Assessing the Impact of “Green” Concrete Mixtures on Building Construction

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PROVIDING THE MEANS TO ADVANCE CONCRETE CONSTRUCTION

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Building Innovation through Research

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PROVIDING THE MEANS TO ADVANCE CONCRETE CONSTRUCTION

Preface

The American Society of Concrete Contractors submitted a proposal to the Charles Pankow Foundation for preparation of a *Users' Guide to "Green" Concrete in Building Construction.* In this context, "green" refers to concretes made with supplementary cementitious materials (SCMs) replacing varying amounts of portland cements to reduce the carbon footprint. As part of the preparation, several green concrete mixtures were tested and the data presented to provide information about mixture composition that is usually proprietary and not available to the industry or public. The relationship between cylinder strength and strength of cores from the same batch of concrete was of particular interest. The data was intended to supplement the limited amount of published data related to field experience with green concrete.

Bruce Suprenant and Ward Malisch, the authors of this report, became interested in this topic as a result of Suprenant's troubleshooting work on projects that utilized green concrete with acceptance testing done at 56 or 90 days rather than the standard 28 days. This later testing compensates for slower rates of strength gain when large amounts of portland cement are replaced by SCMs, but with a downside: more concrete has been placed when the test results become available at test ages greater than 28 days. If a strength test result is lower than allowed, and subsequent core testing indicates that the in-place strength is also lower than allowed, repair or removal and replacement generally costs more because of the larger volume of concrete in place. Schedule delays resulting from needed decisions on acceptance may also be more critical at this point.

As our test results became available, we realized that the relationships between strengths of field-cast, wet-cured cylinders and cores from large blocks cast in the field were particularly puzzling. At ages of 28 to 180 days, the core/cylinder ratios ranged from about 0.40 to 0.90, with an overall average of about 0.65 for all but one of the 11 mixtures studied. Core/cylinder relationships for a control mixture containing no SCMs followed the same trend as those for fly ash, slag, ternary, and quaternary mixtures. This led to a literature search related specifically to the core/cylinder strength ratios for normal or high-strength concretes made with straight cements and varying SCM contents. That search resulted in questions concerning the ACI 318 code requirement that the average core strength of three cores must equal 0.85 times the design strength of the concrete, with no core in the set of three lower than 0.75 times the design strength.

As a result, we changed the title of our report to "Assessing the Impact of "Green" Concrete Mixtures on Building Construction." The report still covers construction rather than performance. But we acknowledge that our test results are for a combination of one cement, SCM, and admixture source and that, in field tests, the control mixture containing no SCMs performed similarly to mixtures made with varying percentages of SCMs. The scope for our field experiments did not include a factorial approach to evaluating the interactions between the cement and admixtures, nor do we suggest that the results of the field experiments can be generalized to include all green concretes. We do believe that, for confirmation, the results require further research of green concretes using differing cements and admixtures and in differing geographic regions. We also agree with the following recommendation in ACI 363.2R-11,"Guide to Quality Control and Assurance of High-Strength Concrete:"

"… a correlation curve should be established for each high-strength mixture to relate the strength of extracted cores (normally 4 in. [102 mm]) in diameter) to the strength of specimens used for acceptance testing, that is, 6 x 12 in. (152 x 305 mm) or 4 x 8 in. (102 x203 mm) cylinders. Then, if coring becomes necessary, the relationship has been established, agreed upon, and is ready for conclusive interpretation."

The correlation between core strength and the strength of specimens used for acceptance testing should be discussed at a preconstruction conference so the engineer of record, concrete producer, and concrete contractor are in agreement on steps to be taken when core tests are needed.

As the title of our original proposal implied, we were primarily interested in topics related to construction as opposed to performance of the green concrete after construction. Thus our testing program did not directly address durability because assessment of durability requires long-term testing or development of models for predicting durability, neither of which was within the scope of our proposal.

Bruce A. Suprenant
Ward R. Malisch

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Assessing the Impact of “Green” Concrete on Building Construction

Chapter 1 Introduction

Concrete is an inherently “green” material as is indicated by many measures of construction-material sustainability. Well-proportioned, properly placed concrete reduces the life-cycle costs of buildings because it is a durable material. The effects of transportation on the environment are reduced because concrete is produced locally, as are the largest proportions of its ingredients, and its producers often use recycled materials that would otherwise be a part of the waste stream destined for landfills. Concrete’s thermal mass can reduce the energy required for heating and cooling buildings and its light color can reduce interior lighting demands in buildings. The demonstrated safety and security of fire- and storm-resistant concrete buildings is enhanced even further by structural design that emphasizes resilience—the ability to maintain building functionality after an earthquake or other natural disaster. But because carbon dioxide is emitted during manufacture of portland cement, green concrete building construction often involves reducing the carbon footprint by replacing some or all of the portland cement.

One approach is completely replacing portland cement with carbon-neutral cements that can be produced by using a magnesium oxide feedstock instead of calcium carbonate, adding a liquid activator to coal ash, or other such methods for reducing carbon dioxide emissions. But the most widely practiced approach is to simply replace increasingly larger percentages of the portland cement with supplementary cementitious materials (SCMs) that have been used in concrete for many years. The most common SCMs used green concrete include fly ash, slag cement, and silica fume.

1.1 Materials for “Green” Concrete

Ready-mixed concrete is used in nearly all concrete for building construction. A survey by the National Ready Mixed Concrete Association indicates that SCMs were used in about two-thirds of all ready-mixed concrete produced in the U.S. (Obla et. al. 2012). Most of the SCMs are byproducts of energy production or manufacturing processes that, if not used in concrete, would be destined for landfills or other storage facilities for industrial wastes. This enhances their value as sustainable alternatives to portland cement consumption.

<u>1.1.1 SCM Sources, Usage, and Effects on Concrete Properties</u>

Fly ash —This most widely used SCM is a byproduct of coal combustion in electric power plants and has been used to replace cement and to improve many desirable properties of concrete for more than 75 years. ASTM C618, “Standard Specification for Coal Fly Ash and Raw of Calcined Natural Pozzolan for Use in Concrete,” describes two classes. Class F fly ash is produced by burning anthracite or bituminous coal and is a pozzolan that reacts with byproducts of portland cement hydration to form additional calcium silicate hydrates—the basic cementing compound in concrete. Class C fly ash is produced by burning lignite or subbituminous coal and reacts with byproducts of portland cement hydration to form additional calcium silicate hydrates. It also has some cementitious properties when it reacts with water. Class F fly ash normally replaces 15% to 25% by mass of cementitious material and Class C fly ash is normally used at dosages of 15% to 40% (ACI 232.2R-03).

Table 4.4.2 in ACI 318-11, "Building Code Requirements for Reinforced Concrete," includes the following limits on fly ash in concrete subject to Exposure Class F3—exposed to freezing and thawing, in continuous contact with moisture, and exposed to deicing chemicals:

Cementitious material	Max. % of total cementitious material by mass
Fly ash conforming to ASTM C618	25%
Total of fly ash, slag, and silica fume	50%
Total of fly ash and silica fume	25%*

* Fly ash and silica fume shall constitute no more than 25% and 10% respectively.

Most concrete used in building construction is not in Exposure Class F3. In these cases, ACI 318-11 does not prohibit the use of percentages of fly ash exceeding 50% in green concrete, nor does it limit slag and silica fume.

The reduced volume of pores resulting from formation of calcium silicate hydrates improves concrete resistance to chemical attack by decreasing the rate at which water and aggressive chemicals such as sulfates can enter the concrete. Some Class F fly ash replacement percentages also reduce the harmful effects of alkali silica reactions in concrete. Improved durability has a positive effect on sustainability by extending concrete's service life in addition to reducing its initial carbon footprint.

Slow setting and slow early strength gain are the major disadvantages when replacing large amounts of portland cement with fly ash to reduce the carbon footprint. Steps that can be taken to offset this effect are discussed later in this report.

There is some anecdotal evidence that concrete containing fly ash may reject coatings such as paint and flooring adhesives, thus resulting in loss of adhesion between the coating or adhesive and the concrete substrate (Lick, 2013). No studies have identified the mechanism by which this happens, however, or a critical percentage replacement at which the adhesion loss occurs. This Pankow study did not include adhesion of coatings or adhesives within the project scope.

As of the date of this report, the future use of fly ash in concrete may be impacted by pending regulatory action by the U.S. Environmental Protection Agency, which is considering regulating fly ash as a hazardous waste rather than an exempt waste.

Slag cement — Previously called ground granulated blast furnace slag (GGBFS), slag cement is a byproduct of iron production for use in steel making. ASTM C989, “Standard Specification for Slag Cement for Use in Concrete and Mortars,” provides for three strength classifications based on a slag activity index: Grades 120, 100, and 80 are ranked in order of decreasing slag activity, which is related to 7- and 28-day strength results. When used in building construction, slag cement may comprise 30% to 70% of the cementitious material with the larger percentage replacements used in mass concrete, especially in foundations. Delays in setting time can be expected when more than 25% slag cement replaces portland cement in concrete mixtures. When compared with portland cement concrete mixtures, concrete containing Grade 120 slag cements typically results in reduced strength within one to three days of placement and increased strength at seven days and beyond (ACI 233R-03). Slag cement reacts with sodium and calcium hydroxides present in fresh concrete, with that reaction producing calcium silicate hydrates that reduce porosity and pore size. This reduces the ability of aggressive chemicals to penetrate concrete and thus improves durability in a way similar to that of fly ash. Again, improved durability enhances sustainability by extending concrete’s service life. Light color is a further sustainability advantage of slag cement concretes because the lighter color reduces lighting requirements.

Because sodium and calcium hydroxides released during portland cement hydration serve as activators for slag cement hydration, replacing all of the portland cement with slag cement would result in very slow setting and strength gain. As with fly ash, slow setting and slow early strength gains resulting from large portland cement replacements with slag cements can be offset by methods discussed later in this report.

Silica fume **—** Also known as condensed silica fume or microsilica, this SCM is an extremely fine, highly reactive pozzolan that is an industrial byproduct of the silicon metal and ferrosilicon industry. ASTM C1240, “Standard Specification for Silica Fume Used in Cementitious Mixtures,” describes three forms in which silica fume can be added to concrete: as-produced, as a slurry mixed with water, or as a densified product. The densified product is most commonly used in the U.S.

Whereas fly ash and slag cement can replace large quantities of portland cement, silica fume is not used primarily as a cement replacement. Dosage rarely exceeds 10% by weight of cementitious material. It is commonly used in concretes that incorporate portland cement and either fly ash or slag cement because its small particle size and chemical reactivity improve the rate of early strength gain and durability of these concretes (ACI 234R-06). Because of its small particle size and resultant large surface area, the water demand of concrete containing silica fume increases with increasing amounts of silica fume. Addition of a water-reducing or high-range water-reducing admixture is mandatory to retain the strength and durability improvements.

To achieve desired performance objectives while reducing the carbon footprint, some green concretes are ternary mixtures containing portland cement, fly ash, and slag cement, while other quaternary mixtures contain three SCMs—fly ash, slag cement, and silica fume—plus portland cement. Results from testing ternary and quaternary mixtures, a binary mixture containing only fly ash and portland cement, and a straight portland cement mixture are discussed in this assessment.

1.2 Significance of High-volume SCM Effects on Concrete Properties

Replacing large amounts of portland cement with fly ash or slag cement can result in slower concrete setting and slower strength gain at both early and later ages. Unless the potential early-age effects are offset by using admixtures or altering the proportions of cementitious materials, the following effects are likely:

- Finishing delays that increase the contractor's labor costs
- Plastic shrinkage cracking or settlement cracking as a result of slower setting.
- Damage to formed surfaces or delayed form removal, creating repair costs, reducing the economies from form reuse, or both
- Delays in stressing post-tensioning tendons that also delay forming and shoring removal

All of these possible effects can slow progress on project completion, which also increases costs.

Slower than normal strength gain at ages later than 7 days is sometimes accommodated by specifying design strengths based on cylinders tested at 56 or 90 days. This has drawbacks because on some green concrete projects, low 56- or 90-day cylinder breaks for field-cast laboratory-cured cylinders have led to core testing that resulted in low core strengths. Some sets of three cores on these projects did not reach 85% of the specified 56-day design strength as required by ACI 318-11, even at ages of 90 days or more. Strength retrogression in shear wall and column cylinder strengths was also noted (See Section 3.2.1).

In ACI 318-11, strength test results for field-cast, laboratory-cured cylinders are used for *acceptance* of the concrete. For concrete design strengths of 5000 psi or more, strength level is considered satisfactory if:

(a) Every arithmetic average of any three consecutive strength tests equals or exceeds the design strength.
(b) No strength test falls below the design strength by more than 10% of the design strength.

If either requirement is not met, steps must be taken to increase the average of subsequent strength test results. If the (b) requirement is not met, further analysis or testing is indicated, as discussed in Section 3.1 of this report.

Also ACI 318-11 indicates that strength test results for field-cast cylinders field-cured in accordance ASTM C31 may be required to check the adequacy of curing and protection of concrete in the structure. If strength of field-cured cylinders at the test age designated for determination of the design strength is less than 85% of that of companion laboratory cylinders, procedures for protecting and curing concrete must be improved. See Section 2.6 of this report for a discussion of this requirement as applied to the Pankow research laboratory data.

1.3 Selected References and Association Websites

- ACI 232.2R-03 "Use of Fly Ash in Concrete"
- ACI 233R-03 "Slag Cement in Concrete and Mortar"
- ACI 234R-06 "Guide for the Use of Silica Fume in Concrete"
- ASTM C 311 "Standard Test Methods for Sampling and Testing Fly Ash or Natural Pozzolans for Use in Portland-Cement Concrete"
- ASTM C 595-13 "Standard Specification for Blended Hydraulic Cements"
- ASTM C 618-12a "Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete"
- ASTM C 989-12a "Standard Specification for Slag Cement for Use in Concrete and Mortar"
- ASTM C 1697-10 "Standard Specification for Blended Supplementary Cementitious Materials"
- ASTM C 1240-12 "Standard Specification for Silica Fume used in Cementitious Mixtures"
- ASTM C 1709-11 "Standard Guide for Evaluation of Alternative Supplementary Cementitious Materials (ASCM) in Concrete"
- Lick, David, "Best Practices: When the Substrate Rejects the Coating," *MPI Newsletter,* Oct.2 , 2013, http://www.durabilityanddesign.com/news/?fuseaction=view&id=10308&nl_versionid=3543
- Obla, K., Lobo, C., and Kim, H., "The 2012 NRMCA Supplementary Cementitious Materials Use Survey," *Concrete InFocus Magazine*, Fall 2012

Associations

- American Concrete Institute www.concrete.org
- American Coal Ash Association www.acaa-usa.org
- American Society of Concrete Contractors www.ascconline.org
- American Society of Testing and Materials www.astm.org
- National Ready-Mixed Concrete Association www.nrmca.org
- Silica Fume Association www.silicafume.org
- Slag Cement Association www.slagcement.org

Chapter 2 Compressive Strength

The procedures for selecting and verifying compressive strength when proportioning green concrete for buildings are similar to those followed for traditional concrete used in buildings. There are, however, a few notable exceptions for green concrete mixtures because using them may result in:

- The need for laboratory trial mixtures containing SCM materials and proportions for which no historical field strength data are available.
- Specified strengths being measured with field-cast cylinders tested at 56 days, 90 days, or later because of slow strength gain.
- Concrete mixtures with lower water-cementitious ratios to compensate for lower strength-gain rates of SCMs. Cylinder strengths for these mixtures are more sensitive to the differing effects of curing with external water or sealing the cylinders so internal water is the only source for curing.
- Core strengths not meeting the common acceptance requirement that the average of three cores must equal 85% of the specified strength.

2.1 Laboratory Trial Mixtures

Specifying strength levels at 56 or 90 days allows either the use of lower cementitious contents to achieve a desired strength or allows a higher strength to be achieved for a given cementitious content. Green concrete often requires laboratory trial mixture proportioning based on strength data obtained at 56 or 90 days, which can increase the lead time for starting concrete construction. Pre-planning is essential for setting construction schedules when no off-the-shelf green concrete mixtures are available.

The later age strength verification requires the design and construction team to be prepared for trial mixtures that might take 3 to 4 months to produce strength results. This time frame is based on having to do only one trial mixture; multiple trial mixtures performed consecutively would add to the required lead time. One ready-mixed concrete producer in California starts developing green concrete mixtures about a year before construction is scheduled to begin. Also in California, owners and the design team anticipate required lead times for multiple laboratory trial mixtures by an early addition of the ready-mixed concrete producer to the construction team.

In other cases, ready-mixed concrete producers develop green concrete mixtures in advance of getting a contract for a project. They understand that having both laboratory and field strength data available for their green concrete mixtures prior to initiation of a project gives them a distinct competitive advantage. Consider the advantage to concrete contactors if they can avoid starting a 3 to 4 month laboratory trial mixture process by:

- Selecting a green concrete mixture with laboratory and field strength data already available
- Initiating a concrete mixture design submittal and, when the submittal is approved,
- Starting concrete placement.

2.1.1 Early Age Strength Data**:** Cylinder strengths are obviously needed at the specified age, whether 28, 56 or 90 days. Knowing early-age strengths, however, is important for planning construction operations such as formwork removal, post-tensioning, and continuing construction. Laboratory cylinder strengths at 1, 3, 7, 14 and 28 days should be determined. Contractors can use these strengths to assist in planning and scheduling construction operations. Also these earlier strength values can be used as check points when cylinders are to be tested at ages greater than 28 days.

Even if the acceptance is at 90 days, it is wise to have field-made cylinders cast and tested at earlier ages to judge the potential of the concrete to reach the specified strength at 90 days. These "early-warning" cylinders can be used to gauge the amount of construction that can continue based on the current strength of the concrete.

While early-age laboratory cylinder strengths are useful for pre-planning and scheduling, the contractor must utilize field-made and field-cured cylinders to determine how to sequence and continue construction operations. For green concrete buildings, it is best to set a minimum strength level as a criterion for continuing construction rather than using a concrete age criterion.

2.1.2 Core-to-Cylinder Relationships**:** Core-to-cylinder relationships are discussed in Chapter 3. This Chapter recommends making field specimens with minimum dimensions that match those of the structural members, then taking cores at the test age for cylinder strength acceptance to establish the core-to-cylinder strength relationship for that mixture. It's preferable to do this during the laboratory trial mixture phase so these results can be discussed prior to starting construction.

2.2 Consequences of Later-Age Strength Verification

Later age strength verification affects the cost of remedial action when acceptance testing indicates that the concrete strength does not meet specification requirements. Section R5.6.5 of ACI 318-11 includes procedures to be followed if strength test results don't meet the requirements of Section 5.6.3.3(b). Section R5.6.5 suggests that the building official should apply judgment as to the significance of low test results and whether they indicate need for concern. Nondestructive tests of the in-place concrete may be may be used to confirm the strength tests results from field-cast cylinders. Per

Section 5.6.5.2 states that if the likelihood of low-strength concrete is confirmed, and calculations indicate that load-carrying capacity is significantly reduced, tests of cores drilled from the area in question shall be permitted. Section R.5.6.5 indicates that cores are to be taken in extreme cases, but doesn't define extreme cases. It is our experience that core tests are common if strength tests are low. If the average core strength doesn't meet the requirements of Section 5.6.5.4 of ACI 318-11, and the structural adequacy remains in doubt, Section 5.6.5.4 states that strength evaluation in accordance with Chapter 20 is permitted or other appropriate action may be taken. Following these requirements and suggestions is complicated when later age strength verification is required.

At 56 or 90 days, as opposed to 28 days for acceptance testing, more concrete has been placed when the test results become available. If a strength test result is lower than allowed, and subsequent core testing indicates that the in-place strength is also lower than allowed, repair or removal and replacement will cost more because of the larger volume of concrete in place. Schedule delays resulting from needed decisions on acceptance may also be more critical at this point. Thus, later age strength verification increases risk for both the owner and contractor. This increased risk should be discussed prior to construction. Possible considerations for reducing this risk would be:

- Having check cylinders tested at earlier ages.
- Having plans for core testing to be implemented when a low strength test triggers an investigation and other options have been investigated.
- Having a plan to alter the sequence or staging of construction when time is needed to address a low-strength investigation.
- Having in place a preliminary strengthening plan that could be implemented effectively and efficiently to minimize any work delay.

Specifications for a few green building projects have incorporated multiple strength requirements at different ages. For instance, one project in California had both 28- and 365-day strength requirements. The concrete met the strength requirement at 28 days. It is unclear whether cylinder strength tests were conducted at 365 days. But if they had been, and the results failed to meet specification requirements, the remedial measures available would have been severely limited because the concrete work was completed and other trades were almost finished with their work.

For traditional projects, contractors are accustomed to multiple field strength requirements needed to continue construction, such as minimum strengths for post-tensioning, form stripping, or shoring removal. But it's best to avoid multiple acceptance strength requirements for concrete based on laboratory cylinders. Otherwise, the concrete could be accepted at 56 days but rejected at 90 days. This creates problems for the owner and contractor with respect to determining how to proceed.

