively short service life. Concrete, in general, will provide a durable riding surface, but many concrete systems are not totally impervious and thus cannot prevent future corrosion of the deck reinforcing. Furthermore, most concrete products that have the strength, durability and impermeability characteristics that are essential to the ideal overlay are expensive to produce and difficult to work with in the field

Even so, and in spite of their high cost and inherent construction difficulties, concrete overlays using specialized materials and placement techniques have been considered a viable rehabilitation procedure by many states and the FHWA. Concrete overlays have not been widely used in California, because during the early years of the deck rehabilitation program, the concrete overlays were viewed as experimental, primarily because the two specialized concrete overlay systems in general use throughout the 1970s (the so-called "low slump" system and the latex-modified concrete system) have many construction drawbacks. Today, the preferred treatment is with methacrylate overlays to "heal and seal" surface cracks and polyester concrete overlays that provide bonded wearing surfaces and also serve to correct for variations in the bridge profile.

Surface Preparation

The actual depth of removal depends on the condition of the existing concrete. Where the deck is in generally good condition except for isolated areas of unsound concrete, removal to a depth of about 1/8 inch will be sufficient to remove surface contaminants and weathered aggregate particles. Deeper removal (up to 1/2 inch or more) may be specified where scaling has occurred or where design considerations require removal to a greater depth. For the typical deck rehabilitation project, the deck preparation item will include removal of a specified depth of concrete over the entire deck area, and cleaning of the area to receive the overlay.

In most cases when the rehabilitation strategy requires only a thorough cleaning of the deck surface, sandblasting may be specified. Sandblasting is a cost-effective means of removing surface contaminants and weathered aggregate particles, and will assure a clean, sound substrate for the overlay. However, sandblasting has several drawbacks. First, there is a large volume of residue that must be removed and disposed of. Second, it is often necessary to provide a protective barrier between the area being sandblasted and an adjacent lane open to public traffic. Finally, while the dust generated by sandblasting a bridge deck is not considered hazardous, under certain environmental conditions it can be an objectionable atmospheric contaminant, and this may cause a problem in urban areas.

To mitigate some of the problems associated with sandblasting, current overlay contracts, primarily those involving construction of polymer concrete overlays, have specified the use of shotblasting equipment for the deck preparation work when only a minimal depth of concrete removal was necessary. Shot blasters are self-contained units that use small-diameter (1/16 of an inch) steel shot to abrade the concrete surface. The shot is continuously recycled, and the units include a dust collector to capture the concrete residue.

Shotblasting is the most effective means of assuring a clean, sound concrete surface. Shotblasting equipment was originally developed to clean concrete floors prior to applying a protective coating; these were small units powered by electric motors and intended primarily for indoor use. Later, gas-driven units became available and as technology improved, larger high-production shot blasters suitable for construction use were developed. These large units are now widely available and the older, smaller units are not cost-effective for bridge work. If shotblasting is specified, it is essential to use the appropriate equipment; otherwise, the intended results will not be obtained.

When more than surface cleaning is required (i.e., when it is necessary to remove more than about 1/8 inch of concrete) the means by which the concrete is removed will, in most cases, be left to the contractor doing the work. Although various types of abrasive impact and/or cutting equipment have been used in the past, today most contractors use milling equipment (roto-milling machine or equivalent) for this purpose. This equipment is effi ient and cost-effective, but it leaves a micro-fractured surface that may adversely affect overlay bond unless the fractured particles are removed before the overlay is placed. For this reason, the specifications will require sandblasting (or shotblasting) of the entire deck area before placing the overlay, but after completing all other preparatory work.

To take advantage of developing technology in the concrete demolition field, the use of hydro-blasting demolition equipment utilizing high pressure water blasting jets has been specified for deck preparation work on selected projects, since the early1980s. While early efforts were only marginally successful, in recent years hydro-blasting equipment has become increasingly more sophisticated, and units are now available that are capable of removing concrete to a preselected depth with little difficulty and without damaging the concrete remaining in place. When used for surface preparation, water blasting has the added benefit of removing unsound concrete at the same time, at little increase in total cost.

Hydro-blasting equipment that is suitable for concrete demolition work may not be suitable for deck preparation. This is the case because hydro-demolition equipment is designed to remove large pieces of concrete by fracturing and spalling the concrete mass. Current specifications require demolition equipment with rotating, or oscillating heads and a nozzle pressure of 30,000 psi. The effectiveness of 15,000 psi equipment is currently being evaluated and may be optional by change order in the future. When the proper equipment is used, a water blasted surface will expose sound coarse aggregate particles with excellent bonding characteristics.

Because of the normal distribution of coarse aggregate within a concrete mass, and because water blasting is not abrasive and therefore does not cut or fracture individual aggregate particles, a water blasted surface will not be a smooth plane, but will appear relatively rough and irregular. For compliance with a depth-of-removal specification, results will be satisfactory if the measured depth to the mortar line is the specified depth, plus or minus 1/2 the diameter of the largest coarse aggregate particles.

Water blasting has some disadvantages, notably the need for a water supply and the need to control surface runoff. Containing and removing surface runoff is an environmental concern and must receive adequate attention. Additionally, it is necessary to thoroughly wash down the water-blasted concrete before the residue dries and adheres to the exposed surface; otherwise the surface must be sandblasted before the overlay is placed. Water blasting is also more expensive than roto-milling. However, from a quality control standpoint, the superiority of a water blasted surface outweighs the disadvantages.

Low-Slump Portland Concrete Overlays

Low slump concrete was originally developed as a repair material for Portland cement concrete pavement, and during the 1960s was widely used for this purpose by many mid-western states. Although low slump concrete was first used as a deck overlay material in Kansas in 1969, its use was actually pioneered in Iowa during the early 1970s, and the system is often referred to colloquially as the "Iowa" method of overlay construction.

In the Iowa method, a low water-cement ratio concrete is placed on a prepared deck surface. The typical application involves the following steps:

- Remove surface concrete to a depth of approximately 1/4 inch
- Remove all remaining unsound concrete
- Sandblast the concrete and any exposed reinforcing steel
- Apply a mortar-bonding agent to a dry concrete surface
- Place the low-slump overlay. Nominal overlay thickness is 2 to 2 1/2 inches
- Cure the concrete using burlap and water

Most authorities recommended a mix design with cement content between 750-850 pounds per cubic yard and a water-cement ratio between 0.32 and 0.34. To achieve this, a water-reducing admixture was used.

The mortar-bonding agent consisted of approximately equal parts of fine sand and cement mixed with sufficient water to form slurry of creamy consistency. The bond coat was applied to the deck in a thin, even coat using stiff brooms. Typically, the neat cement (mortar) was mixed in a mortar mixer and delivered to the point of application by pumping through a flexible hose. The rate of delivery was controlled to prevent drying prior to applying the overlay material. A number of mixing methods were used, ranging from mobile, continuous mixers to jobsite paving mixers to transit mixers. All were capable of producing satisfactory results, although transit mixers required larger admixture dosages than the other mixers.

Because the relatively stiff concrete mix could not be pumped, the mixed concrete was delivered to the point of placement using buggies, or by direct discharge from a transit mixer. Conventional deck finishing equipment was too light to handle the stiff, low-slump concrete; consequently, a finishing machine specifically designed for use with low-slump concrete was used. Such machines, which were much heavier than conventional drum finishers, were equipped with two transverse screeds, one of which was a vibrating screed. In their normal configuration, the finishing width was limited to about 13 feet. Although the units could be extended, the practical limit was about 3 feet because of the weight and rigidity of the equipment.

Vibrators were used ahead of the finishing machine to level and consolidate the overlay material, and to bring grout to the surface to facilitate finishing. Even so, some hand finishing was required behind the finishing machine to smooth irregularities and to seal surface cracks. Surface texturing was usually accomplished with a tined wire-grooving tool.

The overlay surface was covered with wet Burlene® as soon as possible after texturing. Curing compound was not used because the thin overlay is susceptible to both plastic and drying shrinkage, particularly when placed during warm weather. Also, because of the initial low water content, it was necessary to apply water to the surface to ensure that sufficient water will remain in the concrete to hydrate the cement.

Low water-cement ratio overlays used the relatively inexpensive materials normally associated with Portland cement concrete construction. However, the work is labor intensive and required specialized equipment for placing and finishing, and sometimes for mixing as well. Success was greatly dependent on the ability and experience of the construction crew, and on rigid adherence to specification requirements. The few low-slump overlays constructed in California have generally performed well and have been relatively maintenance free. This may be attributed to the fact that the projects were quasi-experimental in nature and therefore were the subject of considerably more engineering attention and involvement than is typically the case. California concluded that low-slump overlay installation required more expertise than most contractors have, and therefore continued success would necessitate a higher degree of State involvement than was considered appropriate. Accordingly, the construction of low-slump overlays was discontinued. Other states have apparently reached this same conclusion, as the use of the low-slump system is far less prevalent today than it was in the past.

Latex-Modified Concrete Overlays

Latex-modified concrete is conventional Portland cement concrete with the addition of approximately 15% latex solids. The latex, which is a polymeric material, is added as an emulsion during the mixing cycle. Polymeric latex is a colloidal suspension of synthetic rubber particles in water. The particles are stabilized to prevent coagulation, and antifoaming agents are added to prevent excessive air entrapment during mixing. The water in the emulsion hydrates the cement, and the latex polymer provides additional binding due to the adhesive and cohesive properties of the latex. This results in a superior concrete having very good strength and durability properties, and low permeability.

In theory, the structural properties of latex modified concrete (LMC) make it an ideal material for bridge deck overlay construction. In practice, however, the properties of LMC overlays vary widely depending on the type and amount of the latex used, the type of cement and aggregate used, the cement factor and the water-cement ratio. Additionally, LMC is sensitive to temperature variation, and is difficult to place and finish under all but the most benign environmental conditions. For these reasons, the desirable properties of LMC are not always obtained under field conditions.

Placement procedures for LMC overlays differ in many respects from the procedure used for low-slump concrete overlays. The principal differences are:

- The deck must be kept wet for at least 1 hour before placing the overlay
- The mixing equipment must have a means of storing and dispensing the latex emulsion into the mixture
- The LMC mix is not air-entrained
- A separate bonding agent is not used
- The nominal overlay thickness is usually 1 to 2 inches
- The LMC mixture has a high slump
- Conventional deck finishing equipment may be used
- A proper cure requires a combination of water curing and controlled air drying

Almost without exception, latex-modified concrete is produced in mobile, continuous mixers that are fitted with an additional storage tank for the latex emulsion. While in storage, the emulsion temperature must be kept between 45°F and 85°F. This may present some difficulty, particularly during the summer months.

The bond coat consists of the mortar fraction of the latex-modified concrete, which is worked onto the deck using stiff brooms. If additional mortar is needed, it is easily obtained directly from the mobile mixer by cutting off the aggregate flow for a short period of time.

Mix proportions differ from low-slump concrete in that relatively more fine aggregate is used and the cement content seldom exceeds 650 pounds per cubic yard. The final mix design is selected after trial batches using various cement and aggregate combinations. This procedure is necessary because the latex is sensitive to, and may react differently, when different brands of cement and different aggregate sources are used.

The LMC mixture is highly fluid and usual slump (or penetration) requirements do not apply. Although the mixture is fluid, the water-cement ratio is relatively low, ranging from about 0.35 to a maximum of 0.40. The fluid nature of LMC makes for a workable, nearly self-leveling mixture, but this property also makes it difficult (and often nearly impossible) to maintain grade and thickness control on decks having even a moderate (say 6%) cross fall.

No special finishing procedures are required, and conventional finishing equipment can be used. For curing, wet Burlene® is applied as soon as possible after finishing, and left in place (and kept continuously moist) for 24 hours. This initial period of wet cure is necessary for the hydration of the cement and to prevent shrinkage cracking. After 24 hours the Burlene® is removed, and the overlay is permitted to air dry for the remainder of the curing period. Air drying is necessary to permit the latex to dry out, which then enables the latex particles to coalesce and form a continuous film within the overlay. The latex film, which fills cracks and pores within the mortar matrix, gives LMC its structural properties of high strength and low permeability. However, the film forming characteristics of latex are temperature sensitive, and develop very slowly at temperatures below 55°F. Curing at a lower temperature will require an extended curing period. (Note that all latex polymer manufacturers recommend air drying for at least 72 hours, and additional drying time during periods of cold weather.) Film formation ceases altogether at about 45°F.

The Department's experience with latex-modified concrete, although limited, suggests that a properly constructed LMC overlay will provide adequate deck protection and a relatively maintenance-free riding surface during a long (probably 25 years or more) service life. However, LMC overlays have not been considered, in California at least, to be a practical deck rehabilitation strategy, for several reasons. To begin with, the LMC mix design is dependent on trial batches which are evaluated by the latex manufacturer. Trial batches are needed to verify compatibility of the latex polymer with the cement and aggregate to be used in the LMC, which normally will be the cement and aggregate being used for other concrete on the project. However, there have been cases when it was necessary to change the brand of cement or the aggregate source, or both, to assure that the materials were compatible.

The time required for trial batch evaluation and mix design, which can take several months, must be factored into the project schedule. Furthermore, LMC overlays can be placed only during a very narrow construction window, since the material cannot tolerate hot, cold, dry, wet or windy weather conditions. After placement, it is necessary to prevent moisture contamination during the period specified for air drying. As previously noted, LMC is quite fluid when first mixed, and therefore is impractical for overlay placement on decks on a steep grade or decks with highly super-elevated cross slopes. These factors, along with a relatively high initial cost, suggest that LMC, based on realistic considerations, is not a practical material for deck-overlay construction.

Conventional Portland Cement Concrete Overlays

During the late 1960s and early 1970s, California's construction strategy for new structures located in frost areas included deck protection consisting of a preformed or liquid membrane covered with a 3-inch asphalt concrete overlay. Where the approach pavement was concrete, the decks were depressed 3 inches to make the new deck surface (after placing the Asphalt Concrete (AC) overlay) match the pavement profile. When the membrane/AC systems reached the end of their service life (usually 10 to 15 years after installation), the repair strategy for those structures built with depressed decks included the use of conventional

Portland cement concrete to fill the space originally occupied by the AC overlay. Typically, the concrete used is a conventional air-entrained bridge mix with a cement content of 650 pounds per cubic yard, with minor modifications. The specified mix design provides for 50 to 55% fine aggregate; the maximum size for coarse aggregate is 1/2 inch; the water-cement ratio is limited and a water-reducing admixture is required. (Initially, the water-cement ratio was limited to 0.36, but this resulted in a stiffer mix than was considered desirable. More recent projects have specified a water-cement ratio of 0.40 maximum.) Nominal reinforcing (epoxy-coated #3 bars at 12 centers in both directions) is provided, and the concrete is placed on an epoxy-bond coat. This successful rehabilitation technique for structures with depressed decks led to the use of conventional concrete overlays at other locations as well.

Portland cement concrete has many advantages for overlay construction. It is relatively inexpensive when compared to other rigid overlay materials, and it can be produced, delivered, placed and finished using conventional equipment and procedures. And while rehabilitation technology now favors thin polymer concrete overlays, other factors being equal, Portland cement concrete still is an appropriate choice where the deck configuration or other design considerations dictate the use of a thicker overlay than is economical with the polymer systems.

It is interesting to note that the first deck rehabilitation project in California, in 1968, included a 2-inch nominal concrete overlay placed on an epoxy-bond coat. The overlay concrete was a 7-sack air-entrained mix with pea gravel aggregate, and no special quality control measures were employed. Fractures below the overlay, which were either undetected when the overlay was placed or developed subsequently, were filled by epoxy injection in 1978. No other maintenance has been necessary, the deck-riding surface remains in good condition and the overlay shows no signs of failure, all of which indicates that overlay construction using conventional Portland cement concrete may be a more viable rehabilitation strategy than most authorities once believed.

Methacrylate Overlays

High molecular weight methacrylate, a low viscosity polymer resin, is a healer/sealer. Methacrylate flows by gravity into cracks less than 0.02 inches wide to seal cracks, on Caltrans bridge decks and as a prime coat for polyester concrete overlays. As a healer, methacrylate fills cracks from the bottom up, preventing water from entering cracked concrete. After setting, cracked concrete will be restored to 75-90% of original strength. Methacrylate is not a wearing surface; it is quickly abraded by vehicle traffic. A recent METS program checked cored samples of methacrylate work; one of the cores is shown in Figure 6-12. The core on the left is shown under ultraviolet light, with the lighter blue color being the area penetrated by methacrylate. The core on the right of Figure 6-12 is the same core under regular lighting. Under regular lighting, it is difficult to see more than 4 inches of crack depth, but under ultraviolet, it can be seen that the crack extended at least 5 inches deep.

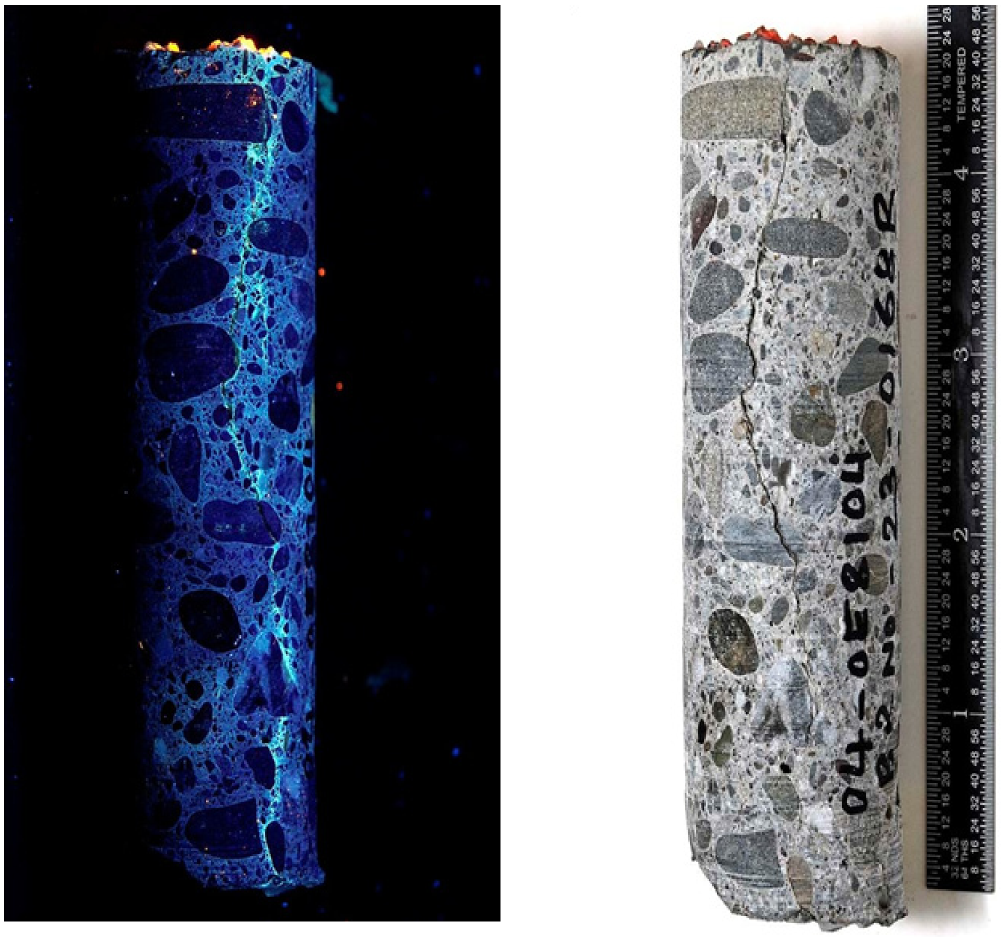

Figure 6-12. Methacrylate Treated Bridge Deck Cores.

Methacrylate lots are tested by METS; test results are available through the Materials Engineer. Methacrylate resin will not start the polymerization process without an initiator like cumene hydroperoxide or methyl ethyl ketone peroxide and a promoter like cobalt naphthenate or cobalt octoate. Initiator and promoter should only be combined in resin; mixing the two together without resin will result in a fire or explosion. In an example of how not to transport hazardous materials, Figure 6-13 includes a 250 gallon container of methacrylate, 4-gallon boxes of cumyl-hydroperoxide and 5-gallon buckets of promoter. In the event of an accident, the materials in the truck bed could mix with explosive results.

As part of the placing plan required by specifications, the contractor should explain how the materials will be safely transported.

Figure 6-13. Methacrylate Components.

Prior to bead blasting the bridge deck, the deck is sounded with a chain to identify delaminated concrete, which is removed and replaced with a rapid setting patch. When patching is complete, the entire deck is bead blasted to remove approximately 1/8 inch of material and then swept clean of dust. Note, if magnesium phosphate patching materials are used, there must be a 72-hour wait before placing an overlay.

Specifications require a test panel prior to actual placement to verify under field operating conditions, that the contractor has control of the process, that the work can be finished within a work window and that the surface will meet skid resistance requirements. From a contract administration perspective, the test panel should not be waived because it could then be argued that risk transfers to the structure representative. If field conditions such as materials in use, crew leadership, or time of operation are changed by the contractor, another test panel would be warranted. As part of the test panel installation, a certified industrial hygienist will perform air quality testing. The air quality report must be accepted by the structure representative before the contractor can proceed with the contractual deck overlay.

Respirators may be required when applying methacrylate, all personnel should be fitted with and possess respirators while on the job site. Avoid physical contact with polymeric materials. Refer to the material safety data sheets for decontamination procedures. The work area used for mixing methacrylate should include protective measures to contain material spills. If possible, the mixing should be done off the traveled way.

Applied materials should be pushed forward with a squeegee and the area swept with a broom to minimize material in grooved surfaces. The average coverage is about 1 gallon for an area of about 90 square feet. Fresh methacrylate is slippery, like walking on ice; if you must walk on the surface, tread carefully. Before the methacrylate sets, sand must be spread over the surface to increase friction for vehicles. After set, loose sand should be swept away.

Some methacrylate resins include wax to limit odors and control cure time; this wax can lead to "tracking," where vehicles leave methacrylate residue on untreated roadway surfaces. In situations where the wax might cause tracking or reduce friction, diatomaceous earth has been spread to absorb wax and accelerate set time.

Polyester Concrete Overlays

A relatively new construction material, polyester concrete, was first used as an overlay in California in 1983. (A typical polyester overlay is shown in Figure 6-14.) Polyester concrete and PCC are similar in that before setting, they are plastic materials, capable of being cast as structural elements. Both materials consist of aggregate and a binder. Where PCC initiates hydration with water, the polymeric reaction of polyester resin is initiated with promoter and initiator like used in methacrylate.

When compared to PCC with 750 pounds of cementitious per cubic yard, polyester concrete has twice the flexural strength and compressive strengths are nearly equal, abrasion loss is 10%, and chloride permeability is 20% of PCC. Polyester concrete is also used as a patching material and as expansion dams when replacing joint seals.

Figure 6-14. Newly Applied Polyester Concrete Overlay.

The first polyester concrete overlays were placed as a multilayer system, also called broom and seed, where a layer of polyester was spread over the bridge deck and followed by a layer of aggregate. The process continued until the specified thickness was developed. Current practice is to mix aggregate and polyester and place the material in one lift, like concrete either in small batches by mobile mixer or with volumetric trucks, as shown in Figures 6-15 and 6-16. After placing, grading could be controlled with a "Texas" screed, or a laser-guided screed as shown in Figures 6-17 and 6-18.

Polyester materials may be used after being checked by the Caltrans Materials Engineering and Testing Service (METS). Test results can be obtained from the local METS Structure Materials Representative.

As with methacrylate overlays, a test panel is required along with the air quality report from the Certified Industrial Hygienist before the contractual overlay can proceed.

Figure 6-15. Mobile Mixer Operation.

Figure 6-16. Volumetric Operation (Note HAZMAT Placard ID # 1866).

Figure 6-17. "Texas" Screed with Ski Controlled Grading.

Figure 6-18. Laser Guided Screed.

Much of the preparatory work is similar to that when placing methacrylate.

- Surface preparation consists of removing any existing overlay material
- Removing and replacing unsound concrete
- Bead blasting the surface
- Removing all dust and placing a methacrylate primer coat

The bridge deck must be clean and dry when the methacrylate primer coat is placed. If magnesium phosphate patching materials are used, there must be a 72 hour wait before proceeding with the overlay. Polyester overlays will not bond to dirty or wet bridge decks or freshly placed mag-phosphate patches. A recent overlay on the I-580 in Oakland failed 3 months after being placed on a deck that was still damp from grinding. In another overlay failure, the spalling shown in Figure 6-19 is attributed to inadequately applied material (1/4 inch instead of a 1-inch thickness) placed over a wet surface.

Figure 6-19. Polyester Overlay Spalling.

Respirators are required for all Caltrans employees when working within 50 feet of the polyester concrete operations. Avoid physical contact with the materials; if contact occurs, treat the contact according to the material safety data sheet for decontamination.

Grade control under direction of the engineer, is required when placing material in the traveled way. Figure 6-20 is a photo of a rail guided system. Other methods could include a laser-guided screed like Figure 6-18 or a ski like Figure 6-17. Prior to approving the placing method, determine the nature of the overlay. If there is a profile correction, a ski would not correct the profile. If the purpose of the overlay is to provide a wearing surface, and the existing deck has an adequate profile, then a ski may be appropriate if allowed by specifications.

Polyester concrete cures in approximately 3 hours; work is often broken into segments that can be completed in single work shifts. Small operations with mobile mixers and "Texas" screeds can place 200 linear feet of overlay in a shift. Combining volumetric mixers with polyester paving machines can result in 1,000 linear feet in a shift.

Figure 6-20. Rail Grade Control.

When placed, the material must achieve 97% compaction in accordance with CT 552. Proper compaction and resin content is physically indicated by a surface sheen which occurs when resin rises to the surface. As with all overlays, the surface must pass profilograph testing and friction testing. Paving machines generally pass profilograph testing while the surface left by a "Texas" screed may require grinding to meet smoothness specifications.

