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16. Abstract Self-consolidating concrete (SCC) is an advanced type of concrete that can flow under its own mass without vibration, pass through intricate geometrical configurations, and resist segregation. The use of SCC in precast structural applications can result in increased construction productivity, improved jobsite safety, and improved concrete quality. Certain changes in mixture proportions, which are necessary to achieve SCC workability, may affect hardened properties. A joint research project (TxDOT Project 0-5134: <i>Self-Consolidating Concrete for Precast Structural Applications</i>) was conducted at the Center for Transportation Research (CTR) and the Texas Transportation Institute (TTI) to evaluate the suitability of SCC for prestressed concrete bridge beams. The CTR researchers related SCC workability to materials and mixtures proportions, developed a series of SCC mixtures expected to be representative of SCC produced in Texas for prestressed concrete bridge beams, evaluated the early-age engineering properties (up to 24 hours) and shrinkage of these mixtures, and developed recommendations for specifying and inspecting SCC. The TTI researchers evaluated the longer-term engineering properties of these mixtures and tested full-scale beams. This report describes the research conducted by the CTR researchers.					
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Self-Consolidating Concrete for Precast Structural Applications: Mixture Proportions, Workability, and Early-Age Hardened Properties

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Dr. Ken Stokoe and Min Jae Jung performed the dynamic modulus of elasticity measurements. Jon Poole and Kyle Riding advised on the calorimetry measurements and *Concrete Works* temperature simulations.

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1. Introduction

Self-consolidating concrete (SCC) is an advanced type of concrete that can flow and consolidate under its own mass without vibration, pass through intricate geometrical configurations, and resist segregation. It has been used successfully for a wide range of precast and ready mixed concrete applications. The use of SCC can result in increased construction productivity, improved jobsite safety, and enhanced concrete quality. These benefits, however, must be measured against the potentially higher material costs and the need for greater technical expertise and quality control measures. In addition, certain changes to the mixture proportions required to achieve SCC workability may affect hardened properties adversely. The extent of these changes in mixture proportions depends on the local materials and application requirements. Therefore, research with local materials is needed to ensure that SCC can be produced to meet application requirements and to compare the early-age and long-term engineering properties of SCC and conventionally placed concrete mixtures.

This report presents partial results of a research project conducted to evaluate the suitability of SCC for prestressed concrete bridge beams in Texas (TxDOT Project 0-5134: *Self-Consolidating Concrete for Precast Structural Applications*). The research project was conducted jointly at the Center for Transportation Research (CTR) at the University of Texas at Austin and the Texas Transportation Institute (TTI) at Texas A&M University. The CTR researchers related SCC workability to materials and mixtures proportions, developed a series of SCC mixtures expected to be representative of SCC produced in Texas for prestressed concrete bridge beams, evaluated the early-age engineering properties and shrinkage of these mixtures, and developed recommendations for specifying and inspecting SCC. The TTI researchers evaluated the longer-term engineering properties of these mixtures and tested full-scale beams. The distinction between early-age and longer-term engineering properties was generally defined as the time of the release of tension in prestressed beams (16 to 24 hours). This report describes the research conducted by the CTR researchers. The research conducted by the TTI researchers is described in a separate report.

1.1 Background

SCC is defined in terms of its workability. The three essential properties of SCC are its ability to flow under its own mass (filling ability), its ability to pass through congested reinforcement (passing ability), and its ability to resist segregation (segregation resistance). The American Concrete Institute defines SCC as a “highly flowable, non-segregating concrete that can spread into place, fill the formwork, and encapsulate the reinforcement without any mechanical consolidation.” The Precast/Prestressed Concrete Institute (2003) defines SCC as “a highly workable concrete that can flow through densely reinforced or geometrically complex structural elements under its own weight and adequately fill voids without segregation or excessive bleeding without the need for vibration to consolidate it.”

It is generally accepted that SCC was originally developed in Japan in the 1980s in response to the lack of skilled labor and the need for improved durability. According to Ouchi (1999) the need for SCC was first identified by Okamura in 1986 and the first prototype was developed in 1988. Collepardi (2003), however, states that self-leveling concretes were studied as early as 1975 and used in commercial applications in Europe, the United States, and Asia in

the 1980s. The use of SCC has gradually increased throughout the world since the 1980s, gaining particular momentum in the late 1990s. One of the first high profile applications of SCC was the Akashi Kaikyo bridge in Japan (Tanaka et al. 2003). Major international symposia on SCC were held in 1999, 2001, 2003, and 2005. Originally, the main application for SCC was in precast plants; however, the use of SCC in ready mixed concrete applications has grown.

The development of mixture proportions often requires more effort for SCC than for conventionally placed concrete. The exact choice of proportions depends on material availability and application requirements. For instance, passing ability may be of little or no importance in some cases whereas segregation resistance is needed in all cases. Hardened property requirements can vary widely. SCC mixture proportions, in comparison to conventionally placed concrete mixture proportions, typically exhibit some combination of higher paste volume, higher powder content, lower water-powder ratio, finer combined aggregate grading, and smaller maximum aggregate size. SCC mixtures always include a high-range water-reducing admixture (HRWRA) to ensure concrete is able to flow under its own mass. Supplementary cementitious materials (SCMs) and mineral fillers are commonly utilized to decrease cost, improve workability, and improve hardened properties. Viscosity modifying admixtures (VMAs) may be used to ensure stability, especially in mixtures with relatively high water-powder ratios. Ozyildirim (2005) found that although SCC can often be made with local materials, shipping materials—even from long distances—may be cost effective for SCC. Proportioning methods for SCC have traditionally been classified into three general categories depending on the predominate change in mixture proportions. These categories include use of high powder content and low water-powder ratio (powder-type SCC); use of low powder content, high water-powder ratio and VMA (VMA-type SCC); and use of moderate powder content, moderate water-powder ratio, and moderate VMA (combination-type SCC).

SCC is highly sensitive to changes in material properties and proportions and, therefore, requires increased quality control. Further, the consequences of deviations in workability are more significant for SCC. For instance, a slight change in water content may have minimal effects on conventionally placed concrete but lead to severe segregation and rejected work in SCC.

The typical characteristics of SCC mixture proportions, which are necessary to ensure adequate workability, can have significant consequences for hardened properties, including strength, dimensional stability, and durability. The same trends associated with conventionally placed concrete typically apply to SCC. The relatively low water-cementitious materials ratios, use of SCMs, improved consolidation, and improved quality control measures can result in improved hardened properties. The finer combined aggregate gradings and increased paste volumes may result in changes such as increased shrinkage and reduced modulus of elasticity and shear strength. The higher powder content, when composed of increased cementitious materials content, may result in increased heat of hydration. Because the extent of these changes in mixture proportions can vary widely depending on local materials and application requirements, hardened properties should be evaluated in reference to specific changes in mixture proportions and not to SCC in general. With the difficulty of achieving compaction resolved, the availability of SCC can enable the development of new types of concrete systems with novel structural designs (Ouchi 1999; PCI 2003).

The advantages and disadvantages of SCC must be evaluated for each producer and application. In general, the advantages of SCC may include:

- Improved ability of concrete to flow into intricate spaces and between congested reinforcement.
- Improved form surface finish and reduced need to repair defects such as bug holes and honeycombing.
- Reduced construction costs due to reduced labor costs and reduced equipment purchase and maintenance costs.
- Increased construction speed due to fewer construction tasks.
- Faster unloading of ready mixed concrete trucks.
- Improved working conditions with fewer accidents due to elimination of vibrators.
- Improved durability and strength of the hardened concrete in some cases.
- Reduced noise generated by vibrators.

The disadvantages of SCC may include:

- Increased material costs, especially for admixtures and cementitious materials.
- Increased formwork costs due to possibly higher formwork pressures.
- Increased technical expertise required to develop and control mixtures.
- Increased variability in properties, especially workability.
- Increased quality control requirements.
- Reduced quality of hardened properties in some cases—possibly including modulus of elasticity and dimensional stability—due to factors such as high paste volumes or finer combined aggregate gradings.
- Delayed setting time in some cases due to the use of admixtures.
- Increased risk and uncertainty associated with the use of a new product.

Prestressed concrete bridge beams are a potentially promising application for SCC because of the highly congested reinforcement, the need for a smooth surface finish, and the possibility for reduced construction costs. In addition, production conditions can often be better controlled in a precast plant than in a ready mixed concrete setting. Concrete for use in prestressed bridge beams must develop mechanical properties quickly to allow the forms to be stripped and tension to be released in the strands as early as 16 to 24 hours. The long-term concrete mechanical and durability properties are also critical to the successful structural performance of prestressed concrete bridge beams. For SCC to be used in prestressed concrete bridge beams in Texas, it is necessary to answer the following questions:

- Can SCC mixtures be proportioned with the materials and production conditions in Texas to achieve the performance requirements for prestressed bridge beams?
- What changes in materials and mixture proportions relative to conventionally placed concrete must be made to ensure adequate SCC workability?
- What are the consequences of these changes in mixture proportions in terms of early-age and long-term hardened properties, such as compressive strength, tensile strength, flexural strength, modulus of elasticity, shear strength, shrinkage, and durability?
- Do the potential changes in hardened properties affect the structural performance of prestressed concrete bridge beams—such as camber, transfer and development length, and flexural and shear capacity—and require changes in the way prestressed concrete bridge beams are designed?
- How should TxDOT specify and inspect SCC to ensure consistent quality?

These questions were addressed by the CTR and TTI researchers.

1.2 Objectives and Scope

The objectives of the research conducted by the CTR researchers and described in full in this report were as follows:

- Review the literature to identify the state-of-the art in SCC technology.
- Survey precasters in Texas to identify typical materials and construction practices.
- Identify workability and hardened property requirements for SCC used in prestressed concrete bridge beams.
- Relate SCC workability characteristics—including filling ability, passing ability, segregation resistance, and rheology—to materials and mixture proportions.
- Evaluate workability test methods to determine their suitability for use in qualifying mixture proportions and monitoring quality control.
- Develop SCC mixture proportions that (1) represent a range of materials and mixture proportions likely to be used in Texas for prestressed concrete bridge beams and that (2) allow the direct evaluation of the effects of materials and mixture proportions on hardened properties.
- Evaluate the early-age engineering properties of the SCC mixture proportions—including the setting characteristics, heat generation, and development of compressive strength and modulus of elasticity—and compare these to the early-age engineering properties of conventionally placed concrete mixtures.
- Evaluate the effects of hot and cold weather conditions on the workability and early-age engineering properties of the SCC mixture proportions.
- Evaluate the early-age and longer-term shrinkage of SCC and conventionally placed concrete mixtures.
- Develop guidelines for modifying mixtures during production.
- Develop recommendations for specifying and inspecting SCC.

To achieve these objectives, representative materials were identified and shipped to the CTR and TTI laboratories. Testing was conducted at the CTR laboratory to relate workability characteristics to materials and mixture proportions. As part of this process, workability test methods were evaluated. With these relationships well understood, a series of final mixture proportions was developed. These mixtures were then tested for early-age engineering properties and shrinkage in the CTR laboratory and longer-term engineering properties in the TTI laboratory. Because early-age engineering properties are highly dependant on time and temperature, they were evaluated under a range of temperature conditions representing cold, mild, and hot conditions. Guidelines were developed for modifying mixtures during production. Field testing was conducted in a precast plant to verify laboratory test results and to evaluate practices for placing and controlling the quality of SCC. The CTR research results were documented in this report. A separate guide for TxDOT inspectors evaluating SCC was also developed.

2. Literature Review

This chapter summarizes the literature on the materials, fresh properties, hardened properties, and mixture proportioning methods for SCC.

2.1 Materials

2.1.1 Chemical Admixtures

The key chemical admixtures used to produce SCC are HRWRAs and, in some cases, VMAs. Other admixtures—including air-entraining admixtures and set-modifying admixtures—can also be used successfully in SCC.

2.1.1.1 High-Range Water-Reducing Admixtures

SCC is most commonly produced with polycarboxylate-based HRWRAs, which represent an improvement over older sulfonate-based HRWRAs such as those based on naphthalene sulfonate formaldehyde condensate (NSFC) and melamine sulfonate formaldehyde condensate (MSFC). Although SCC can be made with NSFC-, MSFC-, and lignosulfonate-based HRWRAs (Lachemi et al 2003; Assaad, Khayat, and Meshab 2003a; Petersen and Reknes 2003), the introduction of polycarboxylate-based HRWRAs has facilitated the adoption of SCC (Bury and Christensen 2002). Compared to sulfonate-based HRWRAs, polycarboxylate-based HRWRAs require lower dosages, have a reduced effect on setting time, exhibit improved workability retention, and increase stability. In fact, polycarboxylate-based HRWRAs typically enable a 70 to 80% reduction in dosage compared to a typical NSFC- or MSFC-based HRWRAs, based on solids content as a percentage of cement mass (Jeknavorian et al. 2003).

Polycarboxylate-based HRWRAs differ from sulfonate-based HRWRAs in their structure and mode of action. Sulfonate-based HRWRAs consist of anionic polymers that adsorb onto cement particles and impart a negative charge, resulting in electrostatic repulsion. Polycarboxylate-based HRWRAs, by contrast, consist of flexible, comb-like polymers with a main polycarboxylic backbone and grafted polyethylene oxide side chains. The backbone, which includes ionic carboxylic or sulfonic groups, adsorbs onto a cement particle and the nonionic side chains extend outward from the cement particle. The side chains physically separate cement particles, which is referred to as steric hindrance. Polycarboxylate-based HRWRAs may function by both electrostatic repulsion and steric hindrance (Bury and Christensen 2002; Yoshioka et al. 2002; Cyr and Mouret 2003; Li et al. 2005) or only by steric hindrance (Blask and Honert 2003; Li et al. 2005; Hanehara and Yamada 1999) depending on the structure of the polymer. The reduced significance of electrostatic repulsion is indicated by the less negative or near-zero zeta-potential measurements for cement pastes with polycarboxylate-based HRWRAs as compared to cement pastes with sulfonate-based HRWRAs (Blask and Honert 2003; Li et al. 2004; Collepardi 1998; Sakai, Yamada, and Ohta 2003). In fact, zeta-potential measurements are frequently insufficient to justify dispersion of cement particles by polycarboxylate-based

HRWRAs on the basis of the DLVO theory for electrostatic repulsion (Sakai, Yamada, and Ohta 2004).

Compared to sulfonate-based HRWRAs, polycarboxylate-based HRWRAs generally produce rheological characteristics that are more favorable for the production of SCC. Polycarboxylate-based HRWRAs are able to reduce the yield stress to a greater degree than NSFC- and MSFC-based HRWRAs (Cyr and Mouret 2003). For a given decrease in yield stress, the reduction in plastic viscosity is less for polycarboxylate-based HRWRAs than for sulfonate-based HRWRAs (Golaszewski and Szwabowski 2004). Yamada et al. (2000) found that polycarboxylate-based HRWRAs reduce plastic viscosity at high water-cement ratios but result in only slight reductions in plastic viscosity at low water-cement ratios. Similarly, Golaszewski and Szwabowski (2004) found that differences in rheological performance between polycarboxylate-based and sulfonate-based HRWRAs were most pronounced at lower water-cement ratios. According to Hanehara and Yamada (1999), polycarboxylate-based HRWRAs begin to affect mortar mini-slump flow at lower dosages than NSFC-based HRWRAs; however, NSFC-based HRWRAs increase mortar mini-slump flow at a faster rate once they begin to have an effect. The relatively high plastic viscosities associated with polycarboxylate-based HRWRAs can make high-strength, low water-cement ratio concrete mixtures impractical (Sugamata, Sugiyama, and Ohta 2003; Golaszewski and Szwabowski 2004). As a consequence, Sugamata, Sugiyama, and Ohta (2003) developed a new polycarboxylate-based HRWRA that incorporated a new monomer in order to reduce plastic viscosity and thixotropy.

The unique structure of polycarboxylate-based HRWRAs contributes to their improved performance. Polycarboxylate-based HRWRAs can be designed at the molecular level for a particular application by changing such characteristics as the length of the backbone, or the length, density, or type of the side chains (Bury and Christensen 2002; Schober and Mader 2003; Comparet et al. 2003; Sakai, Yamada, and Ohta 2003). These changes can affect water reduction, workability retention, setting time, and early strength development (Bury and Christensen 2002). As such, not all polycarboxylate-based HRWRAs are intended for SCC. Those intended for SCC typically provide a higher plastic viscosity for a similar slump flow and dosage (Berke et al. 2002). Velten et al. (2001) suggest blending multiple polycarboxylate-based polymers to create a single admixture that exhibits improved workability retention and reduced sensitivity to changes in cement characteristics.

Numerous studies have been published describing the development of polycarboxylate-based polymers to optimize water reduction, workability retention, setting time, and strength development. In general, water reduction can be increased by increasing the side chain length, reducing the side chain density, reducing the backbone length, or increasing the sulfonic group content (Yamada et al. 2000; Plank and Hirsch 2003; Sakai, Yamada, Ohta 2003; Schober and Mader 2003). In general, the workability retention for polycarboxylate-based HRWRAs is superior to that for sulfonate-based admixtures for two main reasons (Flatt and Houst 2001; Cerulli et al. 2003). First, the side chains of polycarboxylate-based polymers are active at longer distances away from the cement grain and are, therefore, not incorporated into hydration products as soon. Second, some polycarboxylate polymers can remain in aqueous solution and adsorb onto cement particles gradually over time as hydration progresses. Sakai, Yamada, and Ohta (2003) suggest that workability retention can be improved by increasing the side chain length while Schober and Mader (2003) and Yamada et al. (2000) suggest that workability retention can be increased by decreasing the side chain length. Schober and Mader (2003) suggest the improved workability retention for shorter side chains is due to the fact that shorter

side chains require longer times to adsorb on cement surfaces. Sakai, Yamada, and Ohta (2003) further suggest that workability retention can be improved by reducing the backbone length or increasing the side chain density, while Yamada et al. (2000) found that reducing the backbone length had minimal effect on workability retention. Velten et al. (2001) found that reducing the ionic content of the backbone reduced the adsorption rate, allowing more polymer to remain in solution to be adsorbed at later times. The increase in setting time associated with HRWRAs can be decreased by increasing the side chain length, increasing the backbone length, or increasing the degree of polymerization in the backbone (Yamada et al. 2000). The improved strength gain in polycarboxylate-based HRWRAs is the result of the hydrophilic side chains, which draw water to the cement particles, resulting in uniform hydration and rapid early strength gain (Jeknavorian et al. 2003).

The performance of polycarboxylate-based HRWRAs is strongly influenced by cement characteristics, including specific surface area, particle size distribution, sulfate type and content, C_3A content, alkali content, and the presence of grinding aids (Flatt and Houst 2001). Differences in performance of various cement-admixture combinations are typically more significant at lower water-cement ratios (Schober and Mader 2003). The action of polycarboxylate-based polymers added to concrete can be classified in one of three categories: the polymers may be consumed by intercalation, coprecipitation, or micellization, resulting in the formation of an organo-mineral phase; they may be adsorbed on cement particles; or they may remain dissolved in the aqueous phase (Flatt and Houst 2001). Because the performance of HRWRAs depends on the early-age reactions taking place in the first two hours, the initial reactivity of the cement is critical.

Cements with higher fineness and higher C_3A contents are more reactive and, therefore, require higher dosages (Sakai, Yamada, Ohta 2003; Yoshioka et al. 2002). Yoshioka et al. (2002) found that single synthetic phases of C_3A and C_4AF adsorbed significantly more superplasticizer than C_2S and C_3S ; however, the ratio of superplasticizer adsorbed by C_3A to that adsorbed by C_3S was less for the two polycarboxylate-based admixtures considered than for the NSFC-based admixture. Plank and Hirsch (2003), however, found that the decrease in adsorption observed in cements with lower C_3A contents was more significant for the polycarboxylate-based admixtures than for the NSFC- or MSFC-based admixtures. It should be noted that other differences in the three cements tested by Plank and Hirsch (2003) could have contributed to the differences in performance. Although dispersion of cement particles generally increases with increasing HRWRA adsorption (Schober and Mader 2003), the preferential adsorption by C_3A necessitates a higher dosage for adsorption on other phases. Further, a portion of the HRWRA adsorbed on C_3A is consumed in early age hydration products (Schober and Mader 2003). It is also desirable to have some polycarboxylate-based polymer remaining in solution to provide dispersion over time (Burge 1999). Indeed, Schober and Mader (2003) found that cements with higher C_3A contents exhibited reduced workability retention unless the dosage was sufficiently high to provide polymer for delayed adsorption. The early hydration products such as ettringite increase specific surface area, requiring additional polycarboxylate-based polymers to maintain dispersion (Schober and Mader 2003). Polycarboxylate-based polymers are not intercalated into ettringite, but can be intercalated into monosulfoaluminate, C-S-H, and possibly brucite-like phases (Flatt and Houst 2001; Plank and Hirsch 2003). Plank and Hirsch (2003) found that ettringite is the preferred phase for adsorption of NSFC-, MSFC-, and polycarboxylate-based polymers, but that calcium hydroxide and gypsum show no adsorption. Although the presence of HRWRAs does not affect the quantity of ettringite formed, the

HRWRAs do reduce the size of the ettringite crystals formed, especially with sulfonate-based HRWRAs (Plank and Hirsch 2003). Cerulli et al. (2003) suggest that the rate of hydration and mechanical strength development are influenced by the difference in the morphological structures of the ettringite crystals.

The presence of sulfate ions in solution reduces the adsorption of polycarboxylate-based polymers because it is generally thought that sulfates compete with polycarboxylate-based polymers for adsorption on cement particles (Sakai, Yamada, Ohta 2003; Schober and Mader 2003). Whereas an optimum sulfate ion concentration exists for sulfonate-based HRWRAs, the sulfate ion concentration should be minimized for polycarboxylate-based HRWRAs (Flatt and Houst 2001; Yamada, Ogawa and Takahashi 2001). Comparet et al. (2003), however, found that the reduction in adsorption was not due to the increase in sulphate ion concentration but instead due to an increase in ionic strength, regardless of whether sodium sulphate or sodium chloride was used to change the ionic strength of a calcium carbonate model system. Ohno et al. (2001) state that the reduction in polycarboxylate performance may be due to an increase in sulphate ion concentration, an increase in ionic strength, or both. They point out that the ionic strength increases at lower water-cement ratios, which may further reduce cement dispersion. In evaluating the effects of sulfates, the source of sulfates should be considered. Sulfates are supplied by both alkali sulfates and calcium sulfates. The type of calcium sulfate matters, as hemihydrate supplies sulfate ions faster than gypsum (Sakai, Yamada, Ohta 2003). Schober and Mader (2003) suggest using low-alkali cements to reduce the availability of soluble sulfates. Hanehara and Yamada (1999) indicate that the content of alkali sulfates is responsible for the majority of the difference in performance between different cements and conclude that the presence of sulfate ions both reduce adsorption and reduce side chain length. Sakai, Yamada and Ohta (2003) suggest that changes in the performance of polycarboxylate-based polymers due to changes in sulfate ion concentration can be minimized most effectively by increasing the carboxylic group ratio in the backbone. As hydration progresses, the gradual decrease in sulfate ion concentration allows any polycarboxylate-based polymer remaining in solution to be adsorbed on cement particles more readily, which helps to maintain or even increase workability (Sakai, Yamada, and Ohta 2003). Yamada, Ogawa, and Takahashi (2001) suggest increasing the backbone length, side chain length, or carboxylic ratio to increase the resistance to changes in sulfate ion concentration and suggest blending polycarboxylate-based admixtures with different adsorbing abilities to ensure initial fluidity and long workability retention.

The performance of polycarboxylate-based HRWRAs is influenced by temperature. Yamada, Yanagisawa, and Hanehara (1999), in evaluating pastes at 3 temperatures, found that at low temperatures, the initial fluidity was low but increased with time due to the slower increase in cement surface area, the high initial sulfate ion concentration, and the larger decrease in sulfate ion concentration with time. At high temperatures, the fluidity decreased more rapidly because of the faster reaction rate for cement, which was associated with a faster increase in cement surface area and a smaller reduction in sulfate ion concentration.

Polycarboxylate-based HRWRAs are less sensitive than sulfonate-based admixtures to the time of addition. Whereas the efficiency of sulfonate-based admixtures can be improved by delaying the addition of the admixture to the start of the dormant period, the time of addition has minimal effect for polycarboxylate-based admixtures (Blask and Honert 2003; Collepardi 1998; Golaszewski and Szwabowski 2004). Plank and Hirsch (2003) and Collepardi (1998) suggest that sulfonate-based admixtures have high adsorption rates during ettringite growth, resulting in the consumption of the admixture such that a lower concentration remains in solution for

dispersion of C_3S and C_2S . Flatt and Houst (2001) suggest that sulfonate-based admixtures are consumed in the organo-mineral phase whereas the side chains in polycarboxylates extend beyond the organo-mineral phase.

Polycarboxylate-based HRWRAs are more sensitive than sulfonate-based HRWRA to the amount of mixing energy. Blask and Honer (2003) found that increasing the mixing energy dramatically reduced the shear resistance of cement pastes with polycarboxylate-based HRWRA but had only minimal effects on cement pastes with naphthalene sulfonate-based HRWRA. Takada and Walraven (2001) found that increasing the mixing energy for cement paste mixtures reduced plastic viscosity significantly but had no effect on yield stress. The difference was attributed to better dispersion of powder particles and to the generation of high air contents.

The presence of certain clays within aggregates can reduce the performance of polycarboxylate-based HRWRAs significantly. Jardine et al. (2002), Jeknavorian et al. (2003), and Jardine et al. (2003) examined the use of polycarboxylate-based HRWRAs with aggregates containing swellable smectite clays. These clays expand when wetted by the mixing water and adsorb polycarboxylate-based polymers, resulting in significantly higher dosage requirements and accelerated loss of workability. One solution to this problem was to change the mixing order so that the water, HRWRA, and part of the cement are mixed prior to the addition of the clay-bearing aggregate. This mixing procedure, however, was not considered practical. Another solution was to utilize a sacrificial agent that would adsorb and intercalate with clay minerals, would be compatible with other admixtures, and would not harm concrete properties. A suggested sacrificial agent was polyethylene glycol, although it was found that this agent did itself reduce workability. Third, a calcium salt such as calcium nitrate could be added prior to the addition of sand. A combination of these three methods could be used. An additional solution was to add a polyphosphate, which could be used independent of the order of addition.

2.1.1.2 Viscosity-Modifying Admixtures

Viscosity-modifying admixtures, also known as anti-washout admixtures, generally increase some or all of the following properties in concrete mixtures: yield stress, plastic viscosity, thixotropy, and degree of shear thinning. They can be used for SCC applications to improve segregation resistance, increase cohesion, reduce bleeding, allow the use of a wider range of materials such as gap-graded aggregates and manufactured sands, and mitigate the effects of variations in materials and proportions (Bury and Christensen 2002). They may be used as an alternative to increasing the powder content or reducing the water content of a concrete mixture. Berke et al. (2002) suggest that SCC should be produced without a VMA whenever possible, but that a VMA can be necessary in certain situations such as where aggregate moisture content cannot be controlled adequately or in mixtures with poorly graded aggregates or low powder content.