Engineers need to consider alternate methods of describing when formwork stripping, removal of shores, and post-tensioning can proceed. It may not be appropriate to specify a requirement for strengths of 75% or 85% of f_c' when f_c' is designated to be 8000 psi at 90 days. This would mean that a strength of 6000 or 6800 psi is required for starting construction operations needed much earlier than 90 days after concrete is placed.

2.3 Sealed- versus Water-Cured Compressive Strengths

Test cylinders molded in the laboratory are cured in accordance with ASTM C 31 “Standard Practice for Making and Curing Concrete Test Specimens in the Field,” which requires final cylinder curing in water storage tanks or moist rooms. These curing options provide external moisture that is not available when curing concrete by sealing the cylinders in the laboratory or using water-retention field curing methods for structural members. Sealed curing of test cylinders represents the best possible field curing. But for low water-cement ratio concretes, the cylinders cured by sealing or field concrete cured using water retention can undergo self-desiccation as described by Meeks and Carino (1999):

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

Several investigators have studied the effect of a water-cure versus a sealed-cure on compressive strength of concretes containing only portland cement and concretes in which some of the portland cement was replaced with SCMs.

2.3.1 Sanjayan and Sioulas Tests on Slag-cement Concrete: This paper reports the strength development of 16 full-scale columns made with 100% general purpose (GP) portland cement and GP portland cement-slag cement blends containing up to 70% slag cement. One slag cement blend also contained silica fume.

The study included mixtures with strength grades of 100, 80, 60, and 40 MPa (14,500, 11,600, 8700, and 5600 psi, respectively). Of the 1250 specimens tested, 800 were cores. The influence of moisture availability was systematically studied by subjecting standard cylinder specimens to water-bath and sealed curing conditions. Table 2.1 shows strength ratios of sealed-cure (SC) to water-bath cured (BC) cylinders at ages 7, 28, 56 and 91 days.

The average ratios of sealed-cure cylinder strengths to water-bath cured cylinder strengths for the 100% GP cement mixtures were 0.90, 0.90 and 0.96 at 28, 56 and 91 days, respectively. The average ratios of sealed-cure cylinder strengths to water-bath cured cylinder strengths for the GP cement-slag cement blends were 0.87, 0.84 and 0.82 at 28, 56 and 91 days, respectively.

Table 2.1 Sanjayan and Sioulas Compressive Strength of Slag-cement Concrete

Ratio of Sealed-cure Cylinder Strength to Water-bath-cure Cylinder Strength				
Mixture	7 days	28 days	56 days	91 days
100GP	1.04	0.96	0.97	1.01
100GB70/30	0.91	0.87	0.88	0.86
100GB50/50	0.95	0.93	0.88	0.89
100GB30/70	0.96	0.90	0.74	0.75
100GB45/45SF10	0.99	0.88	0.85	0.84
80GP	0.99	0.91	0.89	0.98
80GB50/50	0.84	0.82	0.83	0.77
60GP	0.79	0.85	0.86	0.91
60GB50/50	0.93	0.79	0.83	0.77
40GP	0.89	0.87	0.87	0.93
40GB50/50	1.00	0.89	0.86	0.83
Average without 100% GP mixes	0.93	0.87	0.84	0.82

Table 2.1 Note: Description of the labeling system adopted are as follows: xGP; where GP indicates general purpose portland cement and x indicates the strength grade in MPa. xGBy/z; where GB indicates General Blended cement, x indicates the strength grade, y and z indicate the percentage of portland cement and slag in the binder, respectively. 100GB45/45SF10 is a 100 MPa-grade mixture containing 45% portland cement, 45% slag, and 10% condensed silica fume.

Based on their data, Sanjayan and Sioulas concluded:

> "An inspection of the results showed that a lack of moist curing adversely affected compressive strength development of the slag-blended cements irrespective of slag content. The SC cylinders attained medium- and long-term compressive strengths lower than their respective BC cylinders. Although the early-age (7 days) strengths did not significantly differ between the SC and BC specimens, the detrimental effects of isolating specimens from a continuous supply of moisture were realized in the long term. Depriving the GP specimens of moisture did not adversely affect their long-term strength development."

2.3.2 Wu et al. Tests on Fly Ash Concrete: This study used fly ash concrete specimens at 25% replacement with a w/cm of 0.40 and three curing conditions; water-, sealed- and air-dried curing. Reference concretes without fly ash were also tested. Compressive strength was measured at 7, 28, 91, 182, 273 and 358 days. Figure 2.1 shows the compressive strength for the three curing conditions. Note that the compressive strengths of the sealed-cure specimens were lower than those of the water-cured specimens by approximately 10 to 15% at ages from 91 through 358 days. The researcher's data for reference concrete mixtures showed only a slight difference in strength between water- and sealed-cure compressive strength at all ages.

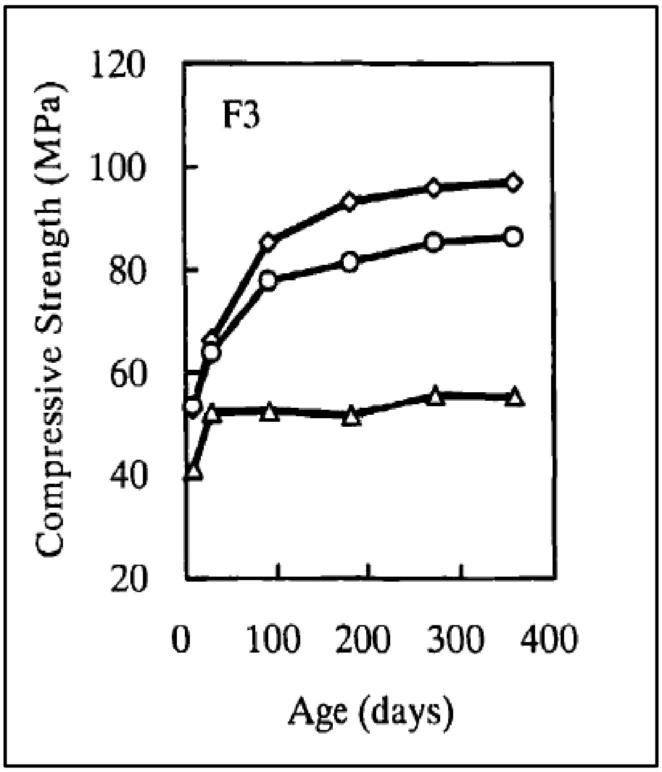


Figure 2.1 Compressive strength of concrete with 25% fly ash replacement tested at ages from 7 to 358 days (Wu et al, 2004). Symbols: diamonds, circles and triangles represent water-, sealed- and air-curing, respectively. Note that, as expected, the compressive strengths of the sealed-cure specimens are lower than those of the water-cured specimens by approximately 10 to 15% at the test ages greater than 28 days.

2.3.3 Aitcin et al. Tests on Concrete Containing no SCM's: This study used ready-mixed concrete made with only portland cement and with target compressive strengths of 5000, 13,000 and 17,500 psi for 4-, 6- and 8-in. diameter cylinders under different curing conditions: water-, sealed- and air-cured. The w/c ratios were 0.45, 0.31 and 0.25. Cylinder compressive strengths were measured at 7, 28, 91 and 365 days. The ratio of sealed- to water-cure compressive strength for the concrete mixtures with the three differing w/c ratios was 0.96.

2.3.4 Pankow Tests on Fly Ash, Slag-cement and Ternary Concrete: This work consisted of two phases: a laboratory and field study for concretes containing varying proportions, types, and amounts of SCMs; types of aggregate; and differing curing conditions. The compressive strength data is summarized in Table L3 and Table F3 in the Phase I report. Table 2.2 summarizes the sealed-cure to water-cure compressive strength ratio for all mixtures used in the study and tested at ages from 7 to 180 days. Tables 2.3 and 2.4 summarize sealed-cure to water-cure compressive strength ratios for individual mixtures in the laboratory and field study.

At 7 days the strengths of water-cure and seal-cure cylinders were about equal. Table 2.2 shows that at 7 days the sealed- to water-cure compressive strength ratios ranged from 0.90 to 1.10, with an average of 0.99 for the field-cast, laboratory-cured cylinders. After 7 days, however, the sealed- to water-cure compressive strength ratios for all mixtures ranged from 0.72 to 0.97 in the laboratory study with the average for 28 through 120 days being 0.86. In the field study ratios for all mixtures ranged from 0.73 to 1.03 with the average for all 28 through 180 days being 0.83. Tables 2.3 and 2.4 show that the ratios for mixtures containing no SCMs followed the same trend as those for fly ash, slag and ternary mixtures. The only difference was for the ternary concrete containing lightweight fines for which the sealed- to water-cure compressive strength ratio remained close to 100% through 90 days.

Figure 2.2 shows a graph of the Pankow field data for sealed- to water-cure strength ratios at varying ages. At 7 days, the ratios for all mixtures fell into a pattern ranging from 0.90 to 1.10. At ages later than 7 days, the range in ratios for any age was greater than that at 7 days, and the average ratios at any age were less than those at 7 days. The decrease in ratios, however, was not proportional to age; ratios were about the same at ages of 28, 58, 90, 120, and 180 days as indicated by a nearly horizontal trend line through the averages. This confirms Meeks and Carino's findings that self-desiccation reduces the internal relative humidity to a point that no further hydration occurs.

2.4 Compressive Strength of Concretes Cured by Air-drying

The compressive strength data for concrete specimens that were cured by air-drying are provided in Table L3 and Table F3 in the Phase I report. Table 2.5 summarizes the air-cure to water-cure compressive strength ratios for all mixtures at ages from 7 to 180 days in the laboratory and field studies. Tables 2.3 and 2.4 summarize air-cure to water-cure compressive strength ratios for individual mixtures in the laboratory and field studies.

As expected, the compressive strengths of air-dried cylinders were much lower than those of seal- or water-cured cylinders. The concrete mixtures containing lightweight fines generally performed better, with higher air- to-water-cure strength ratios than the other mixtures. Concrete containing SCM's followed about the same general trend as the concrete without SCM's. The data suggests that air-curing concrete containing SCM's is as detrimental to strength as air-curing concrete without SCM's.

2.5 Comparison of Researcher's Data

Figure 2.3 compares the average sealed to water-cure and the average air to water-cure strength ratios for Pankow, Sanjayan, Aitcin and Wu. For the sealed to water-cure strength ratios, Aitcin's and Wu's data are about 0.90 or higher. Recall that Aitcin used concrete containing no SCMs and Wu used a single binary concrete mixture containing 25% fly ash replacement. The Sanjayan and Pankow sealed to water-cure strength ratio data are similar for the range of fly ash, slag and ternary mixes. Aitcin's data represents the average of three mixtures, Wu used one binary mixture, Sanjayan used 11 mixtures and the Pankow research included results for 11 mixtures.

For the seal- to water-cure data for concrete mixtures containing no SCM's, Aitcin, Sanjayan and Wu all found strength ratios to be near 1.0. In other words, sealed curing was as good as water curing when portland cement was the only cementitious material used. The Pankow data did not follow this trend. The sealed curing produced significantly lower strength for both the mixtures containing SCMs and those for which portland cement was the only cementitious material. Only one cement source was used in the Pankow research, so this finding may simply be due to behavior specific to that cement source.

For the air- to water-cure strength ratio data, Aitcin again had the highest values while the ratios from the Wu and Pankow studies were similar, with Wu having slightly higher values. Sanjayan had no air- to water-cure data.

2.6 Implications of the Effect of Curing on Compressive Strength

The lack of water curing had a greatest effect on concrete mixtures containing SCM's. For the Sanjayan and Pankow research, the difference between the sealed-cure and water-cure compressive strength ratios was about 0.15, with the water cure resulting in the higher ratio. This is a dramatic reduction for high-strength concretes. Sealed curing represents the best possible field curing. Thus, designers will need to determine whether the water-cured strength or the seal-cured strength should be used for design. This has a significant implication with respect to the economy of using green concrete mixtures.

The sealed-cure data can also be analyzed by comparing the sealed-cured to water-cured strength ratios with ACI 318-11 criteria for comparing strengths of field-cured cylinders to the specified strength, f_c'. As described in Section 1.2 of this report, strength tests of field cured cylinders are sometimes used to check the adequacy of curing and protection of concrete in the structure. ACI 318-11 requires such cylinders to be molded in accordance with ASTM C31 and at the same time and from the same samples as laboratory-cured test cylinders. ASTM C31 requires storing the cylinders in or on the structure as near to the point of deposit of the concrete represented as possible. The cylinders must be provided with the same temperature and moisture environment as the structural work.

Based on the results of tests on field-cured cylinders, Section 5.6.4.4 of ACI 318-11 states the following:

> "Procedures for protecting and curing concrete shall be improved when strength of field-cured cylinders at test age designated for determination of f_c' is less than 85 percent of that of companion laboratory-cured cylinders."

In the Pankow research, with sealed cylinders testing at an average strength about 85 percent of that for the laboratory-cured cylinders, and assuming a normal distribution, one-half of the concrete mixtures tested would be expected to fall below the acceptance level set by ACI 318-11 for field curing. For these mixtures, ACI 318 would require improved procedures for protecting and curing concrete. It is unclear how better curing and protection in the field could duplicate the results from a complete seal provided by a plastic cylinder mold with a lid.

If cores had been removed from the sealed-cure cylinders, the core strength is anticipated to be lower as a result of damage due to drilling. Thus, concrete represented by at least one-half of the cylinders would also not meet the core acceptance criteria in ACI 318-11. This problem is discussed in much more detail in the following chapter.

2.7 References

Aitcin, Pierre-Claude, Miao, Buquan, Cook, William D and Mitchell, Dennis, "Effects of Size and Curing on Cylinder Compressive Strength of Normal and High-Strength Concretes," ACI Materials Journal, July-August 1994, pp. 349-354.

Meeks, Kenneth W. and Carino, Nicholas J. "Curing of High-Performance Concrete: Report of the State-of-the-Art," National Institute of Standards and Technology, NISTIR 6295, March 1999, 191 pages.

Sanjayan, J.G. and Sioulas, Bill, "Strength of Slag-Cement Concrete Cured in Place and in Other Conditions," *ACI Materials Journal*, September-October 2000, pp. 603-611.

Wu, F.-R., Masuda, Y. and Nakamura, S., "Influence of Different Curing Conditions on Strength Development of High-Strength Concrete using Fly Ash", , *SP-221: Eighth CANMET/ACI International Conference on Fly Ash, Silica Fume, Slag, and Natural Pozzolans in Concrete*, American Concrete Institute, 2004, pp. 181-194.

Table 2.2 Pankow Data: Average Ratio of Sealed to Standard-cure Cylinder Strengths for All Mixes

Age	Lab Study			Field Study		
	Minimum	Maximum	Average	Minimum	Maximum	Average
7 days	-----	-----	-----	0.90	1.10	0.99
28 days	0.80	1.01	0.88	0.77	0.94	0.84
56 days	0.74	0.91	0.86	0.77	1.03	0.84
90 days	0.77	0.98	0.86	0.73	0.89	0.82
120 days	0.72	0.93	0.83	0.76	0.91	0.83
180 days	-----	-----	-----	0.74	0.99	0.84

Table 2.3 Lab Study: Ratio of Sealed- and Air-dry Cure 4x8-in. Cylinders to Standard Cure 4x8-in. Cylinders at 28, 56, 90, and 120 days

	100% PC	**20% FA**	**30% FA**	**50% FA**	**50% Slag**	**50% Tern**	**70% Tern**	**Tern+ Silica Fume**	**Tern+ 3/8-in. Ltwt**	**Tern+ Ltwt Fines**	**Tern+ Low w/cm**	**15% FA 4000 psi**			
28 Days	A	B	C	D	E	F	G	H	I	J	k	l			
													Min	Max	Average
Sealed/Standard	0.82	0.89	0.80	0.91	0.86	0.85	0.85	0.89	0.87	1.01	0.87	0.94	0.80	1.01	0.88
Air/Standard	0.79	0.70	0.68	0.69	0.71	0.66	0.68	0.75	0.74	0.84	0.70	0.76	0.66	0.79	0.77
56 Days															
													Min	Max	Average
Sealed/Standard	0.83	0.74	0.79	0.82	0.85	0.90	0.84	0.88	0.91	0.98	0.90	0.94	0.74	0.91	0.86
Air/Standard	0.69	0.60	0.62	0.54	0.73	0.71	0.71	0.59	0.92	0.80	0.79	0.69	0.54	0.92	0.70
90 Days															
													Min	Max	Average
Sealed/Standard	0.77	0.80	0.80	0.79	0.89	0.82	0.80	0.97	0.84	0.98	0.88	0.92	0.77	0.98	0.86
Air/Standard	0.50	0.51	0.55	0.44	0.75	0.62	0.56	0.60	0.70	0.70	0.75	0.65	0.44	0.75	0.61
120 Days															
													Min	Max	Average
Sealed/Standard	0.72	0.76	0.84	0.86	0.86	0.82	0.83	0.90	0.72	0.93	0.88	0.89	0.72	0.93	0.83
Air/Standard	0.47	0.41	0.43	0.64	0.53	0.53	0.66	0.57	0.47	0.67	0.74	0.64	0.41	0.66	0.56

Table 2.4 Field Study: Ratio of Sealed and Air-Dried Cure 4x8 Field-cast Cylinders to 4x8 Field-cast Standard-Cure Cylinders at 7, 28, 56, 90, 120 and 180 Days

	100% PC	**25% FA**	**25% Slag**	**50% FA**	**50% Slag**	**50% Tern.**	**70% Tern**	**Tern+ Low w/cm**	**Tern+ Silica Fume**	**Tern+ Ltwt Fines**	**15% FA 4000 psi**			
7 Days	1	2	3	4	5	6	7	8	9	10	11			
												Min	Max	Average
Sealed/Standard	0.96	0.95	0.93	0.99	0.90	1.00	1.10	0.92	0.97	1.07	1.09	0.90	1.10	0.99
Air/Standard	0.89	0.92	0.84	0.98	0.84	0.98	0.94	0.92	0.84	0.92	1.05	0.84	1.05	0.92
28 Days														
												Min	Max	Average
Sealed/Standard	0.77	0.84	0.77	0.94	0.82	0.84	0.84	0.87	0.82	0.90	0.85	0.77	0.94	0.84
Air/Standard	0.64	0.62	0.63	0.67	0.66	0.71	0.62	0.67	0.60	0.65	0.56	0.56	0.71	0.64
56 Days														
												Min	Max	Average
Sealed/Standard	0.77	0.78	0.82	1.03	0.79	0.84	0.83	0.81	0.84	0.88	0.85	0.77	1.03	0.84
Air/Standard	0.60	0.46	0.61	0.52	0.65	0.60	0.52	0.58	0.52	0.61	0.45	0.45	0.65	0.56
90 Days														
												Min	Max	Average
Sealed/Standard	0.81	0.73	0.81	0.89	0.81	0.84	0.81	0.78	0.81	0.88	0.88	0.73	0.89	0.82
Air/Standard	0.58	0.45	0.58	0.51	0.56	0.46	0.52	0.53	0.45	0.59	0.41	0.41	0.59	0.51
120 Days														
												Min	Max	Average
Sealed/Standard	0.80	0.76	0.79	0.83	0.85	0.84	0.85	0.82	0.80	0.91	0.87	0.76	0.91	0.83
Air/Standard	0.54	0.41	0.53	-----	-----	-----	0.45	0.53	0.41	0.55	0.37	0.37	0.55	0.47
180 Days														
												Min	Max	Average
Sealed/Standard	0.84	0.80	0.76	-----	-----	-----	0.90	0.74	0.79	0.99	0.88	0.74	0.99	0.84
Air/Standard	0.51	0.41	0.57	-----	-----	-----	0.47	0.46	0.40	0.56	0.32	0.32	0.57	0.46