CHAPTER 7
CALTRANS ADVANCEMENTS / HIGH PERFORMANCE CONCRETE

Table of Contents

Caltrans Advancements / High Performance Concrete **7-1**

Introduction **7-1**

High Performance Concrete **7-1**
High Early Strength 7-2
High Strength Concrete 7-7
Self-Consolidating Concrete 7-14
Mass Concrete 7-23
The Basics of Mass Concrete 7-23
Thermal Control Measures 7-24
Identifying Mass Concrete 7-26
Passive Control by Prescriptive Specifications, Caltrans Practice Prior to the 21st Century 7-28
Performance Based Requirements 7-29
The Use of Active Thermal Control by Caltrans 7-32
Noyo River Bridge 7-33
San Francisco-Oakland Bay Bridge East Spans 7-34
Benicia-Martinez Bridge Substructure and Foundation 7-37
Cast-in-Place Segmental Bridge Construction 7-38
Lightweight Concrete Segments & Mass Concrete in Elements with Dimensions as Small as 10 inches 7-38
Devil's Slide / Confusion Hill: A Laconic Approach to Mass Concrete Specifications 7-43
Acceptance of Active Controls And Return to Passive Control 7-45
Back to Prescriptive Passive Control Specifications & Mass Concrete for Concrete Piles 7-46
Mass Concrete Conclusions 7-49
Lightweight Concrete 7-49
Shrinkage Compensating Concrete 7-51
Durability 7-53
Fiber Reinforced Concrete 7-53
Reactive Powder Concrete 7-54

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7 CALTRANS ADVANCEMENTS / HIGH PERFORMANCE CONCRETE

Introduction

Conventional (or classical) structural concrete used throughout the 1930s and still much in use today is a concrete mix design having 500-700 pounds per cubic yard (pcy) of cementitious material that gains its design strengths of 2,500 to 4,500 pounds per square inch (psi) in 28 days. High consistency for classical concrete was a 6 to 7-inch slump. The earliest special performance requirements were:

- Entrained air for freeze-thaw resistance
- Fine grind cement or manufactured cement with high calcium alumina content for rapid strength gain
- Alkali content limitations to mitigate ASR
- Tri-calcium aluminate (C_3A) limitations for durability improvements

The Federal Highway Administration (FHWA) has promoted a broad category of concrete mix designs that meets specific fresh and/or hardened criteria. These concrete mix designs are collectively termed High Performance Concrete (HPC), indicating enhanced particular properties when compared to more conventional concrete.

High Performance Concrete

Currently, with technological advances, concrete can be designed to meet an even larger variety of performance requirements than practical or even possible only 20 years ago. During the 1990s the term high performance concrete evolved into a catch-all name for existing special concretes that met special performance requirements that could not be uniformly attained using conventional materials and handling practices. HPC incorporates specific materials in the mix design and/or specific placement procedures to produce concrete with enhanced specific performance criteria. The Federal Highway Administration (FHWA) through the Research and Technology Report Center has established eight strength and durability performance criteria for evaluating HPC designs:

- Compressive strength
- Modulus of elasticity
- Shrinkage

- Creep
- Freeze-Thaw
- Scaling
- Abrasion
- Chloride permeability

Other special characteristics such as reduced hydration heat for temperature control, low and high unit weights, consistency, workability and curing enhancements can also be considered for HPC specifications. Supplementary cementitious materials and admixtures are almost always needed to achieve the desired results. Aggregate gradations may be more highly optimized for workability or specific hardened concrete characteristics. Extensive suitability testing of a HPC mix design is often required prior to use on a project as the margin for error is more limited with enhanced performance requirements.

FHWA maintains a web site that provides downloads of the High Performance Concrete Structural Designers' Guide, which details the material selection process: http://www.fhwa.dot.ca.gov/publications/research/infrastructure/structures/hpc/12042/index.cfm (visited 6/10/10).

FHWA also maintains a web site which details the criteria for High Performance Concrete for Highway Structures:
http://www.fhwa.dot.gov/bridge/HPCdef.htm (visited 6/10/10).

FHWA also publishes HPC Bridge Views, an electronic newsletter in which Caltrans has provided significant contributions:
http://www.hpcbridgeviews.com/directory.asp (visited 6/10/10).

HPC strength and durability characteristics are further described in the following sections.

High Early Strength

One of the earliest enhanced specific performance criteria that engineers desired was a high early strength gain for a quicker return to service, accelerated construction sequences, and to compensate for low ambient temperatures. Early strength is not an isolated design goal, as durability and shrinkage must also be considered. There are now very high early strength concretes that are durable and have shrinkage properties that are comparable and sometimes superior to classical concretes.

As early as 1918, bauxite and limestone were combined to form High Alumina Cements (HAC), which achieved accelerated strength gain. Rapid strength gain was due to the high reactivity of mono-calcium aluminates, comprising 50-60% of the cement. Unfortunately it became apparent that the hardened cement was unstable and that significant strength loss due to crystalline changes could occur years later under particular temperature and moisture conditions. For years, Type III Portland cement with its extra-fine grinding, was used to reduce the time needed to achieve design strength. Using Type III cement mixes with high-volume cement contents made typical 28-day strengths attainable in less than a week. One drawback with Type III cement use is that the resulting concrete has a much higher dry shrinkage when compared to a classical concrete; this was especially true when used in combination with calcium chloride, which was used as an admixture to accelerate strength gain. Another major drawback to calcium chloride is the corrosion potential of the reinforcing steel is greatly increased. Non-chloride accelerating admixtures have been developed to overcome the corrosive potential but dry shrinkage is still an issue.

The need for early strength concretes increased after most of our infrastructure was built because maintenance and repair became more frequent and required greater attention as the aged highway system approached and exceeded its initial design life. The pre-cast concrete industry also had incentive for early strength concrete. The longer the time required to gain strength, the greater the amount of space required to store the elements. Reducing the time needed before form removal translated to a faster casting cycle and more efficient use of forms.

Including water reducing admixtures with high cement content concrete mixes resulted in lower water-cement ratios. Although the desired early strength was achieved, the ultimate strength was much higher than needed. Precast manufacturers took advantage of high cement contents, fine grind, and water reducers to more quickly meet required strengths for form release. With or without fine grind cement a very high early strength concrete resulted, often with much higher ultimate strength than the structural needs of the pre-cast element. The mixes tended to be less workable with a consistency that is often described as "sticky," i.e., the concrete tends to stick to working tools and not trowel well.

The need to make repairs on existing structures with as little impact on traffic flow as possible inspired interest in the development and control of rapid strength gaining patching materials. Laboratory tests and field trials were performed on some promising high early strength materials and techniques. Evaluation of the field trials was a coordinated effort between material, design and construction engineers. In the 1980s Caltrans Materials and Research Engineer, Leo Ferroni, oversaw a 6-year study which included rapid set patching materials. This work is documented in Caltrans research report "New Materials and Techniques for the Rehabilitation of Portland Cement Concrete" (FHWA/CA/TL-85/16, Paul Krauss, October 1985). California Test 551 as well as current specifications for Caltrans use of bagged rapid setting concretes for patching material resulted from this research project. Material suppliers still modify their bagged rapid setting patching materials to meet standards resulting from the study. These rapid setting patching materials were available for use in 50-100 pound bags, requiring only small equipment and water for mixing at the construction site. The rapid set materials were ideal for spall repair, bridge joint header construction, or for bonding dowels into existing structures proved to be ideal.

The experience with bagged material led to using rapid setting materials for larger elements, such as approach slabs. The basic cementitious materials were similar, but removing and replacing larger highway elements within an eight-hour shift required mix designs that facilitated the mixing and placing of cubic yards rather than cubic feet. Smaller concrete elements such as asphalt expansion dams (headers), deck patches, and paving notches on retrofitted approach and departure slabs had volumes low enough to be conducive to using pallets of 50-pound bagged material and small mixers with capacities of only several cubic feet. However replacing whole approach slabs or panels might require several transit mixers of material making small 50-100 pound concrete bags inappropriate.

Rapid setting concrete was used in approach slabs for the La Cienega structure on the I-10 Santa Monica Freeway after the Northridge Earthquake in 1994. Some may find it interesting that the contractor proposed casting the entire structure with rapid strength concrete. The proposal was denied because of uncertainties concerning placement logistics, heat of hydration and long-term creep. The rapid setting concrete was wet batched into transit trucks, just as with typical conventional concrete, and exceeded compressive strengths over 3,000 psi in just a few hours. Set time was controlled with admixtures to achieve the needed transit and working time.

Performance specifications were later written for the rapid setting concrete material and the material meeting the requirements was termed Fast Setting Hydraulic Cement Concrete (FSHCC). The first use of this specification in California was in 1996 on a contract to replace PCC paving panels for the I-5 "Boat Section[1]" in Sacramento. A continuous volumetric mixer batched the FSHCC on site, with the mixing performed by a chute augur at the point of delivery. Production rates were about 50 cubic yards per 8 to 10-hour shift. At about the same time, smaller approach slab and paving panel replacements were done from transit mixers batched from a central batching plant.

Competitive suppliers of calcium sulfoaluminate cement, the cement used in several bagged patching materials meeting Caltrans and ASTM Type K cement specifications, pursued contracts having slab replacement projects that originally specified finer grind cements with calcium chloride admixtures. With Caltrans encouragement, these rapid setting cements

[1] The "Boat Section" is a three-quarter-mile section of I-5 in Sacramento from R Street to the Capitol Mall overcrossing. Caltrans engineers call this area the "Boat Section" because it is below the water level. It was named back the late '60s when it had to be drained as one of the last portions of Interstate 5 to be completed. Without pumps, the area would fill up with water during storms.

replaced the Portland Cement by contract change orders (CCO). The CCOs were typically at credit to the State as the savings from the increased number of paving panels that could be replaced in a given window more than offset the cost of the higher priced cement. One of the first projects was about 50 cubic yards of pavement panel replacements done on the Los Angeles I-110 Freeway in 1995. Another example was 180 cubic yards of pavement slab replacements on the Los Angeles I-10 Freeway near the I-57 in 1996 over several night shifts.

In April 1997, the Long Life Pavement Team created a Task Force with the objective of reaching a new plateau in roadway reconstruction; replacing a section of freeway with new 40-year concrete during a weekend freeway closure. The publicized slogan was “give us a weekend and we will give you forty years.” The mission for the Task Force was “Get in, Get out, and Stay out.” A construction strategy using very high early strength concrete on the production scale of a conventional large-scale concrete placement was required that entailed solving engineering problems as well as overcoming reluctance to use cements having setting characteristics very different than Portland Cements on such a large scale. There was some resistance from a large portion of the concrete industry, portions of academia and even from some within Caltrans. The reluctance was overcome and use of FSHCC became acceptable and more widespread as a result of the successful work of the Task Force.

The Task Force outlined a well thought out strategy. To develop the expertise and gain experience without risk to major traffic impediment, the first large scale uses were done behind permanent (i.e., duration of the contract) lane closures. The first was done by a CCO written to replace the Portland Cement with calcium sulfoaluminate cement in an existing contract adding a High-Occupancy Vehicle (HOV) Lane on the I-605 Freeway in Los Angeles. In July of 1997, 600 cubic yards were placed in one shift, about the same rate of production as conventional concrete on this project. The concrete was batched from a central batch plant into transit trucks and placed using a 12-ft wide slip form paver, the first time such a mix design was placed with such equipment. Set time was controlled by citric acid to facilitate transportation time and placement needs.

In August of 1998, again where return of traffic was not an issue, an even larger scale paving operation was done by substituting a FSHCC-mix design for the conventional mix design in a construction project that used a 24-ft wide slip form paver for the 60/71 Interchange in Los Angeles. The concrete was central batched and transported by end dump trucks to increase the production rate. Around 4,000 cubic yards were placed over several days. Two different cement suppliers were used under CCO. Daytime air temperatures reached 100°F, making control of set time even more difficult. The concrete in the 60/71 Interchange construction project consistently achieved strengths ranging from 3,000 to over 4,000 psi in 4 hours; a real success under adverse hot weather.

Though two FSHCC projects during this same time period did not fare as well, a contract to replace slabs on the I-5 near Burbank between January and May 1999, was an overwhelming success. The contract required 400 psi in flexure (3,500-4,500 psi in compression) in 3 hours. The contractor was consistently achieving this in 2 hours after placement. During the job, nighttime temperatures sometimes dropped below 45°F. Transit trucks were batched from a central batching plant that was 15 miles from the site. For this job all but the cement itself and a portion of the mix water was batched from the plant into the truck. The cement was added via 1 to 2-ton super bags that were filled and weighed with the amount of cement needed for the predetermined 6 to 7 cubic yard batches used in the transit trucks. The paving panel replacement production rates were around 100 cubic yards per shift. Though not on the same production rates as the I-605 or the 60/71 projects, a significant amount of FSHCC was centrally batched and transported a substantial distance prior to placement. The panel replacement project proceeded without failure, shift after shift.

Smaller jobs were done throughout the State during this time period. Strengths typically expected over weeks were achieved consistently and controllably in a few hours. The plastic and set characteristics were engineered to meet specific needs of placement and design loads under a variety of conditions. Set times were controlled to meet travel time, placement work including anticipation of varying queue times and still meet design strengths before shift end.

In March of 1999 the largest FSHCC project to that date was awarded. Over 20,000 cubic yards of FSHCC were used to replace panels and even whole lane miles on the I-10 Freeway in Pomona. For one section, two lane miles of the existing truck lane were removed and replaced with FSHCC during one continuous operation. On the weekend of October 22, 1999 work began on this portion of the freeway at 10 p.m. Friday night and continued until 3 a.m. Monday morning. About 5,000 cubic yards were placed, all concrete met specification requirements for strength. The placement was continuous save for a few hours when the central plant electrical system broke down and when the roller strike-offs wore out and required replacement. The remaining 15,000 cubic yards were placed during nighttime shifts, lanes being open to traffic daily by 5 a.m. For the duration of this job commuter traffic was interrupted only one time due to a batching error. This project was an overwhelming success winning multiple awards:

- 2001 ACPA Excellence in Concrete Pavement Award for Restoration
- 2001 ACPA Excellence in Concrete Pavement Award for Transportation Management
- 2001 California “Tranny” for Transportation Management
- 2001 Marlin J. Knutson Award for Technical Achievement
- 2001 Caltrans Partnering Award (Bronze)
- 2002 California Excellence in Transportation Award for Innovation

Following the success of the I-10 Pomona project, resistance to using this technology withered. Portland Cements mixed with large dosages of powerful accelerators and High-Range Water Reducers were developed that achieved similar results thus increasing competition among suppliers. A few bridge structural elements have been placed using fast setting rapid hardening concretes. Within a short time this technology became a much more standard tool for Caltrans when faced with repairing or rehabilitating existing facilities.

Today, high early strength concrete tests attain 4,000 psi in 3 hours using mobile mixers. In special situations, as much as 5,000 psi in as little as 2 hours has been achieved using mobile continuous mixers. Current FSHCC specifications include shrinkage restrictions and thermal stability requirements to limit possible HAC crystal changes to improve long-term durability.

High Strength Concrete

High strength concrete (HSC) is defined by ACI as being above 6,000 psi; that being said, HSC typically tests from 8,000 psi up to and exceeding 20,000 psi. Depending on the design application, strength development times can exceed 90 days. Precast manufacturing yards used extensive vibration for low slump mixes placed into well built forms that accommodate rugged treatment as voids are filled during consolidation. But HSC can now also be designed to have high slump or be self-consolidating making HSC more readily available for cast-in-place concrete.

In 1994 "High Strength" concrete was specified for the construction of the Main Street Overcrossing (OC) in District 12 (Orange County Rt. 5, PM 33), where a cast-in-place post-tensioned box girder structure design required that f'_c = 6,500 psi in one of the superstructure frames. Several months of trial batches failed to meet the required 7,100 psi strength (f'_c + 600 psi for the trial batch). The contractor then proposed a mix having 900 pounds of cement per cubic yard. As 800 pounds was the maximum allowed by the Standard Specifications of that contract, a change order was required prior to use. The 900-pound mix ultimately reached 7,800 psi. The contractor made a trial placement of this rich mix in a footing, but still had reservations about being able to consistently attain the 6,500 psi requirement and provide a mix that was workable enough to make a deck that will meet the specifications. Following construction of the superstructure the Structure Representative suggested in a memo to the Chief of Structure Construction that such concrete requiring the large dosages of water reducers to make strength was "only marginally suitable for bridge superstructures". The memo also reported only minor deck cracking which did not require remedial repairs. Over 13 years later, in 2007, a bridge maintenance inspection reported only minor deck cracking; verifying that the "high strength" cement content concrete did not result in long-term or short-term cracking issues.

In 2000, six years after the Main Street project, a concrete strength requirement of f'_c = 5,900 psi for the Gene Autry Bridge, again in District 12, also saw a request by the contractor to increase the cement content above the limit. In this case the recently added 25% fly ash requirement was given as the need for the request. However, the request was denied and the concrete requirements were achieved within the specification limits. Though in this case, also documented in a memo from the Structure Representative to the Chief of Structure Construction, 35 days were given to achieve a slightly lower required strength; one can still conclude some advancement in achieving high strength concrete took place for cast-in-place concrete. However, it was precast concrete elements that brought more rapid advancements in high strength concrete, transmitting this advancement to cast-in-place concrete.

Pre-cast concrete structure elements benefitted greatly from techniques used to achieve HSC. Pre-casters gained experience rapidly through their efforts to achieve high early strength for form removal, storage, and transport. One of the first, if not the first, bridge design to take advantage of pre-casters abilities to achieve high strength was the Sacramento River Bridge in District 2, on I-5 near Anderson, just south of Redding. Construction began in 2001 on the first bridge in California to use spliced high strength concrete pre-cast bulb-tee girders and a cast-in-place concrete deck. Due to environmental constraints the design span length was 154 feet and the structural depth was 6.5 feet. Another restriction was that work could only be done between May and October, and all falsework was to be removed by October 15. Specified concrete strength (f'_c) for diaphragms and the girders including closure pours was 8,700 psi, significantly higher than the Main Street Overcrossing in District 12. The pre-cast yard located in Oregon achieved the required girder strength, and with the aid of the Structure Representative, the local concrete supplier followed suit producing the high strength cast-in-place concrete. Due to the time constraints the cast-in-place concrete needed to reach strength in 10 days; the average 10-day compressive strengths (f_c) were 10,000 psi. In comparison with the District 12 experience, one can see how fast acceptance of HSC technology developed within Caltrans.

The large bridge construction projects of the District 4 Toll Bridge Program accelerated the use of HSC. The new Zampa Bridge that replaced the 1927 Carquinez Bridge was the first new bridge to be built under this program. Work on this new suspension bridge, the first major suspension bridge in the United States since the Verrazano Narrows Bridge in New York, began January 2000. The two cast-in-place concrete towers consisting of two legs tied together with struts rise 426 ft above the water. Though HSC was not specified for any of the structural elements, the lower tower legs utilized HSC to aid the construction schedule. The towers were constructed using jump forms that were supported by attachments to the previous lifts. Approximately 30 lifts using the jump forms were required for each leg, the height of each lift about 13 feet. Each lift required a strength of 1,200 psi before the form could be “jumped” to the level of the next lift. To achieve this strength within a 12-hour period, HSC was implemented by change order since it exceeded maximum cementitious content specifications and used metakaolin for the first time in a major Caltrans structural element. The mix consisted of 940 pounds per cubic yard (or 10-sack) cementitious material and 1/2 in. maximum coarse aggregate. The cementitious materials were 705 lbs (75%) Type II Portland Cement, 188 lbs (20%) class F fly ash, and 47 lbs (5%) metakaolin. The result was an 8 to 10-in. slump (flowing) mix that reached 1,500 psi by 12 hours and exceeded 10,000 psi at 28 days. All aggregate was from local sources.

Both the 1.2-mile Skyway pre-cast segmental box bridge, a portion of the new easterly spans of the San Francisco-Oakland Bay Bridge (SFOBB), and the 1.4-mile Benicia-Martinez light weight concrete cast-in-place segmental box bridge marked a substantially increased HSC volume. For the Benicia-Martinez Bridge HSC concrete was used because of the modulus of elasticity (MOE) requirements and the desire to have early strengths high enough to prestress and move the traveler forms to the subsequent cast-in-place segment. The Skyway was designed with a HSC strength requirement of 8,000 psi for the segment box and pier tables. MOE requirements in the segments increased the need for somewhat higher compressive strengths. Both bridges utilized cementitious contents above 900 pounds per cubic yard. The Benicia-Martinez Bridge segments were as high as 980 pounds per cubic yard while a typical mix for the HSC in the Skyway had 940 pounds per cubic yard. The lightweight aggregate for Benicia-Martinez came from South Carolina while all the aggregate for the Skyway superstructure came from Canada. Figure 7-1 shows the compressive strength results for cast-in-place concrete for the Skyway. Figure 7-2 shows the compressive breaks from the pre-cast yard in Stockton where the segments were fabricated.

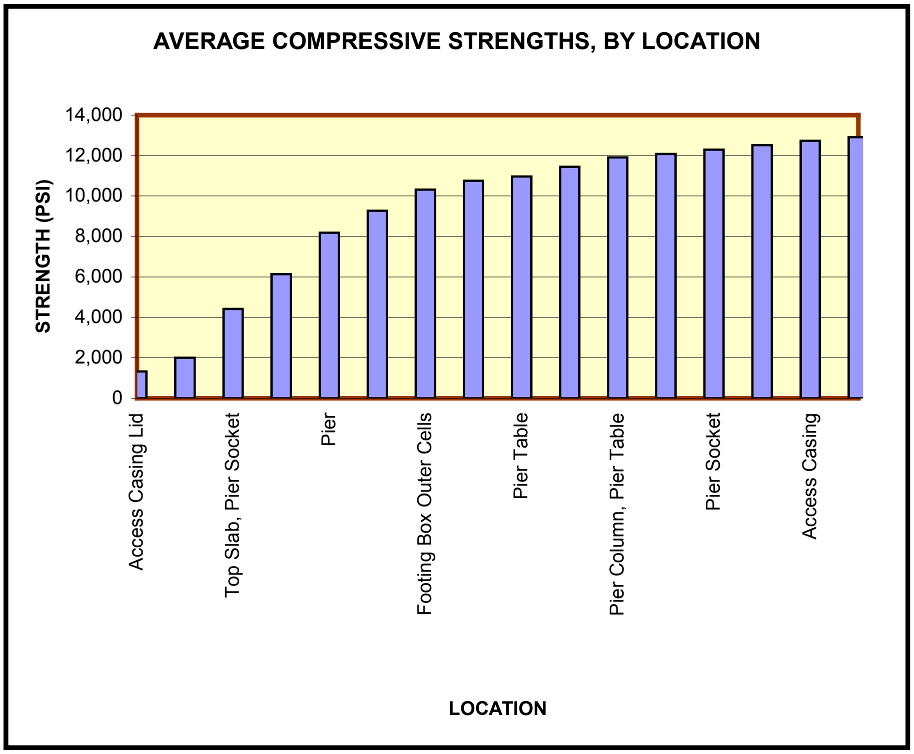


Figure 7-1. Skyway Cast-in-Place Compressive Strengths.

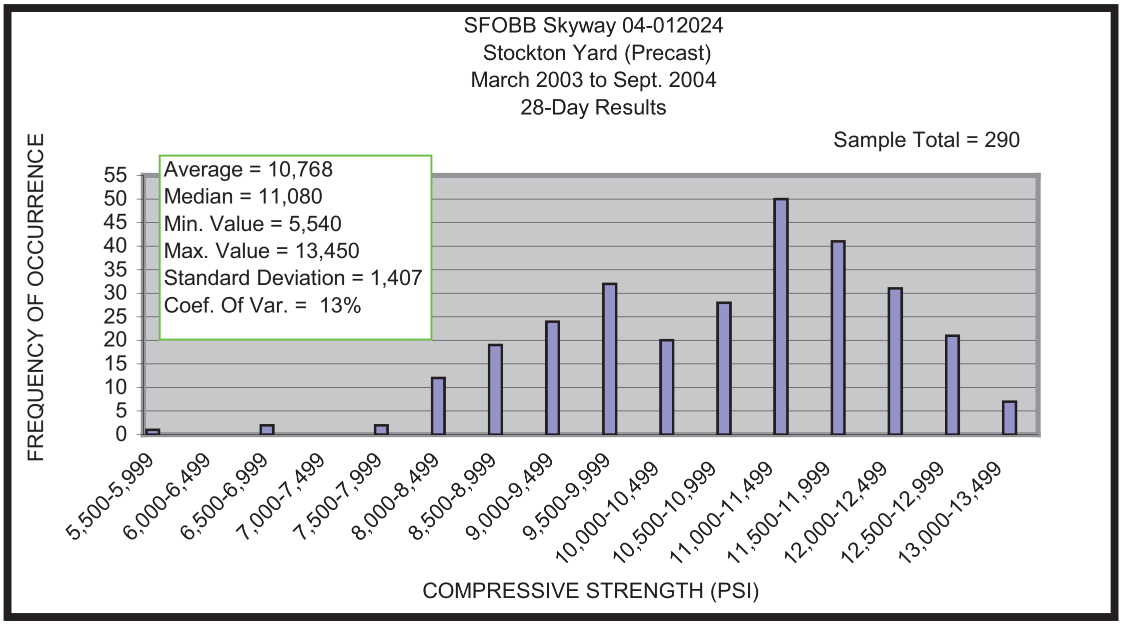


Figure 7-2. Stockton Precast Yard Compressive Strengths for Skyway Elements.

HSC has come into its own since the early Caltrans uses such as the 6,500 psi for the Main Street Bridge. Though by 2008 making high strength cast-in-place concrete was no longer a major concern for Caltrans designers, the HSC was not a given in all areas within the State.

The Angeles Crest Highway Bridge is a 208-ft long single span, precast, prestressed concrete, spliced bulb-tee-girder bridge, spanning an area that was washed out during the spring thaws of 2006 and 2007 along Scenic Route 2, northwest of the city of Los Angeles within the Angeles National Forest. Placement logistics of the precast girders played a significant role in the selection of HSC. The bridge consists of six 96-inch deep girders with a 7.7-inch thick cast-in-place concrete deck. The girders were shipped to the site in three segments with lengths of 56, 92, and 56 ft (See Figure 7-3). With 2 closure pours of 2 ft, the total girder length was 208 feet. The girders were spliced together on the ground in a staging area near the bridge location and then moved onto the abutments. Intermediate and end diaphragms were then cast followed by placement of the concrete deck. The specified concrete compressive strength for the precast girders was 8,500 psi at 56 days. With the bridge located in a freeze-thaw environment at an elevation of 6,500 ft, there was an additional requirement for air entrainment. Air entrainment reducing the strength of concrete meant the concrete mix without air entrainment needed to achieve strength of about 8,500 psi.