The VMAs used for SCC are typically water-soluble polymers; however, other materials such as precipitated silica can be used (Rols, Ambrose, and Pera 1999; Khayat and Ghezal 2003; Collepardi 2003). Water-soluble polymers for use as VMAs in concrete can be broadly classified as natural, semi-synthetic, and synthetic. Examples of each class are provided in Table 2.1. Common VMAs for concrete include cellulose derivatives—which contain nonionic cellulose ether with various substitutes in the ether—and welan gum—which is an anionic, high-molecular weight, natural polysaccharide fermented under controlled conditions (Khayat 1998; Lachemi et al. 2004a; Lachemi et al. 2004b).

Table 2.1 Examples of Water-Soluble Polymers Used as VMA (Khayat 1998)

Natural	Semi-Synthetic	Synthetic
• starches • guar gum • locust bean gum • alginates • agar • gum arabic • welan gum • xanthan gum • rhamsan gum • gellan gum • plant protein	• decomposed starch and its derivatives • cellulose-ether derivatives ○ hydroxypropyl methyl cellulose (HPMC) ○ hydroxyethyl cellulose (HEC) ○ carboxy methyl cellulose (CMC) • electrolytes ○ sodium alginate • propyleneglycol alginate	• polymers based on ethylene ○ polyethylene oxide ○ polyacrylamide ○ polyacrylate • polymers based on vinyl ○ polyvinyl alcohol

VMAs based on water-soluble polymers typically affect the water phase of concrete. Khayat (1995) describes three modes of action by which VMAs function. First, the VMA polymers adsorb onto water molecules, which causes a portion of the water to become trapped and the polymers to expand. Second, the polymers themselves develop attractive forces and thereby block the motion of water. Third, the polymer chains intertwine at low shear rates but break apart at higher shear rates, resulting in shear-thinning behavior. This shear-thinning behavior is desirable because the high apparent viscosity at low shear rates ensures static stability while the lower apparent viscosity at high shear rates results in less energy needed for processes such as mixing, conveying, and consolidating. Bury and Christensen (2002) divide VMAs into two categories: thickening-type and binding-type. Thickening-type VMAs increase viscosity by thickening the concrete but do not significantly increase HRWRA demand. Binding-type VMAs, which are more potent than thickening-types, bind water and result in thixotropic properties and reduced bleeding.

The improvements in concrete properties when VMAs are used are mainly due to increases in viscosity and degree of shear thinning. The increase in yield stress typically must be offset with additional water or HRWRA. For example, the anionic nature of natural polymers may cause them to adsorb onto cement particles, thereby requiring additional HRWRA (Phyffereon et al. 2002). Even with this increase in HRWRA dosage, water content, or both, the concrete will still exhibit increased viscosity and a greater degree of shear thinning. The higher viscosity and greater degree of shear thinning can increase segregation resistance. Bleeding is reduced due to the increase in viscosity and the reduction in free water. This reduction in bleeding, however, can increase the susceptibility to plastic shrinkage cracking (Khayat 1999). Top bar effect, or the reduction in bond between concrete and reinforcing bars higher in a structural element, is reduced due to the reduction in bleeding, segregation, and surface settlement (Khayat 1998).

Welan gum, one of the most common types of VMA, has been shown to increase yield stress, viscosity, and the degree of shear thinning (Khayat and Yahia 1997) while also mitigating the effects of changes in water content (Berke et al. 2002; Sakata, Maruyama, and Minami 1996). Whereas cellulose derivatives are incompatible with naphthalene-based HRWRAs, welan gum is compatible (Khayat 1995). Welan gum, xanthan gum, and guar gum are less affected by changes in temperature than are polyacrylate, methyl cellulose, and hydroxyl ethyl cellulose (Sakata, Maruyama, and Minami 1996). Whereas some cellulose derivatives can entrap relatively

large air volumes, thereby necessitating the use of a defoamer, welan gum does not generally affect the air void system (Khayat 1999). Phyffereon et al. (2002) found that diutan gum was slightly preferable to welan gum because it exhibited higher viscosity and a greater degree of shear thinning, was less affected by changes in cement characteristics, and exhibited a lower charge density so that less HRWRA was required for a constant flow. Despite the many advantages of welan gum, Lachemi et al. (2004a) suggests that the high cost of welan gum relative to other possible alternatives could make the use of welan gum impractical.

Welan gum and cellulose derivative VMAs may delay concrete setting times, while acrylic-type VMAs generally do not affect setting time (Khayat 1995; Khayat 1998). Not only do welan gum and cellulose derivative VMAs themselves increase setting time, the higher dosages of HRWRA required to maintain constant slump flow may further delay setting time. The use of VMA may require significantly higher dosages of air entraining agent due in part to the reduction of available free water (Khayat 1995). Nonetheless, Khayat (1995) found that adequate air void parameters could be achieved in mixtures with VMA.

The presence of VMAs can alter cement hydration, resulting in changes in hardened concrete properties. Khayat (1996) found that welan gum and hydroxypropyl methyl cellulose generally decreased compressive strength, flexural strength, and modulus of elasticity. The reductions in flexural and compressive strengths were more pronounced at lower water-cement ratios while the reduction in modulus of elasticity was more pronounced at higher water-cement ratios. On the basis of x-ray diffraction and scanning electronic microscopy, Khayat (1996) speculated that VMAs interfere with hydration by limiting the water available to cement particles for hydration and reducing the rate of dissolution of cement. Mercury intrusion porosimetry measurements indicated that these VMAs increased the volume of coarse capillary pores for high water-cement ratios but had little effect on the pore size distribution at low water-cement ratios.

2.1.1.3 Air Entraining Admixtures

Air entraining admixtures (AEAs) can be used in SCC to achieve adequate air content, air bubble size, air bubble spacing, and freeze-thaw resistance (Khayat 2000; Khayat and Assaad 2002; Persson 2003; Ozyildirim 2005). Ozyildirim (2005) found that improper air void systems and poor freeze-thaw durability can occur in SCC, but added that these properties are not intrinsic to SCC. If the volume of paste is increased for SCC, the volume of air in the concrete may need to be increased to maintain the same percentage of air volume in the paste.

The potential use of SCMs and multiple chemical admixtures can increase the complexity of entraining an adequate air void system (Khayat and Assaad 2002). Indeed, Khayat (2000) reported using AEA in SCC at a considerably higher dosage than required for conventionally placed concrete. Khayat and Assaad (2002) found that increasing NFSC-based HRWRA dosage increased AEA demand; however, increasing the fluidity of mixtures by other means reduced AEA demand as more free water was available. Khayat and Assaad (2002) further found that the spacing factor increased with increased slump flow, possibly due to the tendency of air bubbles to coalesce. Increasing the binder content; however, decreased the spacing factor. Polycarboxylate-based admixtures may themselves entrain air; however, most commercially available admixtures include a defoamer to offset this effect.

Entrained air bubbles can reduce viscosity, which may reduce the stability of the concrete and necessitate other changes to the mixture such as the use of VMA or reduced water content (Khayat 2000). Similarly, low viscosity in concrete may reduce air void stability. Khayat and

Assaad (2002) found that the air void system in SCC can remain stable even after agitation over time. They concluded that yield stress and plastic viscosity should not be too low, which would cause segregation and a loss of air, nor too high, which would increase the internal pressure in air bubbles and could result in a loss of air content.

2.1.2 Aggregates

Aggregates for SCC are selected in terms of their grading, maximum size, and shape characteristics (including shape, angularity, and texture). Mineral fillers—most commonly finely ground limestone filler—have also been used successfully in SCC. Aggregates mainly influence workability. The effects of aggregates on hardened properties are often indirect—namely the changes in mixture proportions required to achieve a given workability level often have a larger influence than the aggregates themselves on hardened properties.

Packing density is often used to describe aggregate characteristics. The geometrical characteristics of shape, angularity, texture, and grading affect packing density; therefore, packing density can be used as an indirect indicator of aggregate geometrical characteristics. In general, factors that increase packing density also improve concrete flow properties. Aggregates with higher packing density require less paste to fill the voids between aggregates. Equidimensional, well-rounded aggregates typically result in both higher packing density and reduced interparticle friction between aggregates, both of which improve workability.

In general, higher packing density is preferred, although the maximum packing density may not be optimal (Johansen and Andersen 1991; Goltermann, Johansen, and Palbol 1997; Powers 1932; Powers 1968). According to Goltermann, Johansen, and Palbol (1997), concrete mixtures should have more fine aggregate than what is required for the maximum packing density. It must be noted however, that a small change in sand content does not generally result in a large change in packing density. The use of a higher volume fraction of aggregate—especially coarse aggregate—can result in improvements in strength, stiffness, creep, drying shrinkage, and permeability (Johansen and Andersen 1991). The use of higher packing density with continuous grading and a narrow grading span results in reduced segregation (de Larrard 1999a). Khayat, Hu, and Laye (2002) found that SCC with near optimum aggregate packing exhibited lower viscosity, lower HRWRA demand, and similar or greater filling capacity than SCC with slightly lower aggregate packing density. The SCC with slightly lower packing density exhibited better stability due to the higher content of fines smaller than 80 μm and lower coarse aggregate volume. Struble et al. (1998) found that yield stress was minimum near the maximum packing density, but plastic viscosity was minimum at a lower sand content. Johansen and Andersen (1991), however, found that yield stress was minimum at the maximum packing density and that plastic viscosity was minimum at a higher sand content.

Concrete can be considered a suspension of aggregates in paste; therefore, concepts from suspension rheology can be applied to developing mixture proportions in general and to selecting aggregates in particular. Suspension rheology literature had been extensively developed on a wide range of materials (Coussot and Ancey 1999).

It is well known from empirical evidence that the rheology of suspensions depends on the solids volume concentration, the extent of agglomeration and flocculation of the solids, particle shape characteristics, and particle size distribution (Struble et al. 1998; Tsai, Botts and Plouff 1992). Phenomenological models express rheology as a function of solids volume fractions with additional parameters to account for the extent of agglomeration and flocculation, particle shape

characteristics, and particle size distribution. Frequently, these additional factors are accounted for with the maximum solids fraction, which is defined as the solids volume concentration at which particle interference makes flow impossible and the viscosity approaches infinity. A material with higher maximum solids fraction—due to favorable particle shape characteristics, particle size distribution, and lack of flocculation—results in lower relative viscosity at a given solids volume fraction (Barnes, Hutton, and Walters 1999). For example, the Krieger-Dougherty (1959) equation, shown in Equation (2.1), is well-known and widely used. It expresses the suspension viscosity (η) in terms of the viscosity of the suspending fluid (η_s), the solids volume concentration (ϕ), the maximum solids volume fraction (ϕ_m), and the intrinsic viscosity ($[\eta]$).

$$\eta = \eta_s \left(1 - \frac{\phi}{\phi_m} \right)^{-[\eta]\phi_m} \quad (2.1)$$

The intrinsic viscosity accounts for particle shape characteristics whereas the maximum solids volume fraction accounts for particle shape characteristics, degree of flocculation, and particle size distribution (Struble and Sun 1995). The intrinsic viscosity is 2.5 for spheres and increases with particle asymmetry. The maximum solid fraction and intrinsic viscosity vary with shear stress and shear rate (Barnes, Hutton, and Walters 1989; Struble and Sun 1995).

The Krieger-Dougherty equation has been applied successfully to cement paste (Struble and Sun 1995). Szecsy (1997) applied the Krieger-Dougherty equation with modifications to concrete. Martys (2005) suggested Equation (2.2) as an improvement on the Krieger-Dougherty equation:

$$\eta = \eta_s \left(1 - \frac{\phi}{\phi_m} \right)^{-n} \left[1 + K_1 \frac{\phi}{\phi_m} + K_2 \left(\frac{\phi}{\phi_m} \right)^2 + \dots \right] \quad (2.2)$$

where n is termed the critical exponent, $K_1 = \phi_m [\eta] - n$, and $K_2 = \phi_m^2 K_H - n \phi_m [\eta] (n - \frac{1}{2})$.

2.1.2.1 Grading and Maximum Size

The grading, or particle size distribution, of all materials in a concrete mixture—including aggregates, cementitious materials, and other powder additions—are highly relevant to concrete performance. A variety of techniques must be employed to characterize the full grading, which can range from the order of nanometers to tens of millimeters.

For an aggregate source with certain shape, angularity, and texture characteristics, the grading can significantly influence the aggregate's performance in concrete. The importance of a well-graded aggregate with a wide range of particle sizes is well established for producing high-quality concrete. Gradings for SCC typically exhibit higher ratios of sand to total aggregate, have smaller maximum aggregate sizes, and tend to be uniformly graded without an excess or deficiency of material on two consecutive sieves.

As an aggregate size gets smaller, its value to concrete increases because it becomes more costly to produce and its characteristics have a larger influence on concrete properties (Hudson 2002). Shape, angularity, and texture vary for each size fraction (Hudson 2003d).

According to Ozol (1978), sphericity increases with size. The intermediate size fraction, which approximately ranges from a No. 8 sieve to 13 mm, is known to affect workability, finishability, and shrinkage (Shilstone 1990; Hudson 2002). Bager, Geiker, and Jensen (2001) found that increasing the sand fineness increased the yield stress and plastic viscosity of self-consolidating mortars. For high-performance concrete, Aitcin (1998) suggested using coarse sands, with fineness moduli between 2.7 and 3.0, because the use of such sands decreases the amount of mixing water required and because sufficient fine particles are available from the cementitious materials.

The maximum aggregate size affects filling ability, passing ability, and segregation resistance. It is generally known that increasing the absolute sizes of particles does not affect the rheology of concentrated suspensions (Struble and Sun 1995; Mooney 1952). Therefore, increasing the maximum aggregate size improves concrete filling ability to the extent that it improves the grading. Increasing the polydispersity, or spread of sizes, is well known to reduce viscosity. For instance, Farris (1968) found that the lowest viscosity could be obtained as the number of monosized fractions became infinitely large. Decreasing the maximum size improves passing ability by reducing the size of particles that must pass through openings (Bui and Montgomery 1999b). Segregation is reduced by reducing the spread of sizes (de Larrard 1999a). Struble et al. (1998) found that increasing the aggregate size increased yield stress and plastic viscosity. For coarse aggregate, the use of a large maximum aggregate size reduces fresh concrete water demand; however, the hardened properties can be affected negatively because of the increased interfacial transition zone thickness and the fact that larger particles tended to contain more internal defects that would otherwise be removed during crushing (Aitcin 1998). In general, it is desirable to use the largest particle practical to maximize the ratio of volume to surface area (Hudson 2003a).

Numerous ideal particle size distributions have been suggested, most of which are for optimizing packing density. Based on packing model simulations, the optimal packing density of polydisperse mixtures can be achieved with continuous or gap-graded particle size distributions (de Larrard 1999a; Andersen and Johansen 1993). Based on simulations from the compressible packing model, de Larrard (1999a) found that for binary mixes, increasing the size difference between the two fractions increases packing density because interaction is reduced. Despite the improved packing associated with gap-graded aggregates, continuous gradings should be chosen to minimize bleeding and segregation (de Larrard 1999a). Ideal gradings have been suggested by Fuller and Thompson (1907), Andreasen and Anderson (1929), Bolomey (1947), Faury (qtd. in de Larrard 1999a), Druex (qtd. in de Larrard 1999a), and Weymouth (qtd. in Powers 1968). In 1968, Powers (p. 256) wrote that “the hypothesis that there is an ideal size grading for concrete aggregate, or for all solid material in concrete, has now become almost if not entirely abandoned.” Hudson (2003c) cautioned that the ideal grading depends on shape, angularity, and texture and must be selected independently for each aggregate.

In addition to generalized ideal grading curves, mathematical models are available for computing packing density from empirical measurements on individual size fractions. These models vary from simple models of binary combinations of monosized spheres with no interaction between particles to more complex models that incorporate polydisperse blends with interaction between particles. Interaction includes loosening effect, which results from smaller particles reducing the packing density of adjacent larger particles, and wall effect, which results from larger particles reducing the packing density of adjacent smaller particles. Packing models include those of Furnas (1931), Aim and Goff (qtd. in Goltermann, Johansen, and Palbol 1997),

Toufar, Klose, and Born (qtd. in Goltermann, Johansen, and Palbol 1997). Packing models were compared by Johansen and Andersen (1991) and by Goltermann, Johansen, and Palbol (1997). Andersen and Johansen (1993) combined the Aim and Goff model with the Toufar, Klose, and Born model to develop a series of tables to aid in combining aggregates. The compressible packing model (de Larrard 1999a), which is based on a linear packing density model, enables the calculation of the packing density of polydisperse granular mixes with particle interaction. On the basis of the compressible packing model, de Larrard compared the packing densities and segregation potentials of available particle size distributions, as shown in Table 2.2. The compressible packing model has been used by Sedran and de Larrard (1999) and Vachon, Kaplan, and Fellaki (2002) for SCC.

Table 2.2 *Results of Compressible Packing Model for Several Particle Size Distributions (de Larrard 1999a)*

Grading	Packing Density	Segregation Potential
Max. Density	0.929	0.59
Fuller	0.869	0.96
Fauy	0.927	0.59
Dreux	0.914	0.80
Uniform	0.891	0.85
Gap-Graded	0.928	1.00
Minimum <i>S</i>	0.926	0.53

2.1.2.2 Shape, Angularity, and Texture

Shape, angularity, and texture are defined in a variety of ways. Shape generally describes geometrical characteristics at the coarsest scale, texture at the finest scale, and angularity at an intermediate scale. Shape, texture, and angularity are independent of each other, although they may be correlated for certain sets of particles (Ozol 1978). Shape, texture, and angularity are of great interest in many industrial applications and have been the focus of much research (Pons et al. 1999).

Shape is frequently defined in terms of the three principle dimensions of a particle. For example, Powers (1968) defines a shape factor, which is shown in Equation (2.3):

$$\text{shape factor} = \frac{LS}{I^2} \quad (2.3)$$

where L is the longest principle dimension, S is the shortest principle dimension, and I is the intermediate principle dimension. A shape factor less than unity indicates a prolate shape while a shape factor greater than unity indicates an oblate shape. Shape may also be defined in terms of flatness and elongation, which are defined in Equations (2.4) and (2.5):

$$\text{flatness} = \frac{I}{S} \quad (2.4)$$

$$\text{elongation} = \frac{L}{I} \quad (2.5)$$

Sphericity, which represents how close the shape of a particle is to that of a sphere, is further used to describe shape (Powers 1968). The precise definition of sphericity can vary widely.

Angularity describes the sharpness of the corners and edges of a particle. In qualitative terms, particles may be described as angular, sub-angular, sub-rounded, rounded, or well-rounded.

Texture describes the roughness of a particle on a scale smaller than that used for shape and angularity. For instance, Ozol (1978) defines texture as either the degree of surface relief or the amount of surface area per unit of dimension or projected area and points out that texture depends on the amplitude and frequency of asperities.

Numerous other descriptors are available for shape, angularity, and texture. These descriptors vary in the sophistication of imaging techniques and mathematics required (Ozol 1978; Barrett 1980; Pons et al. 1999; Bouwman 2004).

In concrete, the shape, angularity, and texture of aggregates affect the bulk voids and frictional properties of the aggregates (Hudson 2002). The void content in combined aggregates can vary as much as 8 to 9 percentage points due to variations in shape, angularity, and texture, but in practice this range is typically much less (Ozol 1978). Hudson (2003c) suggests that shape and texture are more important than grading and asserts that the focus on grading is mainly due to the historic use of natural sands, which do not vary in shape, texture, and angularity to the degree that manufactured sands do. Hudson (2003b) cites data indicating that mixtures with the same specific area but with different gradings had similar water requirement and compressive strength. Aggregates with cubical shapes are desirable; poorly shaped aggregate may require increased cement content (Hudson 2003e; Goldsworthy 2005). According to Hudson (2003f), texture becomes more important as particle size decreases because more surface area is available. Ozol (1978) indicates that angularity is more influential than shape for workability. Tattersall (1991) and Bager, Geiker, and Jensen (2001) found that texture had little effect on workability. Bager, Geiker, and Jensen (2001) found that increasing the aspect ratio of particles increased yield stress and plastic viscosity in self-consolidating mortars.

2.1.2.3 Mineral Fillers

Mineral fillers—including finely ground fillers and dust-of-fracture microfines—are generally defined as mineral material finer than a certain size—typically between 75 μm and 150 μm . Dust-of-fracture microfines are generated in the production of manufactured sands. Microfines are also present in natural sands, though typically in a smaller quantity. Fine ground filler is most commonly comprised of limestone.

Like aggregates in general, the shape, angularity, texture, and grading of mineral fillers are important. The shape, angularity and texture can vary widely (Stewart et al. 2005). Grading is important in the context of how the mineral fillers affect the grading of the combined powder materials (Yahia, Tanimura, and Shimoyama 2005). In addition, the potential presence of clays and other deleterious materials present in aggregates must be considered. Although they may be in the same size range as mineral fillers, clays have different mineralogy and affect concrete performance differently. Clays may adsorb water in freshly mixed concrete and expand. The

resulting reduction in free water reduces workability (Yool, Lees, and Fried 1998). If the clays later dry and shrink, the resulting voids reduce strength and permeability (Hudson 2002). Further, certain clays may interfere with the performance of polycarboxylate-based HRWRAs (Jardine et al. 2002; Jardine et al. 2003). The effects of clay are a function of the clay's fineness and activity (Yool, Leeds, and Fried 1998). Smectite (montmorillonite) adsorbs more water than illite or kaolinite (Stewart et al. 2005).

The majority of data in the literature regarding the use of mineral fillers is for ground limestone fillers, which are widely used in some parts of Europe (Zhu and Gibbs 2005) but not in the United States. Ground limestone filler, when used to replace cement at levels up to 50%, can improve the economy of SCC by reducing the amounts of portland cement and HRWRA (Ghezal and Khayat 2002). The particle size distribution and fineness of ground limestone fillers vary widely by source. Not only does the particle size depend on variations in grinding, limestone fillers can be classified to produce a certain size range. Ground limestone fillers typically consist predominately calcium carbonate, with few other minerals present. The use of ground limestone filler as a replacement for cement can reduce water demand or superplasticizer demand. The improved workability is typically attributed to the improved particle size distribution, despite the higher fineness (Tsivilis et al. 1999; Nehdi, Mindess and Aitcin 1998; Zhu and Gibbs 2005). In terms of rheology, limestone filler has been shown to decrease both yield stress and plastic viscosity (Svermova, Sonebi, and Bartos 2003; Ghezal and Khayat 2002). Above a critical dosage, however, the addition of ground limestone filler can increase viscosity substantially (Yahia, Tanimura, and Shimoyama 2005). This critical dosage is related to the amount of free space within the solid skeleton and depends on the characteristics of the ground limestone filler, cementitious materials, and aggregates. Once the free space is filled with ground limestone filler, packing is no longer improved and interparticle friction is increased. Ground limestone filler can also increase static stability and reduce bleeding in SCC mixes (Ghezal and Khayat 2002). Finely ground fillers (Kadri and Duval 2002)—including limestone fillers in particular (Pera et al. 1999; Tsivilis et al. 1999; Zhu and Gibbs 2005)—can accelerate hydration, resulting in increased compressive strength at early ages; however, the fillers must generally be much finer than the portland cement. Limestone filler increases the density of the paste, which is particularly important in improving the strength and transport properties in the interfacial transition zone. In cases where limestone filler reduces compressive strength, especially at relatively high cement replacement rates, the improvement in workability may permit the reduction in water content to offset the decrease in strength (Ghezal and Khayat 2002).

Dust-of-fracture microfines have been used to replace either cementitious materials or aggregate. When used to replace cement, the results have generally been favorable (Ho et al. 2002; Bosiljkov 2003). When used to replace fine aggregate instead of cement, the results are generally not as favorable because of the resulting decrease in water-powder ratio, which is partially offset by an increase in paste volume (Celik and Marar 1996; Malhotra and Carrette 1985; Ahmed and El-Kour 1989). If water is added to offset the decrease in water-powder ratio, the hardened properties may be detrimentally affected.

A potentially severe durability problem when limestone mineral fillers are used in cold, sulfate-rich environments is the possibility for the thaumasite form of sulfate attack (TSA). Thaumasite ($\text{CaSiO}_3 \cdot \text{CaCO}_3 \cdot \text{CaSO}_4 \cdot 15\text{H}_2\text{O}$) typically forms from the reaction of sulfate ions, C-S-H, water, and either carbonate ions or carbon dioxide (Santhanam, Cohen, and Olek 2001). Thaumasite may form in a variety of ways (Bensted 2003). Ground limestone filler is a potential source for carbonate ions. Its small size makes it more reactive than larger limestone aggregates.

Other sources of carbonate ions besides limestone aggregates include dolomite aggregates, seawater, and groundwater (Thomas et al. 2003; Sahu, Badger, and Thaulow 2003). Unlike traditional forms of sulfate attack that lead to expansion and cracking, TSA destroys C-S-H (Crammond 2003). Thaumasite is particularly threatening because it is limited only by the availability of sulfate and carbonate ions and can, in theory, continue until the depletion of all C-S-H (Macphee and Diamond 2003). Thaumasite is structurally similar to ettringite, but with silicate in place of aluminate and carbonate ions in place of sulfate ions. Thaumasite formation is generally associated with low temperatures (below approximately 15°C) and has been shown to form at faster rates at lower temperatures. Thaumasite has, however, been observed in warmer climates such as southern California (Diamond 2003; Sahu, Badger, and Thaulow 2003). The increased use of limestone filler in cements around the world has increased the incidence of thaumasite formation (Irassar et al. 2005); however, thaumasite has also been identified in historic buildings (Collepari 1999) and was identified in the United States as early as the 1960s (Stark 2003). Furthermore, the increased availability of analytical techniques has likely increased identification of thaumasite (Thomas et al. 2003).

2.1.3 Cement

SCC often has higher cementitious materials content than conventionally placed concrete in order to achieve adequate flowability. The potential negative consequences of high cementitious materials content include higher cost, higher heat of hydration, and increased susceptibility to shrinkage. All standard types of portland cements are generally acceptable for SCC (EFNARC 2002). Admixture performance can be strongly dependent on cement characteristics. For instance, Vikan, Justnes, and Winnefeld (2005) evaluated 6 different cements and found that the area under a rheological flow curve for cement paste was correlated to the cement characteristic given in Equation (2.6):

$$\text{cement characteristic} = a(\text{Blaine}[d(\text{cubic } C_3A) + (1 - d)(C_3S)]) + b \quad (2.6)$$

where a , b , and d are empirical factors. Cubic C_3A was included because it is considered more reactive than orthorhombic C_3A . Although C_3S is less reactive than C_3A , it was included because it is sufficiently reactive and is available in large quantity. The cubic C_3A and C_3S are multiplied by the Blaine fineness to reflect the amount of reactive material on the surface of the cement grain.

2.1.4 Supplementary Cementitious Materials

Supplementary cementitious materials—including fly ash, slag, and silica fume—are often used in SCC to decrease cost, improve workability, reduce heat of hydration, and improve durability. The use of SCMs with no C_3S , C_3A , or C_4AF can make rheology easier to control (Aitcin 1998). Further, high-fineness powders decrease the size and volume of voids, which results in reductions in bleeding and segregation (Mehta and Monteiro 1993). Due to the reduction in early strength development in mixtures with fly ash or slag, the strength of such mixtures may need to be evaluated at ages beyond 28 days. In some cases, SCMs are used to reduce strength at certain ages because the amount of powder materials needed for workability

would result in excessive strength if composed of only portland cement (Domone 2006). As by-products, SCMs may exhibit undesirable levels of variability.