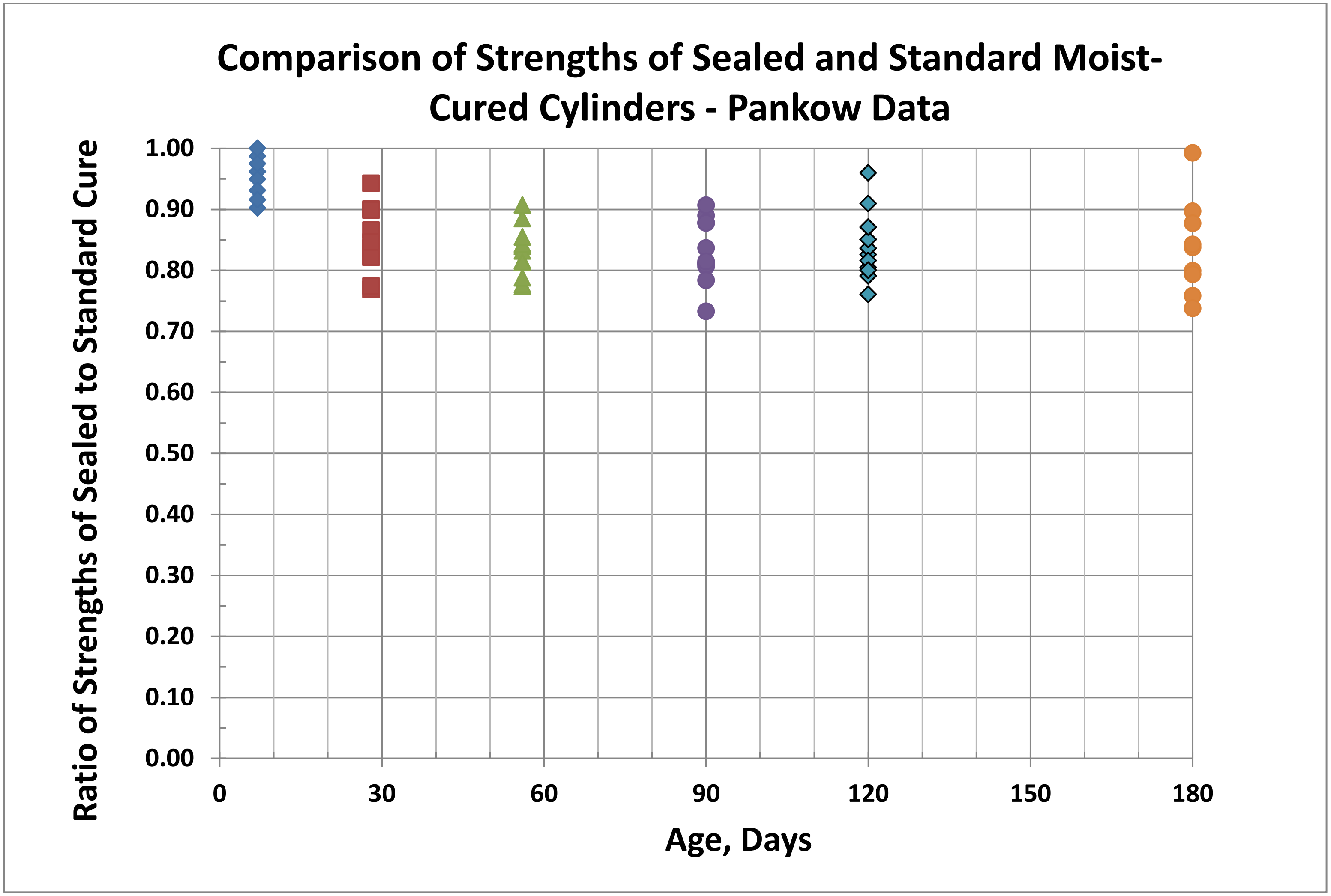


Figure 2.2 Pankow Data: Sealed-cure to water-cure strength ratios for cylinders at ages up to 180 days

Table 2.5 Pankow Data: Average Ratio of Air- to Standard-cure Cylinder Strengths for All Mixes

Age	Lab Study			Field Study		
	Minimum	Maximum	Average	Minimum	Maximum	Average
7 days	-----	-----	-----	0.84	1.05	0.92
28 days	0.66	0.79	0.77	0.56	0.71	0.64
56 days	0.54	0.92	0.70	0.45	0.65	0.56
90 days	0.44	0.75	0.61	0.41	0.59	0.51
120 days	0.41	0.66	0.56	0.37	0.55	0.47
180 days	-----	-----	-----	0.32	0.57	0.46

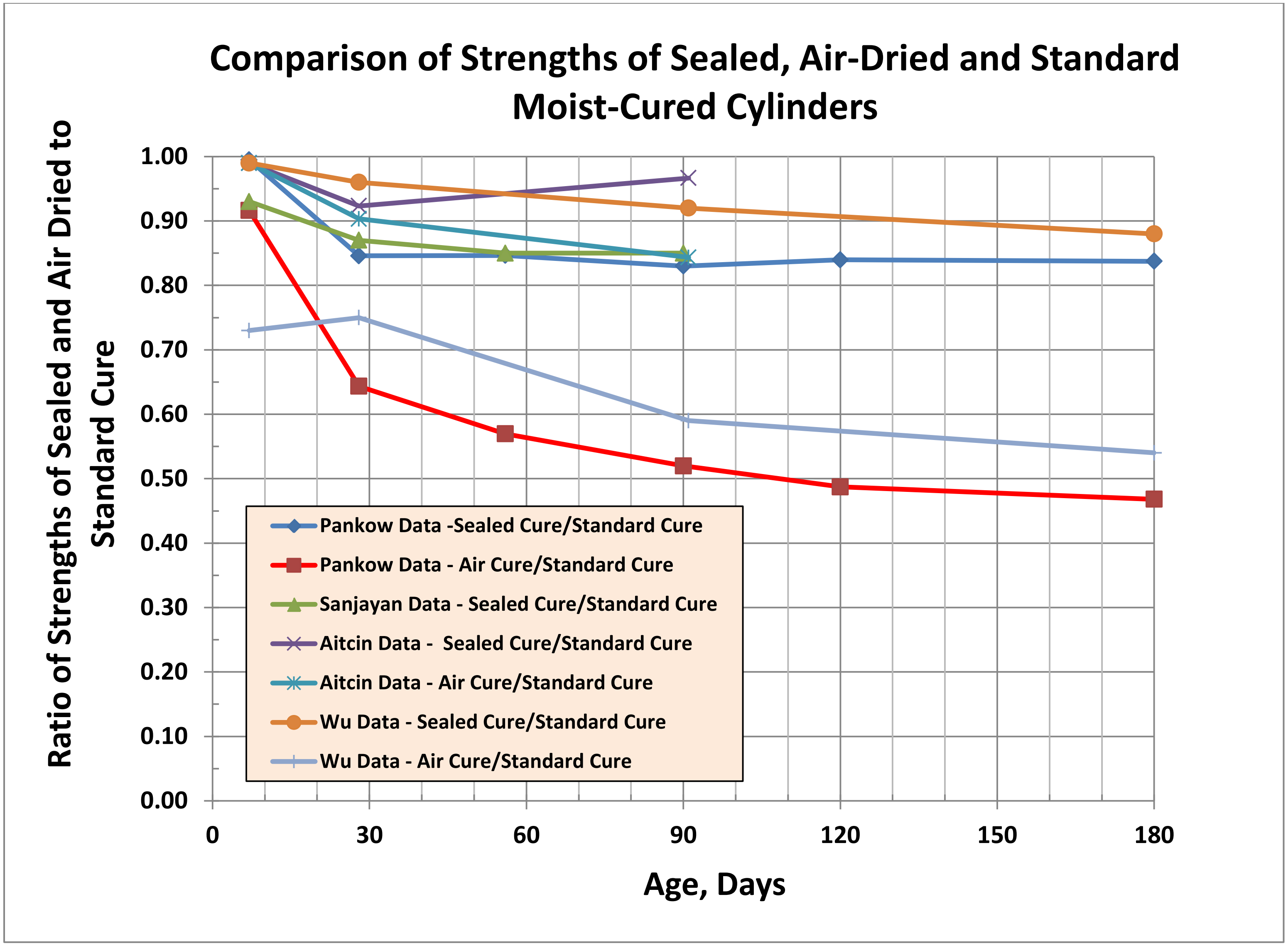


Figure 2.3 Average sealed-cure to water-cure and air-cure to water-cure strength ratios for cylinders at ages up to 180 days.

Chapter 3 Low-strength Investigation: Core-to-Cylinder Relationship

Section 5.6.2.4 of ACI 318-11, "Building Code Requirements for Structural Concrete," defines a strength test as the average of the strengths of at least two 6 by 12 in. cylinders or at least three 4 by 8 in. cylinders made from the same sample of concrete and tested at the age designated for determination of the specified strength, f_c'. Section 5.6.5 provides the procedure to be followed if a strength test result fails to meet specified acceptance criteria. A low-strength investigation isn't required unless a strength test result falls below the specified compressive strength, f_c', by more than 500 psi when f_c' is 5000 psi or less, or by more than 0.10 f_c' when f_c' is more than 5000 psi. In these cases, as stated in Section 5.6.5.1 of ACI 318-11, steps must be taken to ensure that the load-carrying capacity of the structure is not jeopardized. Chapter 2 of this report describes procedures used to confirm the likelihood of low-strength concrete that significantly reduce load carrying capacity, based on calculations. Section 5.6.5.2 states the following:

> "If the likelihood of low-strength concrete is confirmed and calculations indicate that load-carrying capacity is significantly reduced, tests of cores drilled from the area in question in accordance with ASTM C42 shall be permitted."

Note that drilling and testing cores is not required by the Code section quoted. The Commentary states further that, *in extreme cases*, core tests may be conducted:

> "The building official should apply judgment as to the significance of low test results and whether they indicate need for concern. If further investigation is deemed necessary, such investigation may include nondestructive tests or, in extreme cases, strength tests of cores taken from the structure."

Even with these requirements and guidance, however, low strength tests usually result in cores tests, the results of which must meet the following requirements from Code Section 5.6.5.4:

> "Concrete in an area represented by core tests shall be considered structurally adequate if the average of three cores is equal to at least 85 percent of f_c' and if no single core is less than 75 percent of f_c'. Additional testing of cores extracted from locations represented by erratic core strength results shall be permitted."

In buildings using green concrete mixtures, the strength level that initiates a low-strength investigation, the procedures for such investigations, and requirements for structural adequacy are the same as those just described. But with concrete containing large amounts of SCMs—and often with the low w/cm ratios needed to offset slow strength gain—should the core acceptance criteria be the same as for all other concretes? Answering that question requires tracing the background of the ACI 318 core-test acceptance criteria.

3.1 ACI Core Acceptance Criteria: A History

The core-strength acceptance criteria were first published in the 1971 revision of ACI 318. They have remained essentially the same for 40 years, through the 2011 revision.

The ACI 318-71 Commentary provided background details (cited references) and intent (conservatively safe acceptance criteria) as shown below.

> "For cores, if required, conservatively safe acceptance criteria have been provided which, if met, should assure structural adequacy for virtually any type of construction. [4.2, 4.3]"
>
> 4.2. Bloem, Delmar L., "Concrete Strength Measurement- Cores Vs. Cylinders,'' *ASTM Proceedings*, 1965, pp. 668-696.
>
> 4.3. Bloem, Delmar L., "Concrete Strength in Structures," *ACI Journal*, *Proceedings* V. 65, No. 3, Mar. 1968, pp. 176-187.

It's important to note that the core strength acceptance criteria were considered "conservatively safe" for "virtually any type of construction" and were based on work by Bloem in 1965 and 1968. Bloem's work is evaluated in a later section of this document, as an aid in understanding why the core acceptance criteria are conservatively safe. In addition, Bloem's work will be reviewed to compare his concrete materials and properties with those of green concrete and if his test specimens and procedures approximate those currently used.

The ACI 318-83 Commentary added two new references (Malhotra 1976; Malhotra 1977) on cores but did not change the low-strength investigation procedure or core strength acceptance criteria. Unlike Bloem's work, which was still referenced in 1983 with the same sentence shown in the 1971 edition, the two papers by Malhotra were included in the references.

A final reference to the Commentary regarding investigation of low-strength test results (Bartlett and MacGregor, 1994) was added to ACI 318-02. This reference presented data on the effect of moisture condition on core strengths, which did result in Code changes concerning core conditioning. The low-strength investigation procedure and core strength acceptance criteria remained the same.

<u>3.1.1 How Core Conditioning Affects Core Strength-Test Results:</u> It is important to note the effect of core conditioning methods on core strength-test results. Cores are conditioned in accordance with ASTM C 42. ASTM C 42 is explicit with respect to most issues involved in the testing process, but allows the specifying authority to direct a method for conditioning the cores prior to test.

ACI 318-1999 and earlier revisions directed that cores be:

- soaked for at least 40 hr. prior to test and tested wet if the cores represent concrete that will be more than superficially wet under service conditions, or
- tested dry after 7 days of air drying at 60°F to 80°F and less than 60% relative humidity if the cores represent concrete that will be dry under service conditions

Based on the Bartlett and MacGregor research, ACI 318-02 and later editions directed that cores be obtained, moisture conditioned by storage in watertight bags or containers, transported to the laboratory, and tested in accordance with ASTM C42. Testing was required no earlier than 48 hours and not later than 7 days after coring unless approved by the registered design professional. ASTM C42 requires wiping drilling water from core surfaces and allowing surface moisture to evaporate for up to 1 hour after drilling before placing the cores in plastic bags. These revisions were based on Bartlett and MacGregor's research, which indicated that different core conditioning methods resulted in different measured core strengths. The strength of air-dried cores was on average 14 percent higher than the strength of soaked cores. The strength of cores with a negligible moisture gradient, conditioned in accordance with the current ACI 318 requirements, was on average 9 percent larger than that of soaked cores.

Thus, the strengths of cores conditioning by bagging were about 5% lower than the strengths of cores conditioned by air drying. Because ACI 318 still requires the average of three cores to equal or exceed 85 percent of the specified concrete strength, meeting that requirement is now less likely for cores taken from structures that would be dry under service conditions.

The strength differences for cores conditioned by differing methods should be considered when evaluating core-strength test results reported in the literature.

3.2 Project Experience with Green Concrete Core-Strength Test Results

As background information for an in-depth analysis of Bloem's work, experience on construction projects utilizing high-strength green concrete is summarized as follows.

<u>3.2.1 Mat foundation in California:</u> In 2010, the concrete for a mat foundation contained 200 lb of portland cement, 267 lb of slag cement, and 200 lb of Class F fly ash totaling 667 lb of total cementitious materials. The w/cm was 0.36 and the specified concrete compressive strength was 8000 psi at age 90 days. At 90 days, the 32 compressive strength test results for <u>field-cast</u> and lab-cured concrete cylinders averaged 8540 psi with a minimum of 8120 psi and a maximum of 9190 psi. Thus the cylinder strength test results for all 32 cylinder sets met the specified strength of 8000 psi.

Because a question arose about core-to-cylinder strength relationships in another part of the structure, six 6-in. and seven 3-in. nominal diameter cores were removed from the top of the mat foundation 160 days after concrete placement. The core strength test results for the 6-in.-diameter cores averaged 5820 psi with a minimum of 5270 psi and a maximum of 6200 psi. The core strength test results for the 3-inch diameter core averaged 5820 psi with a minimum of 5270 psi and a maximum of 6200 psi. The combined average core strength was 5940 psi.

No cylinders had been tested after 90 days and no hold cylinders were available for testing at 160 days. Thus, in calculating the core-to-cylinder strength ratio, the average 90-day cylinder strength test result of 8540 psi was used. The ratio of the average 160-day core strength to 90-day cylinder strength ratio was 0.70. This was considered to be an overestimate because the green concrete cylinder strength would be expected increase with time. Thus, the estimated core-to-cylinder ratio for equal age cores and cylinders was even lower than 0.70—far below the ACI 318 core strength acceptance ratio of 0.85. For the 13 cores tested, only two of the 3-inch nominal diameter cores had a core-to-cylinder ratio of 0.85 or above. Strength retrogression with age was also noted.

Fortunately, the mat foundation concrete had been previously accepted based on the 32 cylinder strength test results, and the engineer's calculations demonstrated structural adequacy even with the lower than expected core strength results. The broader question remain unaddressed: How do we explain that the 90-day cylinder-test results showed the mat foundation concrete to be acceptable when the core-test results indicated a strength less than 0.85 f_c', for in-place concrete cored 70 days later? Which measure gives the better indication of a structural member's load-carrying capacity?

3.2.2 Columns in Ohio: In 2011, high strength 4 x 4-ft square green concrete columns, were specified for a 15-story building in Ohio. The concrete contained 873 lbs. of portland cement, 218 lbs. of fly ash and 109 lbs. of silica fume totaling 1200 lbs. of total cementitious materials. The water-to-cementitious materials ratio was 0.20. The specified compressive was 10,000 psi at 28 days. Because the contractor and ready-mixed concrete producer were concerned about the strength development in such a high cementitious content mix, a full-size mock-up was constructed and tested prior to construction.

The 28-day standard-cured cylinder compressive strength was 10,925 psi. Six 4-in. nominal diameter cores were removed at 28 days and tested. The average core strength test result was 6435 psi with a minimum of 5700 psi and with a maximum of

7630 psi. The average 28-day core-to-cylinder ratio was 0.59. Again, for this project, the cylinder strengths from concrete placed in the mock-up column met ACI 318 cylinder strength criteria whereas the tested cores from the column did not meet the ACI core-strength criteria.

3.2.3 Personal communication at the NRMCA International Concrete Sustainability Conference: The authors presented a paper, "Comparison of Cylinder and Core Strengths for Low-Carbon-Footprint Concretes" at the NRMCA International Concrete Sustainability Conference held May 6-8, 2013, in San Francisco. After the presentation, one attendee briefly described his company's experience on a construction project for which core-to-cylinder ratios were less than the ACI directed 0.85 even though all of the tests on laboratory-cured cylinders molded at the jobsite yielded compressive strengths in excess of the specified strength. He noted that his project was in litigation over this issue with several million dollars at stake, so he could not discuss further details.

Section 3.2.3 describes anecdotal evidence, whereas the information described in Sections 3.2.1 and 3.2.2 resulted from two field investigations for which specific data were available but with limited sample sizes. In the concrete literature dealing with core strength to cylinder strength ratios, however, conclusions from one study were based on large samples sizes and many different concrete mixtures.

3.2.4 Industry literature confirmation: Strength data from 771 cores taken from large elements cast using 22 concrete mixtures reported in five investigations were analyzed (Bartlett and MacGregor 1997). The researchers concluded that the ratio of in-place strength to standard cylinder strength decreases as the maximum temperature sustained during hydration increases. If the concrete mixture contained silica fume, Class C fly ash, or slag cement, the ratio of the in-place strength at 28 days to the standard 28-day cylinder strength of the same concrete was markedly less than that observed for concretes not containing supplementary cementitious materials. The ratio of the average 28-day core strength to the average 28-day cylinder strength was 0.15 smaller than that for concretes not containing these supplementary cementitious materials.

Note that the cores from the mat foundation in California and the column in Ohio were all conditioned by bagging. The core-test results used by Bartlett and MacGregor were all corrected to equivalent bagged core strengths using correction factors to account for conditioning effects on cores either soaked or dried as recommended in ASTM C42 core testing standards used before 2003.

3.2.5 Is 85 percent of f_c' an appropriate acceptance value for green concrete cores?
Why have core-strength test results for the projects cited not reached a core-to-cylinder

strength ratio of 0.85, even at core ages greater than 120 days? Several factors, alone or in combination, may be a part of the cause.

- Concrete studied in Bloem's 1965 and 1968 research is not representative of today's high-strength concretes.
- Concrete studied in Bloem's 1965 and 1968 research is not representative of concretes with large percentages of cement replaced by SCMs.
- Concrete in Bloem's 1965 and 1968 research is not representative of concretes with much lower w/cms than the w/cs he used.
- Cores taken from Bloem's 1965 and 1968 test slabs and column are not representative for cores taken from much larger structural elements where temperature increases reduce in-place strength.

In this Chapter, the influence of these factors is explored in more detail.

3.3 The Bloem 1965 and 1968 Data

The first step in exploring the factors mentioned is reviewing Bloem's concrete composition, fresh and hardened concrete properties, size of the specimens from which cores were removed, and core conditioning methods. The cylinder and core-test results and the core-to-cylinder relationships are also reviewed.

3.3.1 Basic Test Data: The information for Bloem's 1965 and 1968 research is shown below.

1965 Mix Information (Ingredients, Quantities, Properties) and Size of Test Specimens

- Three mixes: gravel, expanded shale, crushed limestone aggregates
- Cement content: 525, 578, 527 pcy (no SCM)
- No specified water content, all batched to slump: 4.8, 1.3, 3.5 inch
- No admixtures except for air-entraining agent
- Strength Levels, estimated f'c: 4500, 3500, 3500 psi
- Test Specimens: 4- and 8-in thick slabs; 8 x 26-in columns
- Cores tested wet and dry.

1968 Mix Information (Ingredients, Quantities, Properties) and Size of Test Specimens

- Strength Levels, estimated f'c: 4750, 3000, 3500 psi
- Water/Cement Ratios: 0.68, 0.69, 0.77
- Test Specimens: 6-inch thick slabs
- Cores tested wet and dry.

TABLE I—CHARACTERISTICS OF FRESH CONCRETE (SERIES 189)*

Date mixed	Description of cement	Cement content	Water content	Slump	Unit weight	Air content	Temperature
U. S. customary units							
		lb per cu yd	lb per cu yd	in.	lb per cu ft	percent	F
2-21-66	Type III, Sample 1	427	292	1.8	148.5	2.4	71
3-15-66	Type I	478	333	6.8	140.8	7.2	63
4-18-66	Type III, Sample 2	413	317	4.8	142.8	5.3	70

It is obvious that Bloem's materials and methods, while appropriate in the 1960's don't represent many current concrete compositions, in-place concrete masses, and core testing procedures with respect to the following:

- The total cementitious contents were between 413 and 578 lb/yd^3
- No supplementary cementitious materials (SCMs) were used.
- The water-to-cement ratios were high (0.68 to 0.77).
- No water-reducing admixtures were used.
- The minimum specimen dimensions were small for the 4-, 6-, and 8-in.-thick slabs and one vertical specimen; an 8x26-in. cross-section column.
- Cores were tested after conditioning by air drying or soaking in water instead of bagging them to reduce any moisture gradients.