The specified air-entrained HSC for pre-cast girders went without incident. But when the time came for the closure pours of cast-in-place concrete the contractor found it difficult to obtain the 8,500 psi concrete from local suppliers, which could have resulted in a major issue if the opening of the bridge was significantly delayed. With technical guidance of the Caltrans Structure Representative, the problem was solved and closure pour concrete meeting all the specifications was successfully placed. Following completion, the Angeles Crest Bridge won the Precast/Prestressed Concrete Institute (PCI) Design Award for 2009.

Figure 7-3. Angeles Crest Highway Bridge Girder Placement.

The 10,000 cubic yards of HSC used to construct cap beams and bent caps at the eastern and western ends of the Self-Anchored Suspension (SAS) bridge portion (the signature structure) of the San Francisco-Oakland Bay Bridge (SFOBB) used both flowing (8 to 9-in. slump) and self-consolidating concrete (SCC). Figure 7-4 (Bay Bridge May 6, 2008) shows strength vs. time for the first pour of SCC on the W2 Cap Beam. This first placement had 56-day breaks approaching 14,000 psi. One quantitative change in the bent cap concrete was that the high strengths were achieved with only 800 pounds per cubic yard cementitious materials and high performance SCMs such as silica fume or metakaolin were not used. About half the pours were flowing and half were SCC. Pours ranged from 300 to 2,000 cubic yards, the typical pour being 1,500 cubic yards.

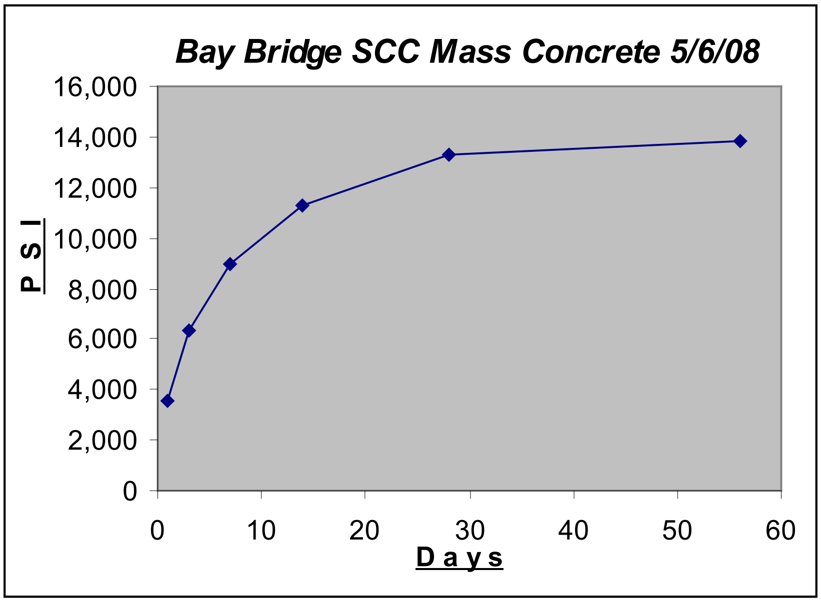


Figure 7-4. Bay Bridge Time vs. Strength Curve.

Workable high strength mixes require inclusion of workability enhancing admixtures. Table 7-1 shows the high strength mix designs for the SAS cap beams. Note the only differences are the admixtures and the coarse-fine aggregate ratios.

Table 7-1. Bay Bridge Concrete Mixes.

Material	High Slump	SCC
Cement, Type II-V	600 lb/yd^3	600 lb/yd^3
Fly Ash, Class F	200 lb/yd^3	200 lb/yd^3
Sand	1,180 lb/yd^3	1,497 lb/yd^3
Coarse Aggregate 1/2″max	1,753 lb/yd^3	1,424 lb/yd^3
Sand portion of aggregate	41%	52%
Type F (HRWR) admixture	35 oz/yd^3	80 oz/yd^3
Viscosity Modifying admixture	0	12 oz/yd^3
Stabilizer / Water reducer	15 oz/yd^3	30 oz/yd^3
Water	264	264
w/cm ratio	0.33	0.33

High strength concrete requires a blend of selected materials and attention to all production aspects. Cementitious materials up to 980 pounds per cubic yard and possibly more have been used on Caltrans projects. As shown above in Table 7-1, high strength concretes, even SCCs, do not necessarily need cementitious contents above 800 pounds per cubic yard. SCMs are useful for conversion of calcium hydroxide to calcium silica hydrates. Because of their small size, SCMs also improve the particle size distribution. Trial batching is needed to determine the optimum cementitious blend and compatibility of cementitious materials.

Aggregate cleanliness, shape, texture, and gradation require attention. Any contaminants like clays and rock dust increase the cementitious paste requirements. Maximum size may be less than one-half inch. Coarse crushed materials provide more surface area to bond with cementitious materials. Trial batching will be needed to determine the optimum gradation. Although it reduces ultimate strength, air entrainment is mandatory for structural elements in freeze-thaw environments.

Trial batching is needed to provide a workable consistent mix. When high strength components are mixed, attention must be given in order of placement in the mixer (as sticky mixes or balling material may happen), temperature, mixing time and the order that admixtures are introduced to the mix. All have impacts on the fresh concrete properties.

Workmanship as in any structural concrete pour is critical when placing high strength concrete. Delivery time must be considered. An axiom in Structure Construction is that workability should not be changed with additional water for any mix; this is even more important for high strength mixes since there is a smaller margin for error as all the mix components have been selected for their high qualities. Although high strength concretes may have large slumps, consolidation is still essential for strength development. Curing is all important; controlling temperature reduces cracking and increases durability. Temperature history is often captured as part of the quality process while the structural element is curing.

Quality control, important for any concrete, is even more important for high strength concrete as there is less tolerance for a mixing or batching error. Product uniformity, achievement of designed strength requires strict enforcement of material standards and placement processes by sampling and inspection throughout the mixing and placing process.

Self-Consolidating Concrete

Self-consolidating concrete is a concrete that flows without segregation to a level state without the use of vibration. Over the 30 years, SSC has rapidly grown in acceptance to the point where the concrete is now part of the 2010 Standard Specifications, as section 90-5. While being developed, SCC was referred to as self-levelling concrete, self-compacting concrete and highly-workable concrete. SCC is allowed for all precast construction, and gaining acceptance in bridge, building and tunnel construction, SCC flow characteristics are achieved by:

- Reducing internal friction through the use of additional fine materials;
- Reducing surface tension by maintaining low water-cementitious ratio with a high-range water-reducing admixture like polycarboxylate ethers;
- Mitigating segregation by reducing the water-cementitious ratio that also can reduce bleeding, and
- Increasing apparent viscosity with a viscosity modifying admixture (VMA) and use of additional fine materials.

Figure 7-5 shows a typical SCC placement tremie.

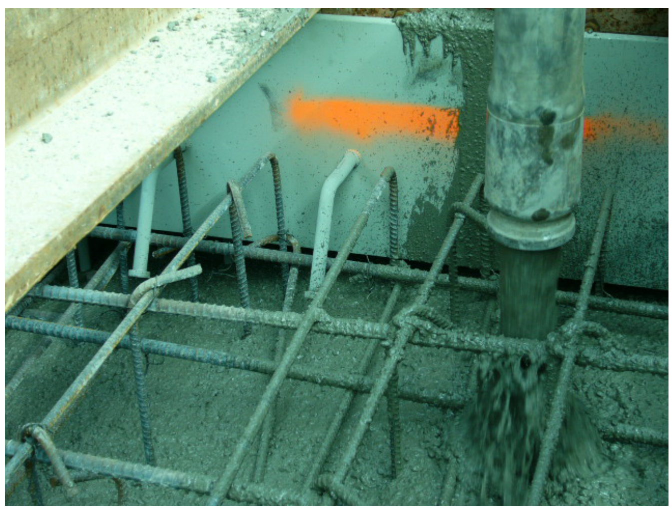


Figure 7-5. SCC Placement Tremie.

SCC resulted from the combinations of two advances in admixture technology. The first was the continued development and refinement of high-range water reducers. The second was the development of anti-wash admixtures. Anti-wash or anti-washout admixtures (AWA) were developed for concrete placed under water to prevent water exposed concrete surfaces from being diluted or even deeper portions of the fresh concrete from being washed away. The AWA makes the concrete more cohesive though still flu d by inhibiting the displacement of free water in the mix by the heavier constituents of aggregate, cementitious material, etc. When mixed with a neat cement paste of medium consistency the paste can be placed in a container of water and the water remains clear. Conversely a neat cement paste without AWA will make the water cloudy. Caltrans used AWA to mitigate cement fines getting suspended and settling in rivers or streams due to placement of concrete. The Caltrans Transportation Laboratory (Translab) tested the admixture by placing a lined paper behind a glass container, placing a mortar mix incorporating the AWA in the mix, and checking to see if the lines on the paper could still be seen through the water. A mortar with AWA placed in the container of water left the water clear keeping the lines on the paper visible. A mortar without AWA immediately clouded the water making it impossible to see through.

AWAs were the predecessor of viscosity-modifying admixtures (VMA). Very fluid mixes whether fluid by excess water or by use of water reducers will cause a concrete mix to segregate. VMAs prevent or mitigate this, and reduce bleeding which is a form of segregation. A very fluid mix that is even self-leveling will still be cohesive with a VMA. Also increasing the fines in the mix mitigates segregation. The aggregate fine-coarse ratios are modified with more fine material for SCC mixes. Additionally, more cementitious materials may be used or other materials such as limestone dust may be added.

Caltrans first used SCC on a large scale during the seismic retrofit of the Richmond-San Rafael Bridge Seismic Retrofit Project when the material was added by CCO. Large underwater steel shells about 14 ft in diameter were filled with concrete without removing water. Due to the size of the shells and the distance a conventional tremie concrete travels, placement of concrete would have required several simultaneous tremie placements to prevent intermixing of water with the concrete mix. SCC allowed for one tremie used down the middle of the shell as the SCC flowed from the tremie to the far perimeters of the placement. SCC was used on the Skyway project, again the material was added by CCO. The footing boxes were filled with both normal weight and lightweight SCC. Figure 7-6 contains two photos; a footing box and the pour front 45 ft from discharge. Note the cohesiveness of the mix. Table 7-2 shows SCC mixes used for the Skyway.

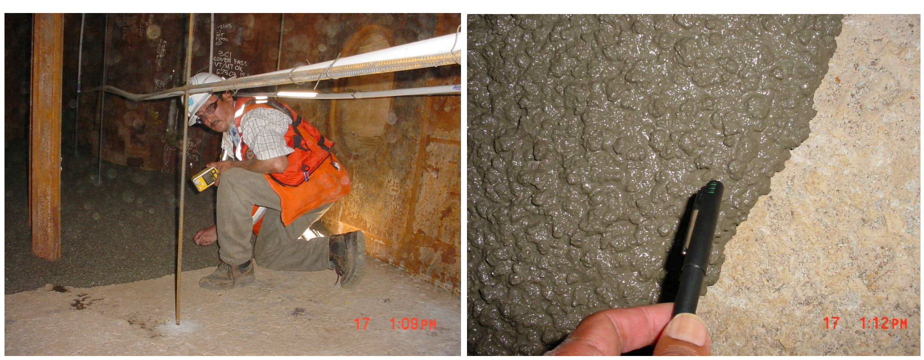


Figure 7-6. Footing Box and Pour Front 45 ft from Discharge.

Table 7-2. Skyway SCC Mixes.

Ingredients	Mix 1	Mix 2
Water, lb	291	267
Portland Cement Type II, lb	291	334
Fly Ash, lb	440	0
Ground Granulated Blast Furnace Slag, lb	0	334
Lightweight "Realite" one-half inch coarse aggregate	875	0
Lightweight "Port Costa" fine aggregate, lb	620	0
Normal weight "Sechelt" fine aggregate, lb	580	1,523
Normal weight "Sechelt" coarse aggregate, lb	0	1,523
HRWR, fluid ounces	240	32
VMA, fluid ounces	100	52
Recover Stabilizer (Type D Retarder)	24	20
Slump Flow Test - Diameter of Spread, inches	28	25

SCC was also successfully used to fill a large void under a steel support plate as part of a sliding plate joint assembly. The SCC was pumped through 2-inch diameter pipes that penetrated the steel plate. Figure 7-7 shows photos of the placement pipe and subsequent grouting of the hole left when the pipe was removed. The SCC performed as a high strength grout, but it was a concrete. SCC became the specified material for this use on all the sliding plate joints to be placed on the east spans of the SFOBB.

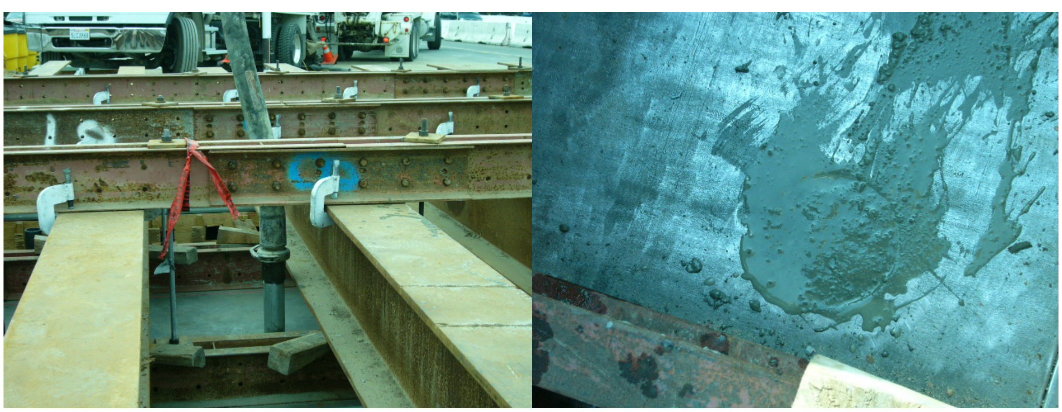


Figure 7-7. Concrete Pump Hose Connected to 2″Pipe and Grouted Pipe Delivery Void.

Figure 7-8 shows formwork for SCC Pours, W2 bent cap of the west end of Self-Anchored Suspension Bridge. Blue Hoses are intake hoses providing water to the internal cooling system for mass concrete and Red Hoses are for the warmer return water.

Figure 7-8. Forms for SCC Placement with Water-cooling Tubes.

The 10,000 psi concrete used for constructing the bent cap beams at the ends of the SAS structure required HSC. Some of the pours were congested enough with rebar to warrant the use of SCC. The cementitious content for the SCC was limited to 800 pounds per cubic yard for both conventional and SCC as discussed in the above section on HSC. The only differences between the conventional and SCC mixes were the fine to coarse aggregate ratio, the amount of High-Range Water Reducer used in each mix, and the addition of VMA in the SCC mix. Cementitious content and water, at least by design, were exactly the same. Upon analysis of the compressive strength cylinders taken from the project, it is notable the SCC had consistently higher strengths. This can be seen in Figure 7-9. The points are the average tests of two separate conventional concrete pours and average tests from four separate SCC pours. This graph does not include the data from the pour shown in Figure 7-4.

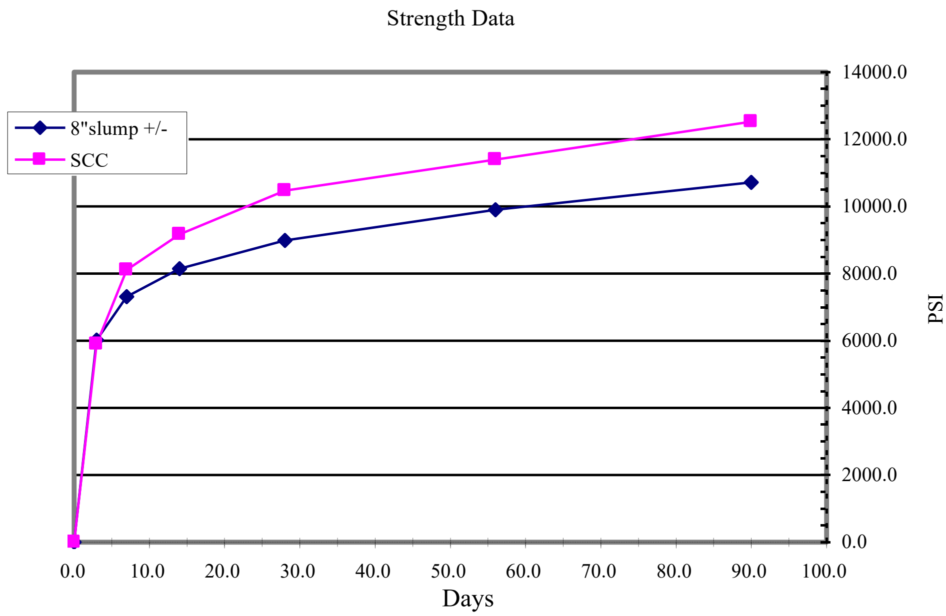


Figure 7-9. Nifty Graph Comparing Strength of SCC and High Slump Concrete.

A SCC early strength mix was used in the closure pours that connected the west-end bent cap beam (W2) to the orthotropic steel boxes to the east making up the superstructure of the Self-Anchored Suspension Bridge. Early strengths were desired to avoid the structure going through a thermal movement cycle at the gap to be filled with reinforced concrete between the concrete bent and steel boxes before adequate strength was obtained in the concrete. Type III cement was used with a Type C chemical accelerator admixture. The slump flow rates ranged from 25 to 29. The first closure pour tying the east-bound orthotropic box structures to the W2 bent was completed about 10 p.m. in one evening. By 5 a.m. the following morning concrete strengths exceeded 3,000 psi. Previous monitoring showed no thermal movement between about 9 p.m. in the evening and 5 a.m. in the morning. The second closure pour done 2 weeks later tying the west bound orthotropic boxes to the W2 bent had similar results though achieving a lower strength of about 1,800 psi by 5 a.m. because the accelerator was cut back to facilitate placement; 1,200 psi was the 5 a.m. target.

Prequalification testing was required to verify flow and segregation characteristics meet specifi ations. A mockup flow test is also included in the current contract special provisions for SCC use. Trial batch test reports document slump flo , flow rate, visual stability index, J-Ring passing ability, column segregation, bleeding, compressive strength and minimum compressive strength.

- Slump flow – Slump flow measures the consistency of fresh SCC. The test measures the spread of a SCC sample after release from a standard slump mold. Spread is the average of flow in two orthogonal directions as shown in Figure 7-10.

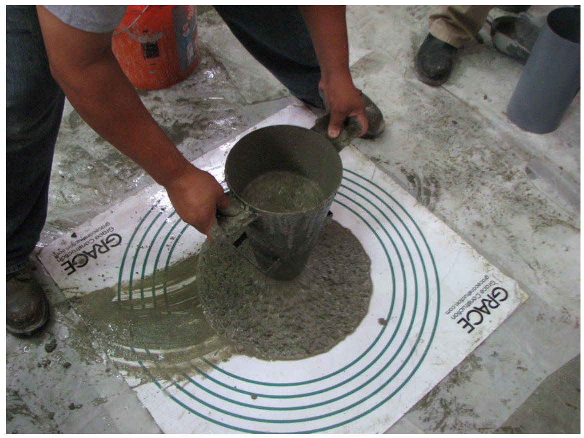


Figure 7-10. Slump Flow Measurements.[2]

- Flow rate – Referred to as T_{50}, flow rate measures the viscosity of the mix. Higher viscosities are indicated by longer times to complete the flow test. High viscosities are associated with "sticky" or "cohesive" mixes. The T_{50} time is the time required for the SCC sample to spread to 20 inches. The use of 50 in the T_{50} symbol represents 500 mm, approximately 20 inches.
- The Visual Stability Index (VSI) is a relative viscosity rating. After the flow test has been performed a visual check is made for segregation and bleeding. A halo is a cement paste ring around the flow that has segregated from aggregate. Aggregate may also segregate as a pile at the center of the flow. The visual stability ratings run from zero to three and are described in Table 7-3.

[2] http://www.hpcbridgeviews.com/i50/Article4.asp (visited 7/20/10)

Table 7-3. VSI Ratings.

Rating	Description
0	Highly Stable – No segregation, no bleeding
1	Stable – No segregation, slight bleeding (surface sheen)
2	Unstable – Mortar halo at exterior ring (< 0.5 inch). There may be an aggregate accumulation at the center of the flow.
3	Highly Unstable – Large mortar halo (>0.5 inch). There may be a large aggregate accumulation at the center of the flow.

- J-Ring Passing Ability – The J-Ring Apparatus, shown in Figure 7-11, is a solid ring supported by 16 dowels. Each dowel diameter is 5/8 inch and the dowels are spaced approximately 2.36 inches apart along the circumference of a 12-inch diameter circle. The test assesses the passing ability of a SCC mix as it flows through the dowel array. Similar to the slump flow test, material is released from a slump test mold at the center of the J-ring apparatus. The flow measurement is the average of two orthogonal measurements of the slump flow. Acceptable results are within one inch of the slump flow tests.

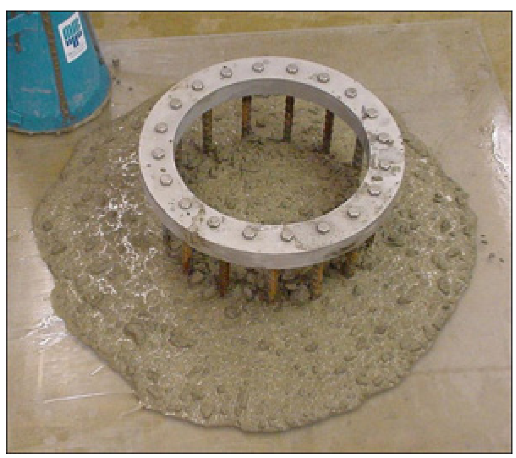


Figure 7-11. J-Ring Apparatus.[3]

[3] http://www.hpcbridgeviews.com/i50/Article4.asp (visited 7/20/10)

- Column Segregation – The column segregation test, shown in Figure 7-12, verifies that coarse aggregate does not segregate in a column placement. After filling a cylindrical mold without tamping or vibration, the mold is separated into upper and lower thirds. The material from each third is washed on a No. 4 sieve and the masses are compared to determine the percentage of segregation.

Figure 7-12. Column Segregation Apparatus.[4]

- Bleeding – ASTM C232 testing determines the percent of bleed water by comparing the mass of water in a specimen to the mass of bleed water drawn off.
- Strength – The strength test is a standard California 521 Test for five cylinders.
- Minimum Strength – The minimum strength test, also California 521, verifies all cylinders exceed minimum specified strength.

After passing all prequalification testing including the mock up QA/QC testing can be limited to slump flow and the typical conventional concrete tests.

[4] http://www.hpcbridgeviews.com/i50/Article4.asp (visited 7/20/10)

Mass Concrete

When measures beyond those typical for concrete construction need to be taken precisely to keep a concrete element from exceeding 160°F, that element is determined by Caltrans to be Mass Concrete. The measures can be applied to the concrete mix design, or to the in-place hardening concrete, or both. The measures taken to prevent the hardening concrete from exceeding 160°F are necessary for the durability of the structural element. It is most efficient from a contractual point of view to detect structural elements having a potential to exceed 160°F before a contract goes out for competitive bid. However, sometimes this potential may be missed during the design phase. If this happens, it is imperative to the durability of the structure to address the issue by CCO; as part of the design and evaluation process for temperature controls, the decision should include cost analysis data for the potential solutions.

Both the practices used to determine in advance potential mass concrete and the measures taken have changed through the years. Besides a basic understanding of the physical characteristics that result in mass concrete, a review of the changes regarding mass concrete over time is useful for grasping the substance of the theory and practice of mass concrete. The better the assimilation of the substance of the theories and practices of the measures taken over the years, the better the structure engineer is able assess solutions to the practical problems that sometimes arise during construction. This is true even when mass concrete is identified and addressed in the contract document.

The Basics of Mass Concrete

As the cementitious materials in concrete hydrate, heat is given off because hydration is an exothermic reaction. The amount of heat produced is proportional to the quantity of concrete placed. As concrete element size increases, the amount of heat produced will also increase. The temperature of the element depends on the rate of heat dissipation to the environment compared to the rate of heat generation. Dissipation depends on the size and geometry of the element while the generation depends on the cementitious material.

High temperatures can affect concrete durability by inducing stresses or chemically altering the cement hydration process. Temperature induced stresses are a result of volume changes that can result in cracking. Delayed Ettringite Formation (DEF) is the temperature induced alteration of the hydration process. It is generally recognized that up to a curing temperature of 160°F, DEF will not occur; while the further above 160°F, the greater the probability of occurrence.

Ettringite is a naturally occurring mineral formed in hydrated Portland cement along with calcium silica hydrates. When high heat occurs, ettringite may not form. However, after the heat caused by hydration subsides, ettringite formation may resume, resulting in an expansive gel within the hardened concrete. This can result in a material related stress similar to Alkali Silica Reaction deterioration.

High peak temperature can also lower compressive strength and related properties by restricting the hydration process due to self-desiccation, a condition where hydration cannot continue because of a lack of available water. Therefore attention to temperature during hardening is all-important.

ACI Committee 207 was established in 1930 for the development of theory and practice of mass concrete for the construction of large dams. Since then, mass concrete theory has become applicable to much smaller concrete elements because of high performance concrete (HPC) incorporating higher cementitious contents to achieve properties like higher strength, self-compaction, and lower permeability. In 1987 ACI Committee 207.1 changed its title from "Mass Concrete for Dams and Other Massive Structures" to simply "Mass Concrete."ACI 116R-00 defines mass concrete as "any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of the cement and attendant volume change, to minimize cracking."

Though Caltrans is interested in preventing cracking due to mass concrete, it is the potential long-term material related distress that triggers mass concrete measures. Therefore the peak temperature potential of 160°F is used by Caltrans to define concrete as Mass Concrete and the need for thermal control measures. Besides, if peak temperatures are controlled the detrimental effect of differential temperature is also mitigated.

Thermal Control Measures

The measures used to control concrete temperature can be either active or passive.

Passive measures control temperature through actions taken before placement. Passive controls may include limiting the amount of cementitious material, using supplementary cementitious materials that generate heat at a lower rate, placing elements in several lifts or segments thus limiting the size of each pour by addition of cold joints, lowering the concrete temperature at placement by adding ice as part of the mix water, chilling stockpiled ingredients prior to batching, or cooling the batched concrete with liquid nitrogen injections. Figure 7-13 consists of two photos of the nitrogen injection process. Figure 7-14 is a photo of ice being added in the batching process.