2.1.4.1 Fly Ash

Fly ash has been used successfully in SCC (Domone 2006), including applications of high volume fly ash (Patel et al. 2004; Christensen and Ong 2005). The use of fly ash generally improves workability and delays strength development. In terms of rheology, fly ash reduces yield stress but may increase or decrease plastic viscosity. For example, Sonebi (2004) found that the use of fly ash reduced both the yield stress and plastic viscosity of SCC. Park, Noh, and Park (2005), however, found that fly ash slightly reduced yield stress but increased the plastic viscosity of superplasticized pastes. Fly ash can also reduce bleeding and improve stability (Shadle and Somerville 2002). The influence of fly ash depends on whether cement is replaced with fly ash on a mass or volume basis. Compared to Class C fly ash, Class F fly ash reduces early strength development to a greater extent but is better for durability. Class C fly ash also delays time of set more than Class F fly ash. The reduction in early strength development for mixtures with fly ash can be offset with the use of an accelerator (Shadle and Somerville 2002). Fly ash may contain unburned carbon. Park, Noh, and Park (2005) found that HRWRA can adsorb onto unburned carbon, reducing the workability.

Ground fly ashes have been used for SCC. For instance, Xie et al. (2002) found that ultra-pulverized fly ash (UPFA) with Blaine fineness of 500-600 m²/kg had an effect on the workability of SCC similar to that of a viscosity agent, in that it improved flowability without reducing viscosity. The UPFA was found to increase mechanical properties and reduce drying shrinkage. Classified fly ash, which consists of small fly ash particles separated from a parent fly ash, is another possibility for SCC. Unlike ground fly ash, classified fly ash retains a spherical shape. In fact, the particles can be more spherical and can reduce water demand to a greater degree than the parent fly ash. The small size of the classified fly ash increases the spread of the particle size distribution of the powder materials, which can also improve workability. For instance, Obla et al. (2003) reported on a classified fly ash with a mean particle size of 3 µm and 90% of material smaller than 7 µm. This smaller particle size increased the reactivity, leading to increased compressive strength and improved durability. The use of classified fly ash reduced water demand and reduced drying and autogenous shrinkage. Even at an age of 1 day, the compressive strength could be maintained by using the classified fly ash and reducing the water-cement ratio to take advantage of the water reducing characteristics. Ferraris, Obla, and Hill (2001) found that classified fly ash reduced yield stress and plastic viscosity of pastes when used at an optimum cement replacement rate.

The ongoing implementation of various environmental regulations for coal-burning power plants continues to change the properties of fly ashes, resulting in important implications for concrete performance. The changes in fly ash quality depend on changes in federal regulations, the implement of regulations by individual states, existing equipment in plants, the approaches used by power plant operators to comply with new regulations, and the type of coal burned.

Regulations requiring the reduction in emissions of nitrogen oxides (NO_x) from coal-burning power plants have had considerable consequences for the use of fly ash in concrete. NO_x emissions can be reduced by changing combustion systems, applying post-combustion treatments, or both (Golden 2001, US Department of Energy 2001, National Coal Council 2005).

Changes to combustion systems aim to reduce the oxidation mechanisms responsible for NO_x emissions by reducing the combustion temperature or reducing the oxygen level. These changes can be accomplished by replacing older, single-stage burners with newer so-called “low-NO_x burners” or with the use of oven-fire air or reburning technologies. The use of low-NO_x burners typically increases the amount of unburned carbon, creates less spherical fly ash particles, increases the coarseness of particles, and increases the variability of the fly ash properties (Golden 2001).

Post-combustion treatments consist of applying ammonia (NH₃) or urea as a part of a selective catalytic reduction (SCR) or selective non-catalytic reduction (SNCR) processes. The ammonia and available oxygen react with NO and NO₂ to form nitrogen and water vapor. The processes are considered selective because they promote this particular reaction over other possible reactions. If a catalyst is used, it is typically solid (heterogeneous catalyst). The amount of ammonia applied to the flue gas must be optimized to reduce NO_x emissions to a sufficient degree while not leaving excessive amounts of unreacted ammonia on the fly ash, which is referred to as ammonia slip. The amount of ammonia slip depends not just on the amount of excess ammonia applied, but also on the capacity of the fly ash to adsorb ammonia. It is often economically advantageous, however, to reduce NO_x emissions to the greatest degree possible to take advantage of tradable emission credits even if it increases ammonia slip. When the fly ash is wetted during concrete mixing, ammonia gas is released. At low concentration levels, ammonia produces a noxious odor. At high levels, it can be toxic. Ammonia contents should generally be less than 50-100 ppm to avoid objectionable odors. It is generally agreed that the presence of ammonia does not detrimentally affect concrete properties; however, limited test data exist (Bittner, Gasiorowski, and Hrach 2001).

Several companies market technologies to mitigate the effects of lower fly ash quality on concrete properties (Golden 2001; Bittner, Gasiorowski, and Hrach 2001). The combustion of coal can be optimized to reduce unburned carbon. Unburned carbon can be removed with carbon burn-out, particle size control, electrostatic precipitation, and wet separation. Several technologies are available to remove the ammonia from the fly ash, with dry processes preferable to wet processes.

2.1.4.2 Silica Fume

Silica fume has been used successfully in SCC (Domone 2006). Silica fume is generally known to increase cohesiveness and reduce segregation and bleeding (EFNARC 2005). It also increases compressive strength, modulus of elasticity, and flexural strength and enhances durability at all ages. This increase in strength may be particularly useful at early ages when silica fume is compared to other SCMs (Mehta and Monteiro 1993).

Silica fume may improve concrete rheology and enhance stability when used at low dosages—typically less than 4-6% by replacement of cement—but have detrimental effects on rheology at higher dosages. Any reduction in workability is generally due to silica fume’s high fineness, which is offset at least partially by its spherical particle shape and any improvements in the particle size distribution of the combined powders. According to Park, Noh, and Park (2005), the high reactivity of silica fume particles can increase adsorption of HRWRA, which reduces the amount available in solution and on cement particles and, thereby, decreases workability. Detwiler and Mehta (1989) found that spherical carbon black with a similar grain size as silica fume resulted in similar workability.

For pastes designed for SCC, Vikan and Justnes (2003) found that adding silica fume at up to a 10% volume replacement increased yield stress. Plastic viscosity, however, was reduced when a polycarboxylate-based HRWRA was used and increased when a naphthalene-based HRWRA was used. The decrease in plastic viscosity was attributed to the displacement of water between cement grains and the spherical shape of the silica fume particles. For superplasticized pastes, Park, Noh, and Park (2005) found that the use of silica fume at cement replacement rates of 5, 10, and 15% increased yield stress and plastic viscosity significantly. They suggested that silica fume be used to increase plastic viscosity to prevent segregation and that the sharp increase in yield stress be offset by the use of a ternary cementitious system with either fly ash or slag. For conventionally placed concrete, Wallevik (1990, qtd. in Vikan and Justnes 2003) found that adding silica fume to concrete at replacement rates up to 6% significantly reduced plastic viscosity but had little effect on yield stress. Higher dosages of silica fume increased yield stress substantially but increased plastic viscosity more gradually.

2.1.4.3 Slag

Slag has been used successfully in SCC (Ozyildirim 2005; Billberg 2000; PCI 2003; Domone 2006). It is typically used at higher replacement rates than fly ash. It is effective in reducing heat of hydration and cost, but does not improve workability to the same extent as fly ash (Park, Noh, and Park 2005; Billberg 2000). Slag can contribute to compressive strength at ages as early as 7 days, which is faster than Class F fly ash but not fast enough for precast applications where release strengths are critical (Mehta 2001).

2.2 Fresh Properties

SCC is defined primarily in terms of its fresh properties; therefore, the characterization and control of fresh properties are critical to ensuring successful SCC performance. Fresh properties encompass all relevant characteristics of SCC prior to final setting, including flow properties, setting time, bleeding, and plastic shrinkage. Fresh properties influence not just constructability but also hardened properties like strength and durability.

2.2.1 Workability

2.2.1.1 Requirements

The workability requirements for SCC are typically defined in terms of three properties: passing ability, filling ability, and segregation resistance (EFNARC 2002). Filling ability describes the ability of concrete to flow under its own mass and completely fill formwork. Passing ability describes the ability of concrete to flow through confined conditions, such as the narrow openings between reinforcement bars. Segregation resistance describes the ability of concrete to remain uniform in terms of composition during placement and until setting. Various test methods are available to measure these properties; however, no test method exists to measure all of these properties at once. Given that these three properties are interrelated, most tests indirectly measure more than one property at a time.

Segregation resistance has been defined in terms of static and dynamic stability (Dazcko 2002; Assaad, Khayat, and Mesbah 2003; Khayat, Assaad and Daczko 2004). Dynamic stability describes the concrete performance during the casting process. It is related to energy input—which may be from pumping, drop heights, agitation, or vibration—and passing ability—which is affected by member dimensions and reinforcement bar spacing. Static stability describes the concrete performance immediately after energy input from casting until setting. It is related to paste rheology, aggregate shape and grading, and the density of the aggregates relative to the paste (Saak 2000; Saak, Jennings, and Shah 2001; de Larrard 1999a).

Other aspects of the workability of SCC are typically improved relative to conventionally placed concrete. In general, the pumpability and finishability of SCC are improved relative to conventionally placed concrete (Bury and Christensen 2002). The formed surface finish is better due to the reduction in honeycombing and the number of bugholes (Martin 2002).

The retention of workability properties over time must be considered. Workability retention is not necessarily associated with setting time. For example, retarding admixtures can increase setting time while accelerating workability loss (Tattersall 1991).

Rheology, or the scientific study of the flow and deformation of matter, can be used to characterize workability. The goal of using rheology is to provide a consistent, repeatable, and scientific description of concrete flow properties. Fundamental rheological parameters are inherent to a material and, in theory, should be independent of the test device used. These rheological parameters can be used to compare the workability of different mixtures, to proportion new concrete mixtures, and to simulate concrete flow in computer models. Rheological parameters may not capture all relevant aspects of workability. For instance, passing ability depends primarily on aggregate characteristics and paste volume—two concrete mixtures with different aggregate sizes could be proportioned for the same rheological properties but would exhibit different degrees of passing ability.

The characterization of concrete rheology is based on the concept that concrete can be considered a fluid. Freshly-mixed concrete is essentially a concentrated suspension of aggregate particles in cement paste. The cement paste is a concentrated suspension of cement grains in water (Ferraris 1999). In contrast to an elastic solid—which undergoes a finite, recoverable deformation upon the application of load—a fluid deforms continuously under a constant shear stress and experiences no recovery of this deformation upon removal of the load. Therefore, in characterizing the fundamental flow properties of a material, the relationship between shear stress, τ , and shear rate, $\dot{\gamma}$, is considered. This relationship is represented graphically with a flow curve. The behavior of a fluid material may be idealized with a constitutive relationship. Six such relationships equations are shown in Figure 2.1 (Hackley and Ferraris 2001).

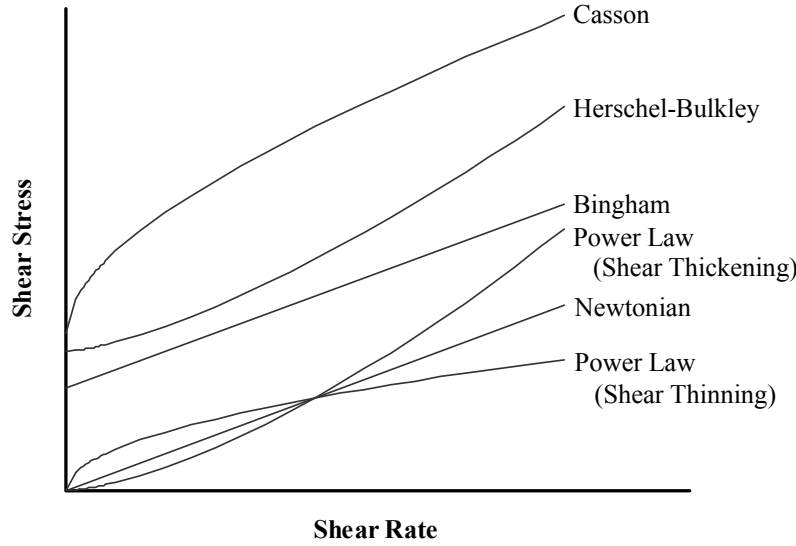


Figure 2.1 *Constitutive Relationships for Fresh Concrete Plotted on a Flow Curve*

The Bingham model is the most widely used constitutive relationship for concrete due to its simplicity and its ability to represent concrete flow accurately for the majority of cases (Ferraris 1999). The Bingham model requires the determination of only two parameters—yield stress, τ_0 , and plastic viscosity, μ —as shown in Equation (2.7):

$$\tau = \tau_0 + \mu\dot{\gamma} \quad (2.7)$$

In practical terms, yield stress represents the amount of stress to initiate or maintain flow while plastic viscosity describes the resistance to flow once the yield stress has been exceeded. Increased plastic viscosity results in greater resistance to flow. The apparent viscosity is equal to the shear stress divided by the shear rate at any given shear rate. Thus, for a Bingham material, the apparent viscosity decreases with increasing shear rate. Fluidity is defined as the inverse of viscosity.

For some concrete mixtures, the linear relationship between shear stress and shear rate is an oversimplification. The Herschel-Bulkley model incorporates two empirical constants, a and b , to represent non-linearity, as shown in Equation (2.8):

$$\tau = \tau_0 + a\dot{\gamma}^b \quad (2.8)$$

If the yield stress is set to zero, the Herschel-Bulkley model describes a shear thinning or shear thickening power law fluid. Because some SCC mixtures can exhibit shear-thinning behavior (Khayat 2000), the variation in viscosity over a range of shear rates may need to be determined.

In terms of the Bingham parameters, SCC must exhibit a proper combination of yield stress and plastic viscosity in order to flow under its own mass and resist segregation. The yield stress is typically near zero to ensure that SCC will flow readily under its own mass; however, segregation can occur if the yield stress is too low. Plastic viscosity must be sufficiently high to prevent segregation, while not being too high that it restricts the speed of flow excessively.

Another important rheological property is thixotropy, which is defined as the reversible, time-dependent reduction in viscosity that occurs when a material is subjected to constant shear (Hackley and Ferraris 2001). Although thixotropy can be beneficial to SCC, its presence can complicate rheological measurements. Thixotropy has specific implications for lateral formwork pressures (Assaad, Khayat, and Mesbah 2003b), stability (Khayat 1999), and pumpability (Barnes 1997).

Thixotropy is usually associated with flocculated suspensions, which typically exhibit a yield stress (Barnes, Hutton, and Walters 1989). When a thixotropic material is at rest, a three-dimensional network structure develops over time due to factors such as bonding and colloidal forces. The application of shear causes a breakdown of this network structure and a reorientation or deformation of particles or flocs, resulting in a reduction in viscosity at a constant shear rate or shear stress. After shear is applied for sufficient time, the material reaches an equilibrium condition where the viscosity is at a minimum for the given shear rate or shear stress. When the application of shear is stopped, the three-dimensional network structure reforms and the original viscosity is eventually restored. This restoration is driven by Brownian motion, which in causing particles to move randomly also causes particles to move close enough to each other such that colloidal forces result in aggregation. Brownian motion applies mainly to particles with sizes less than 1 μm . Colloidal forces may also act on particles larger than 1 μm , resulting in aggregation even without Brownian motion. The at-rest fluid with maximum viscosity is sometimes referred to as a gel, whereas the flowing fluid with minimum viscosity is referred to as a sol. According to Barnes (1997), this concept of a gel-sol transition is more likely attributed to the presence of a yield stress or extreme shear thinning behavior, whereas the reduction in viscosity with time due to shearing is more accurately associated with thixotropy.

The transition between high and low viscosity is illustrated in Figure 2.2 for a stepwise, shear rate-controlled experiment. As the shear rate is increased instantaneously from rest to a constant value, the resulting shear stress in the fluid reaches its maximum value for the given shear rate. Over time, the shear stress decreases due to the thixotropic breakdown and eventually approaches a constant, equilibrium value. Then, when the shear rate is reduced to a lower value, the shear stress immediately decreases but then gradually increases to a new steady-state equilibrium value as the three-dimensional network structure is partially rebuilt. An equilibrium shear stress is associated with each shear rate.

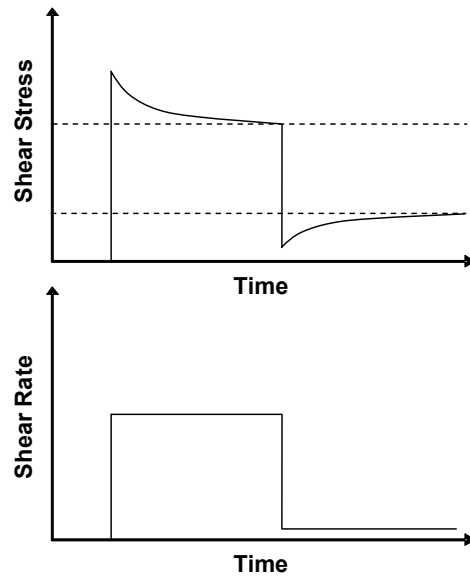


Figure 2.2 Effects of Thixotropy in Rate Controlled Time-Step Experiment

Thixotropy can manifest itself in flow curve measurements, as depicted in Figure 2.3. When the shear rate is initially increased from zero to the maximum value, the presence of thixotropy results in the measurement of shear stresses above their respective equilibrium values. Then, when the shear rate is decreased from a maximum value back to zero, the thixotropic breakdown that occurred during the up-curve measurement causes the down-curve to be below the up-curve. Although they do not explicitly mention thixotropy, Geiker et al. (2002) indicate that this flow curve hysteresis must be minimized when measuring flow curves for SCC by selecting an appropriate amount of time for each point. Doing so will avoid the effects of thixotropy while also minimizing effects due to setting and loss of workability.

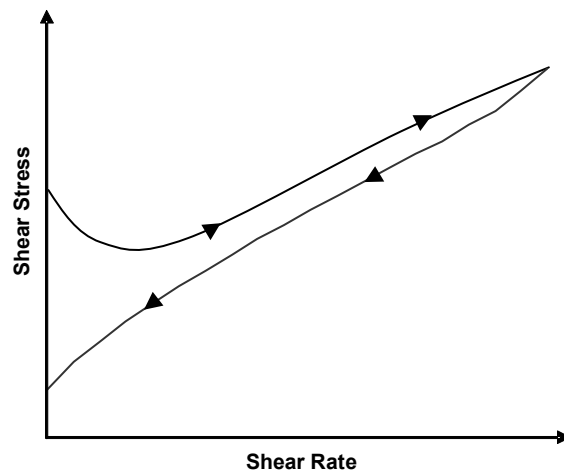


Figure 2.3 Manifestation of Thixotropy in a Flow Curve Measurement

The time for breakdown and recovery to occur depends on the fluid. In general, the time required for rebuilding is much greater than the time for breakdown. Although rebuilding may take considerable time, much of the viscosity may be recovered quickly in the first few seconds or minutes (Schramm 1994). The speed of this initial recovery may be of greater consequence than the time to reach full recovery. Due to the reduced influence of Brownian motion, suspensions with larger particles typically exhibit faster breakdown times and slower rebuilding times than suspension with smaller particles (Barnes, Hutton, and Watlers 1989).

Other fluid properties can result in similar behavior as thixotropy; however, these properties are unique and should be considered separately. First, materials can be both viscoelastic and thixotropic, as illustrated in Figure 2.4. Viscoelasticity causes a delay from the initial application of stress to the resulting final deformation (Barnes 1997). Thixotropy, however, is due to changes in the material structure while viscoelasticity is not. Second, some materials undergo an irreversible loss of viscosity, known as rheomalaxis or rheodestruction, due to such factors as sedimentation (Whorlow 1992). Third, thixotropy should not be confused with shear-thinning behavior, which describes the decrease in viscosity as a function of increasing shear rate, not shearing over time. Thixotropy typically occurs in shear-thinning fluids whereas anti-thixotropy, or the reversible, time dependent increase in viscosity during constant shearing, typically occurs in shear-thickening fluids (Barnes, Hutton, and Walters 1989).

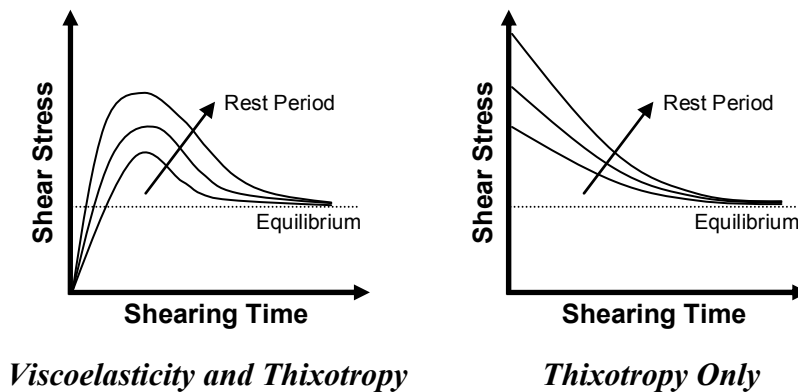


Figure 2.4 Illustration of Distinction between Linear Viscoelasticity and Thixotropy (after Barnes 1997)

No single, standard method is available for measuring thixotropy. Of the methods that are available, each has certain disadvantages and limitations. In fact, Barnes (1997) states that thixotropic behavior, including its experimental characterization and theoretical description, is “one of the greatest challenges facing rheologists today.” Ideally, test methods should isolate thixotropy from effects due to setting or loss of workability and should differentiate between thixotropy and rheomalaxis and between thixotropy and viscoelasticity. In practice, few methods are able to achieve these goals.

Several approaches to measuring thixotropy have been applied to a wide range of different fluids. One such approach is to perform a loop test in a rheometer. In this test, the shear rate is increased from zero to a maximum value and returned to zero. This cycle can be repeated until the down flow curve measurement remains constant. The area between the maximum up and minimum down curve is calculated as an indication of thixotropy. The

imposed shear rates or shear stresses can be changed in a continuous or stepwise manner. To avoid repeated flow curve measurements, Schramm (1994) suggests measuring the up curve, then maintaining the maximum shear rate until full thixotropy breakdown is achieved, and then measuring the down curve. Instead of measuring the area between the up and down curves, Whorlow (1992) suggests monitoring changes in the down curve after different shear histories, provided the down curve can be measured as quickly as possible. The loop test approach to thixotropy characterization suffers several limitations. First, the area between the up- and down-curves depends in part on the measurement time of each shear rate or shear stress point on the flow curve (Whorlow 1992). Further, the initial up curve can be influenced by the initial elastic response of the fluid (Barnes 1997).

A second approach to measuring thixotropy is to perform a step-wise test, similar to that shown in Figure 2.2, where the shear rate or shear stress imposed by a rheometer is changed from one constant value to another and the break-down or build-up in shear stress or shear rate is monitored. The percentage of build-up or break-down and the time for equilibrium to be achieved can be determined (Whorlow 1992). Barnes (1997) calls this approach “simpler and more sensible” than the loop test approach. Still, it is not possible to eliminate the effects of an initial elastic response (Barnes 1997). Whorlow (1992) points out that it is important to check that the material at one shear rate is representative of the behavior at other shear rates.

A third approach is to use a start-up (or stress-growth) test where a constant strain or stress is applied to a material initially at rest (Barnes 1997). The thixotropy is indicated by the overshoot in stress for a strain-controlled test or by the increase in slope in the strain-time curve for stress-controlled tests.

In using any of these three approaches, it is important that the shear history prior to testing is well-known (Schramm 1994, Barnes 1997). Sources of this pre-testing shear could be from mixing, pumping, or filling the rheometer. The variability from shear history can be minimized by using a fixed rest period, by pre-shearing the sample for a certain time followed by a rest period, or pre-shearing the sample at a low speed followed by testing at a higher speed (Barnes 1997).

Researchers have attempted to apply these and several other approaches to concrete. In making measurements of concrete, it is often convenient to test only the mortar or paste fractions because the causes of thixotropy are primarily associated with the paste. In measuring concrete, it is important to distinguish between thixotropy and rheomalaxis, to minimize the effects of setting and workability loss, and to isolate certain viscoelastic effects. The determination of thixotropy in concrete is complicated because the definition of thixotropy is not clear for concrete. For example, non-equidimensional aggregates can reorient under shear, resulting in a reduction in viscosity. Brownian forces do not affect aggregates, so a rest period alone cannot restore the loss in viscosity caused by the reorientation of non-preferentially aligned aggregates. If the concrete is sheared in a direction other than that used in the original test, the aggregates are not in preferential alignment for the new direction of shear and, therefore, can contribute to an increase in viscosity as measured in the new direction. In addition, changes to the aggregates may not affect thixotropy directly; however, such changes may affect the required paste rheology needed to achieve proper concrete workability. These required changes in paste rheology can affect concrete thixotropy.

Assaad and Khayat (2003) and Assaad and Khayat (2004) used the loop test approach for measuring the thixotropy of mortar and concrete, but with some modifications. Structural breakdown curves were measured at various speeds. The initial and equilibrium shear stresses

were taken from the structural breakdown curves and used to compute two separate flow curves. The area between these two flow curves was determined as an indication of thixotropy. Ghezal and Khayat (2003) used a parallel plate rheometer with mortar specimens to make stepwise measurements at alternating shear rates. The difference between the initial shear stress and equilibrium shear stress was taken as an indication of thixotropy. Similarly, Toussaint et al. (2001) measured thixotropy in mortars by imposing a specific shear rate regime in a rheometer. The mortar was pre-sheared at a high shear rate, allowed to rest for variable periods of between 1 and 15 minutes, and then sheared at 0.1 rpm while the gradual build-up in torque was monitored. Billberg and Osterberg (2001) considered four techniques to measure thixotropy. The first technique was referred to as the thixomethod. In this approach a specially designed apparatus was used to monitor how the amount of torque to rotate a vane from rest in an undisturbed concrete sample varied over time. Between each measurement, the vane was lowered to an undisturbed portion of the sample and allowed to remain undisturbed for 30 minutes. In the second technique, the BML viscometer was used to perform stress growth tests after various periods of rest. The third method made use of the RAP-ACT plasticity meter, which consisted of a three-bladed impeller with a tapered bottom. The torque to rotate the impeller, as indicated on a spring-loaded gage, was determined after a period of rest. The fourth technique involved measuring the slump flow from four cones filled at the same time but removed at 30 minute intervals. Wallevik (2003) conducted oscillatory measurements of cement paste and used various equations to model the response.

2.3 Test Methods

The available empirical workability test methods for SCC are categorized in Table 2.3 based on the property measured (filling ability, passing ability, or segregation resistance) and the type of stability considered (static or dynamic). These test methods are described in alphabetical order in the following subsections.