<u>3.3.2 Limitations of Bloem's Research:</u> Although Bloem's research used concrete ingredients and quantities producing properties typical of the mid-1960, they do not represent the concrete mixtures used for green concrete today. For instance, California is a leader in using green concrete for buildings; however, the state is in a very active seismic region with designers using specified strengths, f_c', that range from 6000 psi to 12,000 psi. These strengths are much higher than those for the concrete in Bloem's research, which had *estimated* f_c', values that range from 3000 psi to 4750 psi. The strength levels reported for many current of the green concrete projects fall outside the range of Bloem's test values. In agreement with the lower strength levels are the lower amounts of total cementitious materials used—all less than 600 lb/yd^3. The limitations are further discussed as follows.

Strength Levels—The difference in strength levels may be important. ACI 363.2R-11 "Guide to Quality Control and Testing of High-Strength Concrete" recommends that the current ACI 318 core-strength acceptance criteria be used only if a separate core-to-cylinder relationship for the mixture has not been established. As the following excerpt from ACI 363.2R-11 states, a correlation curve for each high-strength mix should be established. Note that ACI 363 defines high-strength concrete as that which exceeds a specified strength level, f_c', of 6000 psi.

> "A correlation curve should be established for each high-strength mixture to relate the strength of extracted cores (normally 4 in. in diameter) to the strength of specimens used for acceptance testing, that is, 6 by 12 in. or 4 by 8 in. cylinders. Then, if coring becomes necessary, the relationship has been established, agreed upon, and is ready for conclusive interpretation. In the absence of correlation data, the provision 5.6.5.4 of ACI 318-08 should be used."

Supplementary Cementitious Materials—No SCMs were used in any of Bloem's work. Based on the analysis of core data (Bartlett and MacGregor 1997), concretes containing

SCMs show a lower core-to-cylinder ratio (by 0.15) than concrete with no SCMs. This is a significant difference.

Water-Cement Ratio and Curing Method—Bloem reported water-cement ratios (w/c) in his 1968 paper as 0.68, 0.69 and 0.77. Almost all of the current green concrete mixtures have water-to-cementitious-materials ratios (w/cm) less than 0.50. Many have a w/cm ratio less than 0.40 and some approach 0.30. Bloem used what he called good curing and poor curing. Good curing consisted of the concrete being sprayed with a curing compound and later covered with wet burlap and sheet plastic for 14 days. Poor curing consisted of leaving the slab uncovered after placement. Water curing can have a beneficial effect on increasing the strength of small-sized specimens with w/cm ratios less than 0.40. Bloem's work doesn't approximate the w/cms and curing methods used today.

Heat Generation—Bloem's specimens were, for the most part, slabs that were 4-, 6-, and 8-in. thick. One 8x26-in. cross-section vertical column was tested. None of his specimens were massive enough or contained enough cement to generate a high internal temperature gain. Section 3.3 of ACI 214.4R-10, "Guide for Obtaining Cores and Interpreting Compressive Strength Results," describes strength loss with increased internal temperatures as follows:

> "Similarly, data analysis from large specimens reported by Yuan et al. (1991), Mak et al. (1990, 1993), Burg and Ost (1992), and Miao et al. (1993) indicate a strength loss of approximately 3% of the average strength in the specimen for every 10°F increase of average maximum temperature sustained during early hydration (Bartlett and MacGregor 1996a). Maximum temperatures recorded in these specimens varied between 110 and 200°F."

Although Bloem's 1965 paper acknowledges that the maximum internal temperature affects early and later age strengths, the internal temperature was not measured in either of his studies. Based on the analysis by Bartlett and MacGregor (1997) this is a serious limitation. Mat foundations are often designed to be 5 to 10 ft. thick and columns can approach 4 to 6 ft. thick. While SCM's are used to lower the internal temperature, it may still reach the commonly specified maximum temperature of 160ºF.

For instance, assume that current green concrete mixes are used in 4-ft square column. Also assume that the temperature rise the column is 40F higher than that in any of Bloem's specimens. Based on Bartlett and MacGregor's work cited in ACI 214.4R-10, the strength for a core from near the center of the 4-ft. square column would be 12% lower (3% for every 10F or 12% for a 40F difference) than the strength of a core from a smaller size column. Because of the lower core strength, if the original core-to-cylinder ratio was 0.85, it would reduce to 0.73 due to the sustained internal temperature. This is a significant effect that is unaccounted for in Bloem's work and thus unaccounted for in the ACI 318 target core-acceptance value of 0.85.

3.3.3 Bloem's Core-to-Cylinder Relationships

Bloem presented the core and cylinder strength data in both tables and graphical formats. Test results for the 8x26-in. cross-section column (Bloem 1965) are shown in Figure 3.1 and the test results for a 6-in.-thick slab (Bloem 1968) in Figure 3.2. Red lines are shown in each figure to denote the ACI 318 target value for acceptable core strength— 85% of f_c'.

Note the following regarding column data:

- Core strength at 91 days is compared to cylinder strength at 28 days
- Core strength is for water-soaked cores
- Good and poor curing methods were used

It is current practice to take cores almost immediately after receiving the test report stating that cylinder strength is low. In this case, the core strength and cylinder strength are measured at about the same age. The core strength at 91 days represents an expected strength increase due to 63 extra days of curing that is not measured with the 28-day cylinders. The water-soaking results in a decrease in strength when compared with bagged cores (current core conditioning requirement of ACI 318). If these opposite effects are assumed to offset each other, the core strengths could be reasonably close to strengths of same-age and bagged cores. With that as a preamble, it is interesting to note that for the well-cured columns, 4 of the 8 core strengths (85% to 90%) would meet the ACI 318 required 85% value and the other 4 (83% to 60%) would not meet the requirement.

The core-to-cylinder ratio of 0.60 for the core drilled near the top of the column is probably due to water-gain at the top of the 8-ft-tall column. This was noted by Bloem but readers were cautioned not to take cores from the top of a column. Cores are not usually removed from the top of a wall or column, however, because it is difficult to set the core drilling machine that high on a scaffold or anchor it to the wall or column. In practice, cores *are* removed from the top of mat foundations, which could result in a strength decrease caused by water-gain due to the mat depth. This possibility needs to be addressed. Otherwise, it is likely that most cores removed from the top of mat foundations will not meet the 85% ACI 318 acceptance value.

Figure 3.2 shows the core-to-cylinder relationship for a 6-inch thick slab. For the curve labeled 2, at 28 days, air-dried cores from a well-cured slab had a core-to-cylinder ratio of about 0.85 but that ratio dropped to 0.83 at 90 days and 0.74 at 364 days. Curiously, for this slab, core acceptance might have depended on the age at which the core was taken. Also note that curve 2 represents air-dried core strengths that, on average, would be 5% higher than bagged-core strengths.

Bloem's data presents the possibility of two undesirable outcomes:

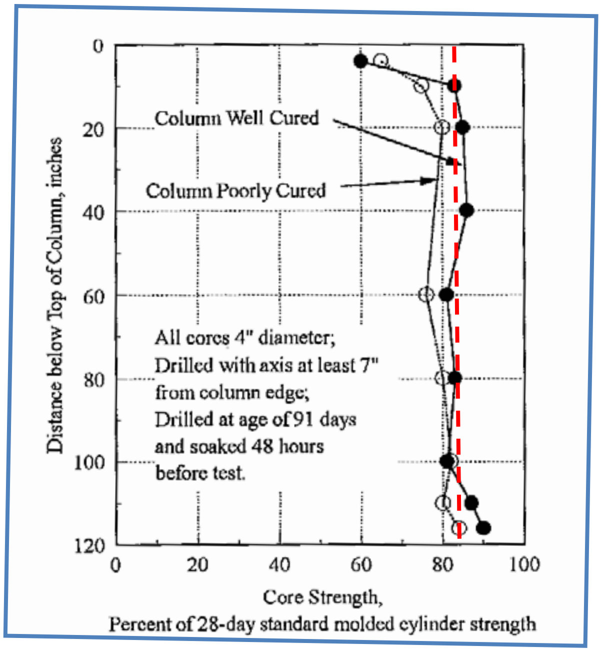
- Core taken from different locations along the same column will sometimes pass and sometimes fail to achieve the strength required by ACI 318 for acceptance
- Cores taken from a slab may sometimes pass and sometimes fail to achieve the strength required by ACI 318 for acceptance, depending only on the age of the cores when core strengths are measured.

These possible outcomes represent an unreasonable construction problem: The concrete is acceptable on the basis of standard-cured cylinder strengths, but the same concrete may not be acceptable based on strengths of cores taken at different locations or tested at different ages.

Figures 3.3 and 3.4 graphically summarize Bloem's core-to-cylinder relationships. These show the core-to-cylinder strength ratios for well-cured slabs and for a column, but with core and cylinder strengths measured at the same age. Note in Figure 3.4 that very few core-to-cylinder ratios exceed 0.85; only 4 out of 48 exceed 0.85 for Bloem's 1968 work. Including all of Bloem's core-to-cylinder ratios for well-cured concrete and air-dried cores puts the average about 0.80. The 0.85 core-to-cylinder ratio chosen by ACI 318 in 1971 represents about the 85th percentile. This summary emphasizes the statement in the ACI 318-71 Commentary that the core strength acceptance criteria is considered to be "conservatively safe" for "virtually any type of construction."

However appropriate this conservatism may have been for Bloem's test results in 1965 and 1968, it might not be suitable for the current green concrete construction in buildings. That was one of the reasons for planning and implementing the extensive laboratory and field research funded by the Charles Pankow Foundation, details of which and described in our companion Pankow Foundation Phase I report entitled "Lab and Field Data for Guide to the Use of "Green Concrete" in Building Construction." In Section 3.4, data from the companion report is used to further explore core-to-cylinder test results for green concrete.

See NEXT PAGE for Figs. 3.1 and 3.2



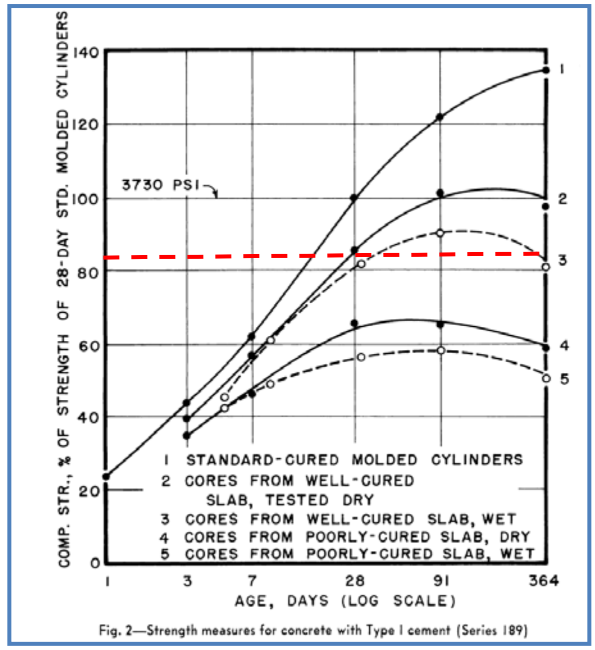


Figure 3.1. From Bloem's 1965 work. Red line shows the 0.85 ACI limit.

Figure 3.2. From Bloem's 1968 work. Red line shows the 0.85 ACI limit.

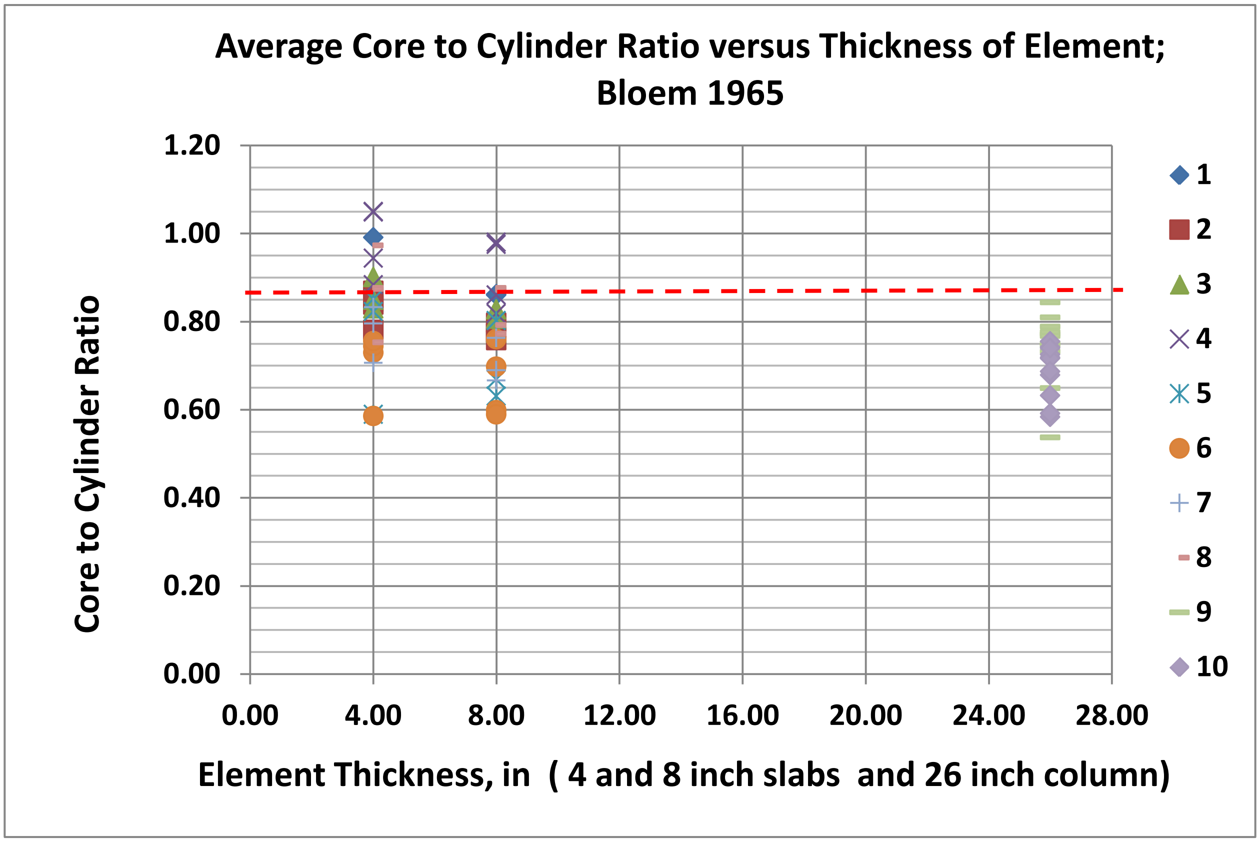


Figure 3.3. Average core-to-cylinder ratios at same age for Bloem's 1965 well-cured slabs and one column. Note column data differs from that presented in Figure 3.2 since that data was for a ratio of 91-day core strength to 28-day cylinder strength.

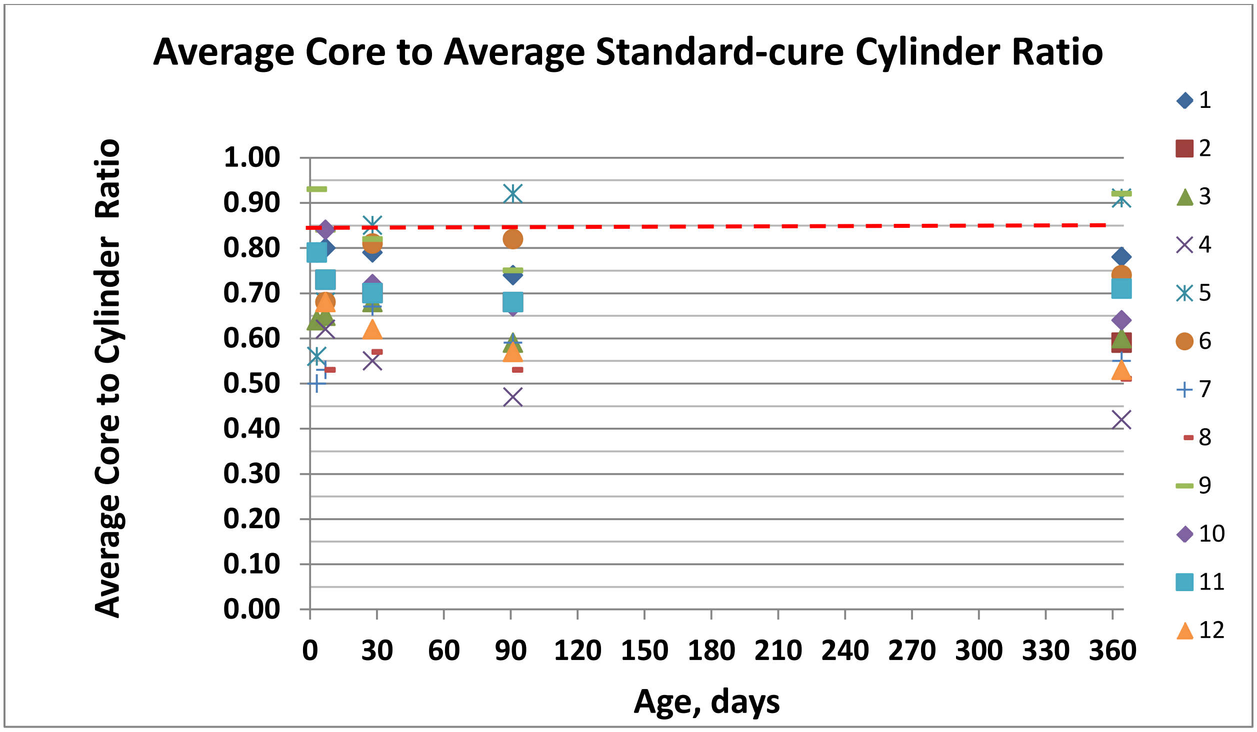


Figure 3.4 Average same-age core-to-cylinder ratios for Bloem's 1968 well-cured slabs. Note that very few results show a core-to-cylinder ratio above 0.85.

3.4 Pankow Core-to-Cylinder Data

The originally planned test data included results of 122 tests on cores and field-cast cylinders to establish the core-to-cylinder ratio for concrete at ages from 28 to 180 days. The mix ingredients, proportions, and properties of fresh and hardened concrete are reported in Phase I of this study. The raw data comparing core-to-standard-cure cylinder strength are shown in Figure 3.5. Figure 3.6 shows the data with sealed- instead of standard-cure (water) cylinder strengths used. Sealed cylinders more closely simulate the curing conditions for structural members. Figure 3.7 shows the core-to-sealed-cure cylinder strengths with an adjustment for internal temperature as recommended by ACI 214.4R-10. Adjusting for the lower strength due to the lack of moisture to cure the concrete (sealed curing) and the lower strength due to internal temperature increases the core-to-cylinder ratio.

<u>3.4.1 Raw Core-to-Cylinder Data:</u> The average core strength (based on three 4-in. nominal diameter cores) versus the average cylinder strength (based on three 4x8-in. cylinders) is plotted in Figure 3.5 versus the coefficient of variation of the three core strengths. Using the coefficient of variation of the core set as the x-axis was intended to determine whether the core-to-cylinder strength ratio was consistent at all levels of core variability. A visual inspection of the data points shows that the core-to-cylinder ratio appears to be unaffected by core-test variability levels. In addition, the regression line

plotted in Figure 3.5 shows that the core-to-cylinder ratio is approximately constant for all x-axis values. The y-axis values include standard-cured core and cylinder strengths. The cores were bag-cured in accordance with ASTM C 42 and the cylinders were water-cured in accordance with ASTM C 31. Refer to Phase I of this study for the details.

Of the 122 core-to-standard-cure cylinder strength ratios, only 4 would pass the ACI 318 required value of 0.85. This pass rate represents about 3% of the tests. The average core-to-cylinder ratio for the 122 test data was 0.65. This is considerably lower than the ACI limit of 0.85. As shown previously, however, this result somewhat mirrored Bloem's results in that the majority of core-to-cylinder ratios in his research did not meet the 85 percent acceptance criteria in ACI 318.

3.4.2 Retest Cores at 282 Days: Using the coefficient of variation of the core set as the x-axis for Figure 3.5 was prompted by the within-test coefficient of variation for the cores ranging from less than 2% to more than 35%. This was a possible indication of poor quality core sampling, moisture conditioning, capping, and testing. The intent of was to determine whether the core-to-cylinder strength ratio was consistent at all levels of core variability. The ratio was consistent, as indicated by the data in Figure 3.5. But there was still some concern about core testing quality.