Figure 7-13. Liquid Nitrogen Coolant Injection.

Figure 7-14. Batching Concrete with Ice.

Active methods control concrete temperatures after placement and during or after hardening. Active controls include the installation of surface insulation, the use of cooling pipes that contain a cooling liquid and act as a heat exchanger within the concrete element or using water curing for evaporative cooling on the surfaces. Figure 7-15 is a typical view of cooling pipe installation.

Figure 7-15. Cooling Pipes Tied to Reinforcing Steel.

Identifying Mass Concrete

The ACI definition does not characterize mass concrete in terms of a specific dimension. Caltrans, as well as other agencies, uses minimum dimensions to identify mass concrete for the purpose of determining when thermal controls are needed. Having a universal standard minimum dimension of an element to identify what would be mass concrete is adequate for large concrete placements with the traditional specified concrete compressive strengths that are no more than 5,000 psi. However with use of higher-strength HPC concretes, minimum dimensions identifying mass concrete have become less predictable.

The dimensions identifying mass concrete in Caltrans bridges changed through the years while the measures used to control temperatures remained similar until the recent increase in the use of HPC. Around 1978-1980, specifications for Potato Slough Bridge and Dumbarton Bridge both identified mass concrete as any structural concrete element with a minimum dimension of 4 feet. By 1992 the minimum dimension identifying mass concrete in structural elements was increased to 7 feet. In 1995 as a soft conversion to metric the minimum dimension was changed 2 meters, or 6.6 feet. With the return to English units, the minimum dimension is 7 feet. This minimum 7-ft dimension has been shown to be adequate for identifying mass concrete in advance when concrete is not high strength concrete.

In October 1999, concrete placed in a 6-ft diameter steel shell shaft rising to the surface of the Carquinez Straits was monitored for temperature; this new pile was part of a seismic retrofit for the 1962 Benicia-Martinez Bridge. The concrete in the pile had no thermal control and was instrumented with thermocouples. The peak temperature occurred along its central axis at 22 feet down. Six feet down from the surface, the temperature reached no more than 122°F. Temperature differentials between concrete near the steel shell that was submerged in the cold bay water and concrete at the center axis of the pile were less than 14°F.

During construction of the Spanish Creek Arch Bridge in District 2, peak temperature measurements on a column with a cross dimension of 4 x 8 ft demonstrated the difficulty in relying totally on the dimensional method to identify mass concrete. The peak temperature was 161°F at 24 hours. This concrete was high strength concrete at a specified 42-day 6,000 psi with also an air requirement since the bridge is in a freeze thaw zone. The mix had the maximum 800 lb/yd^3 of cementitious, which was Type IP cement. This particular Type IP cement utilized a pozzolanic material that was 84% reactive material with a Blaine fineness of 680 m^2/kg; this made the heat rate similar to, if not higher, than Portland cement. In addition, the Blaine fineness of the cement was a relatively high 480 m^2/kg when compared to a more typical being around 400 m^2/kg.

The combination of hot cementitious material, long delivery distance for the concrete, summer temperatures and borderline geometry resulted in high peak temperatures in the curing element. The issue of mass concrete was contemplated during design and based upon the assumptions made by the design team. It was decided that mass concrete would only be an issue at the footing blocks of the arches. Structure Construction staff were diligent in verifying design assumptions and when the temperature was measured on the relatively small column the project staff were able to get a thermal control plan in place for the 8 x 9 ft arch rib.

Figure 7-16 shows the temperature rise for two 5-ft concrete cubes with the only exception to identical mix designs being the Blaine fineness of the cement. As shown in the figure, the temperature of the cube with the finer cement, (380 m^2/kg), rose 10°F higher than the cube with the coarser cement (300 m^2/kg).

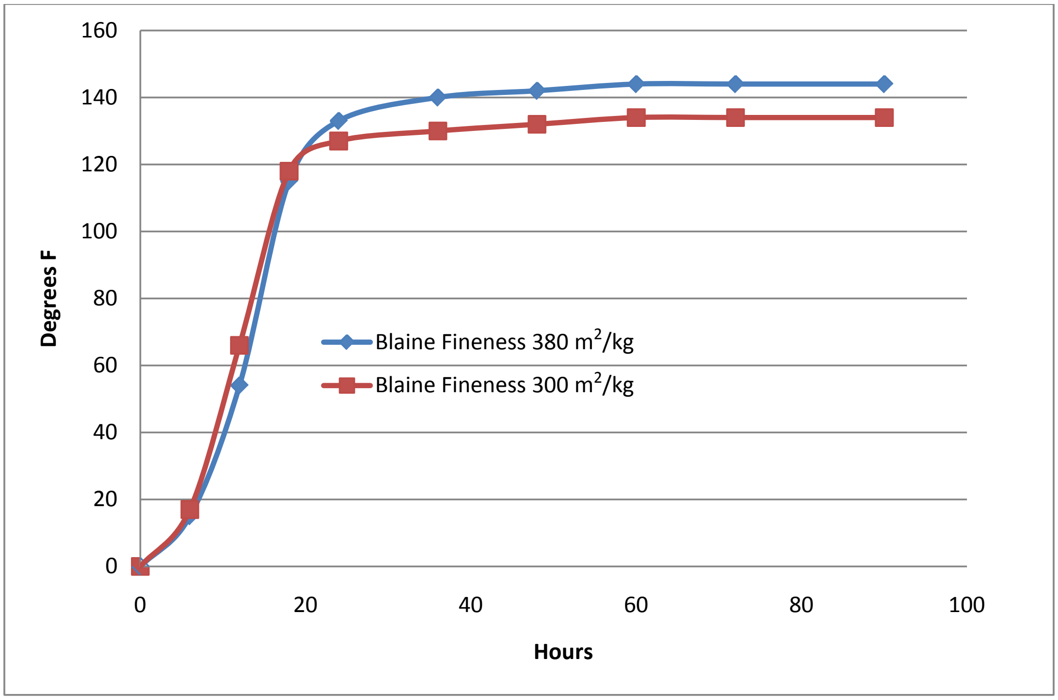


Figure 7-16. Temperature Variation Due to Blaine Fineness.

The column was placed in summer. Therefore with a mix having heat generation characteristics nearly as high as possible within standard specifications and placed during summer, the column did not exceed the 160°F limit used to define mass concrete. Though this is closer to the limit than desirable, the mix design is as hot a mix as can be expected and is high strength. (Note: precautions are called for in this situation but the measures are similar to those taken for concrete placed under hot weather construction discussed in Chapter 5.) Therefore for identifying potential mass concrete in advance, 7 ft serves quite well unless HPC is to be used.

Passive Control by Prescriptive Specifications, Caltrans Practice Prior to the 21st Century

Before 1999, all mass concrete temperatures in California bridges were easily controlled with passive methods specified in prescriptive specifications because mass concrete was generally limited to large footings and columns that had traditional bridge concrete compressive strength requirements seldom more than 4,000 psi. Prescriptive specifications for passive thermal controls were easily employed by special provisions. Initial placement temperature was limited to 65°F when the cementitious material was no more than 564 pounds per cubic yard with the pozzolan being 15% of the cementitious material by weight. For each additional 10 pounds of cement or 20 pounds of pozzolan above the 564 pounds cementitious content, the initial maximum temperature was reduced 1°F. Also, the Type II cement's combined tricalcium silicate and tricalcium aluminate content was limited. Cementitious content was changed to 590 lb/yd3 when Caltrans went to SI units in 1995. The 590 lb/yd3 was due to a soft conversion of class concrete which in this class became 350 kg/m3. Fly ash after 1997 was 25% as required in most structural concrete offsetting the slight increase in cement. The prescriptive specifications were practical for both Caltrans and the contractor.

Field experience showed that these prescriptive specifications of passive measures and having no contractual temperature performance requirements for the in-place concrete were adequate. During reconstruction of the Cypress Bridge structures on Route 880 in Oakland, California, after the Loma Prieta Earthquake, the contract used the prescriptive specifications detailed above with 7 ft as the minimum dimension identifying mass concrete. Six large footings and one column built between 1994 and 1996 were monitored for temperature to check the assumptions that the prescriptive specifications resulted in acceptable control of concrete temperature. The footings were as large as 30 x 45 x 10 ft and the cross section of a column was 9 x 9 ft. The mixture proportions required 564 m^3 of cementitious materials with 15% fly ash for the column and the first footings. Peak temperatures were no higher than 144°F, an acceptable temperature limit preventing DEF. In 1996, fly ash percentages were increased to 30% and then 40% in an attempt to reduce temperature differential within the element to 36°F. The last footing was 10 feet thick and the concrete was 40% fly ash, reducing the peak temperature to 126°F. The additional fly ash reduced the differential to the 36°F target, which Caltrans has since found to be conservative for large elements.

Following the Cyprus project from July to October 1999, five large footings for the 580/680 Interchange bridges in Dublin, southeast of Oakland, California, were monitored for temperatures. The footings were 26 to 36 feet wide, 35 to 40 feet long, and 10 to 12 feet thick. Only the specified passive temperature controls were used. All mixes had 590 lb/yd^3 of cementitious material with either 25 or 35% fly ash. The peak temperatures ranged between 135°F and 140°F, and temperature differentials for the 35% fly ash mixes were near the 36°F limit though somewhat above it. The actual field data again demonstrated that the prescriptive specified passive controls were appropriate.

Performance Based Requirements

Mass Concrete Specifications for the replacement Carquinez Bridge (Alfred Zampa Bridge on Route 80 northeast of Oakland, awarded in 2000 and completed in 2003) were performance based. The contractor was to determine the measures taken to keep concrete temperatures within specified limits. Any element with a least dimension that exceeded 2 meters was limited to a peak temperature to 158°F and temperature differential of 36°F. The tower footing concrete and the massive anchor blocks for the cables were mass concrete and were monitored by measuring temperatures in the concrete via thermocouples connected to data loggers.

The contractor used the Schmidt Analysis Model for thermal predictions and temperature controls. The analysis was reasonably accurate resulting in adequate temperature controls. Cooling pipes were considered by the contractor, but not chosen. However, the contractor chose to use the active measure of applying, maintaining and monitoring thermal insulation blankets to control differential temperature between the center of mass and concrete near the exterior surfaces, and the passive measure of limiting the cement content and increasing fly ash to 35%. Mass concrete segments for the anchor blocks at Carquinez measured approximately 50×20×14 ft. Approximately 40 such segments were cast to produce the 4 anchor blocks, 2 on either side of the bridge. The footing for each of the 4 tower legs that made up the two towers was 59×72×16 ft, with each tower leg joined by a 49×23×16 ft tie beam so that each tower had 1 continuous footing. The insulated concrete, with the required f'_c no more than 4,000 psi, did not exceed the maximum specified peak and differential temperature. Figure 7-17 is an anchor block. The grout tubes extend 58 ft into the blocks. Figure 7-18 shows the footing, or pile cap, form work for the mass concrete. Figure 7-19 is a photo of the Zampa south anchor blocks and towers rising above the pile caps.

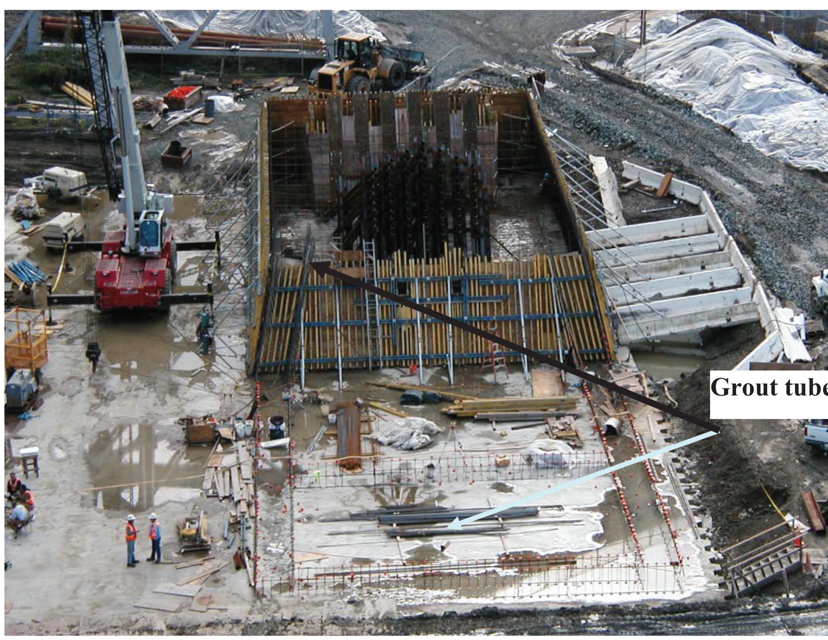


Figure 7-17. Anchor Block, Zampa Bridge.

Figure 7-18. Footing or Pile Cap Prior to Placement of Mass Concrete.

Figure 7-19. Zampa Bridge Anchor Blocks and Towers During Construction.

The Use of Active Thermal Control by Caltrans

With the use of High Strength Concrete, the combination of passive methods with thermal blankets was inadequate for large *massive elements*. During the design of the new Noyo Bridge, the new Benicia/Martinez Bridge, and the new east spans of the SFOBB, internal cooling pipes were discussed for the larger higher strengths concrete elements. The pier tables for the Skyway portion of the new SFOBB east spans, were not only larger than the 580/680 footings, but were to be cast with high strength concrete in the range of 8,000 psi. The large foundation blocks and anchors for the west end of the Self-Anchored Suspension (SAS) portion of the new SFOBB were also of special concern. The planned blocks at Bent W2 where the SAS meets the Yerba Buena Island transition structures were 64 × 64 × 33 ft with corrosion resistant concrete that reached compressive strength of 9,000 psi. The new Benicia and Noyo River bridges also anticipated large elements with higher strength concrete than what was typical though not as high as the 8,000 psi design.

There were concerns that existing prescriptive passive specifications would not work for the anticipated mass concrete envisioned for the three bridges and that active controls, particularly cooling pipes, would be necessary. The unlimited combinations of pipe size and arrangement, water temperature and flow, and possible combination with other passive controls led to the use of a performance specification. Except as noted below, these first performance specifications used 149°F as the limit in order to have some built-in safety factor. However, it was also concluded that practical experience was needed and a cooling pipe thermal control system should be done on some existing project.

To demonstrate to contractors that cooling pipes were a viable option, Caltrans designed a heat exchanger made of steel pipes for one of the large footings at the 580/680 Interchange project. In November 2000, concrete for a 580/680 Interchange footing was changed to a high-strength concrete. This concrete generated significantly more heat than the mixtures used on the other footings at this site that were of a similar size and had previously been monitored for temperature. Active temperature control in the 38 × 38 × 12 ft footing consisted of 1 inch diameter steel cooling pipes that were installed at the same time as the reinforcing steel and configured into 3 square coils with pipes spaced approximately 3 ft apart vertically and horizontally. A 50-ton chiller was used to cool water circulating through the pipes.

With the use of cooling pipes peak temperature of the footing with high strength concrete was comparable to the other normal strength concrete footings on the project previously monitored for temperature. The normal strength concrete had 590 lb/yd^3 of cementitious material, 35% of it fly ash while the high strength had 800 lb/yd^3 cementitious, 20% fly ash 5% metakaolin. This comparable peak temperature is shown in Figure 7-20. Figure 7-21, a time vs. temperature graph comparing the high strength to the normal strength 6 × 12 inch cylinder both cured under identical conditions, shows the difference in temperature generation of the 2 concretes. These two graphs demonstrate the effectiveness of the cooling pipes. A report was written and included with the bid documents on other projects where active controls were expected.

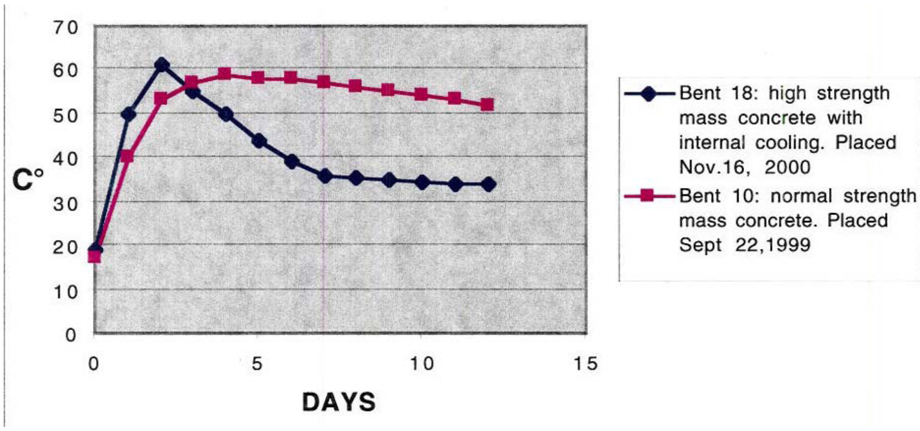


Figure 7-20. 580/680 Footing Concrete Temperature vs. Time.

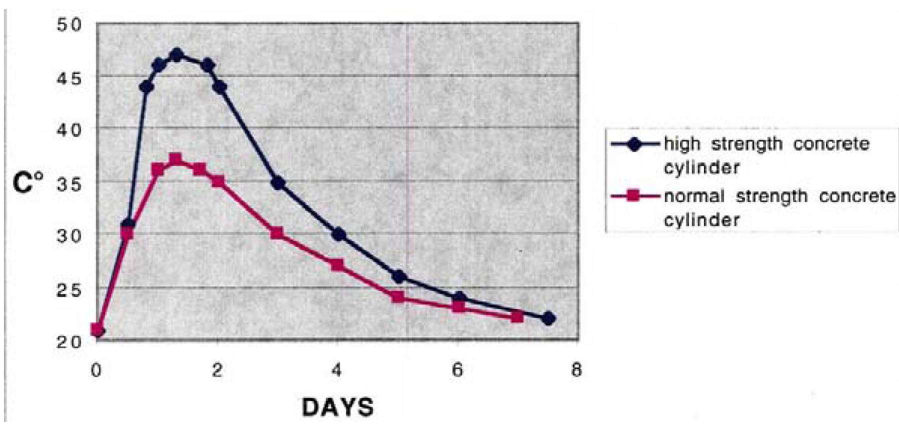


Figure 7-21. 6 x 12 Inch Cylinders Cured Under Same Condition.

Noyo River Bridge

The first bridge with Caltrans specifications requiring an internal cooling system was the new Noyo River Bridge in Fort Bragg, located 140 miles north of San Francisco. The specified concrete compressive strength ranged from 4,300 to 5,700 psi with low permeability required for elements in or near brackish water. Bridge construction began in May 2002 and was completed in August 2005. The mass concrete elements were 39 × 108 × 8.2 ft footings, 9.8 ft diameter columns, 20 × 72 × 12 ft bent caps, and 11 × 85 × 7.2 ft abutments.

The contract special provisions required a thermal control plan that limited peak temperature to 149°F and differential temperature between any two points within the placement was limited to 36°F. Cooling pipes were required in the first placements of the footings, columns, and bent caps. However, if the contractor proposed a plan that did not use the cooling pipes and proved that the plan was effective during the first placements where the cooling pipes were in place only as a back-up system, the cooling pipes would not be required in subsequent placements.

The contractor chose to use 1 inch diameter steel cooling pipes and chilled water from a storage tank as part of the thermal control plan even at the abutments where it was not required in the first placement. The Caltrans Design Engineer for this project noted the ease of fabrication and operation of the system to control temperatures.

San Francisco-Oakland Bay Bridge East Spans

The seismically vulnerable east span of the San Francisco-Oakland Bay Bridge connecting the Yerba Buena Island tunnel to Oakland is being replaced with a new 2.2 mile long bridge. Two of the major structures making up the bridge are the 1.5-mile Skyway and the 2,000 ft self-anchored suspension (SAS) structure. The Skyway, with spans up to 525 ft and box girders up to 30 ft deep and 82 ft wide, was constructed using a pre-cast concrete segment balanced cantilever method. The large bent caps had f'_c of 8,000 psi used cooling pipes to control temperature. All concrete for the Skyway had a specified peak temperature limit of 149°F. The SAS is a single tower signature structure architecturally designed to complement the west span 4-tower suspension bridge linking the island to San Francisco. The span east of the single tower will be 1,263 ft, while the span to the west will be 784 ft. The west end concrete piles, foundation/anchor, and piers were completed in 2004 as a separate contract. The last Skyway segment was placed in December 2006. The 2,000 cubic yards of concrete for the bent cap at the east end (E2) of SAS was placed in 1 continuous pour, while the last of the 5 concrete pours making up the 80,000 cubic yards concrete bent cap on the west end (W2) occurred in February 2009.

Of particular interest are the anchor blocks at W2 at the west end of the structure where the SAS meets the Yerba Buena Island Transition Structure. The two anchor blocks each measuring 63 × 63 × 10 feet help balance the forces in the unequal spans of the signature SAS structure. The maximum peak temperature for the W2 footings/anchors was specified at 122°F to ensure the large mass of concrete would stay intimately in contact with the walls of the excavated rock formation. Low permeability requirement in the concrete for corrosion control resulted in an actual compressive strength that was over 9,000 psi. Each block was poured in one continuous work shift. Liquid Nitrogen as passive control and cooling pipes as active control was used. For one block cooling water was taken directly from the bay while on the other water was chilled water from a storage tank.

The bent caps at W2 on the west end and at the east end of the structure at E2 specified 8,000 psi concrete. (The SAS cable will be looped around the west bent cap through deviation saddles and anchored to the E2 bent cap.) The mass concrete thermal control plan in both bent caps consisted of pre-cooling concrete with liquid nitrogen (passive) and cooling during hydration with active cooling pipes to comply with the temperature limit 149°F.

For these large elements requiring high strength concrete, cooling pipes showed to be a most effective method of controlling temperatures to meet the performance-based specifications. Scaled down mock-ups were required rather than mandatory cooling pipes in the first placement, to validate the contractor's thermal control plans. Figures 7-22 and 7-23 are initial and final photos of the W2 anchor reinforcement and cooling pipe placement. Figure 7-24 is a typical 10 x 10 x 10 ft mock-up cube. Figure 7-25 is a chart of the mock-up temperature curve.

Figure 7-22. Start of Reinforcement / Cooling Pipe Layout W2 Anchor.

Figure 7-23. Complete PVC Cooling Pipe and Reinforcement Layout W2 Anchor.

Figure 7-24. Footing Mock-up 10-ft Cube.

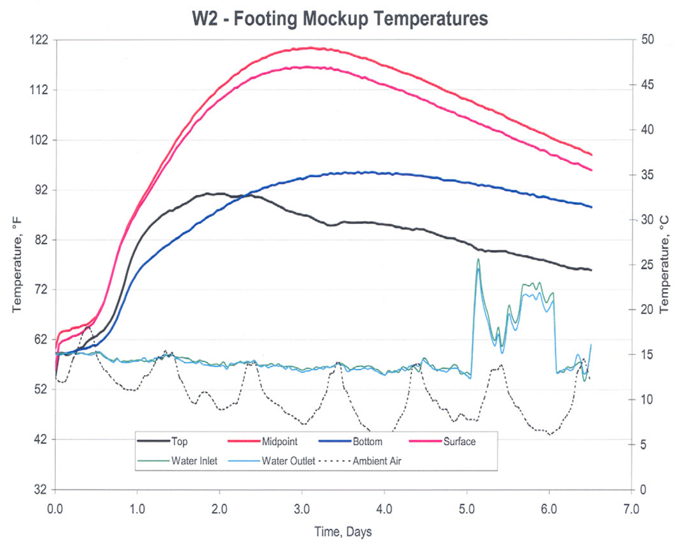


Figure 7-25. W2 Footing Mock-up Temperatures.

Benicia-Martinez Bridge Substructure and Foundation

The new 1.4-mile long Benicia-Martinez Bridge on Route 680 about 15 miles northeast of Oakland, California, has spans up to 660 ft. It was designed as a lifeline structure that will be serviceable after a major seismic event. The portion over water was constructed from 335 cast-in-place single-cell box segments made of high-strength, lightweight concrete. The segments were cantilevered from 11 piers, 10 of which were marine piers rising out of the strait.

The large 8.2 to 9.1 ft diameter piles and massive piers were made of normal weight HPC with a maximum water-cementitious material ratio of 0.40 to limit the permeability of the concrete and protect the reinforcing steel. The specified compressive strength was 5,000 psi for the pier footings, pile caps, and pier walls/columns and 6,500 psi for the pier tables.

There were over 200 normal-weight mass concrete pours. Most, but not all of the pours were cooled with 3/4-inch diameter PVC cooling pipes spaced 2 to 5 ft apart. Overall, thermal control in the substructure and foundations were adequate even though a few placements did exceed the 149°F specified limit, with three placements exceeding 160°F. Cooling pipes were effective, if not indispensable, for these normal-weight mass concrete elements.

Cast-in-Place Segmental Bridge Construction

In constructing a balanced cantilever segmental bridge, a cast-in-place segment, measuring 10 × 20 ft is placed, cured, post tensioned, and then used to anchor and support the traveling forms for the next segment. This is done until the span is constructed. Such construction lends itself to a contractor having concrete achieve a required stressing strength as soon as possible. This leads to a tendency to design high early strength concrete mixes. FHWA guidelines even include steam curing. This tendency if unchecked will lead to concrete mixes that can result in high temperatures during its curing. This turned out to be a particular issue with the lightweight high strength concrete for the Benicia-Martinez CIP segmental bridge. For the Caltrans segmental bridges at Confusion Hill in District 1 and the Devil's Slide Bridge in District 4 that followed, the specifications were written with the idea of limiting concretes that produce enough heat to be Mass Concrete.