Table 2.3 Empirical Test Methods for Flow Properties

Test Method	Properties Measured (EFNARC 2002)	Stability Type (Dazcko 2002)
Column Segregation Test	Segregation resistance	Static
Concrete Acceptance Test	Filling ability and passing ability	Dynamic
Electrical Conductivity Test	Segregation resistance	Static
Filling Vessel Test	Filling ability and passing ability	Dynamic
J-Ring	Passing ability	Dynamic
L-Box and U-Box	Filling ability and passing ability	Dynamic
Penetration Tests	Segregation resistance	Static
Segregation Test (Hardened Concrete)	Segregation resistance	Static
Settlement Column Segregation Test	Segregation resistance	Dynamic
Slump Flow (with T ₅₀ and VSI)	Filling ability and segregation resistance	Static/Dynamic
Surface Settlement Test	Segregation resistance	Dynamic
V-Funnel	Filling ability	Dynamic
Sieve Stability Test	Segregation resistance	Static

In addition to the distinctions made in Table 2.3 between static and dynamic tests, it is also possible to indirectly measure static stability with certain dynamic stability tests. Concrete can be placed inside a dynamic stability test apparatus, such as the v-funnel or l-box, and allowed to rest for a specified period of time. The results for tests with and without the rest period are compared to determine if segregation occurred during the rest period. In the v-funnel, for instance, the collection of coarse aggregate at the outlet of the funnel would result in an increased flow time or possibly a complete blockage. It must be cautioned that such delayed tests can also be influenced by thixotropy and loss of workability.

In evaluating empirical test methods, it must be remembered that empirical tests provide only an index of workability that may or may not be related to fundamental flow parameters. For instance, in testing conducted by Ferraris et al. (2000), the results of the v-funnel and u-box tests were not correlated to yield stress or plastic viscosity measurements as determined with the IBB rheometer and the BTRHEOM rheometer. Nielsson and Wallevik (2003) did find correlations between plastic viscosity and T₅₀, orimet flow time, v-funnel flow time, and l-box flow time and between yield stress and slump flow; however, the scatter was described as “significant”. Utsi, Emborg, and Carsward (2003) found that as long as only one rheological parameter was varied at a time, the rheological parameters measured with the BML viscometer were correlated to the results of the v-funnel and slump flow test; however, the scatter was high. Khayat, Assaad, and Daczko (2004) found that v-funnel results were a function of both yield stress and plastic viscosity.

Many of the available empirical tests measure similar properties and, therefore, are correlated to each other to some degree. For instance, Khayat, Assaad, and Daczko (2004) found correlations between the results of the u-box, l-box, v-funnel, and j-ring tests.

Due to the limited standardization of SCC test methods, the dimensions and details of the empirical test methods can vary within the literature. Daczko (2003) lists dimensions of l-boxes, u-boxes, and j-rings reported by various researchers in the literature. Petersson, Gibbs, and Bartos (2003) found that variations in the amount wall friction, which is affected by test geometry and the smoothness of wall material, can have a significant influence on test results.

2.3.2 Column Segregation Test

The column segregation test (Daczko 2002; Assaad, Khayat, and Daczko 2004; ASTM C 1610), which is shown in Figure 2.5, consists of an 8-inch diameter, 26-inch tall PVC pipe split into four 6.5-inch tall sections. Each section is clamped together to form a water-tight seal. Concrete is placed into the pipe and left undisturbed for 15 minutes. Each section of the pipe is then removed and the concrete inside is collected. Each concrete sample is washed over a 5-mm (#4) sieve to retain all coarse aggregates, which are then dried. The coefficient of variation in coarse aggregate masses present in each of the four pipe sections is calculated as an indication of segregation resistance. Alternatively, the variation between just the top and bottom pipe sections can be determined. Similar tests have been presented by Rols, Ambroise, and Pera (1999); Lowke, Wiegrink, and Schiessl (2003); and El-Chabib and Nehdi (2006). Assaad, Khayat, and Daczko (2004) found that the column segregation test and the surface settlement test were affected by different factors and should be used as complementary tests.

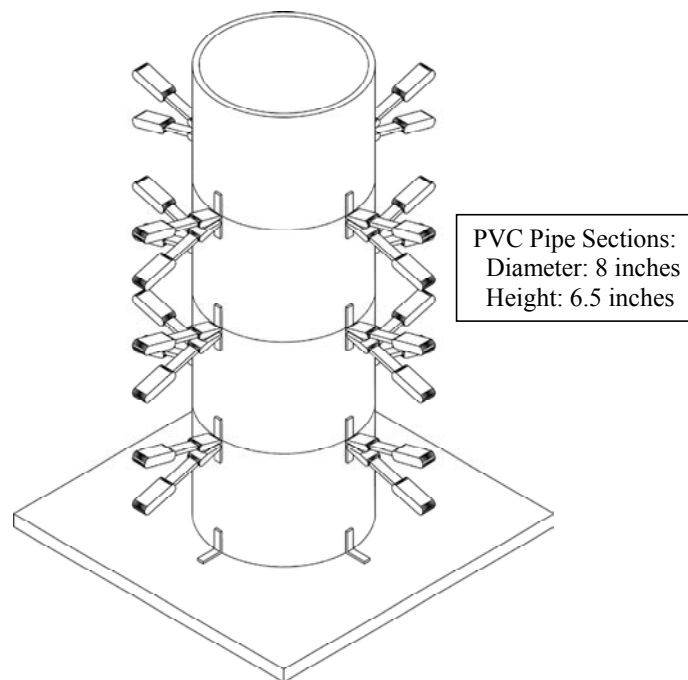


Figure 2.5 Column Segregation Test

2.3.3 Concrete Acceptance Test

The concrete acceptance test (Okamura and Ouchi 1999) is intended for use on a jobsite to verify that all concrete to be used exhibits suitable flow properties. The test consists of a 1200

mm wide, 1200 mm long, and 300 mm tall box that is positioned between the chute of a mixing truck and the hopper of a pump. Three sides of the box are enclosed while the fourth side features a series of staggered reinforcing bars. Concrete is discharged on the side opposite of the reinforcement bars. The concrete is assessed based on whether it flows horizontally and passes through the reinforcing bars. The original device has been modified by Kubo et al. (2001) to add more obstacles for the concrete to pass and by Wantabe et al. (2003) to increase capacity.

2.3.4 Electrical Conductivity Test

The electrical conductivity test (Khayat et al. 2003; Assad, Khayat, and Daczko 2004) measures the bleeding and segregation resistance of mortar by monitoring changes in ionic conductivity throughout a column specimen. The apparatus consists of a vertical probe with 5 stainless steel electrodes spaced 60 mm apart. The probe is immersed into a 100-mm diameter, 350-mm tall cylindrical column of mortar and changes in conductivity between each of the 4 pairs of electrodes are measured for 150 minutes. Changes in conductivity reflect changes in the mortar composition due to segregation and bleeding. Stability is determined quantitatively with two segregation indexes, two bleeding indexes, and two homogeneity indexes.

2.3.5 Filling Vessel Test (Fill Box Test, Simulated Filling Test, Filling Capacity Box, Kajima Test)

The filling vessel test (EFNARC 2002; Bartos, Sonebi, and Tamimi 2002) measures the filling ability, passing ability, and segregation resistance of SCC. The apparatus consists of a clear plastic box with 35 plastic or copper 20-mm diameter bars, as shown in Figure 2.6. An early version of the test featured a wedge shaped box instead of a rectangular box and did not include a funnel. Concrete is poured at a constant rate into the funnel and allowed to flow into the box until the height of the concrete reaches the height of the top row of bars.

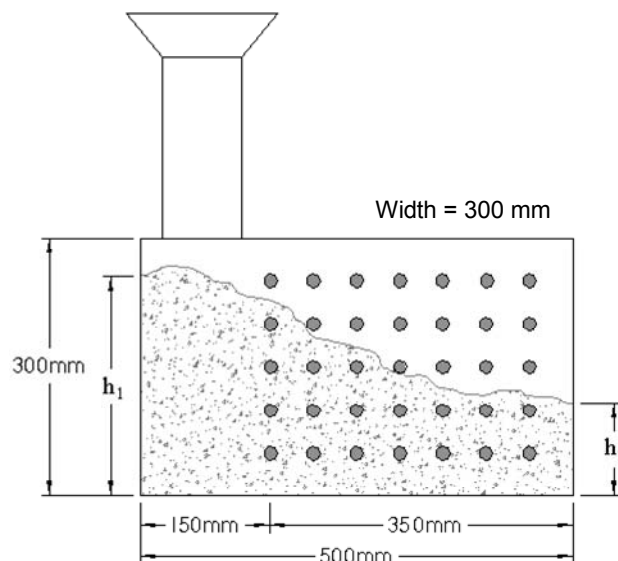


Figure 2.6 Filling Vessel

The height of the concrete at the side nearest the funnel, h_1 , and the height at the opposite side, h_2 , are used to calculate the average filling percentage as shown in Equation (2.9):

$$\text{Filling Percentage} = \frac{(h_1 + h_2)}{2h_1} \times 100\% \quad (2.9)$$

The closer the filling percentage is to 100%, the greater are the filling and passing abilities of the concrete. If a mixture exhibits a high slump flow but low filling percentage, this behavior could indicate that the mixture has high plastic viscosity, poor passing ability, or poor resistance to segregation. The test is a good representation of actual placement conditions; however, it is bulky and difficult to perform on site.

A similar simulated soffit test (Bartos, Sonebi, and Tamimi 2002) consists of a rectangular box with reinforcing bars placed in the box in an arrangement that simulates actual placement conditions for a given job. The reinforcing bars can be both horizontal and vertical. Concrete is placed in the box in a similar manner as with the filling vessel test. After the concrete is allowed to harden, saw-cut sections of hardened concrete are removed to judge how well the concrete filled the box and passed around reinforcing bars.

2.3.6 J-Ring Test

The j-ring test (EFNARC 2002; Bartos, Sonebi, and Tamimi 2002, ASTM C 1621) extends common filling ability test methods in order to characterize passing ability. The j-ring test device can be used with the slump flow test, orimet test, or v-funnel test. The j-ring, as shown in Figure 2.7, is a rectangular section (30 mm by 25 mm) open steel ring with a 300-mm diameter. Vertical holes drilled in the ring allow smooth or deformed reinforcing bars to be attached to the ring. Each bar is 100 mm long. The spacing of the bars can be adjustable.

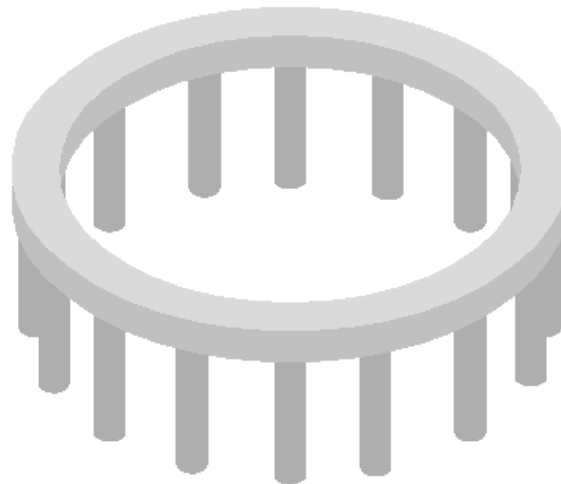


Figure 2.7 J-Ring

To conduct the j-ring test in conjunction with the slump flow test, the slump cone is placed in the center of the j-ring and filled with concrete. The slump cone is lifted and concrete is allowed to spread horizontally through the gaps between the bars. Alternatively, the orimet device or the v-funnel can be positioned above center of the j-ring. Instead of measuring just the time for concrete to exit the orimet or the v-funnel, the concrete is also allowed to spread horizontally through the j-ring.

Various interpretations of the test results have been suggested. The measurements of passing ability and filling ability are not independent. To characterize filling ability and passing ability, the horizontal spread of the concrete sample is measured after the concrete passes between the bars of the j-ring and comes to rest. The horizontal spread with the j-ring can be compared to that without the j-ring. Also, the difference in height of the concrete just inside the bars and just outside the bars is measured at four locations. In addition, Daczko (2003) has suggested assigning a visual blocking index (VBI) rating, in accordance with Table 2.4, based on the appearance of the concrete after the test. Daczko (2003) found that the j-ring was able to distinguish the ability of concrete to flow through obstacles better than the l-box or u-box and suggested using just the j-ring slump flow value for quality control purposes instead of the both the j-ring slump flow and the unrestricted slump flow.

Table 2.4 Visual Block Index Ratings (Daczko 2003)

VBI	Description
0	No evidence of blocking resulting in a pile of coarse aggregate in the middle of the patty and no evidence of bleed streaking behind the rebar obstacles.
1	A slight pile of coarse aggregate in the middle of the patty and slight evidence of bleed streaking behind the rebar obstacles.
2	A clear pile of coarse aggregate in the middle of the patty and significant bleed streaking.
3	Significant blocking of aggregate behind the rebar obstacles, will usually result in a significant decrease in flow value.

2.3.7 L-Box and U-Box Tests

The l-box and u-box tests (Kuriowa 1993; EFNARC 2002; Bartos, Sonebi, and Tamimi 2002), which are shown in Figure 2.8, measure the filling and passing ability of SCC. In the case of the l-box, concrete is initially placed in the vertical portion of the box. The gate is opened and concrete is allowed to flow through a row or reinforcement bars and into the horizontal portion of the box. The times for concrete to reach points 200 mm (T_{20}) and 400 mm (T_{40}) down the horizontal portion of the box are recorded. After the concrete comes to rest in the apparatus, the heights of the concrete at the end of the horizontal portion, H_2 , and in the vertical section, H_1 , are measured to compute the blocking ratio, H_2/H_1 . Segregation resistance can be evaluated visually immediately after the test or the concrete can be allowed to harden and samples can be cut for further evaluation (Tanaka et al. 1993).

For the u-box, concrete is filled into one side of the box, the gate is opened, and concrete is allowed to flow through a row of reinforcement bars and into the other half of the box.

Measurements are made of the time for concrete to cease flowing and of the heights on either side of the box.

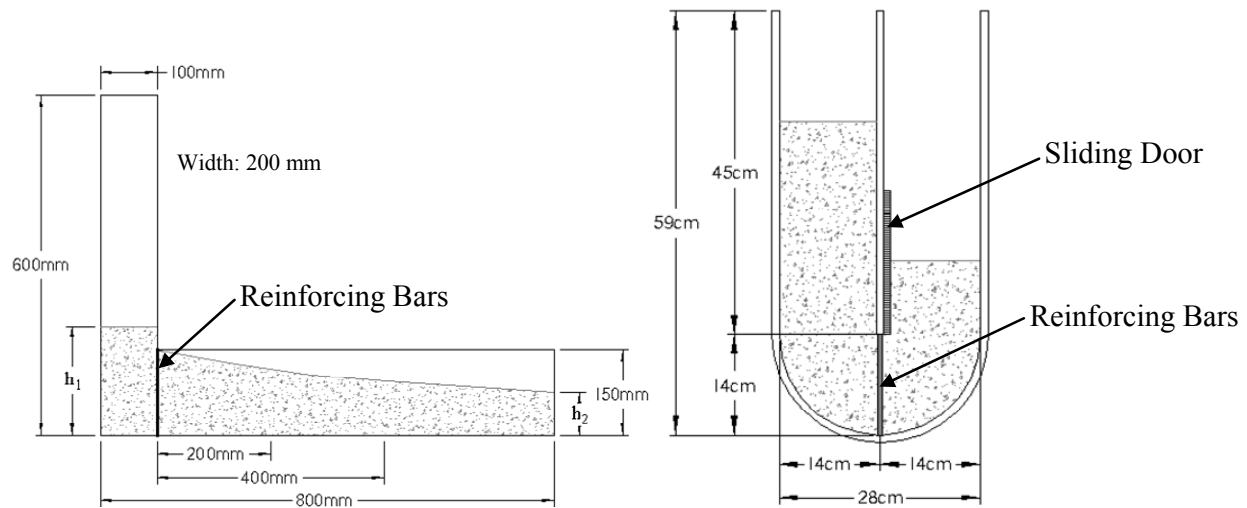


Figure 2.8 L-Box (Left) and U-Box Test Apparatus

Khayat, Assaad, and Daczko (2004) found correlations between the results of the u-box and l-box tests; however, there was much scatter. The l-box was found to be preferable because it gives more information about filling ability. Further, the combination of l-box and slump flow tests was found to be preferable to a combination of j-ring and slump flow tests.

2.3.8 Penetration Tests for Segregation Resistance

Two similar test methods, which were developed independently, measure the penetration resistance of concrete as a means of determining segregation resistance. The penetration resistance should be related to the yield stress of the concrete. The tests further make use of the fact that the settlement of coarse aggregate in SCC results in a mortar-rich region at the top of a SCC specimen, which may reduce the resistance to penetration. The test methods involve placing concrete in a column, allowing the concrete to remain at rest for a specified period of time, and measuring the condition of the material at the top of the column.

In the penetration test (Bui, Akkaya, and Shah 2002; Bui et al. 2002), SCC is placed in a container of sufficient size such that edge effects can be neglected. The top surface of the SCC is leveled and a penetration head, which is depicted in Figure 2.9, is positioned just above the surface of the concrete. In one implementation of the test, the concrete is allowed to remain undisturbed for 2 minutes before the penetration head is released into the concrete. The penetration depth after 45 seconds is recorded. A total of three such measurements are averaged. For a 54-gram penetration head, a penetration depth less than 8 mm was found to indicate acceptable resistance to segregation.

The segregation probe (Shen, Struble, and Lange 2005) consists of a 1/16-inch steel wire wrapped in a 5-inch diameter ring with a 6-inch vertical portion. Concrete is placed in a 6-inch

by 12-inch cylinder and left undisturbed for two minutes. The probe, with a mass of 18 grams, is placed atop the concrete is allowed to settle under its own mass for one minute.

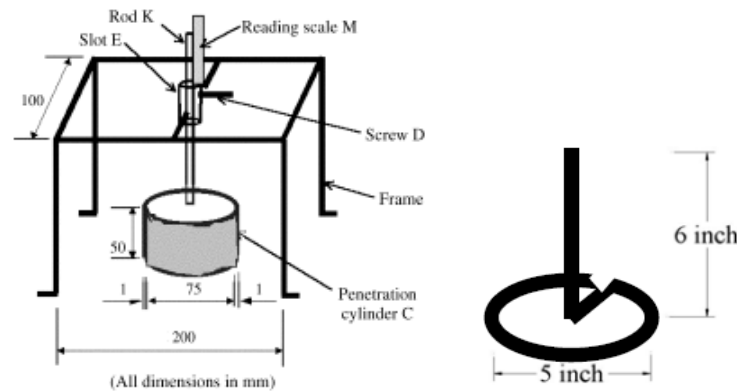


Figure 2.9 Penetration Apparatus (Left) and Segregation Probe (Bui, Akkaya, and Shah 2002; Shen, Struble, and Lange 2005)

2.3.9 Rheometers

Multiple concrete rheometers—with various designs, advantages, and limitations—are available for measuring concrete. Most concrete rheometers are designed to measure a broader range of concrete than just SCC; however, they are particularly well suited for measurements of SCC. Highly fluid concrete mixtures, such as SCC, behave more like homogenous fluids than stiffer, less fluid concrete mixtures and, therefore, can be measured with greater accuracy and repeatability. Unlike concrete, mortar and paste do not require specially designed rheometers.

Concrete rheometers generally function by applying a specified pre-shear regime to achieve thixotropic breakdown and then sweep the shear rate from high to low, during which time the relationship between torque and rotation speed is measured. In traditional rheological measurements, the shear rate throughout the rheometer is known analytically. In concrete measurements, however, the yield stress and the presence of large aggregates make the determination of the distribution of shear stress and shear rate throughout the rheometer significantly more complicated (Mork 1996). The available concrete rheometers take various approaches to converting torque versus rotation speed data to yield stress and plastic viscosity. In general, the approaches can be split between those that provide relative units and fundamental units. To compute relative units, a straight line is fit to the torque (T) versus rotation speed (N) data. In the Tattersall two-point device, the intercept of this line is termed the “ g -value” and the slope is referred to as the “ h -value.” It is assumed that the g -value is related to yield stress and the h -value to plastic viscosity. Other concrete rheometers have used this same naming convention, which is shown in Equation (2.10). This naming convention does not appear to be used in rheological measurements for anything other than cement-based materials. The g -value should not be confused with the shear modulus, which is denoted with a capitalized G .

$$T = g + hN \quad (2.10)$$

The calculation of results in fundamental units, based on the Bingham model (yield stress and plastic viscosity) or Herschel-Bulkley model (yield stress, a , and b), requires calibration or certain assumptions about distributions of shear stress and shear rate throughout the rheometer (Koehler 2004).

Several available concrete rheometers are pictured in Figure 2.10. The BML viscometer (Gjorv 1998; Ferraris and Brower 2001; Bartos, Sonebi, and Tamimi 2002; Ferraris and Brower 2004) and the ICAR rheometer (Koehler 2004) feature coaxial cylinders designs. The BML viscometer, which is intended for use in the laboratory, includes a rotating outer cylinder and fixed inner cylinder. The inner cylinder consists of vertical blades to prevent slippage. The ICAR rheometer is a portable rheometer intended for use in the field. It features a 5-inch diameter vane that is rotated in a container of concrete. The size of the container depends on the aggregate size.

The BTRHEOM rheometer (de Larrard et al. 1997; de Larrard 1999b; Ferraris and Brower 2001; Bartos, Sonebi, and Tamimi 2002; Ferraris and Brower 2004) is a rate-controlled parallel plate rheometer. A simplified version of the BTRHEOM rheometer was developed to eliminate several drawbacks of the original device (Szecsy 1997).

The Tattersall two-point device (Tattersall and Bloomer 1979; Cabrera and Hopkins 1984; Tattersall 1990; Tattersall 1991; Ferraris and Brower 2001; Bartos, Sonebi, and Tamimi 2002; Ferraris and Brower 2004) and the IBB rheometer (Beaupre, Mindess, and Pigeon 1994; Ferraris and Brower 2001; Bartos, Sonebi, and Tamimi 2002; Ferraris and Brower 2004) are impeller-type rheometers. The Tattersall device was one of the earliest attempts to measure the rheology of concrete based on the Bingham model and one of the first devices to use an impeller geometry. It features either a helical or H-shaped impeller and can be calibrated to compute results in fundamental units. The IBB rheometer is essentially an updated version of the Tattersall device. It features an H-shaped impeller and computes results in terms of g and h .

Other available rheometers include the Bertta apparatus (Leivo 1990; Ferraris 1999), the FHPCM (Yen et al. 1999; Tang et al. 2001), the CEMAGREF-IMG (Coussot and Piau 1995), and the falling-ball rheometer (Buchenau and Hillemeier 2003).



Figure 2.10 Concrete Rheometers (Clockwise from Top Left): BML, BTRHEOM, Tattersall, and IBB

2.3.10 Segregation Test (Hardened Concrete)

Multiple researchers have cast concrete in forms of various dimensions, allowed the concrete to harden, and then cut the concrete into sections to assess the distribution of coarse aggregates. For instance, the surface settlement test specimen can be used after it has hardened. Daczko (2002) used a rectangular column measuring 6 by 11 by 33.5 inches. Cussigh, Sonebi, and De Schutter (2003) suggest using an approach developed by Sedran where the depth to the first two coarse aggregates in a 160- by 320-mm cylinder is measured, with depths greater than 10 mm considered indicative of segregation susceptibility. Shen, Struble, and Lange (2005) cut a 6- by 12-inch cylinder in half and evaluated the distribution of coarse aggregate either with image analysis software or by assigning a visual rating on a scale of 0 to 3, with 0 indicating stability and 3 indicating severe segregation.

2.3.11 Settlement Column Segregation Test

The settlement column segregation test (Bartos, Sonebi, and Tamimi 2002) is similar to the column segregation test with the main exception that concrete in the settlement column segregation test is subjected to jolting on a drop table. The test apparatus consists of a tall, rectangular box mounted on top of a standard mortar drop table. The column, depicted in Figure

2.11, is 500 mm tall and has cross sectional dimensions of 100 by 150 mm. Three doors on opposing sides of the column allow sections of concrete to be removed at the conclusion of the test. To begin the test, concrete is placed in the column and left undisturbed for one minute. The concrete is subsequently jolted 20 times in one minute using the drop table and then left undisturbed for an additional 5 minutes. The samples from the top and the bottom of the column are individually washed through a 5-mm sieve to leave only the coarse aggregate. The segregation ratio is calculated as the ratio of the mass of coarse aggregate in the top sample to the mass of coarse aggregate in the bottom sample.

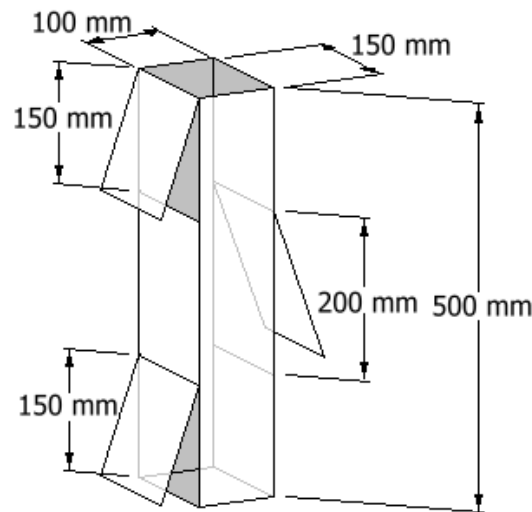


Figure 2.11 Settlement Column Segregation Test

2.3.12 Slump Flow Test (with T_{50} and Visual Stability Index)

The simplest and most widely used test method for SCC is the slump flow test (Kuroiwa et al. 1993; EFNARC 2002; Bartos, Sonebi, and Tamimi 2002; ASTM C 1611), which is pictured in Figure 2.12. To perform the test, a conventional slump cone is placed on a rigid and level non-absorbent plate and filled with concrete without tamping. The slump cone can be placed in the conventional upright orientation or inverted. The slump cone is lifted and the horizontal spread of the concrete and the time for the concrete to spread to a diameter of 500 mm (T_{50}) are measured. Emborg et al. (2003) has suggested measuring the time to flow to a diameter of 600 mm instead of 500 mm, given the availability of more fluid mixtures. It is possible to assess the stability of the concrete qualitatively after performing the slump flow test. The visual stability index (VSI), the criteria for which are shown in Table 2.5, is assigned to the nearest 0.5 based on a visual evaluation of the final test specimen. According to Khayat (1999), the lack of material separation during the slump flow test is not an assurance of stability during and after placement. Khayat, Assaad, and Daczko (2004) recommend using the VSI in conjunction with other tests for stability.



Figure 2.12 Slump Flow Test

Table 2.5 Visual Stability Index Ratings (ASTM C 1611)

VSI	Criteria
0 = Highly Stable	No evidence of segregation or bleeding.
1 = Stable	No evidence of segregation and slight bleeding observed as a sheen on the concrete mass.
2 = Unstable	A slight mortar halo ≤ 0.5 in. (≤ 10 mm) and/or aggregate pile in the center of the concrete mass.
3 = Highly Unstable	Clearly segregating by evidence of a large mortar halo > 0.5 in. (> 10 mm) and/or a large aggregate pile in the center of the concrete mass.