To investigate the accuracy of the core testing, three 30-in.-long cores were drilled from field cure blocks containing five different concrete mixtures at an age of 282 days. Each 30-in. core was to be treated as follows:

- Concrete Central Supply cored, sawed, conditioned, capped, and tested specimens (15-4x8-in. cores)
- Concrete Central Supply cored, sawed, conditioned, and capped specimens, which Inspection Services Inc. (a privately owned laboratory) tested (15-4x8-in. cores)
- Inspection Services Inc. cored, sawed, conditioned, capped, and tested specimens (15-4x8-in. cores)

The results are given in Table 3.1. One of the cores was defective and yielded only one specimen for testing. Note that the coefficient of variation ranged from 3.0% to 22.7%, with the lowest values for four of the five blocks corresponding to the specimens cored, sawed, conditioned, capped, and tested by Inspection Services. In two cases, however, the average strengths for the Central Concrete Cores were more than 1000 psi higher than those from Inspection Services and in two other cases the average strengths were within about 300 to 400 psi of each other. This confirms the absence of a systematic error in Central Concrete's procedures that resulted in lower core strengths. In addition, it confirms the range of coefficient of variation found in the original data.

In further support of the wide range for within-test coefficient in cores tests, review of another research project currently in progress (Darwin 2013) showed that for 30 different concretes, the coefficient of variation of three-core strength tests ranged from about 1% to 20%. For the 122 tests conducted in this study, only 15 percent of the core strength tests had a coefficient of variation more than 20 percent.

3.4.3 Curing Adjustment

Based on the data presented in Chapter 2, sealed-cured cylinders did not reach as high a strength as water-cured cylinders at equal ages. That chapter presents Pankow test data and data from other researchers that show a strength difference of more than 1000 psi between cylinders. Optimum field curing of structural concrete members would usually consist of sealing the concrete, i.e., using a curing compound to minimize moisture loss. The core strength would then represent concrete that has been sealed while the standard-cure cylinder strength would represent concrete with water available for continued hydration of test specimens with a much higher area-to-volume ratio. This difference is especially important for w/cm ratios below 0.42, the approximate ratio at which there is just enough mixing water in the concrete for complete hydration of the cement (Mindess et. al. 2003).

Comparing a core strength from field concrete that is sealed to a water-cured cylinder strength will inevitably lead to a lower core-to-cylinder strength ratio. The lower core-to-cylinder strength ratio is not due to a difference in the concrete but to a difference in the curing. And even if water curing is used for the field concrete, the effect on cover concrete strength is greater than the effect on the interior concrete strength. Thus, to adjust the core and cylinder to the same level of curing, the core strength was compared to the sealed cylinder strength as shown in Figure 3.6

The core-to-cylinder strength ratio with sealed curing for cylinders was 0.75. This is a significant increase over the core to water-cured cylinder strength ratio of 0.65. This increase implies that the core strength represented by the 0.65 ratio simply indicates that the cores did not benefit from the water curing advantage given to the cylinders. It does not seem appropriate to judge the adequacy of the field concrete by comparison to laboratory concrete that continues to gain strength due to water curing.

Without the curing adjustment only 6 core-to-cylinder ratios exceed the 0.85 ACI 318 value. Although the curing adjustment significantly increased the average core-to-cylinder ratio from 0.65 to 0.75, only 11 out of 122 tests would pass the 0.85 ACI 318 value.

3.4.4 Internal Temperature Adjustment: As mentioned earlier in this Chapter, Section 3.3 of ACI 214.4R-10, “Guide for Obtaining Cores and Interpreting Compressive

Strength Results," refers to a strength loss of about 3% of the average strength in the specimen for every 10°F increase of average maximum temperature sustained during early hydration. Internal temperatures were measured in the cylinders and field concrete in this study. As expected, the cylinder temperatures increased by 1ºF or 2ºF. The average internal temperature of the field concrete increased by about 40ºF with a minimum and maximum increase in internal temperature of 27ºF and 54ºF, respectively.

Figure 3.7 shows the core-to-cylinder strength ratios using ACI 214.4R-10 strength loss information. The average core strength loss based on the average 40ºF increase in internal temperature would be about 12 percent. The actual strength loss for each field concrete tested was based on the measured internal temperatures shown in Table F14 of the Pankow Phase I report. Figure 3.7 shows that the core-to-cylinder ratio increased from 0.75 to 0.84 when an internal temperature adjustment was made. With the temperature adjustment, the total number of tests that would pass the 0.85 ACI 318 is 49 out of 122.

While ACI 214.4R-10 recognizes the strength loss due to an increase in internal temperature, the ACI 318 Commentary is silent on whether this adjustment should be considered when judging the adequacy of core strength test results. This adjustment could be a large factor in the decrease in core strengths because some internal temperature increases approach 100ºF.

<u>3.4.5 Comparison to Bloem's Data</u>: Table 4.2 summarizes the Bloem and Pankow data. The average and standard deviation of the core-to-cylinder ratio for the Bloem data were 0.80 and 0.10, respectively. The average and standard deviation of the core-to-cylinder ratio for the raw Pankow data were 0.64 and 0.12, respectively. The variability of the core-to-cylinder ratio as represented by the standard deviation is about the same for each study.

Based on the Bloem and Pankow raw (uncorrected) core data, about 20% of Bloem's core test results and only 5% percent of the Pankow core test results reached 85% of the standard-cured cylinder strength at the same age. While one major difference between the Bloem and Pankow tests was the use of SCM's for all but one of the mixtures tested, other differences such as mixture materials and proportions, differences in curing and temperature, and fresh and hardened concrete properties may also factor into the difference between the Bloem and Pankow data.

<u>3.4.6 Comparison with ACI 363.2R-11 References</u>: ACI 363.2R-11 "Guide to Quality Control and Assurance of High-Strength Concrete" provides core strength test results and conclusions with respect to that data and includes the following:

"A correlation curve should be established for each high-strength mixture to relate the strength of extracted cores (normally 4 in. in diameter) to the strength of specimens used for acceptance testing, that is, 6 by 12 in. or 4 by 8 in. cylinders. Then, if coring becomes necessary, the relationship has been established, agreed upon, and is ready for conclusive interpretation. In the absence of correlation data, the provisions of ACI 318 should be used."

"These data (see references below) indicate that the acceptance criteria for core strengths specified in ACI 318 are also applicable to high strength concretes."

- Cook, J. E., "10,000 psi Concrete," *Concrete International*, V. 11, No. 10, Oct. 1989, pp. 67-75
- Burg, R. G., and Ost, B. W., "Engineering Properties of Commercially Available High-Strength Concretes," *PCA Research and Development Bulletin RD104T*, 1992.
- Bickley, J. A.; Ryell, J.; Rogers, C.; and Hooton, R. D., "Some Characteristics of High-Strength Structural Concrete," *Canadian Journal of Civil Engineering*, V. 18, No. 5, 1991, pp. 885 -889.
- Bickley, J. A.; Ryell, J.; Rogers, C.; and Hooton, R. D., "Some Characteristics of High-Strength Structural Concrete: Part 2," *Canadian Journal of Civil Engineering*, V. 21, No. 6, 1994, pp. 1084-1087.
- Aïtcin, P.-C., and Riad, N., "Curing Temperature and Very High-Strength Concrete," *Concrete International*, V. 10, No. 10, Oct. 1988, pp. 69-72.

The information in this document is confusing and seems contradictory. The statements that an established correlation curve results in a relationship that "is ready for conclusive interpretation," implies that absent a correlation curve, core test results would lead to inconclusive interpretation. But the next sentence recommends that, in absence of a correlation curve, the provisions of ACI 318 should be used. This document also implies, but does not state, that the correlation curve is preferable to ACI 318 provisions. However, the document does not address suggested actions if the correlation curve results in a core-to-cylinder ratio for acceptance that is lower than the 0.*85f'c required* by ACI 318.

The document seems to contradict itself when it states that "these data" indicate that the acceptance criteria for core strengths in ACI 318 are also applicable to high strength concretes. If this were true, it would be unnecessary to establish a correlation curve. "These data" from ACI 363.2R-11 are shown and discussed below.

Cook 1989 — This data shows the range and average core-to-cylinder ratio at the same age. At 7 and 365 days, all cores exceed the ACI 318 limit. This is also reasonably true for the 28 day data where the lower core-to-cylinder ratio is stated at 0.84. However, the same is not true for the data at 56 or 180 days where the average core-to-cylinder ratio is close to the ACI 318 limit, but individual core-test results yielded core/cylinder ratios as low as 0.78. This would mean that 100% of the concrete represented by core strengths would be acceptable at 7, 28 and 365 days, but only about 50% of the concrete (based on a normal distribution) would be acceptable at 56 and 180 days. This anomaly of concrete being acceptable at one age but not another was also found in the Pankow study.

Table 5.2.1—Strength cores from 760 mm (30 in.) square columns (Cook 1989)

Age at test, days	Moist-cured cylinder strength at same age, percent	
	Range	Average
7	94 to 105	99
28	84 to 97	91
56	78 to 94	84
180	78 to 94	86
365	93 to 107	98

Notes: Cement: Type I -- 606 and 602 pcy
Water/cementitious ratio: 0.33, 0.29
Strength Level: 10,000 psi
Class C fly ash at 25 to 30% replacement
Admixtures: Type A and Type F
Core test condition unreported.

Burg and Ost 1992 – These data do not show core-to-cylinder data with both cores and cylinders tested at the same age. The data are for strengths of cores taken at 91 and 426 days and compared to strengths of 28-day-old moist-cured cylinders. In practice, cores are drilled almost immediately after a low-cylinder break is reported. It is not clear how this data can be used to suggest that the ACI 318 criteria applies to high-strength concrete.

Table 5.2.2—Strength of cores from 1220 mm (4 ft) cubes (Burg and Ost 1992)

Cementitious system	Age at test, days	28-day moist-cured 152 x 305 mm (6 x 12 in.) cylinder strength, percent	
		Range	Average
I	91	95 to 106	99
I + SF + FA		93 to 96	95
I + SF		85 to 90	88
I + SF		93 to 104	98
I + SF + FA		102 to 105	103
I + SF + FA		107 to 110	108
I + SF	426	109 to 123	117
I + SF + FA		104 to 106	105
I + SF		94 to 98	96
I + SF		100 to 111	107
I + SF + FA		104 to 113	109
I + SF + FA		122 to 124	123

*I = Type I portland cement; SF = silica fume; FA = fly ash.

Notes:
Water/cementitious ratios ranged from: 0.22 to 0.32
Strength Levels: 10,000 to 20,000 psi
Silica fume and fly ash replacement: 10 to 25%
Admixtures: HRWR and retarder
Core test condition unreported

Bickley et al. 1991, 1994 – This data compares cores tested at 1, 2 and 7 years with average 28-day moist-cured cylinder tests. Waiting this long to take cores is not practical for any ongoing construction. It is not clear how this data can be used to suggest that the ACI 318 criteria apply to high-strength concrete.

Table 5.2.3—Column core strength at later ages (Bickley et al. 1991, 1994)

Age at test, years	Average 28-day moist-cured cylinder strength, percent	
	Range	Average
1	90 to 109	97
2	91 to 107	100
7	97 to 100	99

Notes: Cement is CSA type 10 (Type I) -- 695 pcy
Cement replacement: 25% slag, 7.8% silica fume
Water/cementitious ratio: 0.31
Admixtures: WR and HRWR
Cores from 66.5 x 35.4 in columns
Cores tested dry.

Aitcin and Riad 1988 – This data also shows the results for cores taken at an extended time period, at 2 years. It compares the 2 year core strength to the 28-day moist-cured cylinder strengths.

"Aïtcin and Riad (1988) reported 2-year core strengths from columns made with Type I cement and silica fume. The average 2-year core strength was 97 percent of the strength of 28-day moist cured cylinders."

Notes: Cement is Type I -- 840 pcy
Silica fume replacement at 5.5%
Water-cementitious ratio is 0.26
Admixtures: HRWR and retarder
Cores from 44-in square columns
Cores test condition unreported.

Based on the preceding comments for the data cited, it is unclear how ACI Committee 363 concluded that that the “acceptance criteria for core strengths specified in ACI 318 are also applicable to high strength concretes.” For four of the five references, this is true only if cores can be removed and tested at 3 months to 7 years and compared to the 28-day moist-cured cylinder strength. This is not a realistic construction expectation. For Cook’s data, ACI 318 criteria works well at 7, 28 and 365 days but rejects 50% of the concrete at 56 and 180 days.

Strip 18- 24 hr. Air dry	**Table 3.1 Retest Strength and Variability of Cores from Field Constructed Blocks Core Testing by Central Concrete Supply and Independent Laboratory as Noted**							
Age About 280 days	Mix	#1, psi	#2, psi	#3, psi	Average, psi	Standard Deviation, psi	Coefficient of Variation, %	Comment
100% Cement	1	7786	6942	5872	6867	959	14.0%	Central core, cap and test
100% Cement	1	8440	7530	6280	7417	1084	14.6%	Central core and cap; Lab test
100% Cement	1	7330	6510	5900	6580	718	10.9%	Lab core, cap and test
70% Ternary	7	5472	6059	7412	6314	995	15.8%	Central core, cap and test
70% Ternary	7	4920	5920	7560	6133	1333	21.7%	Central core and cap; Lab test
70% Ternary	7	6540	5730	5420	5897	578	9.8%	Lab core, cap and test
70% Tern+ Low w/cm	8	8261	7834	6294	7463	1035	13.9%	Central core, cap and test
70% Tern+ Low w/cm	8	9030	8250	6450	7910	1323	16.7%	Central core and cap; Lab test
70% Tern+ Low w/cm	8	6450	6210	6080	6247	188	3.0%	Lab core, cap and test
70% Tern+ Ltwt Fines	10	6146	-----	-----	6146	-----	-----	Central core, cap and test
70% Tern+ Ltwt Fines	10	7410	5990	6410	6603	729	11.0%	Central core and cap; Lab test
70% Tern+ Ltwt Fines	10	6660	4600	4570	5277	1198	22.7%	Lab core, cap and test
15% Fly Ash 4000 psi	11	7415	6695	6617	6742	1035	15.4%	Central core, cap and test
15% Fly Ash 4000 psi	11	7170	6060	6990	6740	1323	16.7%	Central core and cap; Lab test
15% Fly Ash 4000 psi	11	5760	5350	5920	5677	188	3.0%	Lab core, cap and test

Table 3.2 Core-to-Cylinder Ratio Statistics				
Data Description	Number of tests	Average	Standard Deviation	Number of Tests Passing 0.85 ACI 318
Bloem 1965	27	0.78	0.09	11
Bloem 1968	56	0.81	0.11	6
Bloem total	83	0.80	0.10	17
Pankow raw data	122	0.64	0.12	6
Pankow sealed correction	122	0.75	0.12	11
Pankow sealed and temperature correction	122	0.84	0.13	49

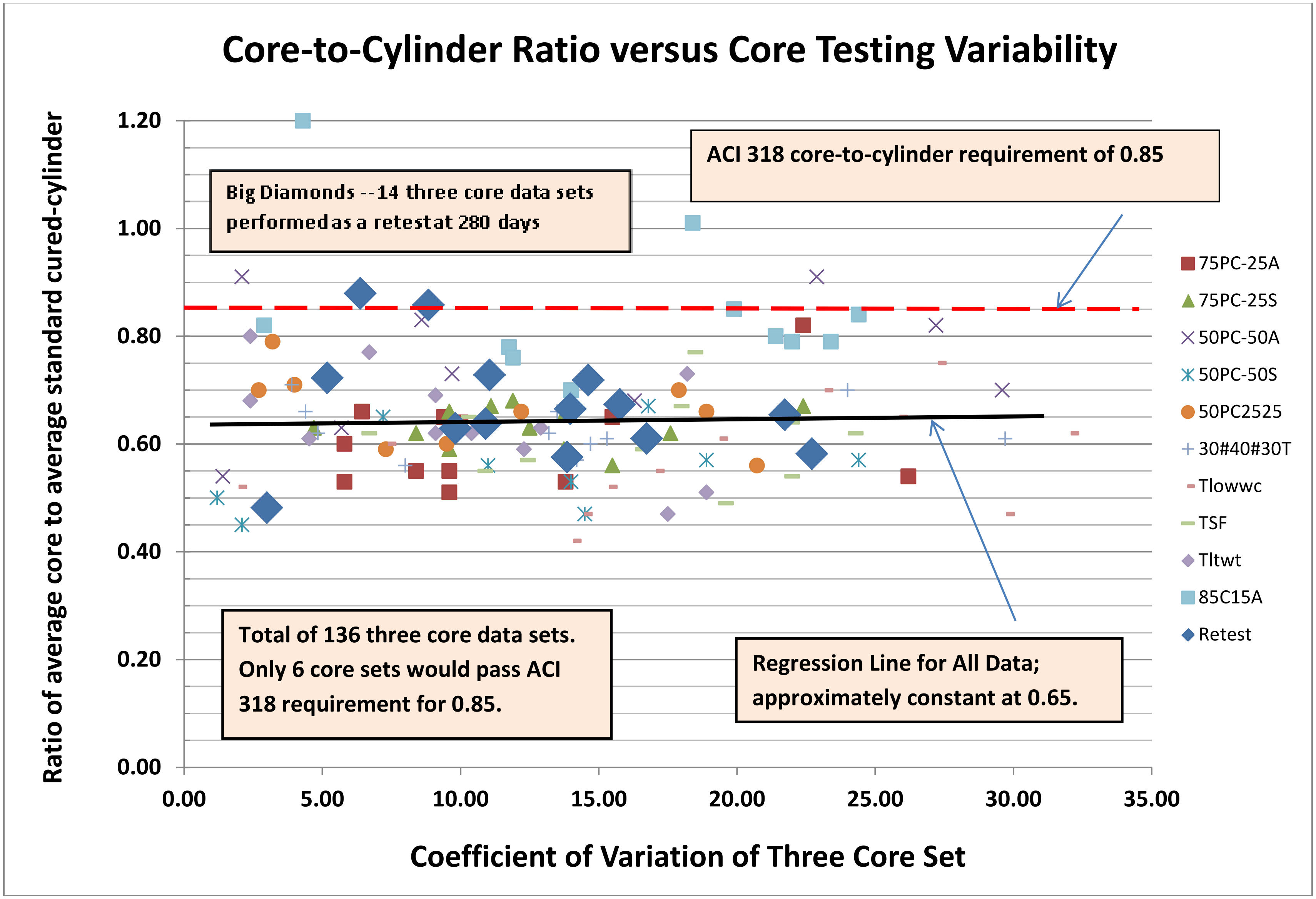


Figure 3.5 Raw (uncorrected) Core/Standard-Cure Cylinder Ratio vs Coefficient of Variation for All Tests at Ages Ranging from 28 to 280 days.

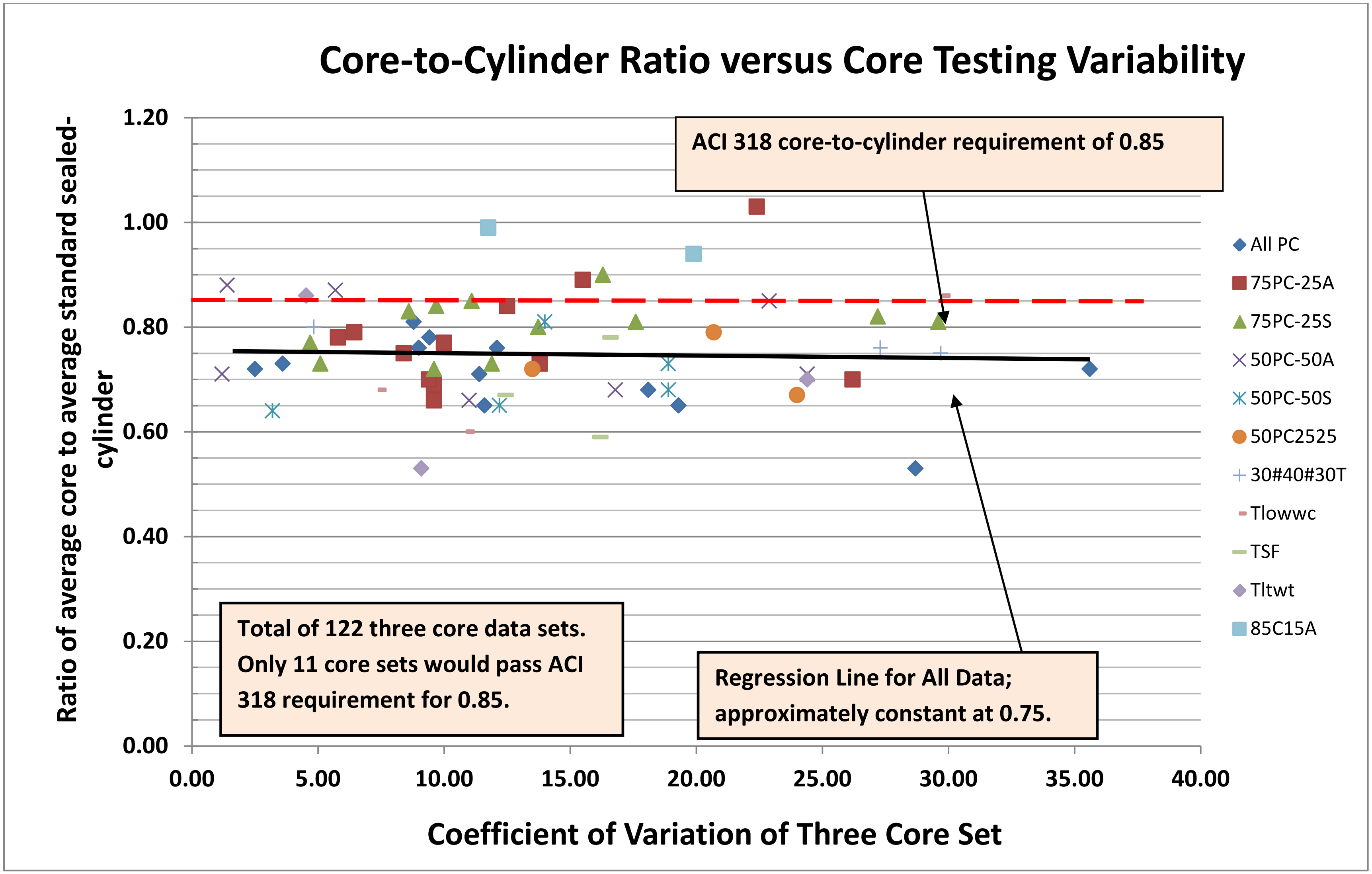


Figure 3.6 Raw (uncorrected) Core/Sealed-Cure Cylinder Ratio vs Coefficient of Variation for All Tests at Ages Ranging from 28 to 180 days.