Lightweight Concrete Segments & Mass Concrete in Elements with Dimensions as Small as 10 inches

The new Benicia-Martinez Bridge construction brought to light the very special thermal properties of high-strength lightweight concrete. With a high cementitious material content of 980 pounds per cubic yard coupled with a low fly ash percentage of 5%, 10,000 psi lightweight concrete used in the segments generated more heat than the normal weight concrete used in the substructure. Based on past experience, it was not anticipated that the decks, which were as thin as 0.83 ft, or the 1.8 ft thick stems would require thermal control. Only soffits that were greater than 3 ft thick were anticipated to require thermal control. The standard 7 ft requirement for mass concrete was reduced to 3 ft in anticipation of the very high cement content which was induced by the economic incentive to produce a very high early strength in the CIP segment so as to be able to stress and move the traveler forms as soon as possible. After the first placements and continued thermal analyses, however, it became apparent that the temperature rise would be an issue with elements much smaller than 3.3 ft, in fact, all of the high-strength lightweight concrete making up the box cell segments needed thermal control. The entire high-strength lightweight concrete was designated as mass concrete with thermal controls required that limited the peak temperature limit to 160°F. Cooling pipes were used throughout the structure, except in some deck and soffit sections near mid-span. The sections without cooling pipes did use liquid nitrogen as a passive control. The soffit manifold, shown in Figure 7-26, used blue hoses for inflow and red for outflow.

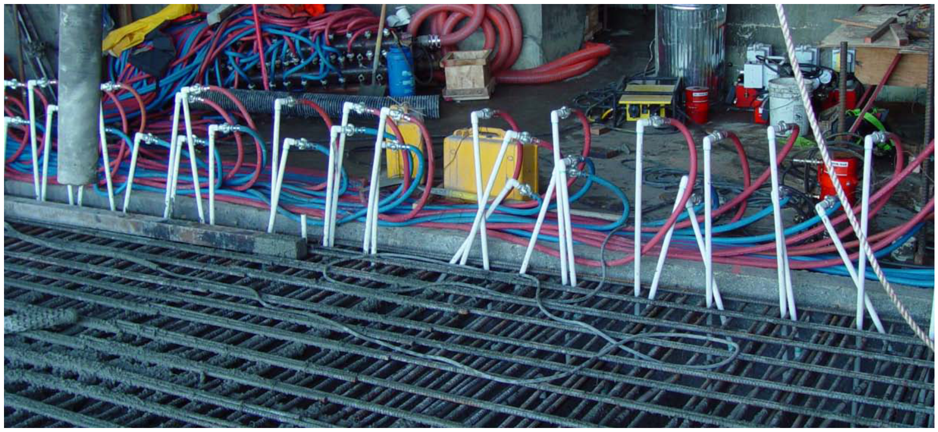


Figure 7-26. Soffit Cooling Pipe Manifold.

The first segment was cast December 31, 2004, and the last was cast October 23, 2006. Of the first 20 segments cast, 15 had elements exceeding 160°F, with four elements over 175°F. Additional temperature measurements at more locations within the segments were taken as construction proceeded. In February 2005, the only segment cast without any cooling pipes reached an alarming 196°F. This was the tenth segment cast, and it became apparent that all segments, though not all deck or soffit components, needed cooling pipes spaced per thermal analyses in addition to precooling to 45°F. After the active controls were implemented, a few element temperatures exceeded 160°F due to occasional problems with the pumping system, but most peaked below 131°F. Precooling with liquid nitrogen and post-cooling with water pumped from the strait flowing through PVC pipes performed quite well, and the cooling pipes appeared to be indispensable for this mix design.

The high strength lightweight concrete for the segments had particular mass concrete properties. The effect of the lightweight aggregate can be seen in the graph shown in Figure 7-27, depicting the temperature profiles of two 3.3 ft concrete cubes. The only difference between the cubes is that lightweight coarse aggregate in one block was substituted with normal weight aggregate on an equal volume basis. All other ingredients were the same (the water and air in the coarse aggregate are assumed to be part of the coarse aggregate).

Both cubes had the same amount of cementitious material and generated heat at the same rate in the same environments; but the cube with the lower mass rose to a signifi antly higher temperature. The variation in temperature occurs because specific heat is the heat capacity per unit mass of a substance. The concrete blocks had almost identical specific heats, but the lightweight concrete had a lower mass. The temperature rise varied inversely with the mass; a larger mass quantity will result in a lower temperature rise as there is more mass to absorb heat.

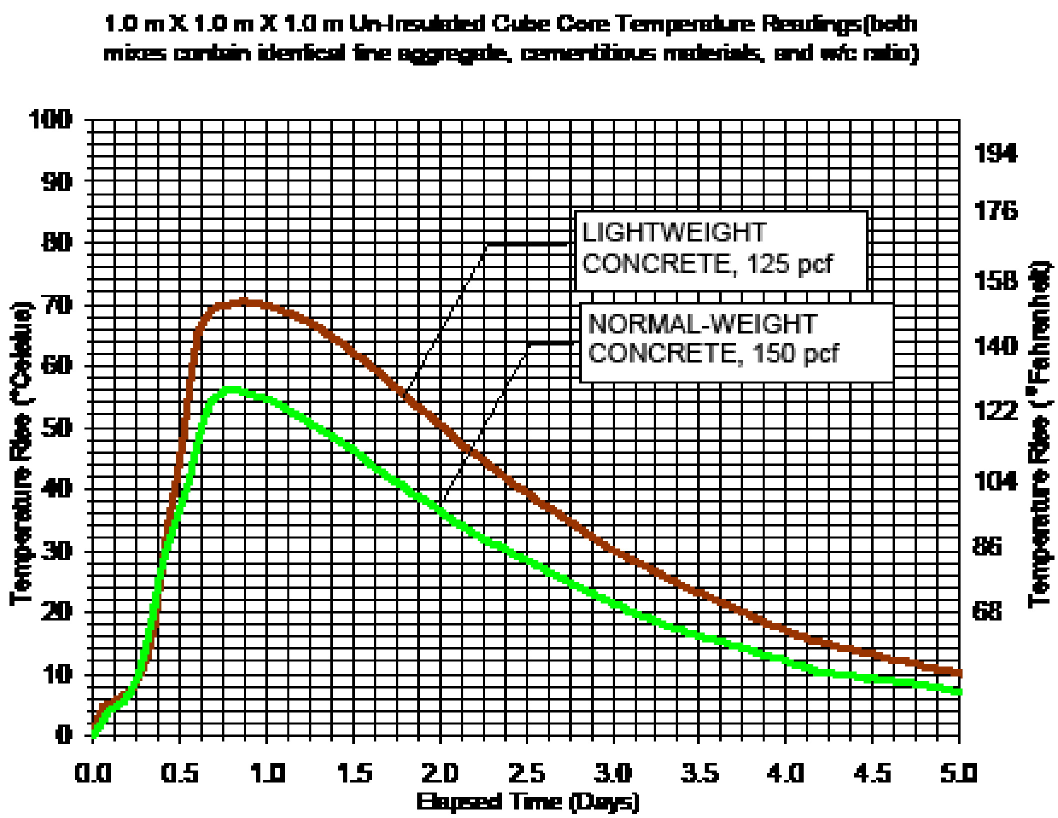


Figure 7-27. Un-insulated Concrete Temperature, Light Weight vs. Normal Weight.

The heat rise ratio and unit weight ratio of lightweight and normal weight concrete are similar. With a unit weight of 125 pcf for the lightweight and 150 pcf for the normal weight, the lightweight cube is 83% (125/150) of the mass of the normal weight cube of the same dimensions. The temperature rise for the lightweight was 128°F, from initial temperature of 66°F to peak of 194°F, while the temperature rise for the normal weight was 99°F, from initial of 66°F to peak of 165°F. The difference in mass resulted in the normal weight having 77% (99/128) of the temperature rise of the lightweight cube.

Since specific heat is the ratio of energy to temperature change for a given material per mass, the energy in the case of the two 3.3 ft cubes is the release of energy from the hydration process. The same cement content and the same type cement (given it was done the same day) are in each cube. This gives the same energy released due to the heat of hydration in each 3.3 ft cube. A similar experiment was performed on 5 ft insulated cubes, but in an effort to mitigate for different amounts of water absorption by aggregates that could affect heat conductivity to the external environment, insulation was placed around both cubes. As shown in Figure 7-28, the heat rise of the normal weight was 83% of the lightweight, exactly the percent of lightweight unit weight to the normal weight concrete. A lesser mass, though possibly having a slightly different specific heat and conductivity value due to water and air in voids in the lightweight aggregate, will have a higher temperature rise. This results in higher temperatures in the low-density concrete. This is an important lesson to keep in mind in pre-identifying mass concrete even though unit weight accounts for this in the equations to predict temperatures.

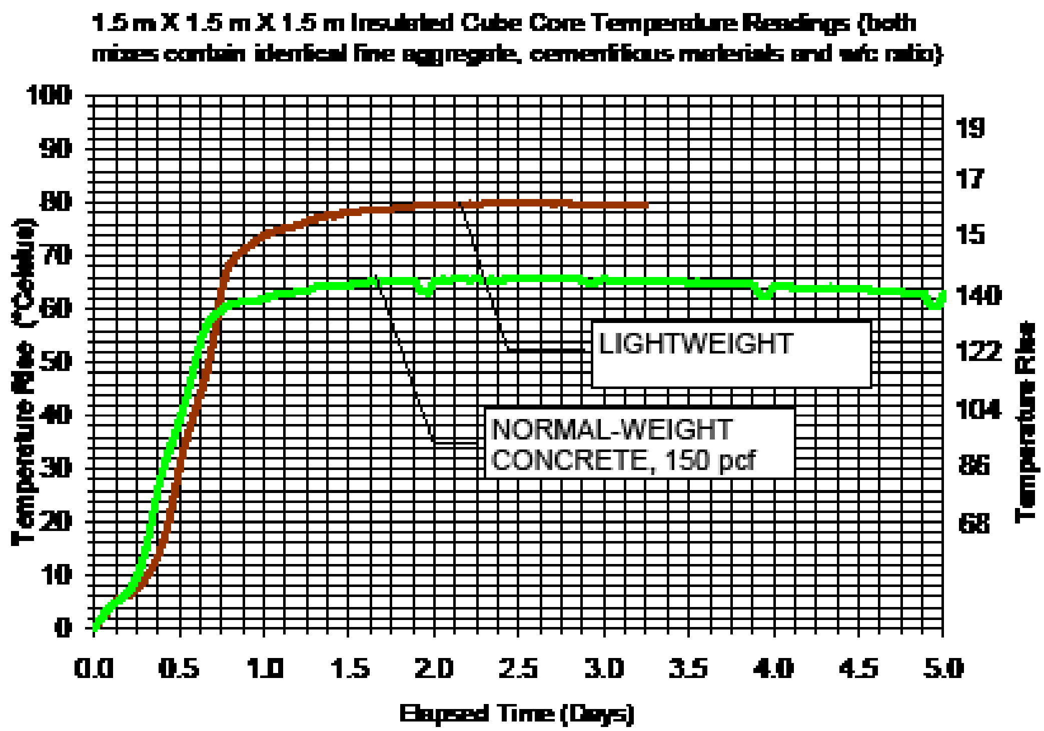


Figure 7-28. Insulated Concrete Temperature, Light Weight vs. Normal Weight.

Figures 7-29 through 7-32 show the thermal histories of the various segment elements. The frequency curve of peak temperatures of stems, soffits and decks are shown. Note the higher temperatures of the deck elements though these are the thinnest sections. This is because cooling pipes were not in many of the deck elements. In contrast cooling pipes were used in about half of the soffits and 318 out of 335 stem pairs. Only the first 17 of 335 stem pairs did not have cooling pipes.

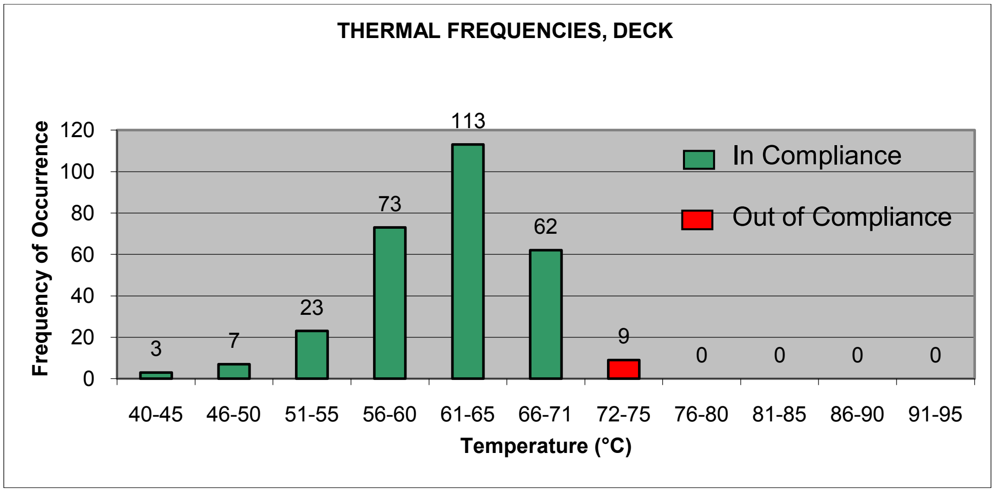


Figure 7-29. Deck Temperatures.

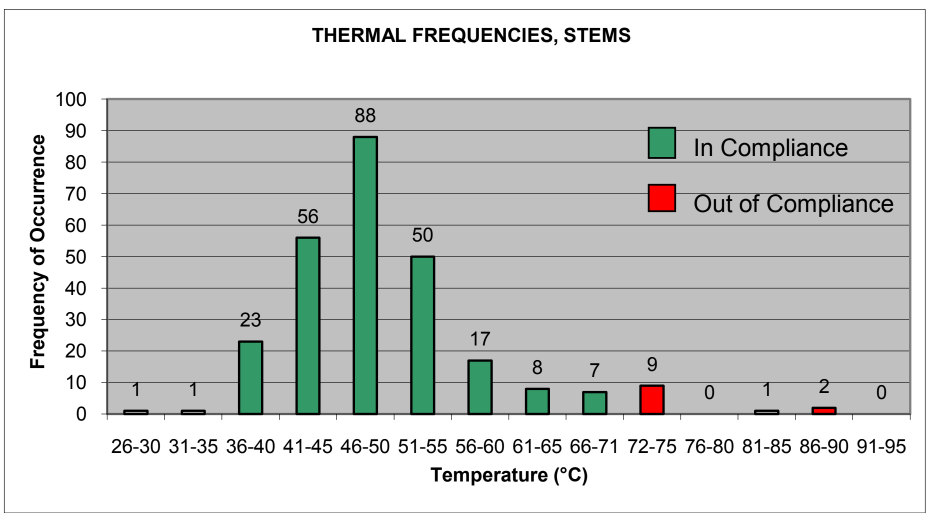


Figure 7-30. Stem Temperatures.

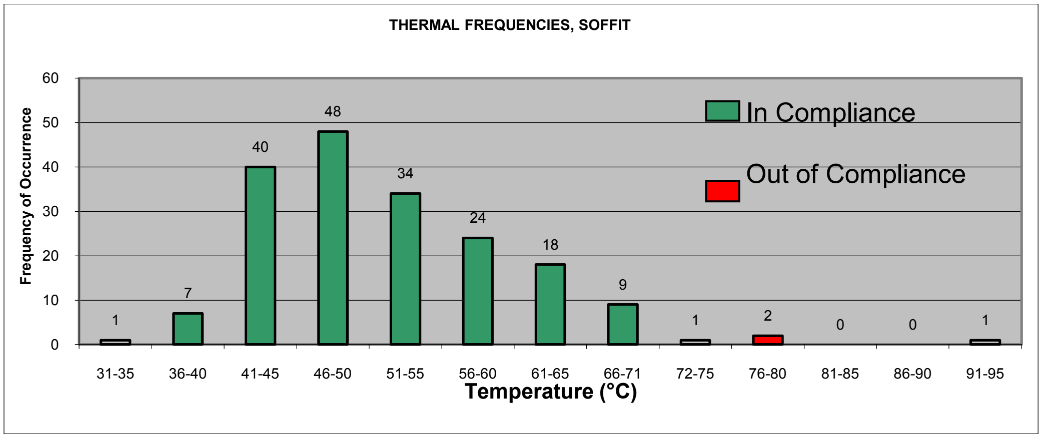


Figure 7-31. Soffit Temperatures.

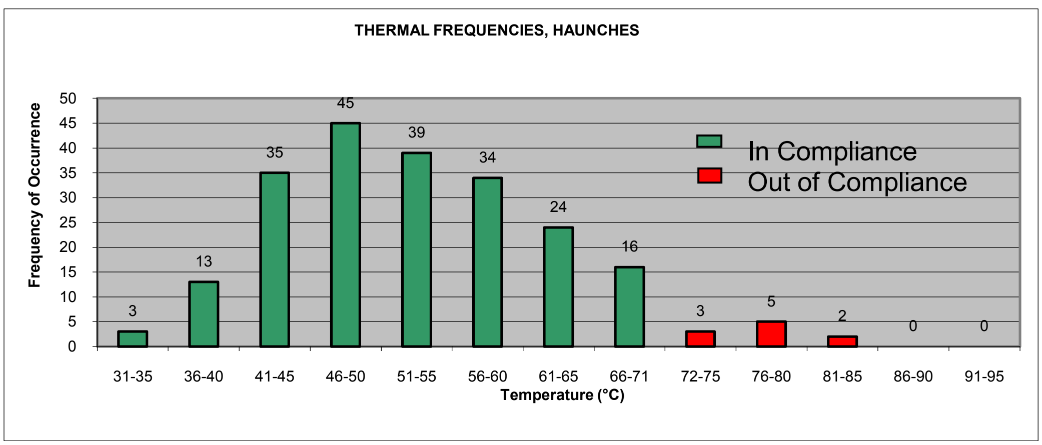


Figure 7-32. Haunches Temperatures.

Devil's Slide / Confusion Hill: A Laconic Approach to Mass Concrete Specifications

Though the issues with the cast-in-place segmental construction of the Benicia-Martinez Bridge were serious, it was recognized that the project's mix design had special thermal characteristics as previously discussed. It was deduced if fly ash was at the typical 25% and cementitious did not exceed Standard Specifications limits of 800 lb/yd^3, elements having a least dimension less than 4 ft would not have a temperature concern. This proved to be a correct assessment when this specification was enforced on the Confusion Hill and Devil's Slide Segmental Bridges. To reduce the incentive for a contractor to gain strength as quickly as possible so as to stress each segment as soon as possible in order to speed up the schedule, a 160°F maximum temperature was specified for any segment. Per specification temperature measurements would be taken in each segment at three locations chosen by the engineer. The maximum measured temperature taken during construction was 135°F for the Confusion Hill Bridge (Figure 7-33) and 142°F for the Devil's Slide Bridge (Figure 7-34). Concrete stressing strength requirements were met within 2 days and sometimes as early as 1 day. The 42-day compressive strength for the segments, which was at 6,100 psi, was exceeded without the need of thermal controls.

Figure 7-33. Confusion Hill Segmental Bridge.

Figure 7-34. Devil's Slide Segmental Bridge.

Acceptance of Active Controls And Return to Passive Control

Given the many factors influencing the characteristics of mass concrete, the methods used to cope with heat are a matter of economics as well as expertise. As contractors became more adept at using cooling pipes to control temperatures in large or high-performance concrete bridge elements, they were selecting this control method on many other Caltrans bridge projects, including some having traditional compressive strength requirements. This was not cost effective.

The Oakland Touchdown is a low-level, post-tensioned, cast-in-place concrete box girder bridge that connects Oakland to the Skyway. In a cost savings move when some large concrete elements were not identified as mass concrete in the bid document, Caltrans initiated change orders that replaced the contractors active thermal control system, which used internal cooling pipes, with a passive thermal control plan based on 50% fly ash concrete mixes. Mock–ups were done to ensure the anticipated outcome occurred in the actual placements. The 1,080 ft long structure has 7 spans over 6 piers. Under the piers are mass concrete pedestals, which sit on mass concrete pile caps that make up the footing. The pile caps vary in size having a footprint from 46 ft square to 52 × 72 ft. The mix had 337 lb/yd^3 of fly ash and 337 lb/yd^3 of Portland cement. The water to cementitious materials ratio was 0.4, the maximum permitted by the specifications for corrosion control. The strength requirement for the pedestal was 5,000 psi at 90 days while the pile cap concrete was 4,350 psi. The average measured strength for all the pedestals was 4,620 psi at 28 days and 5,720 psi at 56 days. Figure 7-35 below shows the 28-day and 56-day strengths for the pile caps. The lowest strengths occurred on samples stored during a 2 to 3-month period when temperature control of the curing room was malfunctioning. A few samples tested at 7 days had average strengths of about 3,000 psi. Concrete from 4 pedestals had average 90-day strengths of 6,225 psi. One 180-day test result was 6,830 psi.

Similar action taken for same reason on the Doyle Drive Bridge in San Francisco resulted in a substantial savings when compared to the contractor proposed thermal controls. For this project a combination of active and passive measures was used. The active system used a limited amount of cooling pipe in the columns (1 to 3 pipes down the approximate longitudinal central axis) along with a low heat mix using limited cement content and 30% fly ash. Savings were also obtained for the footings for the Yerba Buena Island Transition Structure by substituting passive measures for active. Caltrans took responsibility, thus removing risk to the contractor of high temperature issues because of the high degree of confidence in the selected thermal control measures, as was the history prior to the use of High Performance Concrete in mass concrete.

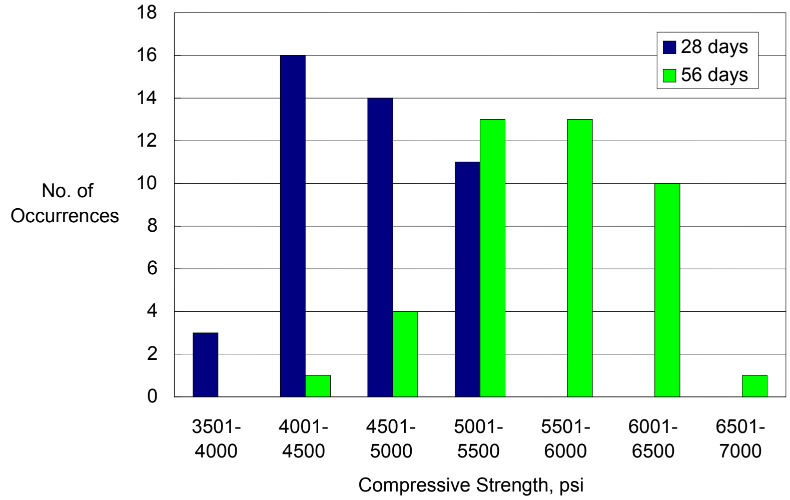


Figure 7-35. Pile Cap Concrete Oakland Touchdown.

Back to Prescriptive Passive Control Specifications & Mass Concrete for Concrete Piles

The data from the larger CIDH piles in the Benicia-Martinez Bridge suggested passive measures might be adequate for CIDH concrete. The pile that exceeded the temperature limits had high cementitious contents. Piles are cylindrical and thus having relatively high surface area to volume dissipate heat to the environment more readily than other common shapes. Also ground temperatures beyond 20 ft deep can be assumed to be no higher than 50°F. Since an assumption for ground temperature can be reliable, researchers at San Jose State University were able to use a finite element analysis, the Schmidt model and actual Q-Drum measurements (heat of hydration for various cementitious contents and combinations), to develop mass concrete specifications for CIDH pile concrete that kept concrete within temperature limits while curing. Subsequently, the accuracy of the calculations was verified against real data where the calculations reasonably matched the results.

The new standard specifications regarding mass concrete for CIDH piles are based on the information generated by the San Jose State research project shown in Figure 7-36.

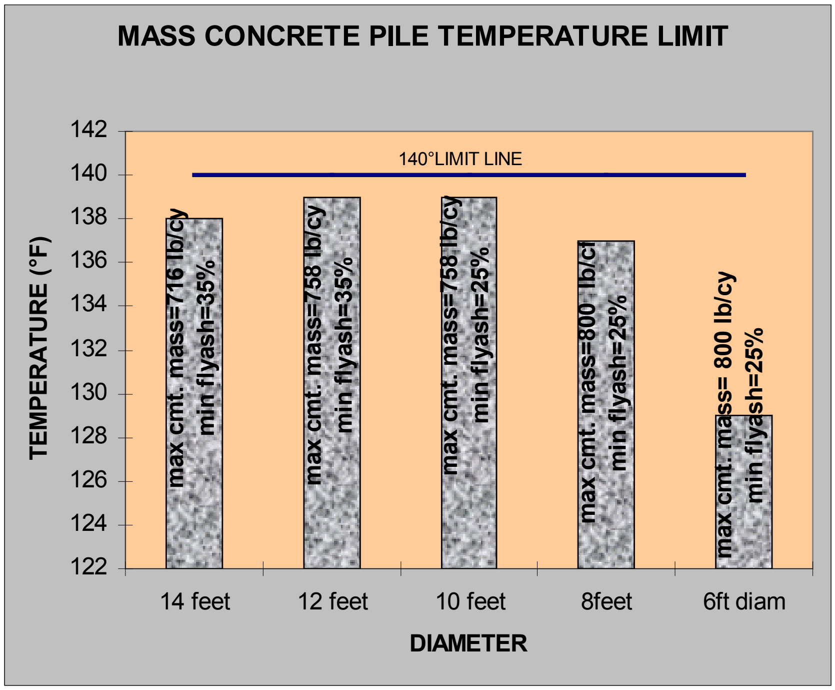


Figure 7-36. CIDH Pile Cementitious Material by Pile Diameter.

The complete set of data is shown in Table 7-4. The specifications require more restrictions on cementitious as the pile diameter increases. Beyond 14 ft in diameter, performance specifications for mass concrete require thermal control plans, mockups, temperature monitoring, and penalties for non-compliance. At the contractor's option, performance specifications may be used for any pile larger than 7 ft in diameter.

Table 7-4. CIDH Pile Cementitious Quantities.