2.3.13 Surface Settlement Test

The surface settlement test (Khayat and Guizani 1997; Assaad, Khayat, and Daczko 2004) measures the settlement of a plate on a column of concrete until setting. Surface settlement is related to segregation resistance. In the test, which is shown in Figure 2.13, concrete is placed in a 200-mm diameter, 800-mm tall PVC pipe and filled to a height of 700 mm. A 4-mm thick, 150-mm diameter acrylic disc is set atop the leveled SCC surface. Three 75-mm screws extend downward from the disc to anchor the disc into the concrete. A dial gage, linear variable differential transformer, or non-contact method is used to monitor the settlement of the disc over time. The first reading is taken at 60 seconds followed by subsequent readings every 15 minutes for the first three hours and every 30 minutes thereafter. The container is covered throughout the test to prevent evaporation. The total settlement—expressed as a percentage of the initial column height—should be less than 0.50% for stable SCC. Assaad, Khayat, and Daczko (2004) found that the results of the test were not correlated to yield stress or plastic viscosity. Unlike the penetration apparatus, the surface settlement test depends on the duration of the dormant period (Assaad, Khayat, and Daczko 2004).

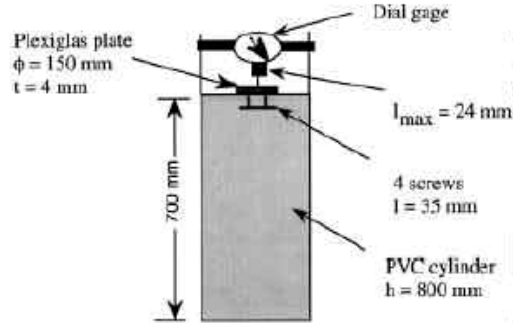


Figure 2.13 Surface Settlement Test (Khayat 1999)

2.3.14 V-Funnel Test

The v-funnel test (EFNARC 2002; Bartos, Sonebi, and Tamimi 2002), which is shown in Figure 2.14, is primarily used to measure the filling ability of SCC and can also be used to evaluate segregation resistance. To perform the test, the funnel is filled with concrete without tamping or vibration and the concrete is left undisturbed for 1 minute. Then, the gate at the bottom of the funnel is opened and the time for all concrete to exit the funnel is recorded.

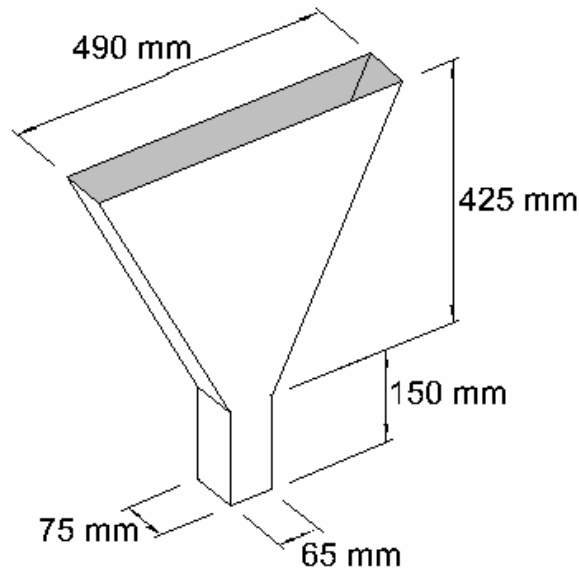


Figure 2.14 V-Funnel

In addition to reporting the flow time, the average flow through speed, V_m , can be calculated as shown in Equation (2.11):

$$V_m = \frac{0.01}{(0.065 \times 0.075) \times t_0} = \frac{2.05}{t_0} (m/s) \quad (2.11)$$

Non-uniform flow of concrete from the funnel suggests a lack of segregation resistance. According to Khayat (1999), a long flow time can be due to high paste viscosity, high interparticle friction, or blockage of flow by coarse aggregates. Likewise, Emborg et al. (2003) found that v-funnel results were related to concrete viscosity, passing ability, and segregation resistance. Therefore, the test results may not identify the true cause of a slow flow time.

The opening size at the bottom is typically 75 x 75 mm or 75 x 65 mm. Emborg et al. (2003) has suggested using a 75 x 49 mm opening to increase the sensitivity of the measurement. In addition, a smaller version of the v-funnel is available for measurements of mortar or paste.

2.3.15 Sieve Stability Test (Vertical Mesh-Pass Tests, GTM Screen Stability Test)

The sieve stability test (EFNARC 2002; Bartos, Sonebi, and Tamimi 2002; Patel 2004), which was developed by the French contractor GTM Construction, measures the ability of SCC to remain uniform under both dynamic and static conditions. To perform the test, a 10-liter sample of concrete is placed in a sealed bucket and left undisturbed for 15 minutes to allow segregation to occur. Then, approximately 2 liters or 4.8 kg from the top of the concrete sample is poured from a height of 500 mm onto a 5-mm (#4) sieve. Mortar from the sample is allowed to flow through the sieve into a lower pan for a period of 2 minutes. The ratio of the mass of material in the pan to the total mass of concrete poured over the sieve is taken as the segregation ratio. It has been reported that the variability of test results is poor, especially when the segregation is severe (Bartos, Sonebi, and Tamimi 2002). Cussigh, Sonebi, and De Schutter (2003) found that the results of the sieve stability test were correlated with the results of the penetration apparatus test developed by Bui.

2.3.16 Setting Time, Bleeding, and Plastic Shrinkage

The setting time of SCC is typically similar to that of conventionally placed concrete; however, given the use of high dosages of chemical admixtures and the possible use of supplementary cementitious materials in SCC, setting time could increase or decrease based on mixture proportions. Polycarboxylate-based HRWRAs generally result in less of a delay in setting time than sulfonate-based HRWRAs.

Given its low water content and high viscosity, SCC typically exhibits minimal surface bleeding (Khayat, Assaad, and Daczko 2004). In particular, the use of fine filler materials and viscosity modifying admixtures can increase the ability of the paste to retain water and result in reduced bleeding (Khayat 1999). Pressure gradients, however, can result in the movement of water through SCC, causing segregation even when surface bleeding is not present (Khayat, Assaad, and Daczko 2004). To measure bleeding, the test method for conventionally placed concrete, described in ASTM C 232, can be used for SCC (Lachemi et al. 2004). Several other available tests are intended primarily for SCC and other highly flowable materials. In the pressure bleed test (Khayat, Assaad, and Daczko 2004), concrete is placed in a pressure vessel with a filter at the bottom. The filter permits the passage of water but blocks most solid particles greater than 1 μm . A pressure of 700 kPa is applied to the top of the concrete for 10 minutes and the amount of bleed water passing the filter is determined and expressed as a percentage of the total water in the concrete sample. In the bleeding test method (PCI 2003), which was developed

in France, SCC is placed inside a volumetric air indicator. Perchlorethylene, which has a specific gravity of 1.59, is filled above the concrete up to the zero mark. The amount of water that rises to the top of the perchlorethylene is measured at regular intervals up to 60 minutes. The total amount of bleed water and the rate of bleeding are determined. Lastly, the electrical conductivity test, described earlier, allows the monitoring of the movement of water within a sample and the computation of bleeding indexes.

SCC can be more susceptible to plastic shrinkage cracking than conventionally placed concrete because of the lack of bleed water and high paste volume (EFNARC 2002; Khayat 1998; Hammer 2003). Turcry, Loukili, and Haidar (2002) and Turcry and Loukili (2003) found that SCC mixtures exhibited plastic shrinkage strains at least two times greater and as much as four times greater than comparable conventionally placed concrete mixtures due mainly to the low water-powder ratio and the delayed setting time induced by the HRWRA. Due to the greater susceptibility to plastic shrinkage cracking, it was recommended that curing be started immediately after casting regardless of weather conditions.

2.4 Hardened Properties

Like conventionally placed concrete, SCC can be proportioned to have widely varying hardened properties. Differences in hardened properties between conventionally placed and self-consolidating concrete can be attributed to three main sources: modified mixture proportions, improved microstructure and homogeneity, and lack of vibration (Klug and Holschemacher 2003). Modified mixture proportions may include higher paste volumes; higher powder contents; lower water-cementitious materials or water-powder ratios; finer combined aggregate gradings; smaller maximum aggregate sizes; and use of SCMs, fillers, HRWRAs, and VMAs. The improved microstructure is related to the higher packing of the bulk paste and the reduced size and porosity of the interfacial transition zone. The lack of vibration eliminates defects due to vibration and ensures uniform distribution of properties. The effects of these changes in mixture characteristics can often be prognosticated based on existing data for conventionally placed concrete. Any changes in hardened properties assume that SCC is properly proportioned for workability; namely that it adequately fills formwork, passes reinforcement, and resists segregation.

Much research has been conducted to evaluate the hardened properties of SCC. In considering the results of this research, the selection of the appropriate baseline for comparing mixtures is crucial and varies by study. Mixtures are often compared at similar compressive strength; similar water-cement, water-cementitious materials, or water-powder ratio; or similar application. According to EFNARC (2005), SCC and conventionally placed concrete with similar compressive strengths should exhibit similar hardened properties. When mixtures are compared at constant water-cement ratio, the SCC mixtures often have large volumes of filler—resulting in lower water-powder ratios and possibly lower water-cementitious materials ratios. These lower water-powder or water-cementitious materials ratios are often, but not always, associated with improvements in hardened properties. For a given application, SCC can often be proportioned to have equal or better hardened properties than conventionally placed concrete by utilizing the tradeoffs associated with different mixture proportioning changes. Further complicating the comparison of conventionally placed and self-consolidating concrete is the fact

that the number of mixtures and the range of mixtures chosen for comparison vary widely by study. In many cases, only a small number of mixtures is compared.

Because of the variety of approaches in comparing conventionally placed and self-consolidating concrete, conclusions vary regarding the hardened properties associated with SCC. Thus, it is necessary to evaluate separately the effects of individual changes to the concrete. As D'Ambrosia, Lange, and Brinks (2005) remark,

...it is best not to treat SCC as a group of materials with comparable mechanical behavior. Different strategies for mixture proportioning may lead to SCC materials that have the common ability to flow into formwork without mechanical vibration, but have very different behavior when considering mechanical performance and early-age cracking risk.

Although low water-powder ratios are usually dictated by workability requirements, the water-cement ratios can be varied much more widely depending on the quantities of fillers used, including fly ash, slag, silica fume, and mineral filler. The rate of development and ultimate values of hardened properties depend on the amount and activity of these fillers.

The potential for improved durability was one of the main original motivations for the development of SCC. The improved microstructure and better consolidation associated with SCC relative to conventionally placed concrete often results in improved durability. The transport properties of concrete depend primarily on the paste volume, pore structure of the paste, and interfacial transition zone (Zhu, Quinn, and Bartos 2001). Although SCC has higher paste volume, the pore structure of the bulk paste and the interfacial transition zone characteristics are often improved due to the low water-cementitious materials ratios and the use of SCMs. The improved stability, reduction in bleeding, and elimination of vibration can lead to a denser interfacial transition zone and improved durability.

2.4.1 Microstructure

The microstructure of SCC is often superior to that of conventionally placed concrete due to the increased packing density of the bulk paste and a reduction in size and porosity of the interfacial transition zone. The low water-powder ratios necessary to achieve adequate workability are responsible for much of the improvement in microstructure. The use of HRWRA results in improved dispersion of cement. Tragardh (1999) compared conventionally placed and self-consolidating concrete mixtures with the same water-cement ratio but with a lower water-powder ratio in the SCC due to the addition of limestone filler. The SCC mixtures exhibited a denser microstructure, with the interfacial transition zone exhibiting a lower porosity and a thinner layer of calcium hydroxide. This improvement in microstructure was attributed to the addition of limestone filler and the reduction in bleeding.

2.4.2 Compressive Strength

Compressive strength is approximately related to the porosity of the concrete, which in turn is related to the water-cement ratio and degree of hydration. Aggregate characteristics can also play an important role in compressive strength. The strength of the aggregate becomes important in moderate- to high-strength concretes. The size, shape, angularity, texture, and mineralogy can affect the quality of the interfacial transition zone and the bond between paste

and aggregate. Although larger aggregates require less mixing water than smaller aggregates, the transition zone around larger aggregates is weaker, resulting in lower compressive strength. Angular and rough-textured aggregates tend to exhibit improved bond to the cement paste. The use of calcareous aggregates generally results in increased compressive strength relative to siliceous aggregates. Other main factors affecting compressive strength include the use of admixtures and SCMs, cement type, air entrainment, and curing conditions (Mehta and Monteiro 1993).

For conventionally placed and self-consolidating concrete mixtures with similar proportions but different workabilities (due to a difference in HRWRA dosage, for example), the SCC should exhibit slightly higher compressive strength due to the lack of vibration, which improves the bond between aggregate and paste (EFNARC 2005), and the improved cement dispersion resulting from the use of HRWRA. Roziere et al. (2005) found that increasing the paste volume from 29.1% to 45.7% while keeping w/cm constant reduced the 28-day compressive strength by 12%. Heirman and Vandewalle (2003) found that when a variety of fillers, including fly ash and mineral fillers, were used and the water-cement ratio (not water-cementitious materials ratio) was held constant, the compressive strength was generally higher.

Klug and Holschemacher (2003) found that the rate of strength development over time was generally similar for SCC and conventionally placed concrete; however, the use of limestone filler could accelerate the early development of strength whereas SCMs could increase the ultimate strength.

2.4.3 Flexural and Tensile Strengths

Flexural (f_r') and tensile (f_t') strengths are often related to compressive strength. The interfacial transition zone characteristics tend to affect tensile and flexural strength to a greater degree than compressive strength (Mehta and Monteiro 1993). Tensile and flexural strengths increase with compressive strength, but at a decreasing rate. Values of flexural strength for lightweight and normal-weight concrete have been reported to range from $7.5\sqrt{f_c'}$ to $12\sqrt{f_c'}$ (ACI 363 1992). ACI 363 (1992) recommends the use of Equation (2.3), which is based on the work of Carrasquillo et al. (1981).

$$f_r' = 11.7\sqrt{f_c'} \quad \text{for } 3,000 < f_c' < 12,000 \text{ psi} \quad (2.12)$$

Tensile strength may be as high as 10% and as low as 5% of compressive strength for low and high strength concrete, respectively. ACI 363 (1992) recommends the use of Equation (2.13), which is based on the work of Carrasquillo et al. (1981).

$$f_t' = 7.4\sqrt{f_c'} \quad \text{for } 3,000 < f_c' < 12,000 \text{ psi} \quad (2.13)$$

Separately, the CEB-FIP model code recommends the use of Equation (2.14), with the value of the constant 1.4 ranging between 0.95 and 1.85.

$$f'_t = 1.4 \left(\frac{f'_c}{10 \text{ MPa}} \right) \quad (2.14)$$

The flexural and tensile strengths of SCC are typically improved relative to conventionally placed concrete due to the improved microstructure of the paste—particularly the improved interfacial transition zone and the denser bulk paste (Klug and Holschemacher 2003). Turcry, Loukili, and Haidar (2002) found that the flexural strength was slightly higher for SCC than a conventionally placed concrete mixture of comparable compressive strength. According to EFNARC (2005), SCC should exhibit similar tensile strength as conventionally placed concrete because paste volume does not have a significant effect on strength. Roziere et al. (2005), however, found that increasing the paste volume of SCC reduced tensile strength slightly. Turcry, Loukili, and Haidar (2002) found that the ratio of tensile to compressive strength was between 0.087 and 0.1 for SCC and 0.075 for comparable conventionally placed concrete. Based on a database of results from around the world, Klug and Holschemacher (2003) found that for a given compressive strength, the tensile strength was comparable to or slightly higher than conventionally placed concrete.

2.4.4 Modulus of Elasticity

For concrete, which can be represented as a multi-phase material, the modulus of elasticity is a function of the volume fractions and elastic moduli of the principle constituents—that is, paste and aggregates—and the characteristics of the interfacial transition zone (Mehta and Monteiro 1993; Nilsen and Monteiro 1993; Alexander and Milne 1995; Neubauer, Jennings, and Garboczi 1996). In general, decreasing the porosity of any of the constituents increases the concrete modulus of elasticity. The paste elastic modulus, which is typically lower than that of the aggregate, is affected by factors such as the water-cement ratio, air content, SCM content, and degree of hydration. In addition, the maximum size, shape, angularity, texture, grading, and microstructure of the aggregates can affect cracking in the interfacial transition zone.

The static modulus of elasticity is frequently related to the square root of the compressive strength. Several such relationships are listed in Table 2.6. The equations shown in Table 2.6 all represent best-fit lines of data, not lower bounds, and actual values may be expected to deviate from the equations by as much as 20% (Oluokun, Burdette, and Deatherage 1991). The equations vary based on the data used for their development. For instance, the widely used equation from the ACI 318 building code was developed based on an analysis conducted by Pauw (1960) of multiple sources of compressive strength, modulus of elasticity, and unit weight data. Much of this data was for concrete with lightweight aggregates. The method of testing for elastic modulus varied between data sources in Pauw's analysis. It is frequently assumed that the unit weight of concrete is 145 lb/ft³; however, this approximation may not always be accurate (Oluokun, Burdette, and Deatherage 1991).

The equations developed for lower strength concrete, such as the ACI 318 equation, have been shown to overestimate modulus of elasticity at higher compressive strengths. According to Carrasquillo, Nilson, and Slate (1981), the modulus of elasticity of high strength concrete is lower than predicted by the ACI 318 equation because compressive strength depends mainly on the mortar properties while modulus of elasticity depends on both the mortar and aggregate properties. Therefore, if the mortar is weaker than the aggregate, any increase in the strength and

stiffness of the mortar results in a larger increase in concrete compressive strength than in concrete modulus of elasticity.

Table 2.6 Models Relating Modulus of Elasticity (E) to Compressive Strength (f'_c) and Concrete Unit Weight (w_c) (all values in psi and lb/ft³, unless noted otherwise)

Reference	Equation	Application Range	Comments
ACI 318 Building Code	$E = (w_c)^{1.5} 33\sqrt{f'_c}$ or $E = 57,000\sqrt{f'_c}$ For “normal-weight concrete”	$90 < w_c < 155 \text{ lb/ft}^3$	Equations taken from Pauw (1960)
ACI 363R State of the Art Report on High Strength Concrete	$E = 40,000\sqrt{f'_c} + 1.0 \times 10^6$	$3,000 < f'_c < 12,000 \text{ psi}$	Based on data from multiple sources, equation originally suggested by Carrasquillo, Nilson, and Slate (1981)
CEB-FIP Model Code	$E = (\alpha)(21,500)\left(\frac{f'_c}{10}\right)^{\frac{1}{3}}$ (values in MPa) $\alpha = 1.2$ for basalt or dense limestone, 1.0 for quartzitic, 0.9 for limestone, 0.7 for sandstone	Valid up to 80 MPa (11,600 psi)	
Ahmad and Shah (1985)	$E = w_c^{2.5} (\sqrt{f'_c})^{0.65} = w_c^{2.5} (f'_c)^{0.325}$	Applicable to low and high strength concrete	
Oluokun, Burdette, and Deatherage (1991)	$E = (w_c)^{1.5} 31.770\sqrt{f'_c}$ or $E = 63,096\sqrt{f'_c}$ for concrete tested	$f'_c > 500 \text{ psi}$	Valid for test ages ranging from 6 hrs to 28 days
Crouch and Pearson (1995)	$E = 41,990\sqrt{f'_c} + 2.299 \times 10^6 \text{ for neoprene capping}$ $E = 37,440\sqrt{f'_c} + 2.531 \times 10^6 \text{ for sulfur capping}$	$2,000 < f'_c < 6,000 \text{ psi}$	
Iravani (1996)	$E = 4,700 C_{ca} \sqrt{f'_c}$ (values in MPa) C_{ca} is selected based on the aggregate type and varies from 0.61 to 0.97	$55 < f'_c < 125 \text{ MPa}$	Based on data from multiple sources

The equations relating modulus of elasticity to compressive strength and unit weight should be used with caution because modulus of elasticity is a function of more than just compressive strength and unit weight (Mehta and Monteiro 1993; Mokhtarzadeh and French 2000; Huo, Al-Omaishi, and Tadros 2001). In particular, the modulus of elasticity has been shown to be strongly dependent on the aggregates (Aitcin and Mehta 1990; Baalbaki et al. 1991; Mehta and Monteiro 1993; Zhou, Lydon, and Barr 1995; Iravani 1996; Cetin and Carrasquillo 1998; ACI 363). The stiffness of aggregates can vary significantly from one source to another.

In most studies, only the coarse aggregates are varied. The difference in modulus of elasticity between aggregate and paste and the physical and chemical bonds between the two influences the micro-cracking that occurs during loading and the associated concrete modulus of elasticity and compressive strength (Carrasquillo, Nilson, and Slate 1981; Neville 1997). For instance, Baalbaki et al. (1991) found that coarse aggregate much stiffer than the mortar increased modulus of elasticity but decreased compressive strength because of the development of stress concentrations at the aggregate-mortar interface. Similar results were obtained by Cetin and Carrasquillo (1998). Aitcin and Mehta (1990) found that the bond between paste and coarse aggregate, which was affected by the aggregate properties, in turn affected concrete modulus of elasticity. Ahmad and Shah (1985) found that increasing the maximum aggregate size or the coarseness of the aggregate grading—with constant consistency and w/c—resulted in higher modulus of elasticity. Compressive strength, however, generally decreases with increasing maximum aggregate size (Neville 1997; ACI 363). Cetin and Carrasquillo (1998) found that decreasing the S/A resulted in slightly higher modulus of elasticity and lower compressive strength but found that reducing the maximum aggregate size had no effect on modulus of elasticity. As concrete strength is increased, the modulus of elasticity of the concrete depends much more on the modulus of elasticity of the aggregates and the relationship between modulus of elasticity and compressive strength is less precise (Neville 1997; Cetin and Carrasquillo 1998). As a result, it is often recommended that modulus of elasticity be measured with the particular job materials.

The test conditions are also highly influential. As concrete is dried, the modulus of elasticity decreases but the compressive strength increases (Ahmad and Shah 1985; Mokhtarzadeh and French 2000). According to Mehta and Monteiro (1993), compressive strength increases 15% and modulus of elasticity decreases 15% when the concrete is dried. The ASTM C 469 standard for modulus of elasticity specifies that cylinders be tested in a moist condition; however, some researchers have allowed specimens to dry. For instance, Carrasquillo, Nilson, and Slate (1981), whose data was used in the ACI 363 equation, allowed their 4- by 8-inch cylinders to dry 2 hours before testing. The method of strain measurement has also been shown to affect results (Ahmad and Shah 1985).

In addition to empirical relationships relating modulus of elasticity to compressive strength, models are available to relate concrete modulus of elasticity to the volume and elastic moduli of the constituents (Hansen 1960; Hashin 1962; Hirsh 1962; Counto 1964; Bache and Nepper-Christensen 1965; Popovics and Erdey 1970; Neubauer, Jennings, and Garboczi 1996). Such models typically represent concrete as a two- or three-phase material. Baalbaki et al. (1992) compared experimental data for high-strength concrete to six such two-phase models and to empirical relationships between modulus of elasticity and compressive strength. They found that both approaches provided reasonable predictions for most aggregates; however, they recommended direct testing with actual materials for better precision. Zhou et al. (1995) compared experimental data to six two-phase models and found the models gave reasonable results for 4 of 6 aggregates. The two aggregate giving poor results were steel beads and expanded clay.

In evaluating the literature on the effects of coarse aggregate on modulus of elasticity, it should be noted that the fine aggregates are typically unchanged in the experiments. The effects of fine aggregates should not be discounted because limited test data are available. Based on the representation of concrete as a three-phase material (paste, aggregate, and interfacial transition zone), the influence of fine aggregates on modulus of elasticity is significant.

The modulus of elasticity of SCC is typically equal to or slightly less than that of conventionally placed concrete due to the higher paste volume and reduced maximum aggregate size (EFNARC 2005). The modulus of elasticity of SCC may be increased, however, by the improved interfacial transition zone. Based on a database of results from around the world, Klug and Holschemacher (2003) found for a given compressive strength that the modulus of elasticity was typically lower than for conventionally placed concrete; however, the vast majority of the data points were within the expected range of the CEB-FIP model code. According to PCI (2003), the modulus of elasticity of SCC may be as low as 80% of that of comparable conventionally placed concrete. Turcry, Loukili, and Haidar (2002) found that the ratio of modulus of elasticity (GPa) to compressive strength (MPa) was approximately 0.6 for SCC and 0.7 for conventionally placed concrete. Roziere et al. (2005) found that increasing the paste volume from 29.1% to 45.7% while keeping w/cm constant reduced the 28-day modulus of elasticity by 14%. Persson (2001) found that at a constant compressive strength level, SCC and conventionally placed concrete exhibited similar elastic moduli. Schindler et al. (2007) found that, for a given compressive strength, the SCC mixtures had similar modulus of elasticity as conventionally placed concrete mixtures at 56 days but slightly lower modulus of elasticity at 18 hours. The lower elastic modulus at 18 hours was attributed to the use of SCMs in the SCC mixtures but not in the conventionally placed concrete mixtures. The S/A was found to have no effect on modulus of elasticity for the majority of SCC mixtures. The values of modulus of elasticity were greater than those predicted by the ACI 318 equation. Naito et al. (2005) found that the modulus of elasticity of one SCC mixture was lower than a conventionally placed concrete mixture for a given compressive strength. The SCC mixture, which was intended for prestressed concrete bridge beams, had a smaller maximum aggregate size, slightly lower w/cm, and higher S/A. Su et al. (2002) found that increasing the S/A from 0.30 to 0.55 in SCC mixtures did not significantly affect the modulus of elasticity because the total aggregate volume was constant and the stiffness of the fine and coarse aggregates were similar.

2.4.5 Dimensional Stability

The risk of shrinkage—including both early-age autogenous and longer term drying shrinkage—may be greater for SCC due primarily to its higher paste content. The more highly refined pore structure of SCC may also increase the risk of autogenous shrinkage. The high cementitious materials contents and low water-cementitious ratios can increase the susceptibility to thermal volume changes. To evaluate the susceptibility of SCC to cracking due to volume changes, the viscoelastic properties and tensile strength of concrete must also be evaluated. The higher volume changes sometimes associated with SCC may not necessarily result in increased cracking risk due to the higher tensile strength, lower modulus of elasticity, and higher creep sometimes associated with SCC.

In general, changes to the mixture proportions that increase the refinement of the pore structure increase autogenous shrinkage. These changes include reducing the water-cementitious materials ratio below 0.40 (Tazawa and Miyazawa 1995b, Aitcin 1999, Li, Wee, and Wong 2002; Zhang et al. 2003), using slag (Tazawa and Miyazawa 1995a; Li, Wee, and Wong 2002), using silica fume (Tazawa and Miyazawa 1995a, Zhang, Tam, and Leow 2003; Jensen and Hansen 2001; Li, Wee, and Wong 2002), and increasing the fineness of cement (Tazawa and Miyazawa 1995a). The use of fly ash has minimal effect on autogenous shrinkage because its particle size is similar to that of cement (Bentz et al. 2001).

Turcry, Loukili, and Haidar (2002) and Suksawang, Nassif, and Najim (2005) found that autogenous shrinkage was higher for SCC than for comparable, conventionally placed concrete mixtures. D'Ambrosia, Lange, and Brinks (2005) found that the autogenous shrinkage of SCC mixtures increased significantly as the paste volume was increased and as the water-cementitious materials ratio was reduced below 0.40. Roziere et al. (2005), however, found the autogenous shrinkage of SCC mixtures to be very low due to the relatively high water-cement ratio of the tested SCC mixtures and because limestone filler and fly ash were found to reduce autogenous shrinkage.