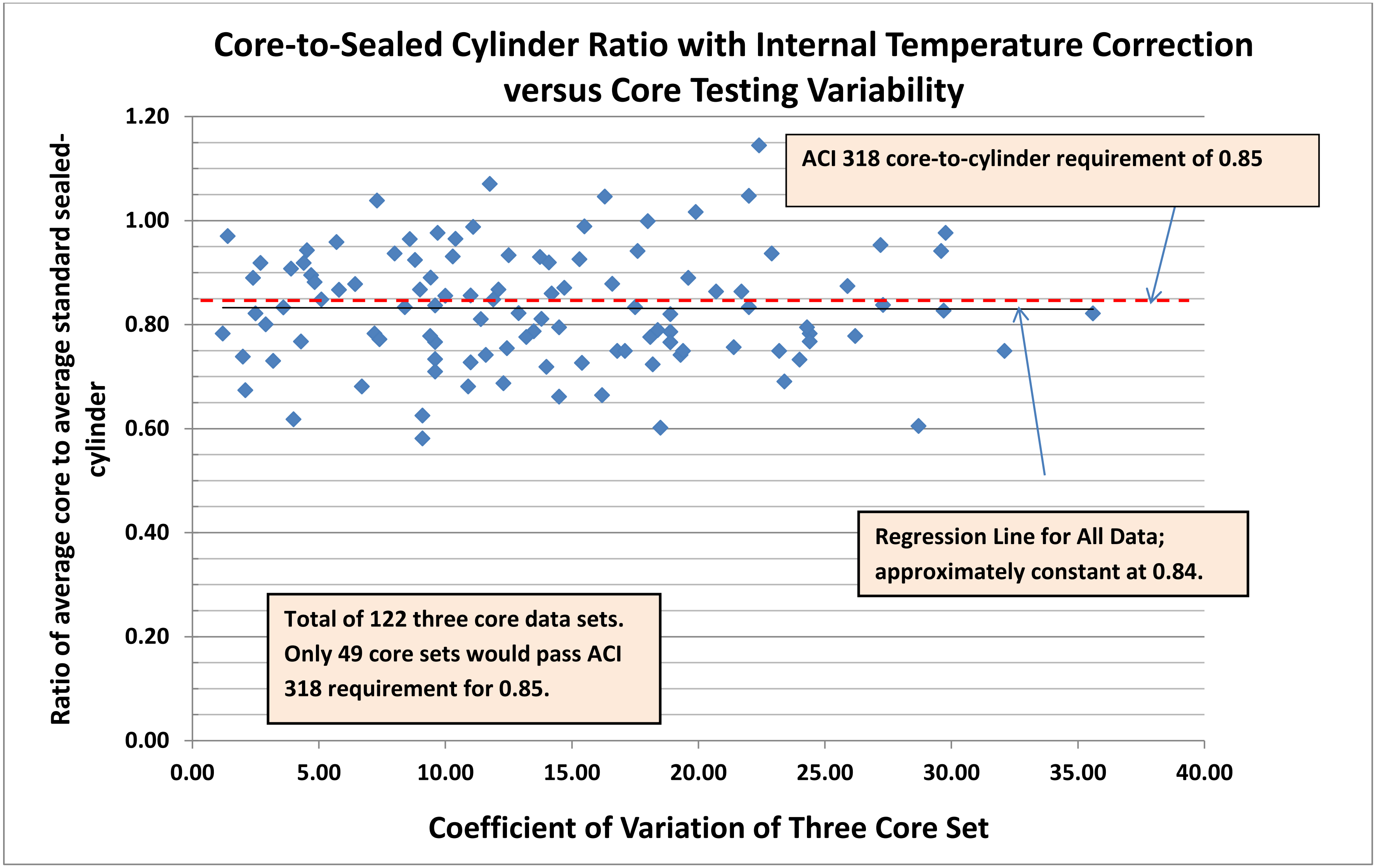


Figure 3.7 Core/Sealed-Cure Cylinder Ratio with Temperature Correction vs Coefficient of Variation for All Tests at Ages Ranging from 28 to 180 days.

3.5 References

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Chapter 4 Early-Age Compressive Strength for Construction Operations

Specification compliance for concrete compressive strength is traditionally set at 28 days. For green concrete, as discussed in Chapter 2, specification compliance testing is often delayed until 56 or 90 days to allow a greater reduction in portland cement content. While the specification compliance testing requirement must be met, there are also early-age strength requirements needed by the contractor for construction operations. The cost and schedule for a project can be considerably altered if the concrete can't achieve the needed strength to sequence construction operations. These early-age strength requirements are first reviewed, discussed with respect to the Pankow data, and then considered with respect to green concrete mixtures in general.

4.1 Early-stage Construction Operations

The following list includes traditional early-stage construction operations that are typically completed within 3 days of placing concrete.

Vertical form removal -- Column and wall forms and beam side forms that don't support formwork for slab or beam soffits are traditionally removed the day after concrete placement. Published research indicates that the minimum compressive strength necessary to prevent mechanical damage to the concrete surface when stripping forms can be as low as about 300 psi or as high as about 450 psi (Malisch 2009). Some project specifications require a minimum strength of 500 psi for a field-cured cylinder to verify that vertical form removal can proceed. ACI 347-04 "Guide to Formwork for Concrete" includes an elapsed-time criteria for form stripping, as follows:

> "Because the minimum stripping time is a function of concrete strength, the preferred method of determining stripping time is using tests of job-cured cylinders or concrete in place. When the contract documents do not specify the minimum strength required of concrete at the time of stripping, however, the following elapsed times [12 hours for walls, columns, and beam sides that don't support formwork for slab or beam soffits] can be used. The times shown represent a cumulative number of… hours, not necessarily consecutive, during which the temperature of the air surrounding the concrete is above 50°F (10°C)."

During warm weather, 12 hours may be enough time to attain the necessary strength. In cold weather, more time may be needed. If forms are to be removed as early as possible to maximize form reuse, the maturity method can be used to determine when the desired strength level has been reached.

On one green concrete building project, the vertical form removal was delayed as long as two weeks to allow the time needed for the concrete to gain enough strength to permit form removal without damaging the concrete surface. This is not the norm, however. Cold weather effects on early-age strength gain for green concrete mixtures increase the possibility of damage to the formed surface during form stripping operations.

Early-age freezing -- ACI 306R-10 “Guide to Cold Weather Concreting” states that one of the objectives of cold weather concreting practices is to:

> “Prevent damage to concrete due to early age freezing. When no external water is available, the degree of saturation of newly placed concrete decreases as the concrete matures and the mixing water combines with cement during hydration. Under such conditions, the degree of saturation falls below the critical level (the degree of water saturation where a single cycle of freezing causes damage) at the approximate time the concrete attains a compressive strength of 500 psi (Powers 1962). At 50°F, most well-proportioned concrete mixtures reach this strength within 48 hours.”

The last statement in ACI 306R-10 is for traditional concrete mixtures with cement replacements by SCMs of less than 20 percent and designed for compressive strength compliance at 28 days. Many green concrete mixtures with cement replacements greater than 50 percent and cured at 50°F are unlikely to reach 500 psi within 48 hours.

Saw-cutting joints -- Sawcutting too early in concrete, before the concrete gains enough strength, causes unacceptable raveling. Most saw operators determine the earliest time to sawcut by judging the degree of raveling in trial cuts made in the slab. FHWA research (Suprenant 1995), however, provides the approximate minimum compressive strength of concrete required before joints can be sawcut with minimal raveling. The compressive strength for acceptable sawcuts varied from a low of 310 psi for rounded, soft coarse aggregates to 1,270 psi for crushed, hard coarse aggregates. While these were the lowest and highest strength values, the compressive strength range needed to produce acceptable sawcuts was between 500 to 1,000 psi for most concrete mixtures.

The American Concrete Pavement Association (ACPA) discusses the effect of using slag cement as a replacement on sawcutting in its R & T Update “Slag Cement and Concrete Pavements.” They state that:

> “For concrete pavements, slag cement is typically used in proportions of 25 to 35 percent. Joints need to be sawed after the concrete has achieved enough strength to keep the sawcut from raveling but before internal stresses in the

> concrete become great enough to initiate an uncontrolled crack. The earlier the concrete can be cut without raveling, the better the chances are that the concrete will not crack before saw cutting. As a rough guideline, for slag cements the time to saw cut is delayed approximately 30 minutes for every 10 percent of slag cement replacing portland cement."

Post-tensioning -- Post-tensioning tendons inside plastic ducts or sleeves, are positioned in the forms before the concrete is placed. Once the concrete has gained strength, but before the service loads are applied, the desired force in the tendons is applied by jacks to produce the required prestress before anchoring the tendons at the outer edges of the concrete member. The Post-Tensioning Manual published by the Post-Tensioning Institute (PTI) states that "when tests of field-cured cylinders indicate that the concrete has reached the proper strength (usually 3000 psi) the stressing operation may begin." In traditional concrete building projects, stressing is usually begun in the first 1 to 3 days.

Elevated form removal -- Supporting forms and shores should not be removed from floor and beam soffits until these structural units are strong enough to carry their own weight and any approved superimposed load. In no case should supporting forms and shores be removed from horizontal members before the concrete has achieved the strength specified by the engineer/architect. ACI 347-04 recommends that:

> "The engineer/architect should specify the minimum strength of the concrete to be attained before removal of forms or shores. The strength can be determined by tests on job-cured specimens or on in-place concrete." Specifications for traditional building projects set the concrete strength requirements at 75% *f'c*. This strength requirement usually allows form stripping and reshore placement to occur between 3 to 7 days.

It may take as long as one month however, to reach 75% *f'c* with green concrete mixtures when the test age for *f'c* is set at 56 or 90 days. Waiting a month for form removal and reshoring significantly increases cost and slows progress, which can put the concrete portion of the project behind schedule. Also note that ACI 301-10 "Specifications for Structural Concrete" defaults to an in-place strength of *f'c* or higher until formwork and shoring can be removed. If f'c is defined at 56 or 90 days, this ACI 301 provision could require leaving forms in place for up to 56 or 90 days.

Although ACI has defined shoring and reshoring design procedures set out in ACI 347.2R-05 "Guide for Shoring/Reshoring Multistory Buildings" some engineers and contractors still use a rule of thumb that it takes 2 floors of reshores and one floor of shores to support a newly placed concrete floor. This is a reasonable rule-of-thumb on most traditional building projects. Depending on the concrete strength at stripping, however, green concrete buildings may require 4 floors of reshores with one floor of shores.

Miscellaneous construction operations -- Contractors often set anchors in previously placed concrete, with temporary braces for forms and other work attached to the anchors. Anchor design is based on the concrete strength. To wait until green concrete reaches an appropriate strength, anchors may have to be drilled and installed at a later age than that for traditional concrete. Alternatively, more anchors than usual may be needed to offset the lower green concrete strength at an earlier age.

4.2 Pankow Early-age Compressive Strength Data

Fig. 4.1 shows the Pankow data for early-age compressive strength of standard-cured cylinders. Note that the compressive strengths range from about 500 to 1,750 psi, about 1,000 to 3,000 psi, and about 1,500 to 5,000 psi. at one, three, and seven days, respectively. The lowest strength at each age is for the concrete mixture with a 28-day design strength of 4000 psi and 15% of the portland cement replaced by fly ash. The green concrete mixes averaged about 6,700 psi at 28 days, 7,900 psi at 56 days and 8,600 psi at 90 days. See Tables L3 and F3 in the Phase I report for the compressive strength data at ages from 1 to 180 days.

The one-day compressive strength level of standard-cured cylinders indicates enough strength to permit vertical form removal, prevent early-age freezing, and allow sawcutting of joints. The three-day compressive strength of 2,500 to 3,500 psi needed to permit post-tensioning or elevated form removal was achieved by only the following 4 of the 11 mixtures:

- Replacement of 50% of the portland cement with slag cement
- Ternary mixture with a low w/cm
- 100% portland cement with no SCMs
- Replacement of 25% of the portland cement with slag cement

This is significant because the strengths of these mixtures are for laboratory-curing conditions. The compressive strengths of field-cured cylinders are lower than those of the laboratory-cured cylinders because of less than optimal curing conditions. This effect would be magnified during cold weather concreting.

The Pankow green concrete mixtures were designed for an *f'c* of about 8,000 psi at 90 days. It is expected that construction operations for these green concrete mixtures in buildings would be delayed as compared with traditional concrete building mixtures with *f'c* equal to about 8000 psi.

4.3 Project Experiences

Previous consulting experiences with green concrete mixtures in buildings in California has shown that relatively cold weather, in the range of ambient temperatures between 50ºF and 60ºF, dramatically affected early-age strength and construction operations. Accelerators were required in the green concrete mixtures to permit early-age construction operations. In a few cases, the green concrete mixture proportions were adjusted by reducing the cement replacement percentage from 50% to 20% in cold weather to avoid delaying early-age construction operations.

The *Concrete International* article, "Sustainability through Strength," (Stevenson and Panian 2009) discusses the impact of strength gain on post-tensioning for the 50% slag cement concrete mixture used at the David Brower Center in Berkeley California.

> "Although in many ways they are similar to conventional concrete, mixtures containing large amounts of slag cement have some unique properties that affect design and construction. These include rate of strength gain, finishing behavior, and ability to form fine details. The rate of strength gain can have a significant impact on the construction schedule. Because the elevated slabs were post-tensioned, the time between concrete placement and slab stressing was a critical-path item. Typically, a 5000 to 6000 psi post-tensioned slab is required to reach 3000 psi before the strands can be stressed. Most conventional mixtures, under typical conditions, can meet this criterion in 3 to 5 days. The 50% slag cement mixtures used in the Brower Center often reached stressing strength within 5 days, but in a number of instances required 7 to 10 days. Because construction continued from late autumn through late spring, a wide range of temperatures was encountered. Placements during colder weather were typically slower to reach strength. As the Brower Center has only four elevated decks and the adjacent plaza portion was on a separate construction track, the net impact on the construction schedule was minor."

Stevenson and Panian also reported that on the same project it was difficult to get sharp concrete corners and edges with the 50% slag cement concrete mixture when the formwork was removed. They reported that "This was problematic where reveals, sharp corners, or other fine features were required in exposed surfaces. Patching provided a good final appearance, but further investigation is needed to determine how to adjust the mixtures to correct this behavior."

4.4 The Need for Early-age Compressive Strength Data Prior to Bidding

The lower early-age compressive strength of green concrete mixtures can significantly impact the schedule and costs of a green concrete building. For traditional concrete buildings, contractors use their experience in understanding and adjusting to concrete mixtures for early-age construction operations. This experience however, may not be applicable for green concrete buildings. To submit a firm bid to the owner, the contractor needs to know the 1-, 3- and 7-day compressive strengths of the green concrete mixtures in order to plan the schedule. It would also be helpful to be able to estimate the effect of cold weather on the green concrete strength and the cost and benefits of accelerators and other green concrete mixture adjustments.

It was shown in section 4.2 that the Pankow early-age strength results would delay normal 3-day post-tensioning. These green concrete mixes were designed for an f'c of about 8000 psi at 90 days. Green concrete mixtures designed for 4000 to 6000 psi at 56 to 90 days will have even lower early-age strengths, creating delays that impact the schedules to a greater extent. The need for early-age strength data for concrete used in green buildings can't be over-emphasized. The data is needed so the contractor can prepare a realistic construction operations schedule that minimizes delays and the costs associated with them.

4.5 Flexible Engineering is Needed

Scattered through-out sections 4.2 and 4.3 are the implications of "flexible engineering" during construction. For the David Brower Center, the engineers adjusted the 50% slag cement concrete mixture to a 40% slag cement mixture when needed. They also recognized the need for patching of exposed concrete at the edges and corners. On other projects in California, green concrete mixtures have been adjusted during construction. As more projects strive to reach up to 70% cement replacement, adjustments may be needed during construction.

4.6 References

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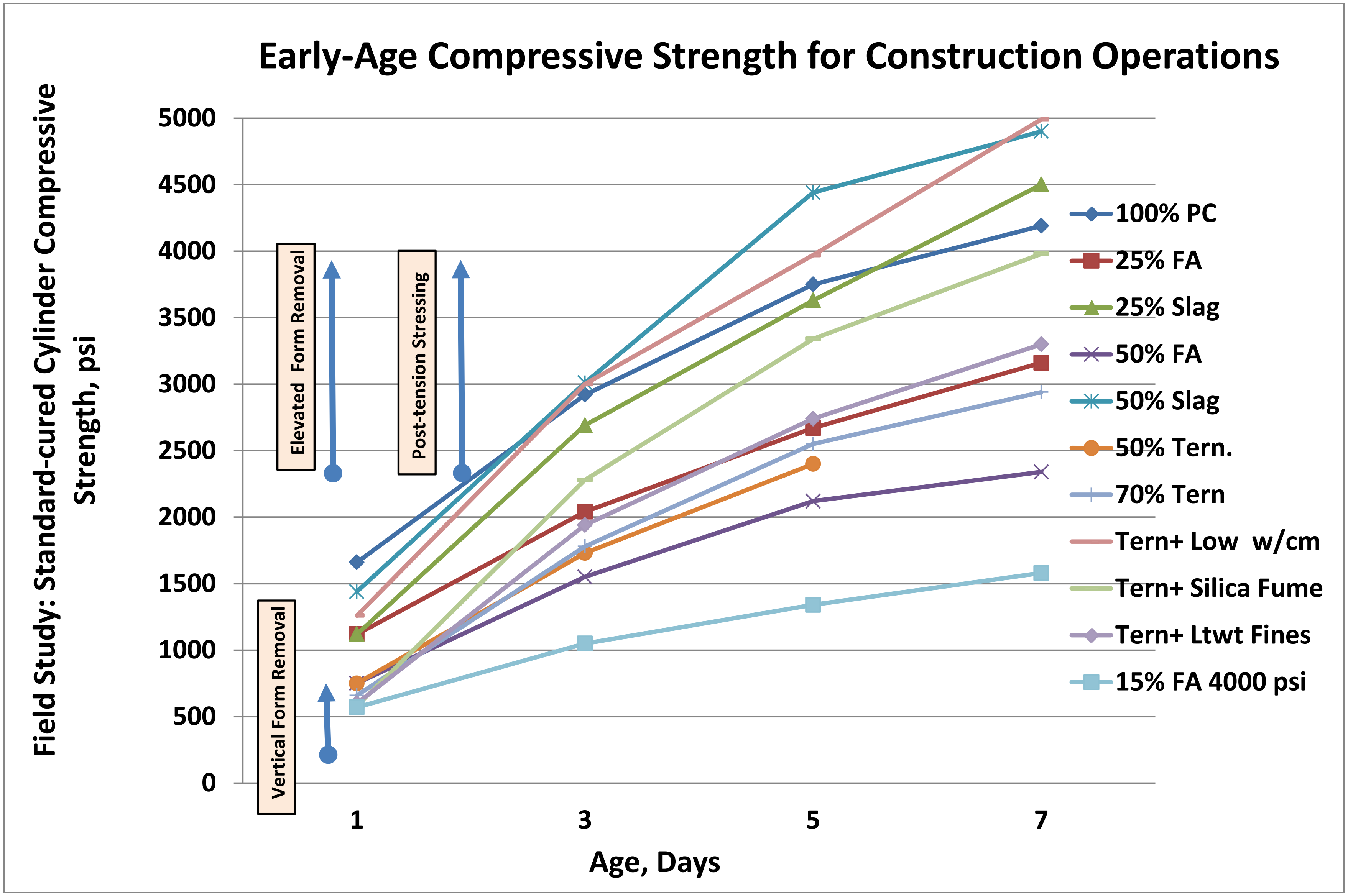

Figure 4.1 Early-age compressive strength of standard –cure, field cast cylinders from 11 Pankow research mixtures.

Chapter 5 Fresh Concrete Stiffening and Construction Operations

Specifications seldom address setting time, although this property of fresh concrete can affect construction operations in several ways when green concrete mixtures are used.