Diameter	Maximum Cementitious		Mimimum Fly ash (%)			
14 feet	kg/ m^3	lb/yd^3	degrees F 15% fly ash	degrees F 25% fly ash	degrees F 35% fly ash	degrees F 50% fly ash
	300	506	126	118	115	112
	350	591	138	127	121	115
	400	675	146	137	133	118
	interpolated 425	717	151	142	138	123
	interpolated 450	760	156	147	143	128
	interpolated 475	802	161	152	148	133
12 feet	300	506	122	116	112	109
	350	591	133	124	119	112
	400	675	141	134	129	116
	interpolated 425	760	146	139	134	121
	interpolated 450	802	151	144	139	126
	interpolated 475	892	156	149	144	131
10 feet	300	506	118	112	109	105
	350	591	128	120	115	109
	400	675	136	129	124	112
	interpolated 450	760	146	139	134	122
	interpolated 475	802	151	144	139	127
8 feet	300	506	113	108	105	102
	350	591	118	115	110	104
	400	675	129	122	118	108
	interpolated 450	760	139	132	128	118
	interpolated 475	802	144	137	133	123
6 feet	300	506	107	103	101	98
	350	591	111	108	104	99
	400	675	120	114	111	103
	interpolated 450	760	130	124	121	113
	Interpolated 475	802	135	129	126	118

Mass Concrete Conclusions

For durable concrete the temperature of curing concrete needs to be evaluated prior to placement. This is a necessary step to ensure the structure will achieve its design life. Caltrans has a vast amount of experience in mass concrete control and has helped to advance the theory and practice. If active or passive thermal control measures are needed beyond normal practices to prevent the concrete from exceeding 160°F, then the concrete is defined by Caltrans as Mass Concrete. The theory and practice of mass concrete is important for evaluation of thermal control plans submitted by contractors, the assessment of plans and specifi ations to determine if mass concrete is applicable, and the determination by Caltrans of the thermal measures to be taken by the contractor when cost effective.

Lightweight Concrete

Lightweight concrete is similar to standard concrete, except that lightweight aggregates (35-70 pcf) are used, resulting in concrete that weighs less than 112 pcf for normal strength concrete and 120-125 lb/ft^3 for strengths above 6,000 psi. Dead loads for girders and bridge decks are lower, offsetting the higher material costs because the spacing of girders and supporting elements such as piers can be increased and foundation demands may be reduced. Seismic loads may also be reduced. Mix designs include Portland cement and SCMs which may reduce permeability and help reduce unit weight, lightweight aggregate, water and admixtures. Lightweight aggregates included in the mix design may be natural like pumice or manufactured by processing natural materials such as expanded shale, slag, clay, pelletized fly ash, or slate into aggregate size particles with a high percentage of void space. Attention is required to specifications when checking density, as oven-dry density and room temperature air-dry density will vary.

The New Benicia-Martinez Bridge completed in 2007, built with balanced cantilever cast-in-place segmental construction used High Performance Lightweight Concrete to meet material design needs which included over water spans exceeding 600 ft as shown in Figure 7-36. The mix design listed in Table 7-5 also had to take into account being pumped as much as 180 ft high and as much as 300 feet horizontally. The f'_c was 6,500 psi, but the Modulus of Elasticity requirements of 3,400 ksi resulted in f'_c over 10,000 psi, shown in Table 7-6. Attention to proper batching and adequate contractor quality control testing resulted in only two failing unit weight quality assurance tests. The compressive strength history shown in Figure 7-37 also indicates consistent batch control.

Table 7-5. New Benicia Concrete Mix.

Material	lb/yd^3
Cement, Type II-V	833
Fly Ash, Class F	49
Metakaolin	98
Normal Weight Sand	1,233
Lightweight Aggregate	858
Water	304
w/cm ratio	0.31

Table 7-6. Lightweight Concrete Properties.

Property	Specified Value	Average Measured Values
Density, lb/cu ft	125±2	125.2
Compressive Strength, psi	6,500 at 28 days	10,370 at 35 days
Modulus of Elasticity at 28 days, ksi	3,400 min.	3,800
Shrinkage after 180 days, %	0.05 max.	0.042
Specific Creep after 365 days, millionths/psi	0.48 max.	0.22
Splitting Tensile Strength psi @ 28 days	450 min.	490 (field tested > 28 days)

Figure 7-37. New Benicia-Martinez Bridge During Construction.

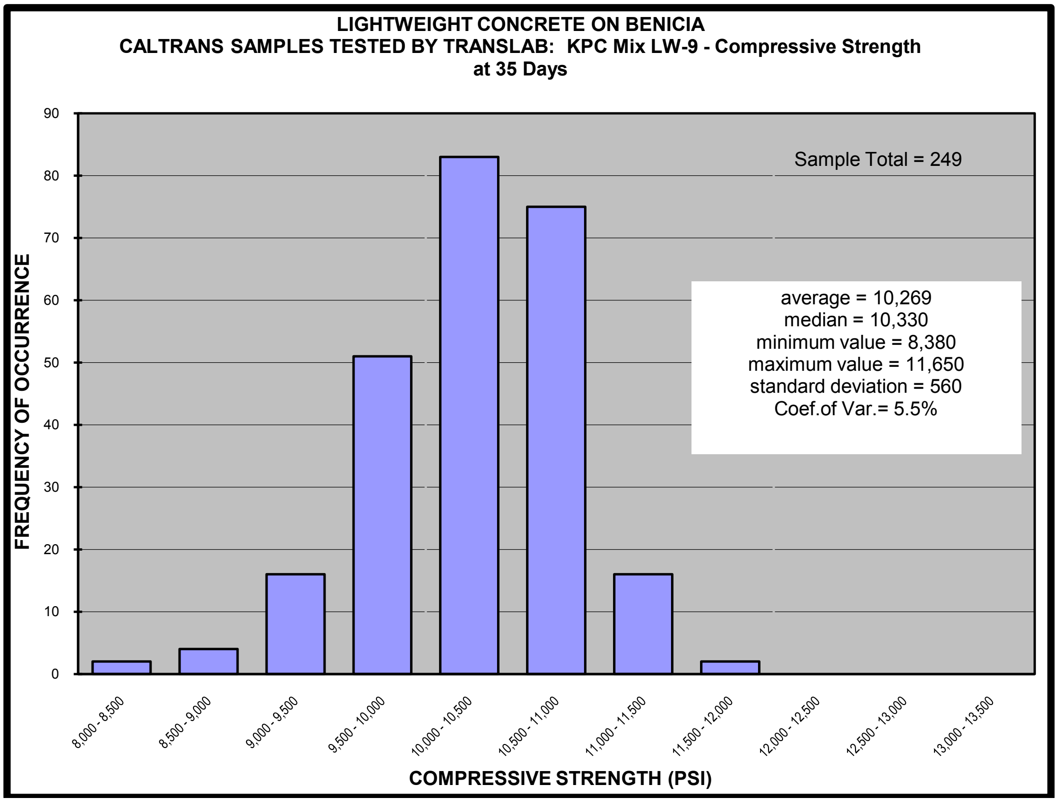


Figure 7-38. New Benicia-Martinez Bridge Concrete Strength History.

Shrinkage Compensating Concrete

After fresh concrete hardens and loses water, shrinkage can occur that will cause surface tension that leads to development of surface cracks. Historical shrinkage controls focused on mix design and curing controls to limit overall shrinkage or reduce the shrinkage rate. The water-cementitious ratio was reduced; free water ultimately bleeds and evaporates, leading to shrinkage. Stiffer aggregates were effectively combined with reduced cementitious materials to limit shrinkage. Cutting expansion joints in newly hardened concrete caused cracking to occur at the joints, leaving the remainder of the panel relatively unaffected by shrinkage. Proper curing practices also limit evaporation-induced shrinkage as does prolonged curing times. The method currently used to control shrinkage is inducing expansion as concrete hardens to compensate for subsequent shrinkage that occurs as concrete dries by use of expansive cement or a shrinkage-reducing admixture.

Expansive cement, containing calcium aluminate (ASTM C 845 Type K), expands after initial set to offset anticipated drying shrinkage. Type K expansive cement is not specified for Caltrans projects. The expansion creates tension in reinforcing steel and compression in concrete. Concrete produced with shrinkage compensating cement possesses properties similar to standard Portland cement concretes. Air-entraining admixtures are also equally effective for freeze-thaw durability. Chemically, the calcium aluminate hydrates with calcium sulfate to produce calcium aluminate sulfate hydrate. The chemical equation is shown below.

$$CA + 3CS + 2CH + 30H \rightarrow C_6AS_3H_{32}$$

Table 7-7 is a summary of concrete characteristics when comparing calcium aluminate concrete to standard Portland cement concrete.

Table 7-7. Type K vs. Portland Cement.

Characteristic	Shrinkage Compensating Cement
Water Demand	Higher
Consistency	Stiffer
Cohesiveness	Better
Final Set	Quicker
Strength	Better
Resistance to abrasion	Better
Sulfate resistance	Similar to Type V Portland Cement

Type K cement requires attention to reinforcement during design to ensure adequate but restrained expansion. The minimum recommended cementitious is 515 lb/yd^3. The inclusion of calcium sulfate leads to higher ettringite formation, which in turn requires additional water and is the source of expansion. Ettringite formation causes fresh concrete to stiffen. Pozzolans may have a negative effect on expansion and should be checked prior to use. As the cohesiveness increases, the risk of segregation decreases. Caution must be taken during placement during extreme weather conditions to avoid water loss because the early stiffening as plastic cracking may occur. After placement, moist cure is required for 7 days to ensure expansion during hydration.

Unlike expansive Type K cements, shrinkage reducing admixtures (SRA) control early-age and long-term shrinkage by reducing the surface tension of fresh concrete. When used, SRAs may slightly retard initial and final set. As expected, when set is retarded, the heat of hydration may also be reduced. Table 7-8 lists SRA effects on concrete.

Table 7-8. SRA Effects on Concrete.

Characteristic	SRA Use vs. Standard Portland Cement Concrete
Shrinkage	30 – 80% reduction
Heat of Hydration	Lower
Final Set	Slower
Strength	Lower to equal
Creep	Lower to equal
Chloride Permeability	Lower to equal
Resistance to frost action	Similar when air entrainment is used

Durability

The construction of expensive structures in various environments with corrosive and abrasive forces requires high durability concretes where design life can exceed 100 years.

- Winter weather with freeze-thaw cycles and heavy vehicle traffic possibly with tire studs or chains results in bridge deck with extreme wear in the wheel pathways.
- Reducing permeability limits the speed that corrosives like chlorides and sulfates can diffuse through concrete pores and attack reinforcing steel. Fine SCMs like silica fume, ultra fine fly ash, and metakaolin combined with low water-cementitious ratios result in concrete with low permeability, which resists flow of corrosive materials.

.

A low water-cementitious ratio is the most important criteria. Where durability is a concern, the water-cementitious ratio should be below 0.4. Carbonation (the combination of calcium oxide with carbon dioxide) which eliminates the protective passivated coating on reinforcing steel, is also impeded by dense concrete. The use of SCMs also increases the electrical resistance of concrete, which inhibits the electrical circuit that develops as corrosion occurs. The presence of corrosion pathways caused by micro-cracking is also reduced with SCM usage. Micro-cracks are caused by variation between surface and internal temperatures while curing is minimized by maintaining a constant temperature throughout the curing process.

Fiber Reinforced Concrete

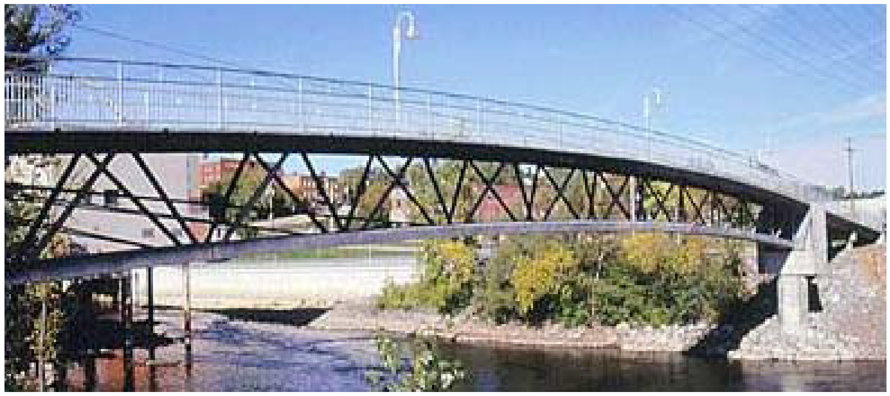
Fiber reinforcement is the addition during mixing of fibrous materials such as steel, plastic, alkali-resistant glass or cellulose to a concrete mix. The use of fiber reinforcement is steadily increasing as a means of increasing concrete durability. The earliest use of fiber dates back to the ancient Egyptians, who included straw in mud bricks for additional strength. After a concrete mix hardens, the fibers bond with hardened concrete and depending on physical properties, provide additional reinforcement. Fibers range from 0.25 to 6 inches in length; they usually have circular or rectangular cross sections, and may be shaped as bars or crimped. Steel fiber use is classified by volume, Low – < 1%, Moderate – > 1% and <2%, and High – >2%. Synthetic fibers with a much lower density seldom exceed 0.2% of volume. When used, fibers improve three-dimensional stability; compressive strengths have exceeded 30,000 psi. Fibers reduce cost by eliminating reinforcing steel but make the mix design less workable. Compared with steel reinforcing, fiber reinforced concretes are less susceptible to chloride attack and carbonation and more durable in freeze-thaw environments.

Reactive Powder Concrete

The Reactive Powder Concrete (RPC) process was patented under the trademark name Ductal® in 1994. Mixes are characterized by compressive strengths in the range of 25,000 to 35,000 psi and flexural strength in the range of 4,000 to 7,000 psi. The surfaces are abrasion resistant. Permeability is very low, with high resistance to carbonation, chloride and sulfate attack due to the small pore structure which also provides for good freeze-thaw durability.

The mix design consists of Portland cement, silica fume, crushed quartz flour, fine silica sand, water, high-range water reducing admixtures and reinforcing by steel or organic fibers. The maximum particle size is 600 microns, and the overall gradation is optimized for maximum density. The low water-cementitious ratio (<0.2) requires use of high range water reducers for workability.

RPC structural elements are more compact and lighter in weight that standard Portland concrete elements. The first bridge made with RPC was a pedestrian bridge in Sherbrooke, Quebec built in 1997. The bridge, a segmental structure, consisted of six precast segments for a 198-ft span as shown in Figure 7-39.



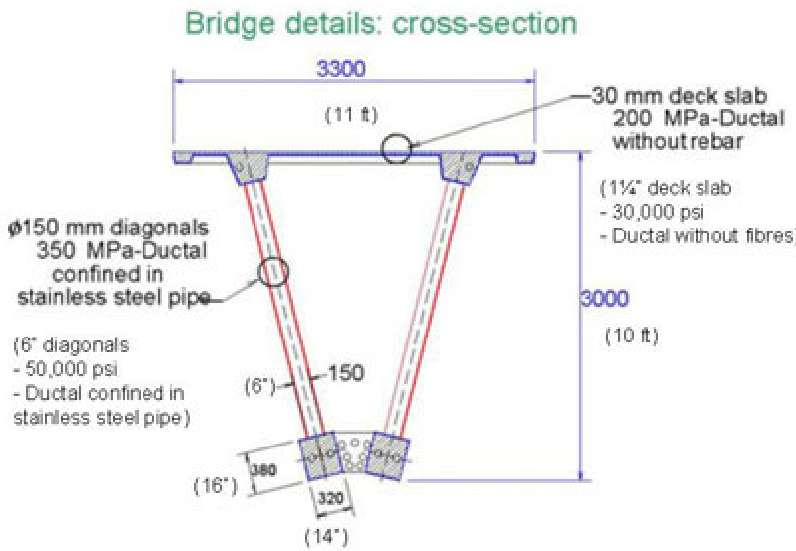


Figure 7-39. Sherbrooke Pedestrian Bridge.[5]

[5] http://www.pci.org/view_file.cfm?file=JL-99-SEPTEMBER-OCTOBER-6.pdf (visited 6/10/10)

CHAPTER 8
MATERIAL SAMPLING AND TESTING

Table of Contents

Material Sampling And Testing **8-1**

Introduction **8-1**

Test Types **8-1**
Initial Samples And Tests 8-2
Acceptance Tests 8-2
Independent Assurance Tests 8-2
FHWA Samples And Tests 8-2
Test Frequency 8-2

Sample Identification **8-3**

Acceptance Testing and Sampling **8-3**
Certification of Project Personnel 8-5

Sampling and Testing Aggregate **8-5**
Test Methods for Aggregates 8-6
Sieve Analysis (California Test 202) 8-7
Percentage of Crushed Particles (California Test 205) 8-7
Resistance to Abrasion Loss (California Test 211) 8-7

Soundness and Durability (California Tests 214 and 229) **8-8**

Sand Equivalent (California Test 217) **8-8**

Surface Moisture (California Test 223) **8-9**

Specific Gravity of Coarse Aggregate (California Test 224) **8-10**

Specific Gravity of Fine Aggregate (California Test 225) **8-10**

Moisture Content by Oven Drying (California Test 226) **8-10**

Cleanness of Coarse Aggregate (California Test 227) 8-10

Alkali Reactivity of Aggregates 8-10
Short Method, ASTM C 1260 8-11
Long Method, ASTM C 1293 8-11

Material Sampling 8-11
Sampling Cementitious Materials 8-11

Sampling Water 8-13

Sampling Admixtures 8-13

Sampling for Polyester Concrete 8-14

8 MATERIAL SAMPLING AND TESTING

Introduction

Chapter 6 of the Caltrans Construction Manual contains information and instructions that govern materials sampling and testing on State highway projects. The administrative provisions and all sampling and testing methods and procedures included therein apply to all construction work including structure construction. Bridge engineers are expected to become familiar with the Construction Manual provisions that apply to sampling and testing of structure concrete materials.

In general, materials used to produce concrete from commercial plants in urban areas will be sampled on a routine basis by personnel assigned to the District Materials Laboratory (Lab), and test results made available to construction projects. For concrete plants in rural areas or projects where portable plants are used, samples will usually be taken by project personnel. Depending on circumstances, to properly control the work it may be necessary for project personnel, including bridge personnel, to sample materials on any project at any time. Accordingly, bridge field engineers must be familiar with sampling procedures and test methods that are applicable to structure concrete work.

Personnel handling, sampling or testing concrete materials need to follow all applicable safety regulations, including Office of Structure Construction (OSC) Bridge Construction Records and Procedures Manual, Section 14, OSC Code of Safe Practices, Caltrans Safety Manual, CalOSHA Construction and General Industry Safety Orders, and the plant's safety regulations and safety precautions addressed in California Test 125.

Test Types

The various tests performed on concrete materials, and all other materials used in the work as well, are classified according to the purpose of the test. Field tests are performed under supervision of licensed engineers in strict accordance with the Code of Professional Conduct of the California Board of Professional Engineers and Land Surveyors. Bridge field personnel are typically concerned with four types of testing.

Initial Samples And Tests

These tests (also sometimes referred to as Process Control Samples and Tests) are made after the award of a contract to determine whether a particular material, such as concrete aggregate, is suitable for use.

Acceptance Tests

These tests are made to verify that the materials being used in the work meet contract requirements. Samples may be taken by lab or construction personnel. Aggregate samples are processed and tested in the District Materials lab or in a construction field lab. Other materials (cementitious materials, SCMs, water and admixtures, if used) will be tested in the Headquarters Offices of Materials Engineering and Testing Services (METS) labs in Sacramento.

Independent Assurance Tests

These tests are made on samples taken (or witnessed) by personnel who are not assigned to the project to verify the reliability of the acceptance test results. These tests are not used to verify compliance with contract requirements. Samples are processed and tested in the District Materials lab or the METS labs in Sacramento, as appropriate for the material sampled.

FHWA Samples And Tests

On Federal-aid projects the FHWA reviewer may request that a particular material be sampled and tested. Such samples are taken as directed by the FHWA, marked for identification as “FHWA Check Sample” and sent for testing to the appropriate District or Headquarters lab.

Test Frequency

The required test frequency for materials used in highway and structure construction is found in Chapter 6 Sampling and Testing of the Construction Manual.

The sample frequency shown in the tabulation is intended as a guide for minimum testing under normal conditions. When materials being furnished on a routine basis are uniformly and consistently within contract requirements, a prudent decrease in the frequency interval may be warranted; however, adequate documentation describing the basis for lower testing frequency needs to be filed in the project files. Materials of marginal quality or where past results are erratic, or materials furnished on an intermittent basis, may require more frequent testing to ensure contract compliance.

Sample Identification

When submitted for testing, all samples must be accompanied by a sample identification card, which is Form TL-101 for all samples (aggregate, water and admixtures) except cementitious materials. The sample identification card for cementitious materials is Form TL-518, which is available from the District Materials Lab.

Form TL-101 is reproduced in Figure 8-1. Although the form is generally self-explanatory, care must be taken to ensure that all entries are completed and that the information entered is correct.

All Form TL-101s are to be marked “normal” or “priority”. Under current practice, the priority designation is used for the first few samples of each construction material submitted for testing, and at any other time when expeditious testing and reporting of results are necessary to properly control the work.

Normally, the lab will e-mail the test results. However, the lab will send results via telephone or FAX results, if requested. If this service is desired, it should be indicated in the remarks section of the sample identification card along with the telephone or FAX number of the person to notify.

Acceptance Testing and Sampling

Present policy requires representative sampling and testing of contractor-furnished materials used in the work. Such tests, because they form the basis of acceptance of the materials, are referred to as “acceptance” tests.

Because acceptance test results are used to accept or reject material, the importance of accurate, representative sampling cannot be overemphasized. It should be obvious that unless the sample is truly representative of the material to be tested, the test will apply to the sample only, and not to the material from which the sample was taken. Likewise, standard test methods and procedures must be followed to ensure the credibility of the test results.

Acceptance testing begins the first day that materials are used, and continues throughout the contract.

STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

SAMPLE IDENTIFICATION CA

TL-0101 (REV. 08/2009)

CARD NUMBER

C 703574

FIELD NO. B3 C7 #2

SAMPLE SENT TO:

- ☑ HDQTRS. LAB
- ☐ BRANCH LAB
- ☐ DIST. LAB
- ☐ PRIVATE LAB
- ☐ PRELIMINARY TESTS
- ☐ PROCESS TESTS
- ☐ ACCEPTANCE TESTS

PRIVATE LAB NAME:

EA: 07-4Y1401

District: 07 Bridge No: 53-1257

County: LA Bridge Name: Bellflower East

Route: 91

PM: 16.54

Sample Description: 3" diameter PCC core from column

Sample Qty: 1 Sample Length: 7"

Date Sampled: 1/28/2010 Date Submitted: 2/1/2010

Date Re'qd: 2/4/2010

Sampled By: Biman Roy

Title: SMR District 07

Contact Name: Michael Mifkovic

Phone #: 227-6939

ASTM Test Req'd: C42

CTM Test Req'd: 404

Test Desc: Compressive strength & chlorides

FORM TL 543 (REV. 3/85)

Certificate No.

VENDOR'S CERTIFICATE OF COMPLIANCE

☐ (PRECAST CONCRETE PRODUCTS) OR ☐ (READY MIXED CONCRETE)

To:

State Highway Engineer:

We certify that the portland cement, chemical and mineral admixtures contained in the material described below are brands stated and comply with specifications for:

CONTRACT NUMBER →

Cement Brand: ______ Mill Location:

TYPE: ______

CHEMICAL ADMIXTURE

1. BRAND: ______ MANUFACTURER: ______
   TYPE: ______
2. BRAND: ______ MANUFACTURER: ______
   TYPE: ______

CHECK BOX IF A CHEMICAL ADMIXTURE WAS NOT USED ☐

MINERAL ADMIXTURE

MANUFACTURER: ______

CLASS: ______

CHECK BOX IF A MINERAL ADMIXTURE WAS NOT USED ☐

Delivery Date (Ready-Mix) Dates of Fabrication (Precast)

List products to which this certificate applies. (Show size and lin. ft. of pipe, etc., delivery slip numbers for ready-mix.)

MANUFACTURER OF CONCRETE PRODUCTS

Orig. to Res. Engr.
Retain Duplicate.

By: ______

AUTHORIZED REPRESENTATIVE

91 91529

Figure 8-1. TL-0101 and TL 543 Forms.[1]

[1] New form is No.8-C78

Certification of Project Personnel

Under Caltrans policy, all project personnel who perform sampling and testing of materials used in the work must be prequalified by the District Independent Assurance (IA), and must possess a current Form TL-0111, "Tester Certificate of Proficiency."

Form TL-0111 is issued by the District IA, and lists the tests which the individual is authorized to perform. Form TL-0111, which is valid in all Districts, must be renewed every year.

While the District IA will issue a Form TL-0111 to qualified project personnel, including bridge personnel, the OSC Structures Representative assigned to the project is responsible to see that all bridge field personnel are qualified and possess a valid certificate before they perform any acceptance sampling or testing.

Sampling and Testing Aggregate

Most sampling of concrete aggregate, and virtually all testing, will be performed by District Materials lab or District construction field lab personnel. However, from time to time there may be circumstances that will require the bridge engineer to obtain aggregate samples; consequently, bridge engineers must be familiar with recommended sampling procedures, and with the applicable test methods as well.

Sampling methods and procedures shall be in accordance with California Test 125, Appendix A, and at sampling rates described in the Construction Manual.

Aggregate, particularly coarse aggregate, should be sampled as close as practicable to the point of incorporation into the work. This is important because segregation and degradation of the natural material along with the addition of deleterious substances may seriously reduce aggregate quality between the point of production and point of use. The Construction Manual lists the following sample locations in suggested order of preference:

- Conveyor belt between the weigh hopper and the central mixer or transit mix truck
- Conveyor belt feeding the batch plant bins immediately preceding the weigh hopper
- Weigh hopper discharge gate
- Discharge gates of bins feeding the batch plant weigh hopper

The required sample sizes for the various primary aggregate sizes are given in Chapter 6 of the Construction Manual. Note that when a sample taken from the weigh hopper discharge gate is to be used for a grading analysis, the sample size is approximately 400 pounds. Such samples are split or quartered to the test size.

Except for samples for grading analysis, samples from discharge gates should consist of the combination of three "grab-sample" increments taken from the entire stream of material. Obtain each increment at random from an amount of aggregate approximating the batch quantity, or mixer capacity if greater, for that aggregate size. The sampling container must be large enough to intercept the entire discharge, and must not overflow.

When sampling from conveyor belts, the sample should consist of the combination of three increments taken at random from an amount of aggregate approximating the batch quantity, or mixer capacity if larger, for that size of aggregate. Use of a pair of templates, preferably conforming to the shape of the belt, is recommended to isolate each increment on the belt. Care must be taken to recover all material between the templates, including all fine particles and dust. Note that the belt must be stopped while the samples are taken.

Sampling from storage bins and/or stockpiles is not permitted for acceptance testing because it is too difficult to obtain a representative sample.

Aggregates for polyester concrete shall comply with the Standard Specifications and the applicable standard special provisions. In such aggregates, low absorption, roundness, and gradation are controlled to minimize resin content. Coarse and fine aggregates are bagged separately. Sampling aggregates for polyester concrete are similar to sampling at Portland cement concrete batch plants and is covered by California Test 125, Appendix A.