The main factors affecting drying shrinkage—aside from exposure conditions and element geometry—are the total contents of water and paste and the aggregate characteristics. Because drying shrinkage is mainly the result of the loss of adsorbed water from the paste, higher paste volumes and total water contents are associated with increased shrinkage (Kosmatka, Kerkhoff, and Panarese 2000). Increasing the water-cement ratio at a constant cement content or increasing the cement content at a constant water-cement ratio will increase drying shrinkage, although this increase is predominately due to the higher paste volume. Bissonnette, Pascale, and Pigeon (1999) found that water-cement ratio had little effect on shrinkage when the paste volume was held constant; however, increasing the paste volume at constant water-cement ratio resulted in increased shrinkage. The fineness and composition of cement generally has negligible effect on drying shrinkage (Mehta and Monteiro 1993; Kosmatka, Kerkhoff, and Panarese 2000). Additionally, SCMs usually have little effect on drying shrinkage. Accelerators and some water reducers can increase drying shrinkage. The use of aggregates with high stiffness and low shrinkage decreases drying shrinkage (Mehta and Monteiro 1993; Kosmatka, Kerkhoff, and Panarese 2000; EFNARC 2005). Other aggregate characteristics primarily affect shrinkage indirectly by controlling the amount of the paste and water needed in the mixture (Mehta and Monteiro 1993; ACI Committee 209 1997).

The drying shrinkage of SCC may be higher than that of conventionally placed concrete primarily due to the higher paste volumes (Hammer 2003; EFNARC 2005). The drying shrinkage of SCC may be reduced, however, due to the denser microstructure (Klug and Holschemacher 2003). The total water content of SCC mixtures may be no greater than in comparable conventionally placed concrete. Based on a database of results from around the world, Klug and Holschemacher (2003) found that the drying shrinkage of SCC was typically 10-50% higher than that predicted by the CEB-FIP model code. Turcry, Loukili, and Haidar (2002) found that the drying shrinkage strains of two different SCC mixtures were similar to comparable, conventional mixtures due to the offsetting effects of increased paste volume and reduced water-powder ratio. Suksawang, Nassif, and Najim (2005) measured increased drying shrinkage in SCC compared to a comparable conventional mixture. Roziere et al. (2005) found that total shrinkage of SCC—including autogenous and drying—increased linearly with paste volume and that limestone filler and, to a lesser extent, fly ash reduced drying shrinkage. Attiogbe, See, and Daczko (2002) found that reducing the sand-aggregate ratio reduced drying shrinkage of SCC. Persson (2001) found that at a constant compressive strength, drying shrinkage was similar in SCC and conventionally placed concrete. Bui and Montgomery (1999a) found that reducing the water-binder ratio and paste volume and the use of limestone filler could reduce the drying shrinkage of SCC; however, the fresh properties had to be appropriate for good compaction and no segregation. Heirman and Vandewalle (2003) found that when a variety of fillers were added to SCC without changing the cement content and water-cement ratio, the shrinkage increased relative to conventionally placed concrete. In comparing a SCC mixture and

conventional mixture with similar water-cement ratios but with higher powder content in the SCC, Vieira and Bettencourt (2003) found the shrinkage to be nearly identical. In evaluating SCC for prestressed concrete applications, Schindler et al. (2007) found that the drying shrinkage strains of 21 SCC mixtures were equal to or less than in two conventionally placed concrete mixtures and that changing the S/A had no effect on shrinkage. Naito et al. (2005) found that the drying shrinkage of one SCC mixture was approximately 40% higher than a conventionally placed concrete mixture of comparable compressive strength. The SCC mixture, which was intended for prestressed concrete bridge beams, had a smaller maximum aggregate size, slightly lower w/cm, and higher S/A.

2.4.6 Permeability and Diffusivity

Permeability and diffusivity are related to the total porosity and the size and continuity of the voids in the concrete. In addition, diffusivity is related to the binding capacity of the cement paste. Permeability and diffusivity are reduced by improving the pore structure—including reducing the volume of pores, the sizes of pores, and connectivity of pores in both the paste and aggregates—and improving the transition zone. The pore structure of the paste can be improved by reducing the water-cementitious materials ratio, reducing the water content, providing proper curing, and using SCMs. While SCMs may not reduce porosity, they refine the pore structure, resulting in less connectivity of the pores. This refinement is due to the fact that the calcium silicate hydrate occupies a greater volume than the calcium hydroxide and pozzolan from which it forms. Very fine particles—such as silica fume—can enhance the physical packing and improve the pore structure. Permeability and diffusivity are reduced with increased hydration. Although higher curing temperatures may accelerate early hydration, they create a coarser structure, resulting in higher long-term permeability and diffusivity than the same mixture cured at a lower temperature. According to Mehta and Monteiro (1993) the paste is not the principle contributor to permeability in well-cured concrete unless the water-cement ratio is excessive (for example, greater than 0.7). Therefore, the properties of the transition zone and any micro-cracking that occurs in the transition zone are of more importance. The capacity of the cement paste to bind ions is enhanced with the use of SCMs and cements with higher C₃A contents. In particular, the hydration products of slag are known to bind chloride ions effectively.

The permeability and diffusivity of SCC may be higher or lower than conventionally placed concrete depending on the mixture proportions. The low water-cementitious materials ratio and frequent use of SCMs are favorable for improving permeability and diffusivity; however, not all SCMs have the same effect. For instance, Suksawang, Nassif, and Najim (2005) found that the rapid chloride permeability test results increased or decreased relative to a comparable conventional mixture depending on the type of SCM used. Similarly, Zhu, Quinn, and Bartos (2001) found that the chloride diffusion of SCC depended strongly on the type of filler used. When the same filler was used in conventionally placed and self-consolidating mixtures, the chloride diffusion was similar. In contrast, the capillary water absorption and oxygen permeability coefficients were found to be significantly lower for SCC regardless of the type of filler used. Audenaert, Boel, and De Schutter (2002) found that decreasing the water-cement and water-powder ratios in SCC mixtures and using fillers with finer gradings reduced the chloride penetration. Tragardh (1999) found that a SCC mixture with similar water-cement ratio as a conventional mixture—but with lower water-powder ratio due to the addition of limestone filler—exhibited lower chloride diffusion.

2.4.7 Freeze-Thaw Durability

Freeze-thaw damage may be caused by internal frost damage or salt-scaling. Resistance to internal frost damage is enhanced by providing an adequate air void system—including proper total air void volume as well as proper air void size and spacing. It can also be enhanced by using low water-cementitious materials ratios and SCMs to reduce both permeability and, in particular, the number of large pores. Increasing the concrete strength also enhances resistance to internal frost damage. Salt-scaling can be prevented by providing an adequate entrained air void system, reducing the water-cementitious materials ratio, and providing proper finishing and curing practices. There is some evidence that the use of fly ash or slag may reduce salt-scaling resistance.

The freeze-thaw durability of SCC is frequently comparable to or better than that of conventionally placed concrete. The low water-cementitious materials ratios and ability to adequately entrain air can enhance the freeze-thaw resistance. The use of fly ash and slag, however, may reduce salt-scaling resistance. Persson (2003) found the internal frost resistance of SCC to be better than comparable conventionally placed concrete and the salt-scaling resistance to be similar. Heirman and Vandewalle (2003) found that when a variety of fillers were used and the water-cement ratio was held constant, the freeze-thaw durability was similar but the salt-scaling resistance decreased relative to conventionally placed concrete. Audenaert, Boel, and De Schutter (2002) found that reducing the water-cement and water-powder ratios in SCC mixtures improved internal frost resistance.

The use of some HRWRAs under certain conditions may detrimentally affect the air content and characteristics of the air void system. Khayat and Assaad (2002), however, found that the air void characteristics of SCC were similar to those of conventionally placed concrete and that air void stability could be improved by increasing the cementitious materials content and reducing the water-cementitious materials ratio or by including a VMA in mixtures with low cementitious materials contents and high water-cementitious materials ratios.

2.5 Mixture Proportioning

Numerous mixture proportioning methods have been proposed for SCC. The methods vary widely in overall approach, in the range of materials and performance characteristics considered, and in the level of complexity.

SCC mixture proportions depend, in large part, on the application. Requirements for hardened properties, filling ability, segregation resistance, and especially passing ability may vary widely by application. These factors must be considered prior to starting the mixture proportioning process. All mixture proportioning methods must ensure adequate yield stress and plastic viscosity of the concrete. According to Yahia et al. (1999), a low yield stress is important for filling ability while high mortar plastic viscosity is needed for placement in highly congested sections and for mixtures with high coarse aggregate contents. High deformability can be achieved by limiting the coarse aggregate volume while segregation resistance can be achieved by controlling the mortar rheology through reducing the w/cm, increasing the powder content, or adding VMA.

Mixture proportioning can be broadly split between three approaches based on the method of achieving sufficient viscosity and segregation resistance: powder-type, VMA-type, and combination-type. In powder-type SCC, the powder content is high and w/p low. In VMA-

type SCC, the powder content is reduced and the w/p is increased relative to powder-type SCC and a VMA is added to ensure segregation resistance. The paste volume, however, may not change significantly between the two types. Combination-type SCC combines both moderately high powder content and the use of a VMA. According to the Japanese Society of Civil Engineers (1999), the powder content in powder-type SCC should be approximately 16%-19% of the concrete volume (500-600 kg/m³ or 850-1000 lb/yd³ based on only cement) and can comprise a wide variety of powders, such as fly ash, slag, and limestone filler. The water-powder ratio of powder-type SCC typically ranges from 0.28 to 0.37. In contrast, the powder content of VMA-type SCC is typically 9.5-16% (300-500 kg/m³ (500-850 lb/yd³ or 9.5 to 16% of the concrete volume based on only cement) and composed entirely of portland cement. The water content of VMA-type SCC may be greater than 18% of concrete volume (300 lb/yd³). For combination-type SCC, the powder content is typically greater than 13% of the total concrete volume and the w/cm is restricted to a narrow range. These powder contents are summarized in Table 2.7.

Table 2.7 Typical Powder Contents of SCC Types Based on JSCE Recommendations (1999)

Parameter	Powder Content	Mass* (kg/m ³)	Mass* (lb/yd ³)
Powder-Type	16-19%	500-600	850-1000
VMA-Type	9.5-16%	300-500	500-850
Combination-Type	>13%	>410	>690

*Based on portland cement only

According to an analysis of 68 SCC case studies conducted by Domone (2006), mixture proportions for SCC vary widely such that there is not a unique solution for any given application. The analysis found that coarse aggregate contents varied from 28 to 38% of concrete volume, paste content varied from 30 to 42% of concrete volume, powder content ranged from 445 to 605 kg/m³, water-powder ratio ranged from 0.26 to 0.48, and fine aggregate content varied from 38 to 54% of mortar volume. The majority of case studies used maximum coarse aggregate sizes of 16 to 20 mm. Nearly all mixtures used some type of non-portland cement powder, with limestone powder the most common addition. In general, the SCC mixture proportions—when compared to conventional, vibrated concrete—were characterized by lower coarse aggregate contents, increased paste contents, higher powder contents, low water-powder ratios, high HRWRA dosages, and the use of VMA in some cases.

Separately, EFNARC (2001) has provided typical values for SCC mixture proportions, as given in Table 2.8.

Table 2.8 Typical Mixture Proportioning Values Suggested by EFNARC (2001)

Parameter	Typical Values
Water/powder (volume)	0.80-1.10
Total powder content	160-240 l/m ³
Coarse aggregate volume	28-35%
Water content	<200 l/m ³

The following sub-sections describe 14 individual mixture proportioning methods. These descriptions are based on the information available in the cited references and, therefore, may not fully represent all aspects of the methods and may not reflect the latest versions of the methods.

2.5.2 Proportioning Methods

2.5.2.1 ACBM Paste Rheology Model/Minimum Paste Volume Method

The ACBM Paste Rheology Model is the result of the input of several researchers. Saak, Jennings, and Shah (2001) originally introduced the concept of a self-flow zone, defined in terms of a range of paste yield stress and apparent viscosity values necessary to achieve both self-flow and segregation resistance. The model was later modified by Bui, Akkaya, and Shah (2002) to include the effects of aggregates by expanding on the Minimum Paste Volume Method, which was developed earlier by Bui and Montgomery (1999) and Bui (2002).

To ensure segregation resistance and self-flow simultaneously, Saak, Jennings, and Shah (2001) developed an analytical model of a single aggregate in cement paste. Based on this model, which was verified experimentally, they defined a self-flow zone in terms of paste yield stress and paste apparent viscosity. The zone was defined by a minimum yield stress and apparent viscosity for segregation resistance and a maximum yield stress and apparent viscosity for self-flow. The paste composition of the SCC mixture was adjusted to be in the self-flow zone.

To incorporate the effects of aggregates, criteria for the solid phase (aggregates) and liquid phase (paste) are considered separately. The solid phase criteria are established to prevent blocking of aggregates while the liquid phase criteria are considered to ensure adequate segregation resistance, flowability, and form-surface finishability. To proportion mixtures, the minimum paste volumes required for the solid phase and liquid phase criteria are computed separately and the limiting case for paste volume is selected. Then, the paste rheology is established to complete the mixture proportions.

The minimum paste volume to satisfy the solid phase criteria is based on the aggregate grading and reinforcement size. The maximum aggregate volume (V_{abmax}) is computed in Equation (2.3):

$$V_{ab\max} = \frac{\rho_g + (\rho_s - \rho_g)N_{ga}}{\sum \frac{P_{vgm} N_{ga} \rho_s}{V_{abm}} + \sum \frac{P_{vsn} (1 - N_{ga}) \rho_g}{V_{abn}}} \quad (2.15)$$

where ρ_g and ρ_s are the specific gravities of the coarse and fine aggregates, respectively; N_{ga} is the ratio of coarse aggregate to total aggregate; P_{vgm} is the volume ratio of coarse aggregate in aggregate group m (i.e. between two sieves) to the total coarse aggregate content; P_{vsn} is the volume ratio of fine aggregate group n to total fine aggregate content; V_{abm} and V_{abn} are the blocking volumes of m and n groups of coarse and fine aggregates, respectively. The blocking volumes are computed with a series of equations based on the aggregate size and the reinforcement size and clear spacing, as described in Bui and Montgomery (1999). The solid phase criteria indicate that increasing the amount of larger particles reduces the volume of total aggregate permitted for a given reinforcement bar clear spacing.

The minimum paste volume to satisfy the liquid phase criteria is based on the average spacing between aggregates, which is computed with Equation (2.16):

$$D_{SS} = D_{AV} \left(\sqrt[3]{1 + \frac{V_p - V_{void}}{V_c - V_p}} - 1 \right) \quad (2.16)$$

where V_p is the paste volume, V_c is total concrete volume (nominally 1 cubic meter or 1 cubic yard), and V_{void} is the volume of voids between densely compacted aggregates (dry-rodded unit weight of combined aggregates, determined in accordance with ASTM C 29), and D_{av} is the average aggregate diameter, computed based on Equation (2.17):

$$D_{AV} = \frac{\sum d_i m_i}{\sum m_i} \quad (2.17)$$

where d_i is the average size of fraction i and m_i is percentage of mass between the upper and lower sieve size for size fraction i . The values of D_{SS} and D_{AV} are assumed to represent the majority of aggregate characteristics. The minimum paste volume (V_{pdmin}) is computed based on the minimum average aggregate spacing (D_{ssmin}), as shown in Equation (2.18):

$$V_{pd \min} = V_t \frac{V_t - V_{void}}{\left[\frac{D_{ss \min}}{D_{av}} + 1 \right]^3} \quad (2.18)$$

The minimum average aggregate spacing must be selected before computing the minimum paste volume for the liquid phase criteria. It is not a standard value, but depends on factors such as the water-binder ratio and the aggregate size. It can be determined experimentally.

For proportioning, the minimum paste volume is computed for various N_{ga} values for both the liquid and solid phase criteria. An example of the results of such calculations is illustrated in Figure 2.15.

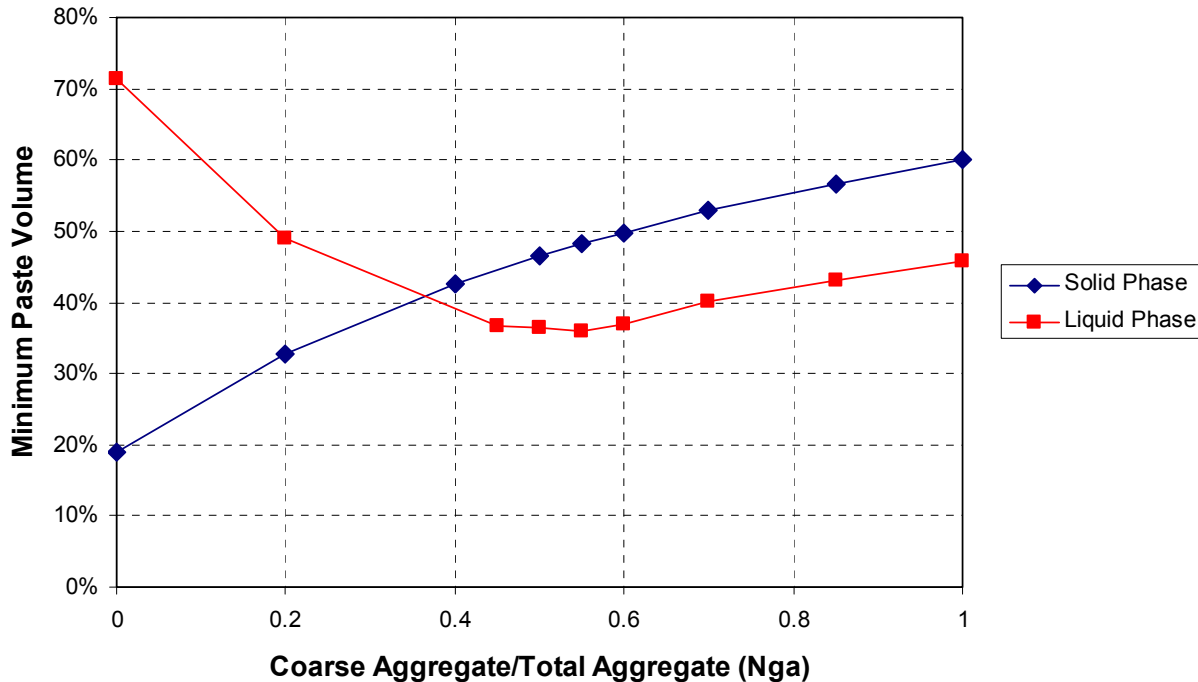


Figure 2.15 Example of Liquid and Solid Phase Criteria

With the minimum paste volume selected, the paste rheology must be optimized. Bui, Akkaya, and Shah (2002) extended the model further by evaluating over 70 concrete mixtures with varying workability. The data was used to develop a rheology model that illustrates trends between paste rheology, average aggregate size, and average aggregate spacing. For example, it was found that as the average aggregate spacing was increased, the optimum ratio of paste mini-slump-flow to viscosity decreased. Thus, as the paste volume is increased for a given aggregate, the paste mini-slump-flow should be reduced and the paste viscosity should be increased. It was also shown that below a certain average aggregate spacing, SCC could not be produced regardless of the paste rheology. For a constant aggregate spacing, decreasing the average aggregate size reduced the optimum ratio of paste mini-slump-flow to viscosity. This rheological model can be used for mixture proportioning to reduce the amount of laboratory work. Bui (2002) adds that the optimum water-binder ratio and ratio of coarse to total aggregate can be selected on the basis of empirical tests to achieve low binder content and low HRWRA dosage.

2.5.2.2 Compressible Packing Model

The compressible packing model developed by de Larrard (1999a) has been applied to SCC (Sedran et al. 1996; Sedran and de Larrard 1999). The intent of the model is to reduce the high paste volumes sometimes associated with SCC. The method includes a detailed packing model to optimize aggregate packing. The model includes equations to compute concrete yield stress, plastic viscosity, and segregation resistance. In addition, a parameter has been developed to predict filling/passing ability.

For proportioning SCC mixtures, the required inputs are the size distributions, specific gravities, and packing densities of the constituents and the saturation dosage of the HRWRA. Because several constants in the compressible packing model depend on the HRWRA, approximately 10 trial batches with different HRWRA and water contents must be tested for rheology and segregation resistance in order to determine these constants. The model equations are used to compute yield stress, plastic viscosity, and parameters describing filling/passing ability and segregation resistance. Limits are established for each of these four parameters. Gap-graded mixtures must be avoided to ensure segregation resistance even though they may result in high packing density. Requirements for hardened properties must also be included. The initial trial proportions are optimized numerically by the model and must then be verified with laboratory trial batches.

2.5.2.3 Concrete Manager Software

The “Concrete Manager” software program utilizes a theoretical model to predict concrete rheology and to optimize the proportions of concrete mixtures (Roshavelov 1999, Roshavelov 2002, Roshavelov 2005). The model used in the software includes both a packing model and Mooney’s equation for the relative viscosity of concentrated suspensions. The packing density is first computed from the packing model and then used in Mooney’s equation to predict the relative viscosity, which can be related to empirical measures of concrete workability.

The development of trial mixture proportions is completed by the Concrete Manager software. First, the desired relative viscosity is selected based on factors such as placement methods, formwork configuration, and reinforcement confinement. Second, the software is used to design an initial trial mixture that both achieves the required viscosity and optimizes proportions. In the third step, a trial batch is mixed and rheological parameters of yield stress, plastic viscosity, and apparent viscosity are measured with a unique capillary rheometer. For the fourth step, the results from the trial batch are compared to the computer calculations and adjustments to the mixture proportions are made as necessary. According to Roshavelov (2005), the predicted apparent viscosities match measured apparent viscosities well.

2.5.2.4 Densified Mixture Design Algorithm Method

The Densified Mixture Design Algorithm (DMDA) for proportioning high-performance concrete (Chang 2004) has been applied to SCC (Hwang and Chen 2002; Li and Hwang 2003; Chen, Tsai, and Hwang 2003; Hwang and Tsai 2005). The DMDA was developed in Taiwan. It aims to maximize the volume of solid materials and minimize the contents of water and cement.

In the first step, the densities of various blends of aggregates are considered in order to select the blend with the maximum density. Fly ash is considered to be part of the aggregate. The blends are evaluated in a multi-step process. First, the blend of fly ash and fine aggregate resulting in the maximum density is determined. Then, this optimum blend of fly ash and fine aggregate is blended with various amounts of coarse aggregate to select the maximum packing density of all three components. In the second step, the volume of paste (V_p) is calculated by increasing the volume of voids between the aggregate (V_v) by a factor (N), which is given in Equation (2.19):

$$N = \frac{V_p}{V_v} = 1 + \frac{St}{V_v} \quad (2.19)$$

where S is the surface area of aggregates and t is the thickness of paste around aggregates. Next, the water-cementitious materials ratio is established based on strength and durability requirements. Finally, the water content, cement amount and HRWRA are determined, subject to a minimum water-cement ratio of 0.42 (to prevent autogenous shrinkage) and a maximum water content of 160 kg/m³.

2.5.2.5 Excess Paste Theory

Oh, Noguchi, and Tomosawa (1999) applied the concept of the excess paste theory, which was originally developed by Kennedy (1940), to SCC. The excess paste theory requires the determination of the excess paste volume, which is the paste in excess of that needed to fill the voids between the aggregates. This excess paste is divided by the surface area of the aggregates to determine the thickness of the excess paste. Multiple methods are available for determining the surface area of the aggregates, including a novel approach suggested by Oh, Noguchi, and Tomosawa (1999). Other methods—including the Minimum Paste Volume Method and Densified Mixture Design Algorithm—incorporate concepts of the excess paste theory. Midorikawa, Pelova, and Walraven (2001) used a similar concept based on the thickness of the water layer around aggregates. Schwartzentruber and Catherine (2000) utilized a similar concept for proportioning concrete based on the mortar fraction.

Oh, Noguchi, and Tomosawa (1999) found that increasing the thickness of the excess paste resulted in decreases in yield stress and plastic viscosity. There was not a unique relationship between the thickness of excess paste and the Bingham parameters for different paste compositions. A unique relationship was found, however, between the relative thickness of excess paste and the relative Bingham parameters. The relative thickness of excess paste (Γ) was defined as the thickness of the excess paste divided by the projected diameter of the aggregate. The relative thickness of excess paste can be computed for an entire aggregate grading by summation of each individual size fraction, as shown in Equation (2.20):

$$\Gamma = \frac{P_e}{\sum_i^n n_i s_i D_{pi}} \quad (2.20)$$

where P_e is the volume of excess paste, n_i is the number of particles in size class i , s_i is the surface area of particles in size class i , and D_{pi} is the projected diameter of the particles in size class i . Alternatively, Hasholt, Pade, and Winnefield (2005) defined the relative thickness of excess paste as shown in Equation (2.21):

$$\Gamma = \frac{1 - \frac{\phi}{\phi^*}}{\frac{f}{k} \phi} \quad (2.21)$$

where ϕ is the actual packing density of the aggregates, ϕ^* is the maximum packing density of the aggregates, and f/k is a factor describing the shape of the aggregates. The value of f/k is 6 for spheres and increases as the shape deviates from that of a sphere.

The relative Bingham parameters are calculated by dividing the Bingham parameters of the concrete by those of the paste. The relationships between relative thickness of excess paste and the relative plastic viscosity (η_r), and relative yield stress (τ_{y_r}) are given in Equations (2.22) and (2.23):

$$\eta_r = 0.0705\Gamma^{-1.69} + 1 \quad (2.22)$$

$$\tau_{y_r} = 0.0525\Gamma^{-2.22} + 1 \quad (2.23)$$

Therefore, by determining the specific surface area of the aggregates and the rheology of the paste, the rheology of the concrete can be computed for any paste volume.

Hasholt, Pade, and Winnefield (2005) evaluated the work of Oh, Noguchi, and Tomosawa (1999) for a range of concrete mixtures and found that it was not possible to link concrete rheology to paste or mortar rheology using the excess paste theory when the concrete, paste, and mortar were measured with different rheometers. The model did perform satisfactorily, however, when inverse calculations were used to compute paste rheology from concrete rheology measurements.

2.5.2.6 Gomes et al. (2001) High Strength SCC Method

Gomes et al. (2001) presented an empirical method for developing high strength SCC mixture proportions. The method considers SCC as a two-phase material consisting of paste and aggregate. Each phase is optimized separately.

In the first step, the paste composition is optimized by determining the optimum ratios of water/cement, silica fume/cement, HRWRA/cement, and filler/cement. The value of water/cement is set at 0.40 and decreased progressively to obtain the desired compressive strength. The value of silica fume/cement is fixed at 0.1. The ratio of HRWRA/cement is selected by determining the saturation dosage of HRWRA with the Marsh funnel test. The saturation dosage is defined as the dosage beyond which the flow time does not change substantially. The optimum value of filler/cement is determined with the mini-slump flow test by measuring pastes with various filler/cement values at the saturation dosage of HRWRA. The optimum value of filler/cement is selected as the value resulting in a certain mini-slump spread diameter and spread time when tested with the saturation dosage of HRWRA.

In the second step, the aggregates are selected by determining the blend of fine and coarse aggregates that results in the lowest voids content. The voids content is determined based on the shoveling procedure in ASTM C 29.