- Forming: Form pressures in walls and columns increase with increases in setting time. Increased form pressures increase forming costs because either the pour rate must decrease resulting in a longer wall placement or the wall must be placed in two separate formed pours.
- Consolidation: Fast setting can reduce the ability to thoroughly consolidate concrete, but slow setting is a more likely problem because it can result in settlement cracking in deep sections.
- Finishing: Again, slow setting is most likely to cause construction problems. Increased finishing costs are the result of delays in finishing caused by slow setting. Surface defects such as blisters, delamination, and plastic shrinkage cracking are often the result of a slow setting concrete.
- Curing: Initiation of initial and final curing can be delayed by slow setting. This too can result in plastic shrinkage cracking or in premature drying of the surface.

As indicated by the above bulleted list, cost for a project can increase considerably if the concrete sets too slowly. The effect of setting time on fresh concrete construction operations is first reviewed. Then the Pankow data on setting time is presented.

5.1 Fresh Concrete Construction Operations

The list below presents traditional fresh concrete construction operations, those typically completed while the concrete is still plastic.

Placing concrete in vertical forms (form pressures) -- Fresh concrete placed in wall or column forms acts like a fluid to exert a lateral pressure against the formwork. The formwork must be designed to withstand this lateral pressure. The maximum lateral pressure is equal to the height, h, of the fluid concrete times the unit weight, w, of the concrete – for normalweight concrete usually considered to be 150 pcf. ACI 347-04 “Guide to Formwork for Concrete” recommends that, unless other conditions are met, the formwork should be designed for a lateral pressure, $p = wh$. This equation considers lateral pressure to be a hydrostatic (fluid) pressure of concrete.

ACI 347-04 states that “The set characteristics of a mixture should be understood, and using the rate of placement, the level of fluid concrete can be determined.” The concrete set time determines the rate at which the full hydrostatic lateral pressure reduces due to concrete stiffening. As the concrete stiffens, the lateral pressure against the formwork decreases.

Concrete is usually placed in lifts of 4 ft. thickness or less. The hydrostatic lateral pressure against the formwork for the first lift is the height of that lift times the unit weight of the concrete. If the next lift is placed quickly, the maximum lateral pressure on the formwork is the total height of both lifts times the unit weight of the concrete. If the second lift placement is delayed, the concrete in the first lift stiffens and no longer exerts a full hydrostatic lateral pressure on the formwork. Typical industry practice is to place the lifts such that the formwork pressure is reduced to a value below maximum hydrostatic pressure. Thus, the set time or stiffening of the fresh concrete is an important factor when determining the formwork pressure used design.

ACI 347-04 indicates that "When working with mixtures using newly introduced admixtures that increase set time or increase slump characteristics, such as self-consolidating concrete, ….the maximum hydrostatic pressure… should be used until the effect on formwork pressure is understood by measurement." ACI 347-04 provides information on form pressures for concrete mixtures containing less than 70% slag or 40% fly ash. Without a formwork pressure measurement, ACI 347-04 recommends that for concrete mixtures that exceed these limits (more than 70% slag or 40% fly ash) the formwork should be designed for full hydrostatic pressures.

Consolidating concrete – Most concrete is consolidated by using immersion vibrators or external form vibrators while the concrete is still in a plastic state. If the concrete sets too fast, as it might when silica fume is used, it can't be adequately vibrated and a cold joint can form between successive placements. British Standard BS 5075 (Dodson 1994) uses a setting time measurement of 72 psi as the upper limit for placing and compaction of concrete. In other words, the faster this setting time value occurs the shorter time the contractor has to place and vibrate the concrete.

Slow setting is a more likely concern, because green concretes normally set more slowly than traditional concretes. This slower setting is a possible problem in deep sections of mat foundations, beams, or walls because the initial vibration doesn't fully consolidate the concrete. Further consolidation that occurs as a result of subsidence—settlement of solids—is likely in slow-setting mixtures. When restrained by reinforcing steel or other embedded items, settlement cracking may occur. Revibration is one method for reducing settlement cracking, but for large area placements such as base mats, it is seldom practical to revibrate the concrete. In addition, revibration increases form pressures by more than 40% above the ACI 347 formulas (Douglas et. al.) For such placements, if slow setting produces settlement cracks, a change in the concrete mixture may be appropriate.

Finishing concrete – After concrete floors or slabs are placed, finishing operations usually include screeding (sometimes accompanied by vibration), bullfloating, power floating and power troweling. The concrete must be plastic enough for screeding and bullfloating but must then stiffen prior to power floating and troweling. Setting time measurements can be used to estimate the amount of time available for finishing operations, and the measurements can also be used as a basis for needed mixture adjustments when concrete will be finished in hot or cold weather. Accelerating admixtures may be useful, especially in cold weather, to reduce setting time and the possibility of top-down setting, which can result in surface crusting and blistering or delamination.

Finishers observe loss of a bleedwater sheen and the depth of their footprint as rough measures of the time at which power floating should begin. Suprenant and Malisch (1988) used 6-in. thick by 4-ft-square slabs to check the depth of a finisher's footprint while also measuring setting time on a mortar sample tested in accordance with ASTM C 403. They found that a finisher's ¼-in.-deep footprint indentation corresponded to a penetration resistance of between 15 to 25 psi. Abel and Hover (2000) found that the decision to start brooming the concrete surface was when the penetration resistance was about 10 psi. Bury et al. (1994) found that the ¼-inch-deep footprint indentation occurred with penetration resistance less than 50 psi.

Curing during finishing – Fresh concrete loses moisture due to evaporation of bleed water. The longer the concrete remains plastic the more moisture it loses. When the evaporation rate exceeds the bleeding rate, plastic shrinkage cracking can occur. This occurs most often during hot weather concreting, but sometimes during cold weather as well. Slow setting exacerbates this problem because of increased bleeding duration and a longer time needed for the concrete to stiffen enough so finishers can start floating. Concretes containing supplementary cementitious materials (SCM's) usually set more slowly and may bleed less than straight portland cement mixtures. Thus, they are more susceptible to plastic shrinkage cracking. Spraying evaporation reducers on the fresh concrete surface assists in reducing moisture loss by evaporation and thus reduces the probability of plastic shrinkage cracking.

Final curing – Concrete floors or slabs are usually cured by spraying curing compounds on the surface or by using sheet materials such as plastic or specialty sheeting to reduces moisture loss or by. These curing activities take place when the concrete is hardened sufficiently so that no footprints are visible when workers walk on the surface. The Federal Highway Administration (Poole 2005) indicates that final curing should commence at initial set.

5.2 Understanding Time of Setting Measurements

Time of setting is determined with penetration resistance measurements on mortar sieved from the concrete mixture, as set forth in ASTM C 403-08 “Standard Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance.” The test method includes a brief summary as follows:

> “A mortar sample is obtained by sieving a representative sample of fresh concrete. The mortar is placed in a container and stored at a specified ambient temperature. At regular time intervals, the resistance of the mortar to penetration by standard needles is measured. From a plot of penetration resistance versus elapsed time, the times of initial and final setting are determined.”

The information determined in this test method is plotted as shown in Figure 5.1 (from ASTM C 403-08). By definition, initial set and final set correspond to penetration resistances equal to 500 psi and 4000 psi, respectively. Tuthill and Cordon (1955) showed that the concrete is usually stiffer than the mortar being tested and that the 4000 psi final set determined by penetration resistance is approximately equal to a compressive strength of 100 psi.

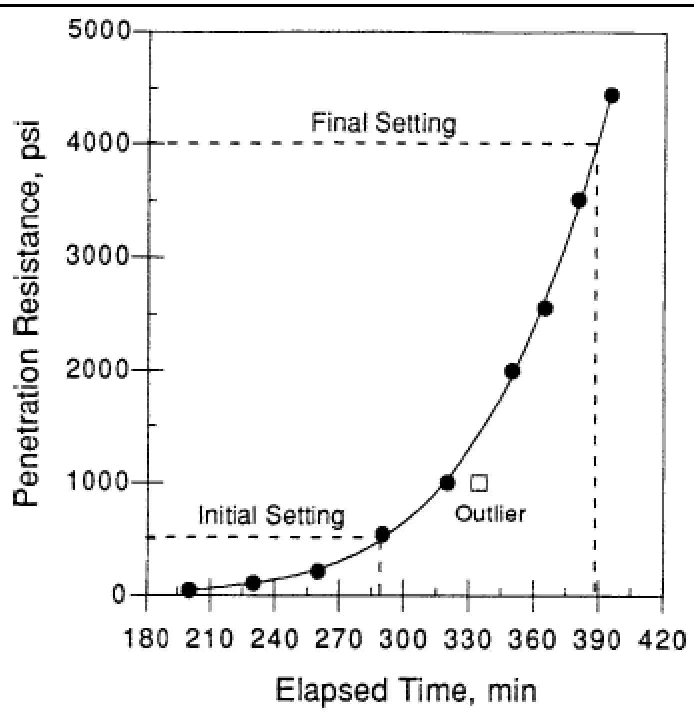

FIG. X1.1 Plot of Penetration Resistance Values Versus Elapsed Time and Hand Fit Curve Used to Determine Time of Setting (Note: Not drawn to actual scale)

Figure 5.1 Penetration resistance versus time showing initial and final set (from ASTM C 403).

Changes in the concrete mixture and the surrounding environment change the setting time. For instance, adding an accelerator to the concrete mixture speeds setting, while adding a retarder would slows setting. Water-reducing admixtures can be formulated from set neutral to acting as a retarder. Concretes containing SCM's are generally slow setting, but some admixtures can make the mixture set neutral. It's best to obtain setting time information for such mixtures from ASTM C 403 tests on samples with and without the admixture. Likewise, cold weather will slow setting time and hot weather will speed setting time. Most testing laboratories test concrete mixtures at about 70^{o}F so the effect at different temperatures needs to be evaluated. The next section gives a procedure for evaluating setting times at different temperatures.

5.3 Evaluating Time of Setting at Different Temperatures

FHWA (2005) notes that setting time measured in accordance with ASTM C 403 is conveniently done during mixture verification work prior to starting construction. The setting time is strongly affected by the concrete temperature, and therefore the field time of setting will differ from the laboratory-determined time if the two temperatures differ. This is important in field applications, since in-place concrete temperatures can differ significantly from laboratory concrete temperatures, and the effect can be substantial. Laboratory values can be adjusted for actual concrete temperature using the following equation.

$$TOS = TOS_{StdTemp} \bullet e^{R\left(\frac{1}{CT}-\frac{1}{StdTemp}\right)}$$

where:

- *TOS* = time of setting at temperature of in-place concrete, same units as in standard test
- $TOS_{StdTemp}$ = time of setting under standard conditions, any units
- *CT* = temperature of in-place concrete, *K*
- *StdTemp* = temperature of concrete during laboratory test, *K*
- *R* = constant

The constant, *R*, can be determined empirically, but a value of 5,000 Kelvins (K) works well. This equation can be programmed into a spreadsheet to simplify the calculation for use in exploratory work.

Temperature, F	Set Time Factor by Equation	Set Time Factor from Pankow Data	
40	2.78	Initial Set	Final Set
50	1.95	-----	
60	1.39	1.26	1.24
70	1.00	1.00	1.00
80	0.73	-----	----
90	0.54	0.63	0.65
100	0.40	-----	----

5.4 How Setting Time Influences Plastic Shrinkage Cracking

Poole's work for the FHWA describes how the combination of cumulative bleeding and cumulative evaporation affects the onset of plastic shrinkage cracking (PSC). "PSC occurs when concrete is still plastic (i.e. before time of initial setting), and when excessive loss of mixing water causes shrinkage sufficient to crack the plastic concrete. PSC may take the form of relatively large, parallel, well-spaced cracks that begin shallow but may penetrate deeply into the concrete. In other cases, PSC may take the form of a fine pattern of map cracks that penetrate only ½ to 1 inch into the concrete."

Figure 5.2 shows that bleed water rising to the surface offsets the evaporation loss. When cumulative bleeding exceeds cumulative evaporation, no plastic shrinkage cracks occur. When cumulative bleeding decreases and cumulative evaporation rate remains the same, a critical point can be reached at which plastic shrinkage cracks occur. When the concrete sets there is no more bleeding. The concrete can set before the fresh concrete reaches the critical bleed-evaporation point. If however, the concrete set time is delayed, the evaporation continues but the fresh concrete does not have enough bleed water to keep up with the evaporation rate. Thus the delayed set increases the potential for plastic shrinkage cracking.

To offset some of the delayed setting time resulting from reduced portland cement contents, green concrete mixtures are often proportioned at water-cementitious ratios below 0.40. This reduction in water-cementitious ratio reduces the amount of water in the concrete. Thus the longer the concrete remains plastic the more likely the bleed water needed to offset evaporation will be depleted. The combination of a lower water content and increased setting time makes green concrete mixtures more susceptible to plastic shrinkage cracking.

Fortunately, plastic shrinkage cracking is relatively harmless in reinforced concrete buildings.

5.5 Setting Time and Form Pressures

While the effect of setting time on form pressures is discussed in ACI 347-04, setting time is not used directly in any of the form pressure equations. Instead, ACI 347-04 modifies the form pressure equation based on concrete temperature, rate of placement, and concrete chemistry (mixture ingredients and quantities). The effect of these variables in combination could be more directly assessed by measuring setting time. In our literature search, however, we could find no research results in which an investigator included setting time as a variable in any proposed formwork pressure equation.

During construction, workers can estimate the degree of stiffening for each lift by inserting a #4 reinforcing bar into the top of the lift. If the bar penetrates more than 6 to 12 in. into the lift, the concrete has not stiffened enough to place the next lift. This field check more accurately estimates setting because form pressure calculations are based on anticipated, not actual, concrete temperature and rate of placement. Using the #4 bar test helps field personnel to evaluate form pressures under conditions that can differ from those assumed in formwork design.

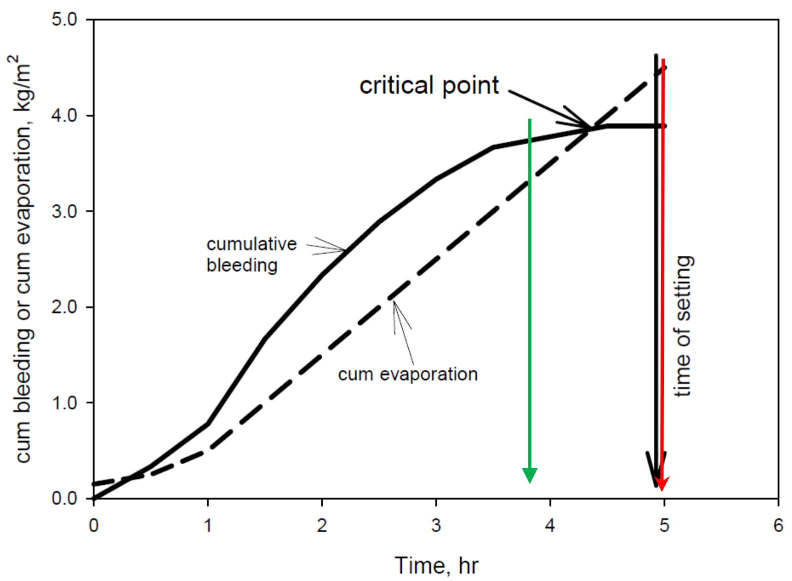

Figure 8. Graph. Plot of cumulative bleed and cumulative evaporation v. time.

Figure 5.2 Delaying the set time, moving from green to red line, can cause the concrete to reach a critical point where the cumulative evaporation exceeds the cumulative bleeding and plastic shrinkage cracks occur. (Modified from Poole 2005)

5.6 Evaluation of Pankow Setting Time Data

Table 5.1 shows the Pankow setting time data, developed in accordance with ASTM C 403 for concrete temperatures of 60^{o}F, 72^{o}F and 90^{o}F. Figure 5.3 illustrates the effect of different concrete mixtures on the initial setting time. Figure 5.4 illustrates the effect of different mixtures on setting time: 100% portland cement, 50% fly ash and 70% ternary with low water-cementitious ratio. Note that the use of SCM's delayed setting.

As previously discussed, fresh concrete construction operations are influenced by concrete setting times below the 500 psi initial-set value. An example of the importance of cold and hot weather on initial setting times for the Pankow data is shown below. Note that at 60^{o}F, the average setting times for all concrete mixtures were about 90 minutes greater than those at 72^{o}F. And that at 90^{o}F, the average setting times for all concrete mixtures were about 120 minutes less than those at 60^{o}F. Note that the range of initial setting times at a cold temperature (60^{o}F) varied by more than 240 minutes; while at a hot temperature (90^{o}F) the variation was much smaller at about 40 minutes.

Initial Setting Times (500 psi) from Pankow Data

- At 60^{o}F, setting time varies from 233 to 480 minutes; average is 360 minutes with a range of 247 minutes.
- At 72^{o}F, setting time varies from 190 to 363 minutes; average is 286 minutes with a range of 173 minutes.
- At 90^{o}F, setting time varies from 156 to 192 minutes; average of 175 minutes with a range of 41 minutes.

As expected the initial setting time for the 100% cement concrete mixture was the lowest at all three temperatures. The highest initial setting time was for the 70% ternary concrete mixture using a carbohydrate-based water-reducing admixture. This result was expected and was chosen to illustrate the importance of choosing a water-reducing admixture that does not retard set when the cementitious material is largely composed of SCMs. Because the large amount of SCM's typically used in green concrete mixtures delays setting, including admixtures that also delay setting can result in a concrete that is unsuitable for construction operations. As noted in this study, initial setting time for the 70% ternary concrete mixture using the carbohydrate-based water-reducing admixture exceeded 780 minutes (more than 13 hours). To show that this extraordinary setting time is primarily due to the high portland cement replacement percentage (70%), initial setting time was measured for another mixture containing 100% cement and the carbohydrate-based water-reducing admixture. This mixture had an initial setting time about 20 minutes slower than the 100% cement mixture containing a polycarboxylate admixture.

Table 5.1 Lab Study: Summary of ASTM C 403 Set Times at 60F, 72F and 90F

	100% PC	**25% FA**	**25% Slag**	**50% FA**	**50% Slag**	**50% Tern**	**70% Tern***	**70% LW**	**75% Quad**	**70% In. C**	**5 sk 15% FA**
	1	2	3	4	5	6	7	8	9	10	11
Initial Set Time, min											
60 F	233	333	250	397	380	357	>> 780	480	450	395	327
72 F	190	233	230	290	300	297	>> 780	333	363	357	270
90 F	156	170	163	170	171	175	197	180	185	188	192
Final Set Time, min											
60 F	357	450	400	535	510	510	>> 780	626	663	600	462
72 F	267	363	317	428	423	412	>> 780	478	527	537	388
90 F	228	240	235	262	260	253	300	289	286	300	281
*	Polycarboxylate water-reducing admixtures used except for the 70% Tern which used a carbohydrate admixture										

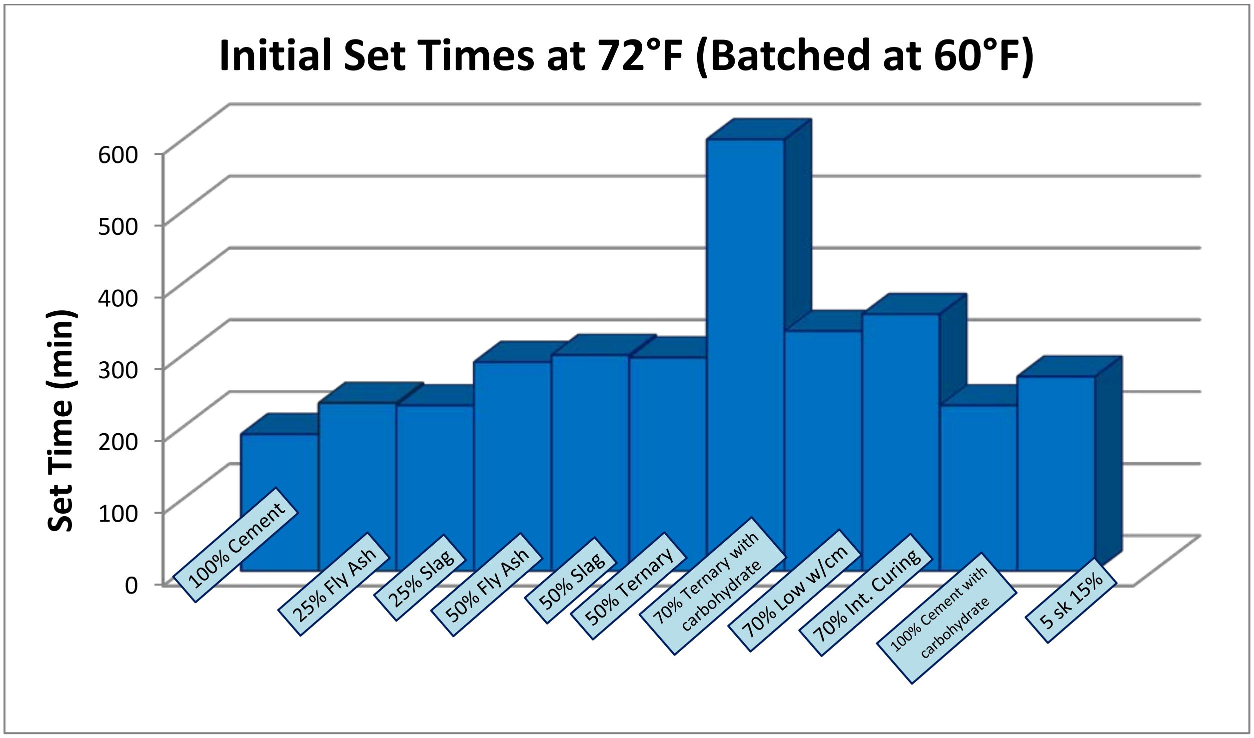

Figure 5.3 Initial setting times for concrete mixtures as measured by ASTM C 403. Initial setting times vary from a low of 190 minutes for the 100% cement mixture to greater than 780 minutes for the 70% ternary mixture with a carbohydrate-based admixture. Note that the carbohydrate-based admixture did not substantially increase initial setting time when used with the 100% cement mixture. All other concrete mixtures contained a polycarboxylate water-reducing admixture.