When sampling aggregates all safety precautions need to be observed including awareness of hot aggregate and equipment, dust, ascent and descent of ladders and stairs, and lifting of heavy bags or buckets of aggregate samples. California Test 125 also contains helpful advice on safe practices of aggregate sampling. The plant's safety manual also needs to be reviewed for applicable safety regulations.

Test Methods for Aggregates

Tests normally performed in a District or construction field lab to check aggregate for compliance with specification requirements are briefly described in the following sections. The tests are identified by their California Test numbers. A complete description of each test may be found at Caltrans Materials Engineering and Testing Services – California Test Methods web site http://www.dot.ca.gov/hq/esc/ctms/index.html. Certain ASTM or AASHTO test methods may also apply or may be used. Such tests may be accessed at Caltrans Office of Structural Materials web site at http://onramp.dot.ca.gov/hq/oscnet/, click "Field Resources" and select "ASTMs, etc." from the drop-down menu. For reference, a table is provided below showing California Tests and comparable ASTM or AASHTO tests where available.

Table 8-1. California Test Methods for Aggregates.

Material	California Test	Comparable Test	Description
Coarse Aggregate	202	ASTM C136	Sieve Analysis
	205	ASTM D5821	Percent of crushed particles
	227	ASTM C117	Cleanness of coarse aggregates
	211	ASTM C131	Resistance to abrasion loss
	214	ASTM C88	Soundness (sodium sulfate)
	229	------	Durability index
	224	AASHTO T85	Bulk specific gravity (field method)
	223	------	Surface moisture (field method)
Fine Aggregate	202	ASTM C136	Sieve analysis
	217	ASTM D2419	Sand equivalent
	213	ASTM C40	Organic impurities
	515	------	Relative mortar strength
	214	ASTM C88	Soundness (sodium sulfate)
	225	AASHTO T84	Bulk specific gravity (field method)
	223	ASTM C70	Surface moisture (field method)
Combined Aggregate	202	ASTM C136	Sieve analysis
	226	ASTM C566	Moisture content by oven drying

Sieve Analysis (California Test 202)

This test is used to determine the particle size distribution of both fine and coarse aggregates by separation with standard sieves. Average time to be allowed for testing in the lab: 1/2 day.

Percentage of Crushed Particles (California Test 205)

This test determines the percentage, by weight, of particles that by visual inspection have the characteristics of crushed aggregates.

Resistance to Abrasion Loss (California Test 211)

For concrete used in highway and bridge construction, one of the most important factors in aggregate selection is the ability of the aggregate to withstand wear. This aggregate property is commonly referred to as "abrasion resistance". Abrasion resistance is often used as a general indication of overall aggregate quality.

The abrasion resistance of a given aggregate is determined by California Test 211, which is commonly known as the Los Angeles Rattler Test. This test measures the ability of coarse aggregate to resist disintegration caused by impact in a rotating cylinder containing small steel balls (note that only coarse aggregate is tested for abrasion resistance). Average time to be allowed for testing in the lab: 1/2 day.

Soundness and Durability (California Tests 214 and 229)

The term "soundness" is used to describe the ability of an aggregate to withstand splitting and fracturing when exposed to severe weather conditions. This aggregate quality is measured by California Test 214, which is referred to as the "soundness" test. This test is used to determine the soundness of both fine and coarse aggregates.

In California Test 214, a sample of aggregate is immersed in a sodium sulfate solution for at least 16 but not more than 18 hours. During this period salt crystal growth in the aggregate pores creates a pressure that is similar to that produced by freezing water. After the immersion period, the sample is oven-dried until a constant weight is obtained. The test sample is then cooled and the test repeated. Average time to be allowed for testing in the lab: 7 days

The immersion, drying, weighing, and cooling cycle is repeated five times. The sample is then shaken through sieves having openings one-half the size of those on which the aggregate was originally retained. The percentage loss is determined on each individual fraction of the original sample and a weighted average loss is calculated. Total material loss is limited to not more than 10%.

The Durability Index provides a measure of the relative resistance of an aggregate to producing clay-sized fines when subjected to prescribed methods of interparticle abrasion in the presence of water. California Test 229 provides direction for evaluating the Durability Index. Four procedures are provided for use with materials with various nominal sizes and specific gravities. Average time to be allowed for testing in the lab: 1/2 day

Sand Equivalent (California Test 217)

This test provides a rapid means of determining the amount of detrimental fine particles (silt or clay) present in a sample of fine aggregate. The test result is expressed in terms of a "sand equivalent" value, or "SE".

In this test, a test specimen prepared from the aggregate sample is placed in a graduated cylinder containing a calcium-chloride solution, and allowed to soak for 20 minutes. The cylinder is then secured in a mechanical shaker and agitated for 45 seconds. Additional solution is added, using an irrigator to flush the clay-size particles from the sample and into suspension in the solution. The cylinder is then allowed to stand undisturbed for 20 minutes. Following the 20-minute settlement period, the clay sediment and sand sediment height "readings" are determined in accordance with the procedure explained in the test method. The sand-equivalent value is the sand reading divided by the clay reading multiplied by 100, and then rounded up to the next higher whole number.

Since the sand-equivalent value is inversely proportional to the percentage of fine particles present in the sample, higher values indicate a better (cleaner) material. Average time to be allowed for testing in the lab: 1/2 day

Surface Moisture (California Test 223)

Surface moisture is defined as moisture in excess of the moisture present in the aggregate when the aggregate is in a saturated surface-dry condition.

This test is used in the field as a rapid means of determining the amount (percent) of surface moisture present in a sample of fine or coarse aggregate having a known specific gravity. The test requires only 2 or 3 minutes to perform and is accurate within about 0.2% (plus or minus) of the true value.

In this test a sample weighing approximately 18 lb is placed in a pail with enough water to cover the sample, and stirred to remove any entrapped air. The pail is then filled with water, suspended from a weighing device, and the pail with sample is immersed in a container of water, and weighed. The weight of the sample in water is equal to the weight in water of the pail and sample minus the weight in water of the pail alone (previously determined). From this information the approximate surface moisture percentage can be calculated, using the formula given in the test method instructions.

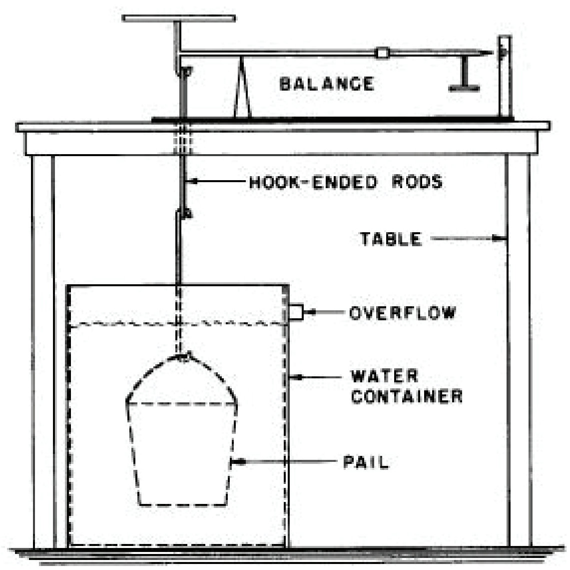


Figure 8-2. Surface Moisture Test.

Specific Gravity of Coarse Aggregate (California Test 224)

This test provides a rapid means of determining the bulk specific gravity of coarse aggregate. The test procedure is similar to California Test 223 for determining surface moisture by the displacement method. In this test, however, the sample is first brought to an approximate saturated surface-dry condition by wetting the sample and then rolling the wet sample in an absorbent cloth to remove excess moisture. The sample is then placed into the pail, and the California Test 224 procedure is followed.

Specific Gravity of Fine Aggregate (California Test 225)

This test provides a rapid means of determining the bulk specific gravity of fine aggregate. Except for the method of drying the sample, the test procedure is similar to California Test 224 for coarse aggregate.

Moisture Content by Oven Drying (California Test 226)

This test is used to determine the water content of a material by drying the sample to a constant weight at a specified temperature. The water content is expressed as a percentage, by weight, of the dried sample. Average time to be allowed for testing in the lab: 1 day.

Cleanness of Coarse Aggregate (California Test 227)

This test is used to determine the quantity of detrimental fine particles present in a sample of coarse aggregate. It is similar in purpose and procedure to the sand equivalent test which is performed on fine aggregate; however, in this test the fine material is removed from the aggregate by washing and the exact test procedure varies with the size of the coarse aggregate being tested.

Test results are rounded up to the nearest whole number, which is the “cleanness value” of the tested material. Average time to be allowed for testing in the lab: 1/2 day.

Alkali Reactivity of Aggregates

Two ASTM test methods are acceptable methods of determining the susceptibility of an aggregate to react with alkali materials.

Short Method, ASTM C 1260

This test method provides a means of detecting, within 16 days, the potential of an aggregate intended for use in concrete for undergoing alkali-silica reaction resulting in potentially deleterious internal expansion. It is especially useful for aggregates that react slowly or produce expansion late in the reaction. However, it does not evaluate combinations of aggregates with cementitious materials nor are the test conditions representative of those encountered by concrete in service. Since the specimens are exposed to a NaOH solution, the alkali content of the cement is not a significant factor in affecting expansions.

Long Method, ASTM C 1293

This test method covers the determination of the susceptibility of an aggregate or combination of an aggregate with pozzolan or slag for participation in expansive alkali-silica reaction by measurement of length change of concrete prisms. This test method is considered the most reliable test method for assessing ASR on an aggregate. Its main disadvantage is the 1-year duration needed for the testing.

Material Sampling

Sampling Cementitious Materials

Cementitious materials are one of few construction materials accepted for use in the work on the basis of a certificate of compliance. However, cementitious materials samples are still required, as explained in the following section. Cementitious materials to be used in Caltrans projects are required to be on the Authorized Materials List (AML) at the time of mix design submittal. The list can be found at http://www.dot.ca.gov/hq/esc/approved_products_list/.

For cementitious materials used in precast concrete products or in ready-mixed concrete, the certification is made on Form TL-543[2], Vendor's Certificate of Compliance. A copy of this form is reproduced in Figure 8-1. The certificate will be signed by the manufacturer of the precast product or by the ready-mix concrete supplier, as the case may be.

Cementitious materials used in ready-mix concrete may be certified by a single certificate, for each brand of cementitious materials used, covering all concrete delivered to the project on a given day. The certificate must show the brand name and mill location, and each delivery covered by the certificate must be identified by the delivery slip number.

[2] New form is No.8-C78

For precast products, such as precast concrete piles, the required certificate of compliance will be furnished to the Caltrans' representative who is inspecting the manufacturing of the product. The certificate of compliance will form part of the required documentation for acceptance and release of the precast product.

If cementitious materials are delivered directly to the site of the work, as will be the case if the contractor or a supplier has set up a portable batch plant at the job site, the certificate of compliance is signed by the cementitious materials manufacturer. One certificate is required for each shipment.

For cementitious materials delivered directly to the job site, the certification may be on a State form or on the manufacturer's own form. In either case, the certificate must show the name of the cementitious materials mill, the date of shipment and quantity shipped, and a serial number traceable to a specific silo, bin or lot of cementitious materials as identified by the manufacturer. The certification must show the contract number and type and brand of cementitious material, and must state that the cementitious material meets contract requirements. Sampling cementitious materials shall be in conformance with California Test 125, Appendix C.

Even though the cementitious material is covered by a certificate of compliance, samples must be taken periodically and submitted to the METS lab in Sacramento. Cementitious materials sampling frequency is given in Chapter 6 of the Construction Manual. The lab does not test all samples submitted, but samples are tested randomly to monitor the manufacturer's quality control procedures and to independently verify that the cementitious material used in the work meets all specification requirements.

Cementitious material samples may be taken from the weigh hopper or from the conveyor belt or feed line leading to the weigh hopper. Note that the full 8 lb sample should be taken at one time, not in smaller increments. The sample bag should be closed immediately and a flexible tie placed high on the bag to leave room for the cement to shift in the bag. The sealed bag is placed in a second plastic bag with the white copy of Form TL-518, and the outer bag closed with a flexible tie.

Cementitious material samples should be shipped to the lab in special cartons (which are designed to hold a single 8 lb sample) provided for this purpose. If more convenient, samples may be shipped in concrete test cylinder cartons which will conveniently hold six samples. However, no more than six samples should be shipped in any one container.

The outside of the shipping carton should be marked “Cementitious Material Sample”. Cementitious material samples, if tested, will be tested soon after being received at the lab; therefore, samples should be shipped promptly rather than accumulated merely to facilitate packaging.

Supplementary cementitious materials are sampled similar to cementitious materials if in powdery form. If in liquid form, supplementary cementitious materials will be sampled similar to admixtures.

Sampling Water

Samples of water to be used in Portland cement concrete should be sent to the lab for analysis unless there is definite evidence of suitability. For example, water intended for domestic use need not be tested unless it is suspected of having a high chloride or sulfate content. If there is doubt, a sample should be submitted prior to use.

Water obtained from a non-commercial source, such as a well or a river, should be sampled and tested, even though the water may be potable and free of obvious impurities. Testing will be in accordance with California Test 405 (methods for chemical analysis of water), 417 (testing for sulfate content), and 422 (testing for chloride content). Water reclaimed from mixer wash-out operations may be used in mixing concrete. The alkali content and specific gravity requirements are specified in Section 90 of Standard Specifications. Reclaimed water, also known as recycled water, has been recovered from domestic, municipal, and industrial wastewater treatment plants, where impurities were removed. The treated water must meet the cleanliness standards for use in concrete as specified in the Standard Specifications. Chapter 2 of this manual discusses the water requirements in detail.

Samples of water are shipped in special plastic containers which are available for this purpose. Leave room in the container for expansion.

Sampling Admixtures

Most commercially available admixtures in common use have been tested for their suitability in structure concrete work on State highway projects, and those that are acceptable are included in the Authorized Materials List (AML), referenced before.

Admixtures on the authorized list, if accompanied by a certificate of compliance stating that the admixture furnished is the same as the admixture previously authorized, may be used prior to testing. Admixtures not on the authorized list, shall not be used but may be submitted for inclusion in the AML, see the website for the prequalification process. For admixtures on the authorized list but supplied without the required certificate of compliance, the sample must be received in the lab at least one week prior to the intended use.

Samples of liquid admixtures should be taken from the delivery tube at the batch plant by filling a 1 quart can or plastic bottle [3]

The sample identification card (Form TL-101) must include the manufacturer's lot number and the type of work in which the admixture will be used; i.e., prestressed concrete, reinforced concrete, etc. This information is needed by the lab to determine the suitability of the admixture for the intended use and to determine the maximum allowable dosage.

Sampling for Polyester Concrete

Initially, manufacturers need to send samples along with available QC documents to the METS lab at least 15[4] days prior to shipping their products to the jobsite. The sample shipment shall include:

- Polyester Resin
- Promoter
- Initiator
- Aggregate

The Manufacturers will include the specific polyester concrete mix design for that project to the METS lab.

For sampling at the site, it is recommended that polyester concrete samples be taken from the back of the machine. The samples should be no larger than 2″ deep by 4″ in diameter. When sending the sample to the METS lab for processing, attach the TL-101 to the sample. Additionally, request unit weight and burn off tests for resin content. The resin content information on the dispensing machine at the time that the sample is pulled should be provided on the TL-101. For gradation analysis, aggregate samples taken prior to addition of the resin can be sent to the METS lab. Typically no more than 2 pounds of dry material is required.

[3]2006 Standard Specifications 90-4.10 require each liquid admixture dispensing system to be equipped with a sampling device consisting of a valve located in a safe and readily accessible position, or 2010 Standard Specifications 90-1.02 F(4)(b)

[4]2010 Standard Specifications 15-5.06 A(2).

Sampling in the field for quick verification of aggregate ratios will require 3/8" and No.4 sieves, pan and scale for weighing. A sample of dry aggregate that has been combined but not yet mixed with the resin can be pulled from the belt, hand shaken and weighed to check the coarse to fine ratio of the aggregate. There should be no material retained on the 3/8" sieve.

The resin, catalyst and promoter are not sampled in the field. That work is done in advance as it is not possible to sample these items and receive results once the material has arrived on the project. Test methods for resin for polyester concrete are addressed in the applicable Specifications[5]. These test methods attempt to measure a number of properties of the resin including viscosity, specific gravity, elongation, tensile strength, bond strength and styrene content.

The aggregate should be delivered to the site in kiln-dried sealed bags or buckets. Should a bag be torn or a bucket damaged on arrival and open to the elements, the material is to be rejected.

[5] 2010 Standard Specifications 15-5.06 B.

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CHAPTER 9
JOB CONTROL SAMPLING AND TESTING

Table of Contents

Job Control Sampling And Testing **9-1**

Introduction **9-1**

Sampling Procedure **9-2**

Field Control Tests **9-2**

Ball Penetration Test (California Test 533) **9-4**
Lightweight Concrete 9-4
Calibration 9-4

Test Procedure **9-5**

Density Of Fresh Concrete (California Test 518) **9-5**

Air Content by Pressure Method (California Test 504) **9-7**

Air Content by Volumetric Method (California Test 543) **9-8**

Proportion of Coarse Aggregate (California Test 529) **9-9**

Tests for Compressive Strength (California Test 521) **9-9**
Number of Test Cylinders 9-10
Frequency of Sampling 9-10

Making and Handling Test Specimens (California Test 540) **9-10**
Identification of Test Cylinders 9-12
Shipping Test Cylinders 9-13
Reporting Test Results 9-15
Safety 9-16
Concrete Pour Records 9-16
Trial Batches 9-19

Concrete Compressive Strength by Maturity Method 9-19
Sampling and Testing of Self-Consolidating Concrete 9-21
Slump-Flow Test (ASTM C1611) 9-22
Static Segregation Test (ASTM C1610) 9-22
Passing Ability Test (J-Ring, ASTM C1621) 9-23

References 9-24

9 JOB CONTROL SAMPLING AND TESTING

Introduction

In addition to all materials being tested and approved for use, the resulting concrete mixture must be tested to verify that the basic ingredients, when combined, produce concrete having the specified properties and characteristics.

All concrete mixes are tested in the plastic state to determine yield, cementitious material content, density[1], consistency, and uniformity. In addition, certain mixes (such as air-entrained concrete mixes) require tests to verify that the desired special properties are being obtained.

The normal frequency of acceptance sampling and testing is addressed in Chapter 6 of the Construction Manual. Note that the frequencies shown in the tabulation are minimums for average conditions. In actual practice concrete should be tested as often as necessary to verify compliance with specification requirements.

Sampling and testing of the plastic concrete must be performed in accordance with the applicable California Test or ASTM Test methods. Tests normally performed in the field are briefly discussed in the following sections. Complete descriptions and instructions may be found at the following web locations:

California Test Methods: http://www.dot.ca.gov/hq/esc/ctms/

ASTMs: http://onramp.dot.ca.gov/hq/oscnet/
Click on "Field Resources", then select "ASTMs, etc." from the dropdown menu.
Contact the Structure Materials Representative for additional assistance.

The importance of sampling and testing in strict accordance with the applicable procedure cannot be overemphasized. To ensure reliable test results, samples must be taken carefully, and they must be truly representative of the material to be tested. Likewise, all test methods and procedures must be followed explicitly.

---

[1]Density and Unit Weight are being used interchangeably throughout this chapter.

Sampling Procedure

California Test 539 describes the procedure for obtaining samples of fresh concrete. The following points are emphasized:

- The minimum sample size for compressive strength tests is about 8 gallons. Where appropriate, a smaller volume may be used for other tests.
- Samples for tests to verify compliance with a compressive strength specification should be taken at, or as close as practicable to, the mixer discharge. Samples for tests to determine actual in-place strength at a particular time (such as strength to control prestressing) should be taken at the point of placement in the work.
- When sampling from truck mixers, the sample should be made from portions taken at three or more intervals through the discharge of the entire batch, taking care to avoid the start and end of the discharge. Concrete thus obtained should be remixed with a shovel or trowel before casting test cylinders or performing field concrete tests.
- If water is needed to be added to a truck mixer to adjust slump at the job site, the sample should be taken after the water has been added and the concrete thoroughly remixed. In no case, however, should any water be added beyond the required mix design amount.
- When sampling from forms, the test sample should consist of unvibrated concrete from the same batch taken at several different locations within the forms. Individual portions of the sample should extend deep enough into the mix to assure a representative distribution of the ingredients.
- Field control tests should be made as soon as practicable after the sample is taken.

Field Control Tests

Field control tests routinely performed by Caltrans personnel include the unit-weight test, the ball penetration test and the test to determine air content. On rare occasions, a test to determine coarse aggregate proportions may be warranted.

Under current Caltrans policy, all project personnel who perform tests on material being used in the work must possess a valid Form TL-0111, "Tester Certificate of Proficiency" listing the tests the individual is authorized to perform. This form and its applicable requirements may be found in Independent Assurance Manual, Procedures for Accreditation of Laboratories and Qualification of Testers at:

http://www.dot.ca.gov/hq/esc/Translab/ofpm/IA_reports/IAP.htm
Click on IA Program Manual.
Form TL-0111 is reproduced below in Figure 9-1.

Concrete field control tests are briefly discussed in the following sections. The tests are identified by their California Test numbers. A complete description of each test may be found at http://www.dot.ca.gov/hq/esc/ctms/. Table 9-1 provides a list of California Test Methods (CTMs) with comparable ASTM standards. It is recommended that personnel preparing for certification testing use the CTM rather than the comparable ASTM to ensure familiarity with the CTM.

TL-0111 (REV.06/00)

CALIFORNIA DEPARTMENT OF TRANSPORTATION

Presents this

CERTIFICATE OF PROFICIENCY

to

Caltrans

____

____

who is qualified to perform the following tests:

____________________	____________________
____________________	____________________
____________________	____________________
____________________	____________________
____________________	____________________
____________________	____________________
____________________	____________________
____________________	____________________
Test method & expiration date	Test method & expiration date

____

Certified Independent Assurance (IA)

Date Issued: ____________________

Note: This certificate is valid as long as the Tester complies with applicable requirements in Caltrans' Independent Assurance Program Manual.

Figure 9-1. Form TL-0111.

Table 9-1. Applicable California Test Methods and their ASTM Comparable.

California Test Methods	Comparable Test	Description
CTM 504	ASTM C231	Air Content (Pressure Method)
CTM 518	ASTM C138	Test for density, yield and air content (gravimetric)
CTM 529	-------	Proportions of coarse aggregate in fresh concrete
CTM 533	-------	Ball Penetration
CTM 539	ASTM C172	Sampling Fresh Concrete
CTM 540	ASTM C31	Making and Curing Test Cylinders in the field
CTM 543	ASTM C173	Air Content (Volumetric Method)
CTM 556	ASTM C143	Slump of Hydraulic Cement Concrete

Ball Penetration Test (California Test 533)

This test method describes the procedure for determining the consistency of fresh concrete by measuring the depth of penetration of a metal mass into plastic concrete under the force of gravity.

The ball penetration apparatus consists of a 6-inch cylinder with a hemispherical shaped bottom which is machined to a smooth finish. The penetrator is attached to a shaft graduated to measure penetration to the nearest 1/4 inch. The mass of the apparatus (ball, shaft, and handle), exclusive of the yoke, is 30 ± 0.1 pounds.

Lightweight Concrete

A modified ball is used for determining the consistency of fresh lightweight concrete. The modified ball is identical in shape and size to the 30 lb ball, but the mass of the lightweight apparatus (ball, shaft, and handle), exclusive of the yoke, is 20 ± 0.1 pounds.

Calibration

Zero reading is established by placing the ball and the feet of the yoke on a plane surface. The shaft is then adjusted by turning the threaded shaft in the ball penetrator to obtain a zero reading at the top of the sleeve. The locknut at the top of the penetrator is then tightened.

Test Procedure

- The ball penetration test may be made on concrete in a wheelbarrow, buggy, or other container, or after it has been deposited in the forms or on the subgrade. The depth of the concrete above the bottom of the container or reinforcement shall be at least 6 inches for 1-inch maximum size aggregate or smaller, and 8 inches for larger maximum size aggregate.

- The surface of the concrete to be tested is struck off level over an area of about 3 square feet. Do not tamp, vibrate or consolidate the concrete. Screed the minimum amount required to obtain a reasonably level surface. Overworking may flush excess mortar to the surface and cause erroneously high penetration readings.

- Hold the device by the handle; lower it slowly over the prepared area until the feet of the yoke touch the surface of the concrete. Make certain the shaft is in a vertical position and free to slide through the yoke. Gradually lower the ball penetrator into the concrete, maintaining enough restraint on the handle so that penetration is due to the dead load of the ball only and not to any force generated by acceleration of the mass. When the ball comes to rest, release the handle and read the penetration to the nearest 1/4 inch. Penetration of the feet of more than 1/8 inch may indicate that the concrete has been overworked in screeding the surface, or that the yoke is binding on the shaft.

- Take a minimum of three individual readings for each penetration determination. Individual readings shall be at least 9 inches between centers. The minimum horizontal distance from the centerline of the handle to the nearest edge of the level surface on which the test is made shall be 6 inches. The reported penetration shall be the average of the first three successive readings, which agree within 1/2 inch of penetration.

Report the average of the three readings as penetration in "___ inches of penetration." To ensure such accurate results, keep in mind that accuracy is impaired if the surface of the ball is roughened by scratches, dents, or adhering mortar. It should be cleaned carefully after each test and always kept in the carrying case when not in use to prevent damage.

Density Of Fresh Concrete (California Test 518)

Section 90 of Standard Specifications requires the cementitious material content of a concrete mixture to be determined in accordance with California Test 518. This test, previously referred to as Unit Weight test, could be used to determine the true cementitious material content of a given batch of concrete when the actual batch weights are known. Based on the test results, batches may need to be adjusted in order to be in compliance with cementitious material content requirements.

In this test method a sample of concrete with known volume is taken and weighed. The density is then calculated in lb/yd^3 as the ratio of the weight to volume of the sample. From the calculated density and the known total batch weight provided in the batch ticket, in pounds, the volume of the batch is determined from the following formula:

$$S \text{ (volume in yd}^3\text{)} = \text{total batch weight/density}$$

The total weight of cementitious material is also provided by the batch ticket. Therefore, cementitious material content (CC), expressed in lb/yd^3, could then be calculated as follows:

$$CC = \frac{\text{(total weight of cementitious materials, lbs)}}{S}$$

When performing the density test, field personnel should keep in mind that the test does not check batching accuracy. The test procedure assumes that scale weights shown on the batch ticket are correct.