With the optimum paste composition and aggregate blend selected, the third step involves selecting the appropriate paste volume. Concrete mixtures with various paste volumes are measured for filling ability, passing ability, and compressive strength. The minimum acceptable paste volume is selected.

2.5.2.7 ICAR Mixture Proportioning Procedure

The ICAR mixture proportioning procedure (Koehler and Fowler 2007) was developed to incorporate more fully the effects of aggregate characteristics on SCC. The method is based on a representation of SCC as a concentrated suspension of aggregates in paste, which provides a consistent framework for evaluating and selecting mixture proportions. The method consists of three steps: select aggregates, select paste volume, and select paste composition. The steps are conducted in this order because the paste volume depends primarily on the aggregates and the paste composition depends on the aggregates and the paste volume.

First, aggregates are selected on the basis of maximum size, grading, and shape and angularity. The combined aggregates—instead of individual fine and coarse fractions—are considered. The maximum size should be selected as large as possible for filling ability but should be limited for passing ability and segregation resistance. The optimal grading depends on the aggregates and the application. In general, uniform gradings and gradings near the 0.45 power curve or finer are favorable. Aggregates for SCC can vary widely in shape and angularity; however, selecting equidimensional and well-rounded aggregates can significantly improve workability.

Second, the paste volume is selected for filling ability, passing ability, and robustness. A minimum paste volume, which depends primarily on the aggregate characteristics, must be provided to achieve SCC workability. An equation is provided for computing the minimum paste volume for filling ability and testing is conducted to determine the minimum paste volume for passing ability. The larger of the minimum paste volumes for filling ability or passing ability is selected. Additional paste volume can be provided for robustness with respect to aggregate properties.

Third, the paste composition is established for workability and hardened properties by selecting the relative amounts of water, powder, and air and the blend of powder. The water-powder ratio is used for evaluating workability, the water-cement ratio for early-age hardened properties, and the water-cementitious materials ratio for later-age hardened properties. Supplementary cementitious materials and mineral fillers are used to ensure sufficient paste volume, minimize cement content, enhance durability, and modify workability.

2.5.2.8 Particle-Matrix Model

The Particle-Matrix Model was originally developed by Ernst Mortsell for conventionally placed concrete (Mortsell, Maage, and Smeplass 1996) and has since been extended to SCC with mixed success (Smeplass and Mortsell 2001; Pedersen and Mortsell 2001; Reknes 2001). The model splits concrete between the matrix phase—which consists of water, admixtures, and all particles smaller than 0.125 mm—and the particle phase—which consists of all particles larger than 0.125 mm. Workability is assumed to depend on the matrix rheology, the characteristics of the particles, and the volume of matrix. The matrix rheology is described with the flow resistance ratio (λ_Q) and the particle characteristics are described with the air voids modulus (H_m).

The matrix flow resistance ratio is measured with the FlowCyl, which is a modification of the Marsh funnel test. An electronic ruler with data logger is added to measure the flow rate as a function of the height of the matrix in the FlowCyl. The flow rate versus height of matrix in

the FlowCyl is plotted for an ideal fluid and for the tested material. The difference between these two curves is computed as the loss-curve. The flow resistance ratio is defined as the ratio of the area under the loss-curve to the area under the ideal fluid curve. The value of the flow resistance ratio varies from 0.0 for an ideal fluid with no loss to 1.0 for a fluid that does not flow. The flow resistance ratio is typically 0.1 for water and between 0.6 and 0.8 for SCC (Pedersen and Mortsell 2001). The flow resistance ratio has been said to be correlated to plastic viscosity (Pedersen and Mortsell 2001); however, Mortsell, Maage, and Smeplass (1996) asserted that it is a better measurement of paste properties than viscosity. The flow resistance ratio does not capture the effect of yield stress, which may be a major limitation in applying the measurement to SCC (Pedersen and Mortsell 2001).

The air void modulus (H_m) is computed based on the characteristics and volume fractions of the fine and coarse aggregates, based on Equation (2.24):

$$H_m = v_1 \left[\frac{H_s}{Fm_s^{0.5}} + Ts \right] + v_2 \left[\frac{H_p}{Fm_p^{0.5}} + Tp \right] \quad (2.24)$$

where v_1 and v_2 are the volume fractions of sand and coarse aggregate, respectively; H_s and H_p are the void contents in the compacted sand and coarse aggregate, respectively; T_s and T_p are the aggregate parameters for sand and coarse aggregate, respectively; and Fm_s and Fm_p are the fineness moduli of the sand and coarse aggregate, respectively. The air void modulus is intended to equal the paste volume when the mixture changes from no-slump to a small slump. The fineness modulus is included to adjust for the fact that the sand has a greater effect on workability than coarse aggregates do. The value of T_s and T_p can be found by regression analysis of multiple mixtures where the water content is adjusted to change from a zero slump to a non-zero slump.

With the matrix composition and aggregate blend characterized, workability is measured for various matrix volumes. An equation is fitted to the plot of a workability parameter—such as slump flow, yield stress, or plastic viscosity—versus matrix volume. Such equations are developed for multiple matrix flow resistance ratios and aggregate air void moduli. Equations can also be developed to relate the matrix composition to the flow resistance ratio. The use of these equations enables the prediction of the effects of changes in mixture proportions and the selection of optimum mixture proportions.

2.5.2.9 Rational Mix Design Method

The Rational Mix Design Method was developed in Japan and has been presented in various forms by multiple authors—including but not limited to Okamura and Ozawa (1995); Ouchi, Hibino, and Okamura (1997); Edamatsu, Nishida, and Ouchi (1999); Okamura, and Ouchi (2003). The use of this method has been suggested in Europe by EFNARC (2001) and in the US by the Precast/Prestressed Concrete Institute (PCI 2003).

The method generally consists of six steps. First, the desired air content is established. Typically, the air content is set at 2% unless air entrainment is required. Second, the coarse aggregate volume is set at 50 to 60% of the coarse aggregate bulk density. Thus, a coarse aggregate with a dry-rodded unit weight of 100 lb/ft³ would be used at 50 to 60 lb/ft³ in concrete, or 1350 to 1620 lb/yd³. The exact amount depends on the aggregate's maximum size and shape, with smaller aggregates and rounded aggregates used in higher volumes. Third, the sand volume

is set at 40-50% of the mortar volume. Alternatively, Okamura and Ozawa (1995) suggested that equal volumes of sand and coarse aggregate be used. Elsewhere, Edamatsu, Nishida, and Ouchi (1999) suggested a method for determining the optimum sand content. This method involves the use of the mini-v-funnel test and mini-slump flow test for mortars (Figure 2.16). Glass beads are added to these mortars to represent the interaction between sand and coarse aggregate. The ratio of mini-v-funnel flow time with and without the glass beads is evaluated to select the proper sand content. For the purposes of this method, material in the sand below a certain size is considered powder. Okamura and Ozawa (1995) recommend material finer 90 μm be considered powder while EFNARC (2001) recommends 125 μm .

Fourth, the water-powder ratio for zero flow in paste is determined by measuring the mini-slump flow in pastes at various w/p (1.1, 1.2, 1.3, and 1.4 by volume) and extrapolating the w/p for zero flow (β_p). The value of β_p typically ranges from 0.7 to 1.0 depending on the characteristics of the powder, which includes cement and any additions (Okamura and Ozawa 1995). The mini-slump cone typically used has a top diameter of 70 mm, a bottom diameter of 100 mm and a height of 60 mm (Figure 2.16). Fifth, the optimum water-powder ratio and HRWRA dosage are determined in the paste, based on measurements with the mini-slump cone and mini-v-funnel (Figure 2.16). Various water-powder ratios in the range of 0.8 to 0.9 β_p are used to reach a target mini-slump flow and mini-v-funnel flow time. Generally, the water-powder ratio is changed in order to modify mini-v-funnel flow time, which is related to viscosity. EFNARC (2001) suggested a target mini-slump flow of 240 to 260 mm and a mini-v-funnel time of 7-11 seconds. Ouchi, Hibino, and Okamura (1997) suggested a target value for mini-slump flow of 245 mm and a target mini-v-funnel flow time of 10 seconds. Sixth, tests are performed on trial batches of concrete to finalize the mixture proportions.

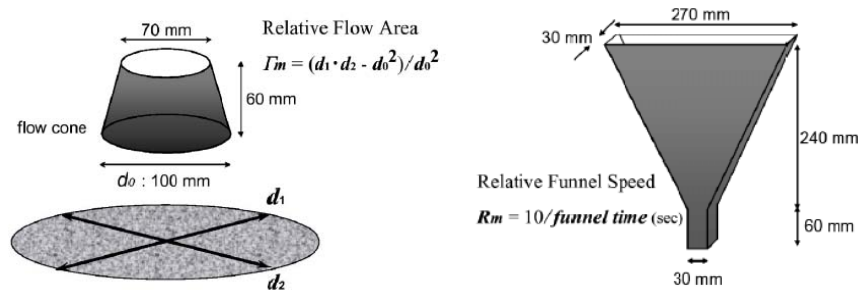


Figure 2.16 Mini-Slump Flow Cone and Mini-V-Funnel Used to Evaluate Paste Properties in the Rational Mixture Design Method (Okamura 2003)

2.5.2.10 Statistical Design of Experiments Approach

Multiple researchers have used statistical design of experiments (DOE) techniques to evaluate the effects of mixture proportions, select trial proportions, and optimize proportions. DOE techniques provide a way to evaluate the effects of different factors in a statistically sound manner and with a minimum number of mixtures. Regression models are fitted to the results of each measured response. A summary of four such approaches is presented in Table 2.9.

A central composite response surface is the most commonly used approach. Some prior knowledge of both the materials to be used and SCC proportioning is required to select the

values of factors used in the experiment design such that all or most mixtures exhibit SCC or near-SCC flow characteristics. Although the absolute values of the modeled responses may change when different materials are used, the general relative trends illustrated for a certain set of materials and proportions may remain consistent when a different set of materials is used (Ghezal and Khayat 2002).

Similarly, Nehdi, Chabib, and Naggar (2001), developed artificial neural networks to predict SCC performance based on mixture proportions. The values of slump flow, filling capacity, segregation resistance, and 28-day compressive strength were modeled. The success of the model was limited by the amount of data used to train the model. It was suggested that the model could be used in mixture proportioning to limit the number of laboratory trial batches. Mixture proportions could be created and tested in the artificial neural network model to select mixtures to achieve the required properties.

Table 2.9 Summary of Statistical Design of Experiments Approaches

Reference	Experiment Design	Factors	Other Parameters	Responses
Khayat, Ghezal, and Hadriche (1999)	Central composite response surface with 5 factors and 32 points (16 fractional factorial points, 10 star points, 6 center points)	Water-cement ratio (0.37-0.50), cementitious materials content (360-600 kg/m ³), viscosity enhancing agent dosage (0.05-0.20% by mass of water), HRWRA dosage (0.30-1.10% by mass of binder), volume of coarse aggregate (240-400 l/m ³)	Fine aggregate content varied to achieve volume	Slump flow, rheological parameters (IBB rheometer), filling capacity, v-funnel flow time, surface settlement, compressive strength at 7 and 28 days
Ghezal and Khayat (2001); Ghezal and Khayat (2002)	Central composite response surface with 4 factors and 21 points (8 fractional factorial points, 8 star points, 5 central points)	HRWRA content (0.12 to 0.65% by mass of powder), cement content (250-400 kg/m ³), limestone filler content (0-120 kg/m ³), water-powder ratio (0.38-0.72)	Coarse aggregate content held constant, fine aggregate content varied to achieve volume; constant dosage of VMA in all mixtures	Initial slump flow, slump flow after 45 minutes, rheological parameters (IBB), v-funnel flow time, surface settlement, compressive strength at 1 and 28 days
Sonebi, Bahadori-Jahromi, and Bartos (2003); Sonebi (2004a); Sonebi (2004b)	Central composite response surface with 4 factors and 21 points (8 fractional factorial points, 8 star points, 5 central points)	Cement content (183-317 kg/m ³), fly ash content (59-261 kg/m ³), HRWRA dosage (0-1% by mass of powder), water-powder ratio (0.38-0.72)	Coarse aggregate content held constant, fine aggregate content varied to achieve volume	Slump flow, loss of fluidity, orimet flow time, v-funnel flow time, l-box H ₁ , l-box ratio, j-ring with Orimet, j-ring with slump cone, rheological parameters (IBB rheometer), compressive strength at 7, 28, and 90 days
Patel et al. (2004)	Central composite response surface with 4 factors and 21 points (8 fractional factorial points, 8 star points, 5 central points)	Total binder content (350-450 kg/m ³), fly ash content (30-60% mass replacement of cement), HRWRA content (0.1 to 0.6% by mass of cementing materials), water-binder ratio (0.33-0.45)	Coarse aggregate content held constant, fine aggregate content varied to achieve volume	Slump flow, compressive strength at 1 and 28 days, rapid chloride permeability (other properties were measured but not modeled statistically)

2.5.2.11 Su, Hsu, and Chai (2001) Method

The mixture proportioning method developed by Su, Hsu, and Chai (2001) consists of selecting the aggregate volume and then filling the voids between aggregates with paste of the appropriate composition.

In the first step, the coarse and fine aggregates are proportioned based on their loosely packed densities, which are determined in accordance with ASTM C 29 but with the aggregates dropped from a height of 300 mm. The masses of coarse and fine aggregates in concrete are increased by a packing factor (PF), which reflects the increase in packing density of the aggregates in actual concrete mixtures. The packing factor is defined as the ratio of the mass of tightly packed aggregate in concrete to the mass of loosely packed aggregate. It is chosen by the designer, with higher packing factors associated with higher aggregate contents. In an example, Su, Hsu and Chai showed that increasing the packing factor resulted in a lower paste volume with a higher w/cm and a lower concrete strength. The masses of coarse (W_{coarse}) and fine (W_{fine}) aggregates are calculated based on Equations (2.25) and (2.26):

$$W_{coarse} = PF \times UW_{coarse-loose} \times \left(1 - \frac{S}{A}\right) \quad (2.25)$$

$$W_{fine} = PF \times UW_{fine-loose} \times \frac{S}{A} \quad (2.26)$$

where $UW_{coarse-loose}$ and $UW_{fine-loose}$ are the loosely packed densities of coarse and fine aggregates, respectively, and S/A is the sand-aggregate ratio. In the second step, the cement content is selected based on strength requirements, as shown in Equation (2.27):

$$C = \frac{f'_c}{20} \quad (2.27)$$

where C is the cement content in kg/m^3 and f'_c is the compressive strength in psi. This relationship is based on empirical data from Taiwan and may vary for other regions. In the third step, the water content required by the cement is calculated from the water-cement ratio needed for strength. Thus, the water content for cement is the required water-cement ratio multiplied by the cement content. In the fourth step, the total volume of fly ash paste and slag paste to fill the remaining volume of the concrete is determined. The flow table test is used to determine separately the water-fly ash and water-slag ratios to achieve the same flow as the cement paste already selected. In step 5, the total mixing water is calculated as the sum of the water contents required for the fly ash, slag, and cement pastes. Lastly, trial batches are evaluated and the proportions are adjusted.

2.5.2.12 Swedish Cement and Concrete Research Institute (CBI) Model

The Swedish Cement and Concrete Research Institute (CBI) Model is based on the assumption that SCC is a suspension of aggregates in paste (Billberg 2002). The model incorporates aspects of the Minimum Paste Volume Method developed by Van Bui (Bui and Montgomery 1999). The CBI model splits concrete between the solid fraction, which consists of all particles greater than 0.125 mm, and the “micro-mortar” fraction, which consists of water, admixtures, air, and particles smaller than 0.125 mm. The model can be used to fulfill requirements for rheology and passing ability.

First, design and detailing criteria and contractor requirements are considered. Design criteria may include requirements for strength and durability, which may impose limits on parameters such as water-cement ratio, sand-aggregate ratio, and air content. Detailing

requirements involve geometrical limitations due to formwork geometry and reinforcement spacing. Contractor requirements may include the rate of strength development and rate of slump flow loss.

With these criteria evaluated, the first step in selecting mixture proportions is to set the ratio of coarse-to-total aggregate and the minimum micro-mortar volume. These two parameters are based on the blocking criteria and the void content of the aggregate. The minimum paste volume for blocking is selected based on the criteria presented by Bui and Montgomery (1999) and subsequently modified by CBI. Petersson and Billberg (1999) found that adding a viscosity modifying admixture enabled only a small reduction in paste volume. The minimum volume of micro-mortar for blocking criteria increases with increasing coarse aggregate-to-total aggregate ratio, decreasing clear spacing between reinforcement, or increasing aggregate angularity. The dry-rodded void content is measured at various ratios of coarse-to-total aggregate in order to evaluate the ratio with the minimum void content. The actual minimum micro-mortar content is greater than the dry-rodded void content in the aggregate. Second, the micro-mortar rheology is established based on rheometer measurements. Third, the performance of trial concrete mixtures is evaluated. The slump flow with T_{50} and VSI and the l-box are used to evaluate fresh concrete properties.

2.5.2.13 Technical Center of Italcementi Group (CTG) Method

The CTG Mixture Proportioning Method was developed at the Technical Center of Italcementi Group (CTG) in 1997 and has been used worldwide by Italcementi Group (Vachon, Kaplan, and Fellaki 2002).

The method involves four steps. First, the paste composition is designed for strength requirements. Second, the paste volume is selected to achieve necessary fluidity and resist segregation. This paste volume—which constitutes water, air, and all particles smaller than 80 μm —is set at 37% in most cases as a starting point. Third, the aggregate is selected to prevent segregation and blocking. Fourth, the HRWRA dosage and, if necessary, the VMA dosage are selected. Values for the paste content and aggregate grading are established empirically based on testing or previous experience.

2.5.2.14 University of Rostock (Germany) Method

The mixture proportioning method developed at the University of Rostock in Germany aims to determine the optimum water content for SCC based on the water demand of the individual solid components (Marquardt, Diederichs, and Vala 2001; Marquardt, Diederichs, and Vala 2002). Additionally, the paste volume is selected based on the voids content of the aggregates and the HRWRA dosage is adjusted to achieve sufficient fluidity.

In the first step, the aggregate grading is selected. The aggregate grading should have sufficient sand volume and high packing density. In the second step, the volumes of paste and aggregate in the concrete are determined. The concrete is assumed to comprise three volumes: the volume of aggregate (V_g), the volume of the paste required to fill the voids between the aggregates (V_{LHP}), and the volume of surplus paste (V_{LU}). The total paste volume ($V_L = V_{LHP} + V_{LU}$) is related to V_{LHP} by a factor κ :

$$\kappa = \frac{V_L}{V_{LHP}} \quad (2.28)$$

The value of κ depends on the shape and size of the aggregates and is normally set between 1.9 and 2.1 for SCC. To determine the total paste volume needed, the compacted volume of aggregate (V_{G0}) and volume of voids between the compacted aggregates (V_{HP0}) are measured for the selected grading. The values of V_{LHP} , V_G , and V_L are calculated based on Equations (2.29) to (2.31):

$$V_{LHP} = \frac{1000}{\kappa + \frac{V_{G0}}{V_{HP0}}} \quad (2.29)$$

$$V_G = \frac{V_{LHP} V_{G0}}{V_{HP0}} \quad (2.30)$$

$$V_L = \kappa V_{LHP} \quad (2.31)$$

In the third step, the cement type and quantity is selected based on hardened property requirements. For the fourth step, the types of additives, such as fly ash or limestone powder; the type of HRWRA; the type of VMA; and the air void content are selected. In the fifth step, the water demand of all solid components is determined. For the aggregates, the water demand is calculated as the water needed to cover all particle surfaces with a thin layer of water and to partially fill the space between particles. It can be determined either by centrifugation of water-saturated aggregates or based on the specific surface area of the aggregates. The water demand of powder constituents is determined by measuring the power consumption of a mixer as water is gradually added to the powder. As water is added, the power consumption increases from a minimum value when only powder is in the mixer to a maximum value before beginning to decrease. The water content corresponding to the maximum power consumption is considered the water demand. The water demand is determined separately for each powder constituent.

The sixth step involves calculating the mixture proportions. The total water content is the sum of the water demand from the aggregate and each individual powder constituent. The volumes of additives such as fly ash are adjusted to achieve proper total concrete volume. In the final step, trial mixtures are evaluated. The flowability of the mixture is adjusted by changing the dosage of HRWRA.

2.6 Summary

The mixture proportioning methods described in this chapter are summarized in Table 2.10. Although each mixture proportioning method takes a different approach, the methods do share some similarities. Most methods—with the exception of the Rational Mix Design Method and the referenced statistical design of experiments test plans—assume that SCC is a suspension of aggregates in paste. These methods must establish three things: the paste volume, paste composition, and aggregate blend. The paste volume is set to be greater than the volume of the

voids between the compacted aggregates. The methods of compacting the aggregates and of selecting the paste volume vary with each method. The paste composition is usually designed independently of the rest of the mixture based on measurements of flow properties, hardened properties, or both. Each method uses a different series of tests and has different target values for selecting the paste composition. Some methods are very specific about the target paste properties while others are much more open-ended. The aggregate blends are often, but not always, selected to achieve the minimum voids between the aggregates. In the final step, the paste volume, paste composition, and aggregate blend are combined for the preliminary, trial concrete batch or batches.

The approach of assuming that SCC is a suspension may be limited in some but not all cases due to the ways this approach has been implemented. For example, when the optimized paste volume and paste composition, which are typically determined separately, are combined in the concrete mixture proportions, the concrete rheology may not be optimum. Furthermore, when the aggregate blend is selected on the basis of minimizing the voids content, the resulting concrete flow properties may not be ideal. Some of the mixture proportioning methods are not flexible or provide limited guidance in allowing the paste volume, paste composition, and aggregate blends to be modified when tested together in the combined concrete mixture proportions.

In some methods, the aggregate void content is assumed to account for all aggregate properties—including packing, size, grading, shape, angularity, and texture. Other methods assign an additional factor or measure additional properties (such as surface area) to account for some of these other aggregate properties. It is not clear whether these approaches are sufficient for capturing the aggregate properties. For instance, a crushed aggregate and rounded aggregate, each with the same voids content, would likely result in much different workability.

Given the wide range of materials that are used in producing SCC, the ability to modify concrete mixture proportions efficiently once the initial trial batch is computed is crucial to ensuring successful mixture proportions. In fact, it is unreasonable to expect a mixture proportioning method to result in the optimized proportions initially without subsequent modifications based on measurements of concrete mixtures. Most methods, however, provide little if any guidance on modifying the initial trial proportions.

The methods also vary widely in their level of completeness. Some of the methods provide limited guidance for selecting and varying the values of some key parameters, which increases the number of concrete tests required to establish the effects of these parameters. Other methods focus on specific applications, such as high strength concrete, and do not provide guidance for other applications.

Table 2.10 Summary of SCC Mixture Proportioning Techniques

Method	Basic Concepts	Development	Unique Features	Limitations
ACBM Paste Rheology Model/Minimum Paste Volume Method	The minimum paste volume is selected based on either the solid phase (blocking) or liquid phase (segregation, flowability, form surface finishability) criteria. The paste rheology is then determined on the basis of laboratory testing. The concept of a self-flow zone, defined in terms of paste yield stress and apparent viscosity, is introduced to ensure segregation resistance and flowability.	The method was developed by multiple researchers working at different times. Bui pioneered Minimum Paste Volume Method and combined it with other work done at ACBM, including that of Saak.	The method provides detailed equations to compute the paste volume required for blocking resistance. Equations are also available for liquid phase criteria; however, assumptions must be made regarding average spacing between aggregates.	Limited guidance is available for selecting the average spacing between aggregates and for optimizing paste rheology.
Compressible Packing Model	Proportioning is based on a packing model. Equations are available for computing yield stress, plastic viscosity, a parameter representing filling/passing ability, and a parameter representing segregation resistance.	The method is based on the compressible packing model published by de Larrard. It has been expanded for SCC with the inclusion of a parameter describing filling/passing ability.	The method uses a detailed packing model to optimize aggregates and includes the ability to compute yield stress, plastic viscosity, and parameters for filling/passing ability and segregation resistance.	The use of the model requires proprietary software. The calculation of yield stress and plastic viscosity is based on empirical measurements with the BTRHEOM, which typically gives higher values than other rheometers.
Concrete Manager Software	The method combines a packing model and the Mooney equation for relative viscosity to predict workability. The software program optimizes the trial mixtures.	The method was developed by Roshavelov and incorporated into a software package. It is similar to the solid suspension model/compressible packing model proposed by de Larrard.	The method includes the ability to predict apparent viscosity. A unique capillary rheometer is used to evaluate trial concrete batches.	The selection of proportions must be completed in the software. The necessary calculations are complex.

Summary of SCC Mixture Proportioning Techniques (Continued)

Method	Basic Concepts	Development	Unique Features	Limitations
Densified Mixture Design Algorithm	The optimum blend of aggregate and fly ash resulting in the lowest voids content is selected. The paste volume is set as the volume of voids between the aggregates and fly ash, increased by a factor N . The composition of the paste is selected for hardened properties.	The method was developed in Taiwan for high-performance concrete and has been extended to SCC.	Fly ash is considered as part of the aggregate and not the paste.	The method is primarily intended for high-strength concrete. The aggregate/fly ash combination giving the minimum voids content may not be optimal for workability.
Excess Paste Theory	The relative thickness of excess paste is computed and used to predict the yield stress and plastic viscosity of the concrete relative to the paste.	The method is based on the excess paste theory originally proposed by Kennedy in 1940. This theory has been used by other researchers since then for both conventionally placed concrete and SCC.	The model has been shown to predict both yield stress and plastic viscosity of SCC accurately, based on the aggregate properties, paste volume, and paste rheology.	Various approaches are available for determining aggregate surface area. The approach suggested by the authors is computationally intensive, especially when fine aggregate is considered. The yield stress and plastic viscosity must be determined in a consistent manner on both the paste and concrete so that they can be related.
Gomez et al. (2001) High-Strength SCC Method	The optimum paste composition is determined with the Marsh funnel and mini-slump cone, subject to limits on strength. The blend of aggregates resulting in the lowest voids content is selected. Various paste volumes are tested to achieve the optimum workability and compressive strength.	The method was developed based on previous concepts for proportioning high-strength and high-performance concrete.	The application of procedures to optimize the paste composition to achieve high strength is unique for SCC.	The method is mainly intended for high-strength SCC. The aggregate combination giving the minimum voids content may not be optimal for workability. The approach for selecting the optimum paste composition may not result in the lowest paste volume. Accordingly, in selecting the paste volume, it may be appropriate to alter the paste composition to achieve lower paste volume.