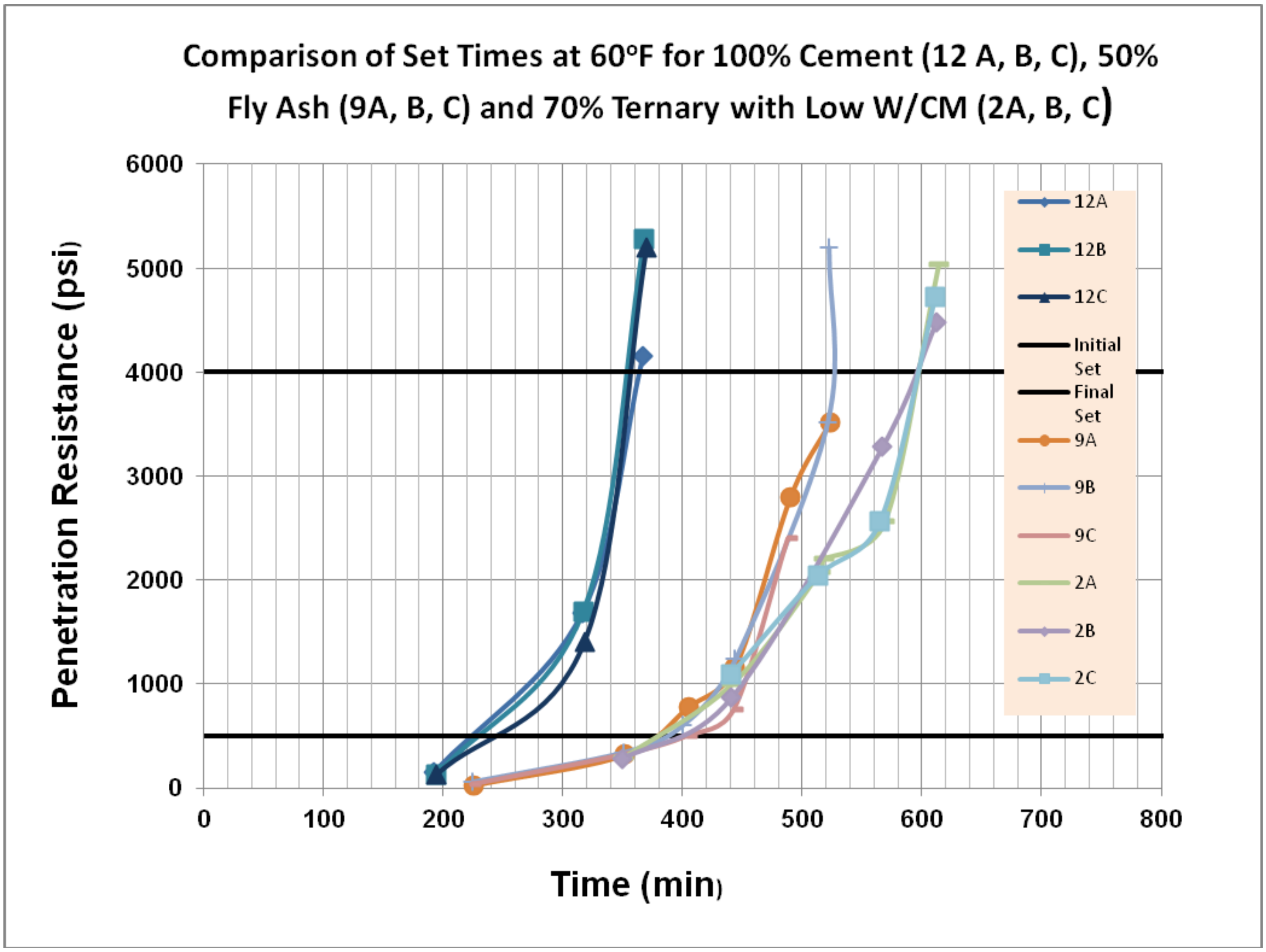

Figure 5.4 Penetration resistance versus time for three different mixtures at 60°F: 100% portland cement, 50% fly ash and 70% ternary with low water-cementitious ratio. Note that the use of SCM's significantly delayed setting.

Setting Times at 60°F and 72°F from Pankow Data

- At 72°F and 500 psi, setting time varies from 190 to 363 minutes; average is 286 minutes with a range of 173 minutes.
- At 72°F and 100 psi, setting time varies from 155 to 277 minutes; average is 219 minutes with a range of 122 minutes.
- At 60°F and 500 psi, setting time varies from 233 to 480 minutes; average is 360 minutes with a range of 247 minutes.
- At 60°F and 100 psi, setting time varies from 189 to 321 minutes; average is 256 minutes with a range of 132 minutes.

As previously discussed most fresh concrete construction operations occur when the concrete penetration resistance is less than 100 psi as measured in accordance with ASTM C 403. The information above compares the setting time at 100 and 500 psi penetration resistance at a concrete temperature of 72^{o}F and 60^{o}F. Obviously the time to reach 100 psi penetration resistance is shorter than the time to reach 500 psi penetration resistance. At 72^{o}F, the difference in times is about 60 minutes and at 60^{o}F is about 90 minutes.

The important difference, however, is that caused by differences in mixture proportions. For a 100% cement mixture, the time to reach a penetration resistance of 100 psi at 60^{o}F was 189 minutes. For the 50% fly ash and 50% slag mixtures, the time increased by up to 60 minutes. For other mixtures, for instance 70% low w/cm, the time is almost 120 minutes longer. During cold weather concreting, 60^{o}F may not be the appropriate concrete temperature for determining setting time. ACI 306R-10 recommends that for concrete slab thickness of 12 inches or less, the concrete temperature as placed and maintained should be not less than 55^{o}F. Using the previously cited equation for adjusting setting times based on temperature, it can be shown that at 55^{o}F, the time to reach 100 psi penetration resistance with 100% cement is 320 minutes. For the 50% fly ash and 50% slag mixtures this time increases by slightly more than 60 minutes and the 70% low w/cm mixtures it increases by slightly more than 120 minutes.

Pankow Set Time Data		
Concrete Mixtures	Time to Reach 100 psi	
	At 60^{o}F	At 72^{o}F
100% cement	189	181
25% fly ash	217	197
25% slag	213	155
50% fly ash	254	218
50% slag	282	225
50% ternary	269	229
70% low w/cm	296	258
75% quad	305	277
75% ternary with WR	321	255
100% cement with carbohydrate	210	196
5 sack 15% fly ash	210	200

5.7 References

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Chapter 6 Recommendations for Using “Green” Concrete Mixtures

6.1 Background

Buildings are designed and constructed with knowledge, understanding, and expectations based on what we have called traditional concrete mixtures in this report. Concrete for building construction commonly contained only portland cement or portland fly-ash or blast-furnace slag cement blends as the binder until well into the 1960s. Expectations of designers and contractors were based on their having years of experience with this concrete and researchers having identified factors affecting properties such as setting behavior and strength gain. The 1963 ACI Building Code first allowed the use of fly ash as an admixture in 1963 and slag cement in 1989. The replacement percentage for building construction usually ranged from15% to 35%, and with portland cement contents above 400 lbs/cy. The concrete industry dealt with this change through a combination of research and experience that again set expectations for the effect of this level of SCMs on strength gain, internal heat generation, setting time, and other concrete properties. Based on these information sources, concrete buildings were successfully designed and constructed.

A new direction has emerged as evidenced by the increased concern with sustainability and, specifically, concrete’s carbon footprint. While the carbon footprint is reduced by using the more traditional replacement levels of portland cement with SCMs, there is now an even greater emphasis on making concrete green by increasing these replacement levels. The new trend is for green concrete mixtures with SCMs replacing up to 70% of the cement and holding cement contents as low as 200 lbs/cy. As was the case when SCMs were first being used in building construction, the new green concrete mixtures must also be evaluated to determine their advantages and uses.

6.2 Recommendations

<u>Chapter 1</u>

1.1 Do not expect green concrete to cost less. Even though the total cost of cementitious materials *may* be lower because the unit cost of some SCMs is less than that of portland cement, there may be cost increases due to:

- Adding the production capacity (extra bins or silos) at the ready-mixed concrete plant
- Added quality control for additional cementitious ingredients
- Using more admixtures to balance longer setting times
- Other cost factors discussed in Chapters 2 through 5.

The only way to determine the cost of the green concrete is working with a contractor and concrete producer to assist in developing the concrete mixture and the associated costs. In other words, do not budget to save money because SCMs are being used unless that can be confirmed by the construction team.

1.2 Start with the industry information available from the American Concrete institute (ACI), American Society for Testing and Materials (ASTM), the National Ready Mixed Concrete Association (NRMCA), the American Coal Ash Association (ACAA) and others. Note however, that documents from these industry organizations may have a lag time of several years in publishing the information. The organizations do, however, publish magazines in which more recent trends are often reported.

1.3 Take advantage of the green concrete mixture proportions and test data provided in the Phase I report for this research project. Many concrete producers consider details of their green concrete mixtures to be proprietary information, so the mixture ingredients and quantities are not always available. The information in the Phase I report can be used as an aid in evaluating the ingredients and proportions for your project specific green concrete mixture along with expected fresh- and hardened-concrete properties. Note that the Phase I concrete mixtures were developed based on California experience, where the need for high-strength concrete in seismic regions was balanced with the desire for green concrete. Also note that the Phase I mixtures include cement, fly ash, slag, and silica fume from only one source. Results may differ with differing materials.

1.4 Be cautious with respect to concrete surfaces that will receive additional final finishes. There is some anecdotal evidence that concrete containing fly ash may not be suitable for surfaces to be painted, coated, or covered by flooring installed with adhesives. While there are some references to loss of adhesion in the literature, no proposed mechanisms such as slowed drying or an oily surface residue have been verified to explain the reasons for this phenomenon. It is also unclear if, or how, other SCMs may affect adhesion to a concrete substrate. Thus it is unclear as to how green concrete mixtures will perform with respect to drying and providing appropriate bonding for final finishes.

Chapter 2

2.1 Consider specifying compressive strength, f_c', at a designated test age greater than 28 days. Some green project specifications require compressive strengths to be achieved at 56 or 90 days so the slower rate of strength gain for green concrete mixtures can be accommodated.

2.2 Allow enough lead time for strength testing. The ACI Building Code requires trial laboratory or field mixtures when an acceptable record of field tests is not available to document acceptable strengths. This is often the case for green mixtures made with more than one type of cementitious material. When specifying compressive strengths at a designated test age of 56 or 90 days, the multiple trial batches needed substantially increase necessary lead times. Some specifiers for projects in California collaborate with the concrete producer a year in advance of starting construction.

2.3 Obtain more compressive strength information on green concrete mixtures than is usual for traditional concrete. Because of the construction operations and owner's schedule, contractors need data indicating early-age strength development. If the designated test age is 56 or 90 days, compressive- strength test results at 1, 3, 7, 14 and 28 days help the contractor estimate the time required for critical construction operations needed to stay on schedule. Consider obtaining compressive strength data for concrete that must be used for hot and cold weather concreting because high or low temperatures affect the concrete strength for each age. This additional information is useful to contractors during bidding so they can develop a time schedule and costs that meet the owner's requirements. Early-age strength requirements also provide a benchmark for early-age field tests that can be used as an early warning signal that 28-, 56- or 90-day specified strengths may not be reached.

2.4 Specify strength requirements and not time requirements or percentage of strength for construction operations. Time requirements assume curing conditions that may or may not be suitable for the concrete being placed. Also avoid specifying strengths required for construction operations as a percent of f_c'. For instance, the standard percentage for elevated formwork removal is 75% f_c'. *B*ut if f_c' is specified at 90 days, 75% of f_c' might not be reached until five to seven weeks after placement which must be considered in the schedule. For traditional concrete, 75% f_c', when f_c' is at 28 days, usually occurs in 3 to 7 days.

2.5 Have a plan for dealing with failure to achieve specified compressive strengths. If the designated age for acceptance testing is 56 or 90 days, there can be 4 or 5 floors on a mat foundation or on a lower floor level when strength test results are obtained. How will low strength-test results affect continued construction? Must construction stop? Must all floors be reshored? Early-age strength testing is recommended so action can be taken as soon as a potential problem is identified.

2.6 Use both standard-cure and sealed-cure cylinders in determining compressive strength during the laboratory trial mix-design phase. The Pankow data along with that of other investigators shows a 15% strength reduction when test cylinders are not cured by water immersion or in a moist room. Because the field concrete will not be cured in this manner, the engineer must determine if the design is to be based on water-cure or sealed-cure cylinder strengths.

2.7 Determine the effect of the standard-cure versus sealed-cure on other important design and construction considerations. These considerations include the ACI requirement for field curing in which field strength must be 85% of the companion laboratory-cured cylinders and core-strength acceptance requirement in which the average of three cores must equal at least 0.85 f_c'. If a 15% strength reduction is likely for sealed-cure laboratory cylinders, the effect in the field is likely to be larger.

Chapter 3

3.1 Determine the decision process needed if cylinders and cores for the same concrete are tested and cylinders test results meet ACI acceptance requirements but core test results do not meet ACI acceptance requirements. This has happened on some green concrete building projects and creates much confusion for the owners, designers and contractors.

3.2 Reevaluate Bloem's data, which was the ACI 318 basis for developing core-strength acceptance requirements. Bloem's concrete mixtures, specimen size, curing, and strength levels are unlikely to match those for green concrete projects. Consider developing a core-to-cylinder relationship during the laboratory trial-mixture phase for a project. Knowing this relationship in advance of cores being taken can save valuable time when a low-strength investigation is needed.

3.3 Evaluate the effect of internal heat generation on the strength of cores. ACI 318 is silent on this issue but ACI 214.4R-10 states that "… a strength loss of approximately 3% of the average strength in the specimen for every 10°F increase of average maximum temperature sustained during early hydration." The strength of cores removed from mat foundations and large columns or beams may be reduced due to this effect, which is not accounted for in the ACI 318 core-strength acceptance criteria. Thus when making specimens that will be cored for developing the core-to-cylinder relationship during the laboratory concrete mix design process, use specimen sizes that will reflect the effects of heat generation. If this isn't done, use the ACI 214.4R-10 strength-loss approximation of 3% to correct core strengths for the heat generated during curing in the structure.

3.4 Evaluate the effect of water- and sealed-curing on the strength of cores. ACI 318 is silent on this issue but the Pankow data and the data from other investigators indicates a possible15% strength reduction when green concrete is not cured with water. When making specimens that will be cored for developing the core-to-cylinder relationship during the laboratory concrete mix-design process, use the curing method that is required in the field.

3.5 Note the ACI 363.2R-11 recommendation that a "A correlation curve should be established for each high-strength mixture to relate the strength of extracted cores (normally 4 in. in diameter) to the strength of specimens used for acceptance testing, that is, 6 by 12 in. or 4 by 8 in. cylinders." However, this document also states that: "These data indicate that the acceptance criteria for core strengths specified in ACI 318 are also applicable to high-strength concretes". The cited ACI 363.2R-11 data includes core-to-cylinder strength ratios for cores taken and tested up to 7 years after the construction and cylinders tested at 28 days.

Chapter 4

4.1 Balance the compressive strengths needed at 1 to 7 days for timely construction operations with the desire for green concretes that dramatically reduce the carbon footprint attributable to portland cement replacement. This decision affects the owner's cost and schedule.

4.2 Provide the contractor with as much green concrete mixture data as possible to increase the likelihood of a bid that represents the construction operation sequence necessary for that concrete.

4.3 Stay flexible. Utilizing the green concrete mixtures with up to 70% replacement of portland cement by SCMs is likely to present some challenges. Engineers have sometimes adjusted strength and concrete mix-design requirements during building construction.

Chapter 5

5.1 Understand that formwork costs might increase slightly because some green concrete mixtures require that formwork be designed for full-hydrostatic pressure.

5.2 Measure setting times during the laboratory trial-mixture design process. Measured setting times (ASTM C 403) at temperatures representing hot and cold weather concreting would also be useful. Contractors can use this information to plan and schedule fresh concrete construction operations.

5.3 Be wary of the increased risk of plastic shrinkage cracking and settlement cracking that can occur as a result of green concrete mixtures setting more slowly than traditional concretes. Contractors need to consider construction practices to minimize these issues. Changes in the green concrete mixture design may also be necessary if these types of cracking are noted.

Preface

The American Society of Concrete Contractors submitted a proposal to the Charles Pankow Foundation for preparation of a *Users' Guide to "Green" Concrete in Building Construction.* In this context, "green" refers to concretes made with supplementary cementitious materials (SCMs) replacing varying amounts of portland cements to reduce the carbon footprint. As part of the preparation, several green concrete mixtures were tested and the data presented to provide information about mixture composition that is usually proprietary and not available to the industry or public. The relationship between cylinder strength and strength of cores from the same batch of concrete was of particular interest. The data was intended to supplement the limited amount of published data related to field experience with green concrete.

Bruce Suprenant and Ward Malisch, the authors of this report, became interested in this topic as a result of Suprenant's troubleshooting work on projects that utilized green concrete with acceptance testing done at 56 or 90 days rather than the standard 28 days. This later testing compensates for slower rates of strength gain when large amounts of portland cement are replaced by SCMs, but with a downside: more concrete has been placed when the test results become available at test ages greater than 28 days. If a strength test result is lower than allowed, and subsequent core testing indicates that the in-place strength is also lower than allowed, repair or removal and replacement generally costs more because of the larger volume of concrete in place. Schedule delays resulting from needed decisions on acceptance may also be more critical at this point.

Although the ACI 318 criteria for satisfactory core test results are based on the ratio of core strength to the design strength, f_c***'***, we chose not to use that ratio in our research. Instead of estimating f_c***'*** based on the average strength and standard deviation for our data, we used the ratio of core strength to standard-cured cylinder strength with both cores and cylinders tested at 28, 56, 90, and 180 days.

As our test results became available, we realized that the relationships between strengths of field-cast, wet-cured cylinders and cores from large blocks cast in the field were particularly puzzling. At ages of 28 to 180 days, the core/cylinder ratios ranged from about 0.40 to 0.90, with an overall average of about 0.65 for all but one of the 11 mixtures studied. Core/cylinder relationships for a control mixture containing no SCMs followed the same trend as those for fly ash, slag, ternary, and quaternary mixtures. This led to a literature search related specifically to the core/cylinder strength ratios for normal or high-strength concretes made with straight cements and varying SCM contents. That search resulted in questions concerning the ACI 318 code requirement that the average core strength of three cores must equal 0.85 times the design strength of the concrete, with no core in the set of three lower than 0.75 times the design strength.

As a result, we changed the title of our report to "Assessing the Impact of "Green" Concrete Mixtures on Building Construction." The report still covers construction rather than performance. But we acknowledge that our test results are for a combination of one cement, SCM, and admixture source and that, in field tests, the control mixture containing no SCMs performed similarly to mixtures made with varying percentages of SCMs. The scope for our field experiments did not include a factorial approach to evaluating the interactions between the cement and admixtures, nor do we suggest that the results of the field experiments can be generalized to include all green concretes. We do believe that, for confirmation, the results require further research of green concretes using differing cements and admixtures and in differing geographic regions. We also agree with the following recommendation in ACI 363.2R-11,"Guide to Quality Control and Assurance of High-Strength Concrete:"

"… a correlation curve should be established for each high-strength mixture to relate the strength of extracted cores (normally 4 in. [102 mm]) in diameter) to the strength of specimens used for acceptance testing, that is, 6 x 12 in. (152 x 305 mm) or 4 x 8 in. (102 x203 mm) cylinders. Then, if coring becomes necessary, the relationship has been established, agreed upon, and is ready for conclusive interpretation."

The correlation between core strength and the strength of specimens used for acceptance testing should be discussed at a preconstruction conference so the engineer of record, concrete producer, and concrete contractor are in agreement on steps to be taken when core tests are needed.

As the title of our original proposal implied, we were primarily interested in topics related to construction as opposed to performance of the green concrete after construction. Thus our testing program did not directly address durability because assessment of durability requires long-term testing or development of models for predicting durability, neither of which was within the scope of our proposal.

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