Though the Standards do not allow pay deduction for insufficient cementitious material content based solely on California Test 518, they do require the Contractor to make adjustments on subsequent loads based on the test results. This test can also be an indicator of adequacy of proportioning operations as changes in proportioning can lead to changes in density. An example might be where the water content of a stockpile is underestimated resulting in both more sand and water than called for by design. This will lower the unit weight as water and sand are the lightest ingredients even with the right cementitious content.

The OSC work sheet (Form DS-OS C68), facilitates the unit weight and cementitious material content calculations. Bridge Construction Memo 100-2.0, Control of Cement Content[2] in Concrete, provides filled-in examples of this worksheet along with a more detailed explanation of the procedure involved in making the required calculations.

[2] 2010 Standard Specifications use “cementitious” materials.

Air Content by Pressure Method (California Test 504)

This test describes the method used to determine the air content of fluid concrete by the pressure method using a commercial air meter. The meter, shown in Figure 9-2, operates on the principle of equalizing a known volume of air at a known pressure in a sealed air chamber with the unknown volume of air in the concrete sample, the dial on the pressure gauge being calibrated in terms of percent air for the observed pressure at which equalization takes place.

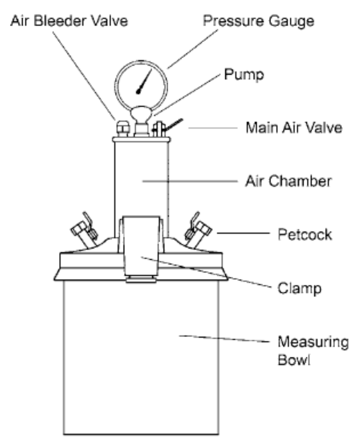

Figure 9-2. Apparatus for Air Content Test by Pressure Method.
(Ref. ASTM C231/C231M – 09b)

An air meter consists of a base unit and a cover containing a pressure chamber. In making the test, the base unit, which is essentially a metal container, is filled with fresh concrete. The concrete is placed into the container and rodded following the same procedure as used when casting concrete test cylinders.

After rodding, the concrete is struck off to the level of the top of the container with a flat bar or other suitable tool, and the top of the container is wiped clean to ensure a tight seal when the cover is clamped into place.

The cover is clamped in place with the petcocks open. Depending on the air meter type, either a funnel or syringe is used to add water in the space between the top of the concrete and the bottom of the cover unit. Water is added through one petcock until all air is expelled through the opposite petcock. Air is then pumped into the pressure chamber until the gauge reaches the initial pressure line. The gauge is allowed to stabilize, the petcocks are closed and the air is released into the base section, thus compressing the concrete in the base.

Since the initial pressure and volume are known, the drop in gauge pressure is directly related to the compressibility of the concrete, which in turn is a function of the amount of air in the concrete mixture. The more air there is in the fresh concrete, the more compression of the mass; hence the greater will be the corresponding drop in gauge pressure. The gauge is calibrated so that the drop in gauge pressure is shown as an increase in air content, which permits direct reading of the percent of air in the mix.

Air meters are precision instruments and should be treated as such. They should be handled with care, and in particular, they should be cleaned thoroughly after each use.

Air Content by Volumetric Method (California Test 543)

Use of a pressure-type air meter to determine the air content of a concrete mixture as described in the preceding section is not applicable to lightweight concrete. This is the case because the porosity of the lightweight aggregate permits water to penetrate the particles when pressure is applied to the concrete mass, thus causing an incorrect (too high) reading on the air gauge.

The volumetric-type air meter used in California Test 543 does not depend on air pressure; consequently, it is not sensitive to aggregate porosity and can be used with lightweight concrete.

The volumetric-type air meter consists of a base unit and a top unit. The base unit is filled with concrete, following the same procedure as described for the pressure method. The top unit, which is essentially a hollow chamber, is clamped in place and the chamber filled with water. The air meter is then inverted and agitated. After the water and concrete are thoroughly mixed, the meter is placed in a tilted position and rolled and/or rocked to further agitate the mixture. This action removes air from the concrete, and the air thus released rises to a calibrated gauge in the neck at the top of the chamber. After all air appears to have escaped from the concrete, agitation is stopped and the meter is placed in an upright position and allowed to stand for several minutes. This allows any remaining air to rise to the surface of the water in the top chamber. The volume of air displaced from the concrete is read directly on the gauge at the top of the upper section of the air meter.

While volumetric type air meters can be used with any concrete, California Test 543 takes much longer and is dependent on the skill of the technician to a much greater extent than California Test 504. Accordingly, the volumetric method is used only with lightweight concrete.

Proportion of Coarse Aggregate (California Test 529)

This test is used to check the effectiveness of concrete mixing equipment by measuring the uniformity of distribution of coarse aggregate in freshly-mixed concrete. The test, which is sometimes referred to as the "uniformity" test or the "washout" test, may be warranted to verify mixer compliance with contract requirements if visual inspection reveals a non-uniform mix or if it is suspected that a mixer is not functioning properly.

Depending on minor variation in procedure, the test method describes three procedures which may be used. In essence, however, each procedure compares the weight of coarse aggregate in two samples taken from separate portions of a batch of concrete or a truck mixer load. The samples are wet-sieved until all material finer than the No. 4 sieve has been removed. The free water is then drained, the retained aggregate weighed, and the proportion of aggregate in the sample (lb/yd^3) is calculated. As a standard of uniformity, Section 90 of Standard Specifications limits the variation in the amount of coarse aggregate to 170 lb/yd^3, as determined from the weight of aggregate in the two samples. It should be noted that this test is not commonly performed.

Tests for Compressive Strength (California Test 521)

Compressive strength requirements are covered in Standard Specifications Section 90. The compressive strength of concrete will be determined from test cylinders that have been fabricated from concrete sampled in conformance with the requirements of California Test 539. The results of this test method are used for determination of compliance with specifications and as a basis for quality control of concrete proportioning, mixing, and placing operations; control for evaluating effectiveness of admixtures; and similar uses.

A nominal Design compressive strength will be specified for all structure concrete, and occasionally for other highly stressed structural elements as well. Historically, minimum design compressive strength was measured at 28 days. With the addition of Supplementary Cementitious Materials (SCMs) and the resulting slow down in concrete strength gain, the design compressive strength may be measured over 42 days or longer. This period will be determined in the mix design review process. Additionally, the specifications prohibit the start of certain construction operations, such as removing falsework and placing backfill material against retaining walls and bridge abutments, until a minimum compressive strength (usually less than the minimum design strength) is obtained.

Since construction operations often depend on the time needed for concrete to reach a specified strength, sufficient information must be available to predict the rate of strength gain. It is good practice early in the project to cast extra test cylinders and have them broken at 7, 10, 14 and 28-day intervals. Using the strength-time curve thus developed, fairly accurate predictions of strength gain are possible.

Number of Test Cylinders

Each compressive strength test requires 2 cylinders taken from the same batch or load of concrete, and the test result is the average strength of the 2 cylinders.

Frequency of Sampling

Sampling frequency will be per Section 6 of the Construction Manual, or as required for acceptance for a specific contract. Where knowledge of early strength is required and at other times where engineering judgment indicates a need, test cylinders may be cast at more frequent intervals.

For concrete designated by compressive strength, the specifications provide that no single compressive strength test shall represent more than 300 cubic yards.

Making and Handling Test Specimens (California Test 540)

This test covers the procedure for casting, handling and curing concrete compressive test specimens. The test method procedures are self-explanatory, and generally are well-understood by field personnel. However, since large quantities of concrete are accepted on the basis of compressive strength test results, it is imperative that the test cylinders are cast, handled and stored in exact accordance with the test method. Therefore, the test method should be reviewed periodically to ensure that all required procedures are being followed.

Department policy requires a penetration test and a unit weight test (to determine the cementitious material content) for each batch or truckload of concrete from which cylinders for strength tests are fabricated. When air-entrained concrete is being used, an air content test is required as well.

Proper care needs to be taken when casting the cylinders. Pictures included in Figure 9-3 are examples of cylinders that were poorly cast, making them unusable for strength testing.

(a) Cylinder Cast with Rock Pockets.

(b) Cylinder Not Filled Properly.

(c) Cylinder Poorly Cast.

(d) Foreign Objects Left in the Cylinder.

(e) Cylinder Top Not level.

Figure 9-3. Examples of Improperly Cast Concrete Cylinders.

After fabrication, the test cylinders should be taken to their field-curing location as soon as practical. Precautions are necessary when moving the cylinders from the site to the curing location to prevent vibration which might result in segregation within the cylinder mold. The curing location should be on a firm level surface, free from vibration and protected from any disturbance during the field-curing period.

Under the current specifications, all test cylinders for concrete designated by compressive strength, other than steam-cured concrete, and all test cylinders taken to verify strength prior to applying loads or stresses, are to be cured by Method 1 as described in California Test 540. If a precast concrete member is steam cured, the compressive strength of the concrete will be determined from test cylinders that have been handled and stored in conformance with Method 3 of California Test 540.

California Test Method 540 also includes a second method for curing concrete cylinders. Method 2 was developed to evaluate in-place concrete strength prior to applying loads and is particularly suited to situations where concrete temperature could be well below 65°F. In 1988 it was decided to eliminate cylinder curing by Method 2 in furtherance of simplification efforts to the Standard Specifications. The basis for this decision was the assumption that most in-place concrete strengths would not be overestimated from cylinders cured by the room temperature water-bath method (Method 1) since most concrete initially cures with an internal temperature of more than 70°F.

For most climates in the State this assumption holds true; rarely would one experience conditions where the initial internal concrete temperatures fall below 65°F. For certain climatic conditions where Method 1 curing may not be appropriate for determining the load carrying capacity, and certain applications, such as cast-in-place segmental construction, the engineer may choose to cure the cylinders in accordance with Method 2. As a rule of thumb, if the ambient temperature falls below 50°F, the potential exists for the concrete temperature to remain low enough to affect the rate of strength gain. Field engineers should always make sure that sound engineering is practiced and that the contract stipulations are respected. At times, contract-testing requirements may need to be modified to comply with sound engineering.

Identification of Test Cylinders

Test cylinders are marked in accordance with a uniform system of identification. Under this system, each cylinder is marked with the contract number, a sample identification number, and the date cast. The sample identification number is a series of digits separated by dashes to indicate the method of field curing, the age at which the cylinder is to be tested, the cylinder number of the pair which is to be tested, and optional job coding. (Use of a flow pen to mark each cylinder is recommended.)

Note the following cylinder identification example:

Contract	03-100844
Sample No.	1 - 28 - 2/2 - - -
Date Cast	08/01/09

In the example, the first digit of the sample number indicates that Method 1 curing procedure was used. The second group of two digits (28) indicates that the cylinder is to be tested at 28 days. The third entry (2/2) indicates that it is the No. 2 cylinder of a two-cylinder test group. The remaining spaces may be used for any desired job coding consisting of number, letters, or a combination of both.

Shipping Test Cylinders

Each pair of cylinders is shipped to the lab for testing in a cardboard carton specifically designed for this purpose, as seen in Figure 9-4. These cartons may be obtained from District supplies.

Figure 9-4. Shipping Test Cylinders.

When submitted for testing, all test cylinders are accompanied by a sample identification card, which is Standard Form TL-502. For easy reference, Form TL-502 is reproduced in Figure 9-5. Only one identification card is needed for each pair of cylinders shipped in the same carton.

This form is generally self-explanatory; however, care must be taken to ensure that all entries are completed and that the information entered is correct.

When completing the card, note the following:

- Under source of aggregates indicate the deposit from which they were obtained, such as “Kaiser-Radum”, or “Chevreau-Bear River”, and not the batch plant.
- Enter the sample identification number, noting that the card covers two test cylinders. (For the example in the previous section, the sample number entry would read: 1-28-1/2 & 2/2 to indicate that the information on the card covers both test cylinders.)
- In the space designated, show the total weight of water used per weight of cementitious material in the mix based on actual weights -- not design weights.
- Show the mix design number or other entry to identify the type of concrete on the first line of the “Remarks” section.
- Use the “Remarks” section to give any special instructions to the lab, such as a request for test results by telephone. (If this service is desired, show the phone number and the name of the person to receive the results.)
- Under “Remarks” show if the unit weight of the hardened concrete cylinder(s) is required. The laboratory will not furnish unit weight data unless it is requested.

STATE OF CALIFORNIA • DEPARTMENT OF TRANSPORTATION

FIELD SAMPLE OF PORTLAND CEMENT CONCRETE
SAMPLE CARD

TL-0502 (REV. 6/97)

NO.

CONTR. NO

FED NO.

DIST.

CO.

RT.

P.M.

LIMITS OF WORK

RES. ENG. OR SUPT

ADDRESS

CONTRACTOR

CEMENT

CERTIFICATE NO.

FINE AGGREGATE

SMARA

COARSE AGGREGATE

SMARA

REMARKS

SAMPLED BY

TITLE

Figure 9-5. Form TL-502 Field Sample of Portland Cement Concrete.

Cylinders may be shipped or delivered either to the Main Transportation Laboratory (Translab) in Sacramento or to the district lab where available, whichever is more convenient. Test cylinders should be shipped within the time period specified in California Test 540, using the least expensive means that will get them to the lab in time for testing on the required date.

Reporting Test Results

The lab uses Form TL-507 to keep a permanent record of all concrete test cylinders received and tested. Form TL-507 contains enough space to record the results of several different tests made for a particular job. A Form TL-507 is initiated whenever test specimens are received by the lab. The sample number, the date on which the test will be made and the age at testing are entered in the first three columns on the master copy of the form. When a test cylinder is broken, the strength of the concrete is entered in the fourth column. Test results are reported by means of a photocopy of the master form.

Note that the sample shipment card (Form DH-TL-502) for the particular cylinder tested will be reproduced on the right side of the report form.

Safety

Safety should be a primary consideration when sampling and molding the test cylinders. Transit-mix trucks, particularly when they are backing, swinging concrete buckets and similar concrete construction equipment can present a danger in unguarded moments, and they deserve special alertness. Molding the test cylinders requires close attention to detail and, in the interest of safety, should always be done away from the construction area.

These safety precautions should also be followed when making the unit weight and air content tests, which are done in conjunction with molding concrete test cylinders.

Concrete Pour Records

To ensure uniformity as well as facilitate and streamline record keeping, Structure Construction has developed a series of forms which are used to record and report certain information pertaining to concrete used in structure construction. These forms are shown in the following tabulation:

Form No.	Description	Purpose
DS-OS C71	Aggregate Grading Chart	Used to plot aggregate grading.
DS-OS C70 A&B	Concrete Mix Design	Used to check contractor’s mix design.
DS-OS C72	Field Record for Concrete Pours	Used by concrete inspector to record pour data.
DS-C73	Concrete Pour Record	Used to summarize and record data for each concrete pour.

These forms may be accessed on Bridge Construction Records and Procedures Manual located on Structure Construction's web page at:

http://onramp.dot.ca.gov/hq/oscnet/sc_manuals/crp/vol_1/crp016.htm

and on the Structure Construction forms page at:

http://onramp.dot.ca.gov/hq/oscnet/downloads/forms.htm.

Please note that on the latter page, forms for aggregate grading and mix design checks are contained in an Excel spreadsheet named "concrete.xls".

Form DS-OS C72 was developed to facilitate recording of test data and other pertinent information by the concrete pour inspector. Note, however, that the use of this particular form is optional and is at the engineer's discretion. Alternatively, any suitable method of record keeping may be used provided all required information is preserved.

Following completion of the concrete pour, any information needed from truck delivery tickets should be entered on Form DS-OS C72, if this form is used, and the completed form (or the alternative record), the vendor's certificate, and the delivery tickets should be stapled together and filed by pour number in a suitable box or holder.

Form DS-C73 is the permanent record of the concrete pour. It is prepared from information previously recorded on Form DS-OS C72, or from an alternative field record-keeping source. When completing Form DS-C73, it is very important to substantiate all waste. Waste due to form variation should be estimated on an attached calculation sheet. Waste outside the forms (concrete that is spilled, left-over, rejected, etc.) must be described and estimated as accurately as possible. See the example form shown in Figure 9-6. For instructions on use of Form DS-C73 as an estimate document, refer to Bridge Construction Memo 4-5.8.

Department of Transportation
CONCRETE POUR RECORD
DS-C73 (Rev. 5-2-77)

File Category / Item Number / This Bridge / Bridge Identification

Sheet No. 48 - (34-0006R)

Job Stamp:
04-01202L4
04-SF, Ala-80-1.6/2.7
New SFOBB East Span

Bridge Name: *Oakland Touch Down* Bridge Number:
Item No. and Description: *102 Structural Concrete*
Date: *8/15/2008* Mix Design No.: *M 418 1283* Cert. of Compliance No.: *17830*
Pour Location: *23 L Abutment*
Admixture Used:
Air Entraining Agent: Amount: Air Required:
Pour Inspected By: *Mahmoud K.* Samples Taken By: *Sami Daouk*
Sample Numbers:

Time	Load Number	Barge Number	Air Temp C	Conc. Temp C		Average Readings Slump (inches)	% Air	Tested By
12:30 PM	8		64'	80'		41-/4"		Sami Daouk

Remarks:

Waste Outside Forms - Explain

Waste Inside Forms - Substantiate

Plan Pay Quant.-This Item-This Br. ______ m^3
Due to CCO's
Final Pay Quant.-This Item-This Br.
Quant. Prev. Paid-This Item-This Br.
Total Quant. To Date-This Item-This Br.

Quantity Delivered: 138.0 M^3
Quantity Returned: *1.5* M^3
Waste in forms: *0.0* M^3
Waste Outside Forms: *0.0* M^3
Net =Pay Quant.
This Pour: *136.5* M^3
Sheet No.
Line No.
Date: *8/15/2008*

Pour Record Info Entered By: *Sami Daouk*

Figure 9-6. Completed Form DS-C73 Concrete Pour Record.

Trial Batches

When concrete has a compressive strength greater than 3,600 psi, the Standard Specifications require the concrete materials, mix proportions, and mixing equipment and procedures to be prequalified based on certified test data or trial batch reports by the contractor. In most cases, the concrete will be prequalified by trial batches.

Compliance with prequalification requirements is the contractor's responsibility. This means the contractor must make all arrangements for producing the trial batch, and must arrange for a testing firm to sample the concrete and make the appropriate tests. All tests must be performed in accordance with California Test methods or the comparable ASTM test methods.

In accordance with Structure Construction policy, the structure's engineer should witness sampling of trial batch concrete, molding of the test cylinders, and all field tests. The engineer may elect to sample trial batch concrete and make test cylinders for testing in the Translab or district materials lab.

Concrete Compressive Strength by Maturity Method

An alternative method of estimating concrete strength is by the use of Maturity Method. This method is based on the principle that concrete properties are directly related to its age and temperature history. Furthermore, it is assumed that samples of a concrete mixture of the same maturity will have similar strengths even though they may have different age and temperature combinations. Maturity Method concept is described in more detail in Chapter 1 of this manual. The advantage of the maturity method is that it uses the actual temperature profile of the concrete in the structure to estimate its in-place strength.

It should be noted that determination of concrete compressive strength by Maturity Method does not apply to all projects. Use of this method would be based on its cost effectiveness and relative complexity or special circumstances of the project.

ASTM C1074 defines Maturity Method as a technique for estimating concrete strength that is based on the assumption that samples of a given concrete mixture attain equal strengths if they attain equal values of maturity index. Maturity index of concrete is a function of its temperature history and age and is used to estimate its strength development based on a pre-determined relationship developed from lab tests for that mixture.

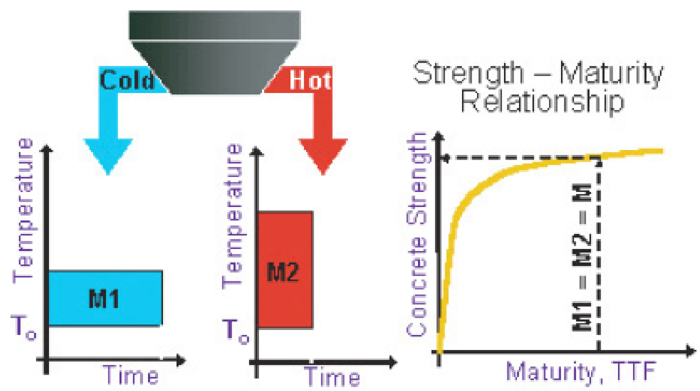

Figure 9-7. Concrete Maturity and Strength[3].

ASTM C1074 provides the procedure for estimating concrete strength using this method. Two types of maturity functions are described in the standard. One is the Nurse-Saul function that assumes a linear relationship between rate of strength development and temperature. Using this method the maturity index is expressed as temperature-time factor (TTF) from the product of temperature and time in °F-hours or °F-days. The accuracy of the Nurse-Saul prediction breaks down when there are wide ranges of curing temperatures, but its accuracy is considered adequate for most applications. Figure 9-7 provides a graphic depiction of the maturity concept. The figure shows that two concrete specimens, with different curing histories, would reach the same strength as long as their TTFs, M1, and M2 are equal.

In the second method, the Arrhenius function assumes that the rate of strength development follows an exponential relationship with temperature. The maturity index is expressed in terms of an "Equivalent Age" at a reference temperature. Actual age is typically normalized to an equivalent age at 68°F or 73°F. The Arrhenius function is considered to be more scientifically accurate although the Nurse-Saul function is more commonly used due to its simplicity.

It should be noted that, like any other test method, there are limitations for accuracy of the maturity method. These include:

- Concrete used in the structure must be representative of the one used to establish the maturity index. Changes in concrete mix design, like cementitious materials, aggregates, air content and w/c ratio could lead to erroneous estimates of concrete strength;

[3] http://www.fhwa.dot.gov/pavement/pccp/pubs/06004/index.cfm

- It does not account for humidity conditions during the curing stage and/or high early age temperatures, and

- It is assumed that concrete is placed, consolidated and cured properly. Additionally, continued cement hydration is assumed by providing adequate curing conditions.

It is a good practice to periodically verify that the established maturity-strength relationships for the specific concrete are still valid by alternative verification methods such as cast-in-place cylinders. Field-molded cylinders instrumented with maturity instruments could also be tested at early ages to serve as another verification method.

Commercial maturity devices are capable of continuous measurement of concrete temperature and calculating maturity index. The technology allows numerous locations to be monitored simultaneously. It is important to select a rugged system with uninterruptable data collection capabilities.

It should also be noted that the Maturity Method is not intended to replace the standard cylinder break method. However, in conjunction with other non-destructive testing methods it can replace field-cured cylinder testing to improve decision making for important construction activities. It can also be a vital quality control and quality assurance method.

Sampling and Testing of Self-Consolidating Concrete

Self-Consolidating Concrete (SCC) is highly flowable, non-segregating concrete that can flow into place, fill the formwork and encapsulate the reinforcement without any mechanical consolidation. Properties and characteristics of this concrete have been addressed elsewhere in this manual.

Testing of fresh SCC is performed to measure the following:

- Flowability and/or filling ability
- Stability and resistance to segregation (both static and dynamic)
- Passing ability (blocking potential)
- Self leveling (if required)

Note that typical fresh concrete tests such as air content, concrete temperature and density are also needed and addressed earlier in this chapter. During the mixture development stage of SCC, more testing may be needed than in the field for quality control (QC) purposes.

Slump-Flow Test (ASTM C1611)

This test method is used to monitor the consistency of fresh, unhardened self-consolidating concrete and its unconfined flow potential. In this test the mean diameter of the spread of fresh concrete is measured using a conventional slump cone, see Figure 9-8. The test method is considered applicable to SCC having coarse aggregate up to 1 inch in size.

Figure 9-8. Slump Flow Test of SCC.

In addition to spread, ASTM C1611 also provides Relative Viscosity (T_{20}) and a Visual Stability Index (VSI). T_{20} is defined as the time taken for the concrete to reach a spread diameter of 20 inches from the moment the slump cone is lifted up and is a measure of the viscosity of the SCC. T_{20} typically ranges between 2 and 10 seconds for SCC.

The resistance to segregation is estimated through a VSI. The VSI is established based on whether bleed water is observed at the leading edge of the spreading concrete, or if aggregates pile at the center. VSI values range from zero for “highly stable” to three for “unacceptable stability.” The test method includes an appendix that provides non-mandatory visual rating criteria to estimate the VSI.

Static Segregation Test (ASTM C1610)

ASTM C1610 is primarily used during the development phase of the SCC mixture. However, it is briefly discussed here in case a need for field application arises. The test method uses a 1/8-inch diameter cylinder filled with SCC and compares the amount of coarse aggregates in the top and bottom sections of the cylinder, when washed over No. 4 sieve, after a 15-minute wait time. A Segregation Index (SI), defined as percent difference in coarse aggregate mass between the top and bottom sections of the cylinder is calculated and compared to the allowable limit, normally 15%.

Passing Ability Test (J-Ring, ASTM C1621)

This test method measures the spread of SCC through reinforcing steel. The test utilizes a J-Ring in combination with a mold. The J-Ring test, shown in Figure 9-9, is a variation to the slump flow, where a simulated rebar cage is placed around the slump cone. The difference between the slump flow and J-Ring flow is an indication of the passing ability of the concrete. A difference less than 1 inch indicates good passing ability and a difference greater than 2 inches indicates poor passing ability. This test method is limited to self-consolidating concrete with nominal maximum size of aggregate of up to 1 inch.

The orientation of the mold for the J-Ring test and for the slump flow test without the J-Ring shall be the same. (Although the ASTM test specifications allow either orientation for the testing, the slump flow test, ASTM C 1611, and the J-Ring test, ASTM C 1621, must agree within 2 inches. Inconsistent orientation of the slump cone could result in a test failure.)

Figure 9-9. J-Ring Test (Ref. ASTM C1621).

References:

- Guide to non-destructive testing of concrete, FHWA, publication No. FHWA-SA-97-105, September 1997
- FHWA Technical Brief: www.fhwa.dot.gov/pavement/pccp/pubs/06004/index.cfm
- National Ready Mix Concrete Association CIP-39, “Maturity methods to estimate concrete strength”