Summary of SCC Mixture Proportioning Techniques (Continued)

Method	Basic Concepts	Development	Unique Features	Limitations
Particle-Matrix Model	The model is based on paste volume, paste rheology, and aggregate properties. The paste rheology is characterized with the flow resistance ratio, which is measured with the FlowCyl device. The aggregates are characterized with the air voids modulus, which depends on the aggregate volumes, fineness moduli, and empirically determined aggregate parameters. Workability is measured at various paste volumes for each flow resistance ratio and air void modulus. The resulting equations are used to predict the effects of changes in mixture proportions.	The model was originally developed by Ernst Mortsell for conventionally placed concrete and has been extended to SCC with mixed success.	The flow resistance ratio and air voids modulus are unique parameters describing the paste rheology and aggregate characteristics, respectively.	The model has been applied with mixed success. The developer of the model has stated that more work is needed in optimizing the paste rheology. The air void modulus is complicated to compute, particularly in determining the aggregate parameters. The flow resistance ratio may not be the best parameter to characterize paste rheology. The flow resistance ratio and air voids modulus have limited physical meanings.
Rational Mix Design Method	The coarse aggregate content in the concrete is set to 50 to 60% of the coarse aggregate bulk density. The fine aggregate content in the mortar is set to 40-50% of the mortar volume. The water-powder ratio and HRWRA dosage are determined with mini-slump flow and mini-v-funnel measurements of paste in order to reach prescribed target values.	The method was originally developed in Japan and has been presented by various researchers. The method has evolved; however, the basic principles remain the same. The use of the method has been suggested by organizations around the world, including EFNARC and PCI.	The fine and coarse aggregate contents are selected based on specific multiples of bulk density. A unique method of selecting optimum paste rheology with the mini-slump flow test and mini-v-funnel is provided.	The method is rather restrictive in the way it sets the coarse and fine aggregate contents and establishes the target paste flow properties. The resulting proportions may not be optimal in concrete.
Statistical Design of Experiments Approach	Statistical design of experiments techniques are used to evaluate the effects of 4-5 parameters in a statistically efficient way. Regression models are used to evaluate data and optimize proportions.	The statistical concepts are well-known and widely used in many industries. They have been implemented for SCC by multiple researchers.		The resulting regression models are specific to only the materials and range of proportions considered. In some cases, many of the mixtures in the reported test plans do not exhibit SCC flow characteristics. Some prior knowledge of the materials and SCC proportioning is required to establish the test plan.

Summary of SCC Mixture Proportioning Techniques (Continued)

Method	Basic Concepts	Development	Unique Features	Limitations
Su, Hsu, and Chai Method	The fine and coarse aggregates are set as the loosely packed densities, increased by a packing factor. The cement content and water-cement ratio are selected based on strength requirements. Fly ash and slag pastes are added to fill the remaining volume. The water demand of fly ash and slag are determined separately with the flow table test.	The method was originally developed in Taiwan.	The method uses a packing factor to select the contents of sand and coarse aggregate.	Not all of the values needed for selecting initial proportions are well defined. Several factors such as the packing factor, sand-aggregate ratio, and relative amounts of slag and fly ash must be chosen <i>a priori</i> by the designer; however, little or no guidance is given. The water is selected in three separate processes and does not take into consideration the combined effect of the total water content on strength or workability until trial concrete proportions are evaluated.
Swedish Cement and Concrete Research Institute (CBI) Model	The blend of fine and coarse aggregate is selected to achieve the minimum void content. The paste volume is selected based on the voids between the aggregate or the blocking criteria. The paste composition is selected based on rheology measurements. The mixture is finalized based on trial concrete batches.	The method was developed at the Swedish Cement and Concrete Research Institute. It is based in part on work done previously by Van Bui in the Minimum Paste Volume Method.	The method has detailed criteria for ensuring passing ability.	Criteria for the selection of micro-mortar rheology and the amount of micro-mortar are not well-established.
Technical Center of Italcementi (CTG) Method	The paste composition is designed for strength and the paste volume is set for workability. The aggregates are selected to achieve segregation and blocking resistance.	The method was developed by Italcementi in France and used by the company throughout the world.		The method is simple and typically relies on previous empirical experience.

Summary of SCC Mixture Proportioning Techniques (Continued)

Method	Basic Concepts	Development	Unique Features	Limitations
University of Rostock (Germany) Method	The aggregate blend is selected and the paste volume is selected based on a factor κ , which depends on the size and shape of the aggregates. The water demand of each solid component is determined separately. These water contents are added to select the total water content and establish the final mixture proportions.	The method was developed at the University of Rostock in Germany.	The method evaluates the voids between the aggregate, but does not suggest that the aggregate blend with minimum voids be selected. The method uses unique approaches for the selection of the paste volume and the determination of the water demand of the solid components	The method computes a single water content, which may produce an inappropriate viscosity. Further, the water demand of the concrete mixture may vary as the paste volume is varied. Limited guidance is given on selecting an aggregate blend. The determination of water demand for aggregates by centrifugation may not be feasible for all labs.

3. Materials and Requirements for SCC Mixture Proportions

SCC mixtures were developed to cover a wide portion of the range of materials and mixture proportions likely to be used by precasters in Texas for prestressed concrete bridge beams. Materials and mixture proportions were systematically altered over this range to evaluate their effects on workability and hardened properties. The specific materials and mixture proportioning requirements are described in this chapter.

3.1 Materials

Materials were selected to be broadly representative of those used by precasters in Texas for prestressed concrete bridge beams. A survey of the precasters producing prestressed concrete bridge beams in Texas was conducted to identify classes of materials and specific sources of materials. The materials selected are shown in Table 3.1. Primary materials were used in all mixtures unless noted otherwise.

- *Aggregates.* Two aggregate sets were selected—one with river gravel coarse aggregate (RG) and the other with crushed limestone coarse aggregate (LS). Both coarse aggregates had maximum aggregate sizes of $\frac{3}{4}$ inch. Separate natural sands were selected for each aggregate set (NS-A and NS-B). Complete series of mixture proportions were developed for each aggregate set. The properties of the aggregates are shown in Table 3.2.
- *Cement.* The two Type III cements (ASTM C 150) most commonly used by precasters in Texas were selected. One was used as the primary cement (PC-A); the other was evaluated as an alternate as part of a sensitivity analysis (PC-B). The properties of the cements are shown in Table 3.3.
- *Fly Ash.* At the time of the survey, Class F fly ash (ASTM C 618) was used predominately. A commonly used Class F fly ash was used as the primary fly ash (FA-A); another Class F fly ash was evaluated as an alternate as part of a sensitivity analysis (FA-B). The primary Class F fly ash came from a coal power plant using “low-NO_x burner” technology and was not treated or modified further to mitigate the changes in fly ash quality due to the use of “low-NO_x burner” technology. In addition, one concrete mixture was tested with a Class C fly ash (FA-C) and three mixtures were tested with an ultra-fine fly ash (UFFA). The properties of fly ashes FA-A and FA-B are shown in Table 3.4.
- *HRWRA.* Only polycarboxylate-based HRWRAs were evaluated. The exact commercial formulations available changed over the course of the project as new products were introduced and older products were removed from the market. A single HRWRA was selected as the primary HRWRA and used throughout the project (HR-A); three other HRWRA were evaluated as alternates (HR-B, HR-C, HR-D). A fifth was used for conventionally placed concrete mixtures (HR-E). All HRWRAs were intended for precast concrete.
- *Mid-Range Water-Reducing Admixture.* One mid-range water-reducing admixture (MR-A) was evaluated.
- *Accelerator.* Three accelerators were evaluated.

- *Retarder*. Retarders were used for the purpose of ensuring workability retention. Four retarders were evaluated for their effects on workability retention. A single retarder was selected for use in all mixtures requiring a retarder (RET-A).
- *VMA*. A polysaccharide-type VMA (VMA-A) and a methyl-cellulose-type VMA (VMA-B) were evaluated.

Table 3.1 Materials

	Type	Source
Aggregate Set 1	Coarse: River Gravel (RG)	Fordyce-Murphy Pit (Victoria, TX)
	Fine: Natural Sand (NS-A)	Fordyce-Murphy Pit (Victoria, TX)
Aggregate Set 2	Coarse: Crushed Limestone (LS)	Hanson Aggregates (Garden Ridge, TX)
	Fine: Natural Sand (NS-B)	Austin Sand and Gravel (Austin, TX)
Cement	Primary: Type III (PC-A)	Alamo Cement (San Antonio, TX)
	Alternate: Type III (PC-B)	Capitol Cement (San Antonio, TX)
Fly Ash	Primary: Class F (FA-A)	Boral-Sandow Plant (Rockdale, TX)
	Alternate: Class F (FA-B)	Headwaters-LEGS Plant (Jewett, TX)
	Alternate: Class C (FA-C)	Headwaters-W.A. Parish Plant (Thompsons, TX)
	Alternate: Ultra-Fine Fly Ash (UFFA)	Boral-Sandow Plant (Rockdale, TX)
HRWRA (polycarboxylate-based)	Primary: Glenium 3400 NV (HR-A)	BASF Construction Chemicals
	Alternate: ViscoCrete 2100 (HR-B)	Sika Corporation
	Alternate: ADVA Cast 530 (HR-C)	W.R. Grace & Co.
	Alternate: ADVA Cast 555 (HR-D)	W.R. Grace & Co.
	Conv. Concrete: PS-1466 (HR-E)	BASF Construction Chemicals
MRWRA	Polyheed 997 (MR-A)	BASF Construction Chemicals
Accelerator	Sika Set NC (ACC-A)	Sika Corporation
	Pozzolith NC 534 (ACC-B)	BASF Construction Chemicals
	Pozzutec 20+ (ACC-C)	BASF Construction Chemicals
Retarder	Primary: Delvo Stabilizer (RET-A)	BASF Construction Chemicals
	Alternate: Pozzolith 300R (RET-B)	BASF Construction Chemicals
	Alternate: Plastiment (RET-C)	Sika Corporation
	Alternate: Daratard 17 (RET-D)	W.R. Grace & Co.
VMA	Polysaccharide, Rheomac VMA 362 (VMA-A)	BASF Construction Chemicals
	Cellulose, Rheomac VMA 450 (VMA-B)	BASF Construction Chemicals

Table 3.2 Aggregate Properties

		River Gravel Set		Crushed Limestone Set	
		River Gravel (RG)	Natural Sand (NS-A)	Crushed Limestone (LS)	Natural Sand (NS-B)
Source		Fordyce Ltd.	Fordyce Ltd.	Hanson Aggregates	Austin Sand and Gravel
Location		Victoria, TX	Victoria, TX	Garden Ridge, TX	Austin, TX
ASTM C 33 Designation		#67	Sand	#67	Sand
Specific Gravity (SSD)		2.59	2.58	2.59	2.60
Absorption Capacity, %		0.78	0.54	1.43	0.56
Dry-Rodded Unit Wt., lb/yd ³		105.4	106.3	93.4	108.9
Compacted Voids Content, %		34.2	33.6	41.4	32.4
Methylene Blue Value, mg/g			18.0		7.1
Fineness Modulus			2.72		2.58
Grading (% Passing)	1" (25 mm)	100.0		100.0	
	¾" (19 mm)	94.8		95.1	
	½" (13 mm)	64.3		70.5	
	⅜" (9.5 mm)	40.4		38.2	
	#4	7.8	99.0	0.2	98.5
	#8	2.2	86.5		86.9
	#16	1.5	73.8		75.1
	#30	1.2	49.3		53.4
	#50		16.0		20.9
	#100		3.8		6.8
	#200		1.2		1.8
Data obtained with following tests: specific gravity and absorption (ASTM C 127 for coarse, ASTM C 128 for fine); unit weight and voids content (ASTM C 29); methylene blue value (AASHTO TP 57 for material passing #200 sieve obtained by dry sieving); sieve analysis (ASTM C 136), % passing #200 sieve (ASTM C 117)					

Table 3.3 Cement and Fly Ash Properties

	PC-A	PC-B	FA-A	FA-B
Source	Alamo Cement	Capitol Cement	Boral Material Technologies	Headwaters Resources
Location	San Antonio, TX	San Antonio, TX	Rockdale, TX (Sandow)	Jewett, TX (LEGS)
Chemical Tests				
Silicon Dioxide (SiO ₂), %	20.6	20.09	52.49	55.11
Aluminum Oxide (Al ₂ O ₃), %	4.9	4.87	21.78	20.42
Iron Oxide (Fe ₂ O ₃), %	3.4	1.87	4.94	8.18
Calcium Oxide (CaO), %	64.1	63.43	13.92	9.90
Magnesium Oxide (MgO), %	0.8	1.24	2.00	2.72
Sulfur Trioxide (SO ₃), %	3.5	4.34	0.79	0.54
Sodium Oxide (Na ₂ O), %			0.32	
Potassium Oxide (K ₂ O), %			0.74	
Total Alkalies (as Na ₂ O _{eq}), %	0.50	0.54	0.81	
Available Alkalies (as Na ₂ O _{eq}), %			0.24	0.46
Free Lime, %	1.5			
Insoluble Residue, %	0.57	0.10		
C ₃ S, %	56.6	57.79		
C ₂ S, %	16.3	14.02		
C ₃ A, %	7.2	9.73		
C ₄ AF, %	10.3	5.69		
Physical Tests				
Fineness				
Wagner, m ² /kg	264	274		
Blaine, m ² /kg	539	552		
Setting Time				
Initial (Gilmore), min	110	105		
Final (Gilmore), min	210	148		
Initial (Vicat), min		63		
Final (Vicat), min		101		
Compressive Strength				
1 day, MPa	24.1	26.8		
3 day, MPa	32.6	37.5		
7 day, MPa	39.1	42.9		
28 day, MPa	46.8	48.8		
Air Content, %	6	7.40		
Moisture Content, %			0.26	0.07
False Set, %	73			
Loss on Ignition, %	2.1	2.47	1.05	0.11
Amount Retained on #325 Sieve, %	0.9	0.9	27.68	28.92
Specific Gravity			2.33	2.39
Autoclave Soundness, %	-0.02	0.00	0.07	-0.03
Strength Activity Index (7 day), %			73.6	80
Strength Activity Index (28 day), %			82.0	96
Water Required, %			93.8	96

The primary fly ash was initially tested both with and without a chemical treatment applied by the distributor of the fly ash. The purpose of the chemical treatment was to mitigate variations in entrained-air content, which can be caused by the use of “low-NOx burner” technology in coal-burning power plants. This chemical treatment is only necessary in air-entrained content. Although concrete used in prestressed bridge beams in Texas is not typically air entrained, the use of chemically treated fly ash was considered because of the possibility that prestressed concrete may be air-entrained in some situations.

Preliminary testing with the treated fly ash indicated that concrete mixtures with high volumes of fly ash (30% replacement rate) and the high HRWRA dosages necessary to achieve SCC flow properties resulted in high air void contents. For example, Table 3.4 shows that the use of 30% treated fly ash increased the air content from 2.0% to 8.0%. In contrast, the use of the same quantity of untreated fly ash resulted in essentially no change in air content. Additionally, the use of treated fly ash without HRWRA did not result in a substantial increase in HRWRA demand. The mixtures shown in Table 3.4 incorporated HR-C; however, the same trends were observed with HR-A. For mixtures with HR-A, the air content was found to increase with increased dosages of HRWRA in a given mixture with treated fly ash and to decrease over time with additional mixing.

It is known that polycarboxylate-based HRWRAs can inherently entrain air in concrete. Therefore, they typically include an anti-foaming agent to offset this characteristic. Based on the laboratory data and discussions with the fly ash distributor and HRWRA manufacturers, it was concluded that the most likely cause of the high air contents was the chemical treatment on the fly ash interacting with the anti-foaming agent in the HRWRA. Consequently, fly ash with the particular chemical treatment applied to the primary fly ash should not be used in SCC applications. For cases where air entrainment is not required, the untreated fly ash can be used. In cases where air entrainment is required, the fly ash distributors and HRWRA manufacturers should be consulted to determine the best available products and techniques for ensuring consistent and proper air void system characteristics.

Table 3.4 Comparison of Effects of Fly Ash on Air Content

ID	Notes	Mixture Proportions (lb/yd ³)					HRWRA (oz/cwt)	Slump Flow (inches)	Air Content (%)
		Cement	Fly Ash	Coarse	Fine	Water			
1	Cement only	710.0	0.0	1776.1	1266.8	248.5	13.0	26.5	2.0
2	30% treated fly ash	490.0	210.0	1751.1	1249.0	245.0	9.5	27.5	8.0
3	30% untreated fly ash	490.0	210.0	1751.1	1249.0	245.0	10	29.0	1.9
4	30% treated fly ash; higher w/cm	470.5	201.7	1681.5	1199.4	302.4	--	--	2.7
HR-C used in mixtures 1-3									

3.2 Mixture Proportioning Requirements

SCC mixtures were proportioned for workability and hardened properties. The 2004 TxDOT specifications, which do not address SCC, were also considered.

3.2.1 Workability

For workability, mixtures were designed for filling ability, passing ability, and segregation resistance. The specific target workability properties are shown in Table 3.5. The workability test methods are presented in Appendix A.

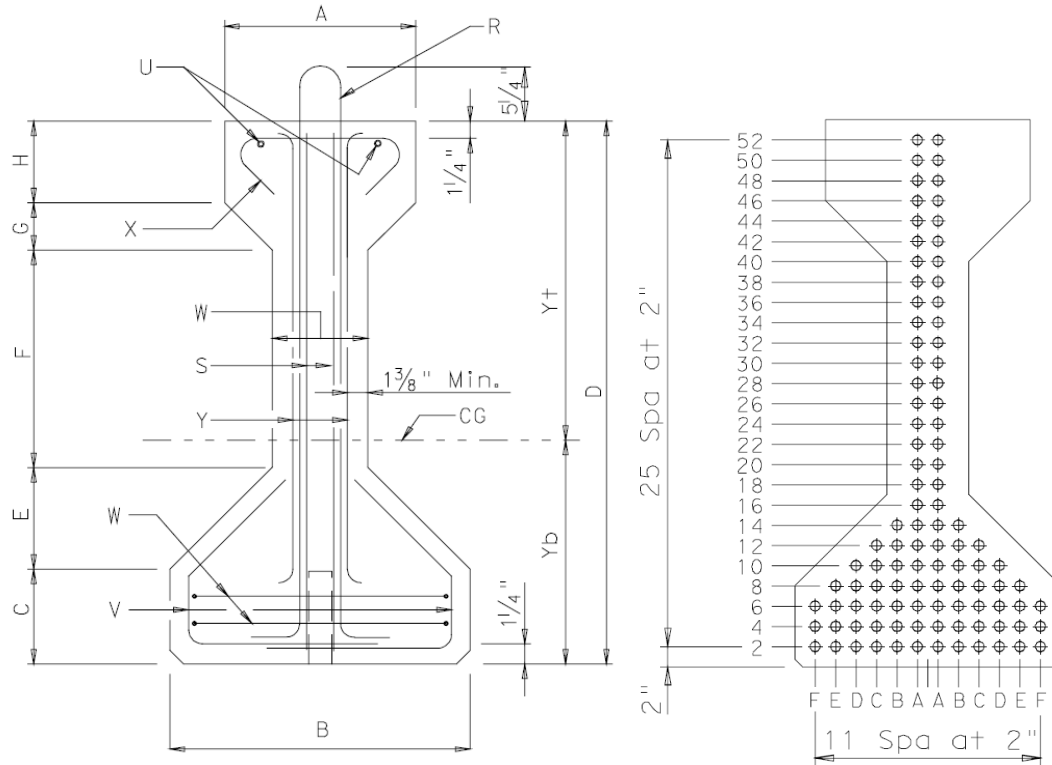
Table 3.5 Target Workability Properties

Property	Test Method	Requirement
Filling Ability	Slump Flow	Achieve a slump flow of 28-30 inches with $VSI \leq 1.0$ and $3 s < T_{50} < 7 s$; maintain a 23-inch slump flow 20 minutes after mixing.
Passing Ability	J-Ring	For an AASHTO Type IV beam, strands ~1.5-inch clear spacing. J-Ring $\Delta H \leq 0.50$ inches
Segregation Resistance	Column Segregation	Exhibit minimal segregation and top bar effect. Static segregation $\leq 15\%$.

Filling ability was specified in terms of the slump flow test. The initial slump flow was set relatively high (28 to 30 inches) to help ensure sufficient workability retention and robustness. The slump flow may not need to be 28 to 30 inches during normal production. If SCC can exhibit high slump flows without segregation, it will be even less likely to segregate at lower slump flows. Therefore, if the actual slump flow used during production is lower, the mixture will be robust and able to accommodate normal variations in slump flow. The difference in HRWRA dosage required to change the slump flow from 23 inches to 30 inches is normally small; therefore, such robustness is important. A minimum slump flow after 20 minutes of 23 inches, which is near the minimum needed for self-flow, was specified to ensure workability retention. The range of acceptable values for T_{50} was specified to ensure stability (minimum T_{50}) and ensure that the concrete was not too sticky and cohesive (maximum T_{50}).

Passing ability was selected based on typical clear spacings in an AASHTO Type IV beam section (Figure 3.1), which is commonly used by TxDOT (TxDOT 2001). Both 0.5- and 0.6-inch diameter strands are approved for use in this beam section. Passing ability was selected based on a maximum difference in height on the inside and outside of the j-ring of 0.50 inches. This difference in j-ring was measured with a slump flow of 28 to 30 inches, which is an important condition because increasing the slump flow can increase passing ability. The clear spacing between bars in the j-ring was 1.5 inches.

Segregation resistance was evaluated with the column segregation test. The maximum specified static segregation was 15%. The static segregation was evaluated with a slump flow of 28 to 30 inches, which is conservative because reducing the slump flow for a given mixture reduces the probability of segregation.



Beam Type	A in	B in	C in	D in	E in	F in	G in	H in	W in	Yt in	Yb in	Area in ²	I in ⁴	Wt/Lf lbs
A	12	16	5	28	5	11	3	4	6	15.39	12.61	275.4	22,658	287
B	12	18	6	34	5 3/4	14	2 3/4	5 1/2	6 1/2	19.07	14.93	360.3	43,177	375
C	14	22	7	40	7 1/2	16	3 1/2	6	7	22.91	17.09	494.9	82,602	516
IV	20	26	8	54	9	23	6	8	8	29.25	24.75	788.4	260,403	821

Figure 3.1 AASHTO Type IV Beam Dimensions and Strand Spacings (TxDOT 2001)

SCC was also evaluated in terms of rheology, which was measured with the ICAR rheometer. SCC must exhibit proper yield stress, plastic viscosity, and thixotropy. The yield stress must be near zero in order to ensure self-flow; however, a small yield stress is needed to prevent segregation. Therefore, a target yield stress of 10-80 Pa was used. The plastic viscosity should not be too low or too high. If the plastic viscosity is too low, dynamic stability can be reduced and static segregation can be exacerbated. If the plastic viscosity is too high, the concrete can be sticky and difficult to place. Increasing the plastic viscosity requires a lower yield stress for a given slump flow, which increases the risk of segregation. Therefore, a target plastic viscosity of 20-50 Pa.s was used. Thixotropy is essential to ensuring segregation resistance. The build-up of an at-rest structure due to thixotropy results in an increase in static, at-rest yield stress. The low dynamic yield stress needed for self-flow is typically insufficient to resist segregation. Therefore, it is important that concrete be thixotropic, resulting in a sufficiently fast build-up in the static yield stress. If the thixotropy is too great, however, the mixture can be impractical to place and result in cold joints.

3.2.2 Hardened Properties

Mixtures were designed for release of tension compressive strength, which was measured at 16 hours. According to TxDOT (2001, 7-78), release of tension strengths of 6,500 psi and 28-day strengths of 8,500 psi are “feasible for usual designs”. Because early-age strength development is highly dependent on temperature, mixtures were cured at a range of temperature histories selected to be representative of beams cured in Texas weather conditions throughout the year (Kehl and Carrasquillo 1998). These temperature histories, which are shown in Figure 3.2, vary in the pre-set time (time from casting until temperature increases), initial temperature, and maximum temperature by 16-hours. The temperature histories were imposed with a match curing system. In addition, 4 by 8-inch cylinders were cured with constant ambient temperatures of approximately 50, 72, and 95°F. Each mixture was identified in terms of its nominal 16-hour compressive strength, which was achieved with an 8-hour pre-set time, 75°F initial temperature, and 120°F maximum temperature. Mixtures were developed with nominal 16-hour compressive strengths of 5,000 and 7,000 psi (35 and 48 MPa). The mixtures varied in S/A, paste volume, and paste composition to evaluate the effects of these parameters on hardened properties.

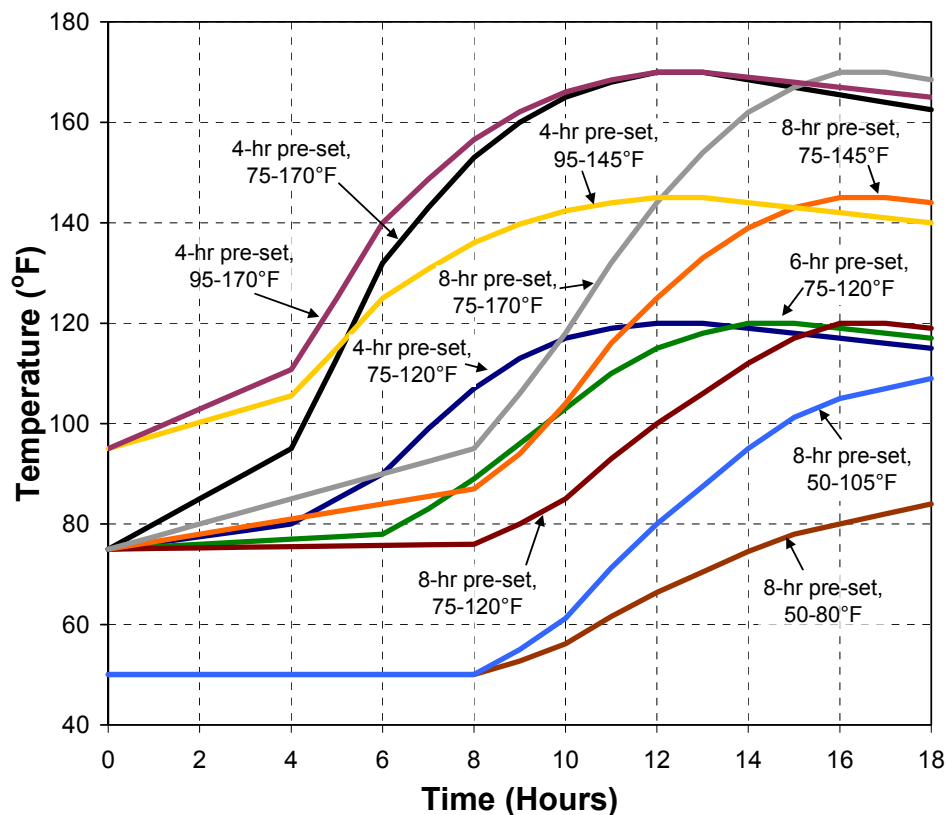


Figure 3.2 Imposed Curing Temperature Profiles

3.2.3 TxDOT Specifications

The 2004 TxDOT specifications were considered when developing SCC mixture proportions. However, these specifications were not developed for SCC, do not address SCC, and may not be appropriate for SCC. The following provisions are directly relevant to concrete mixture proportions for prestressed concrete bridge beams.

- **Section 421.4.A.** Prestressed concrete beams are to be made of Class H concrete, which is subject to a maximum water-cementitious materials ratio of 0.45 and must use coarse aggregate Grades 3-6, which refer to grading. Coarse aggregate Grades 4, 5, and 6 correspond to ASTM C 33 sizes #57, #67 and #7, respectively. Aggregate Grade 3 has a 1.5-inch nominal maximum size.
- **Section 421.4.A.1.** The maximum cementitious materials content is limited to no more than 700 lb/yd³ “unless otherwise specified or approved”.
- **Section 421.4.A.5.** The maximum acceptable placement slump for prestressed concrete members is 9 inches if HRWRA is used and 6.5 inches otherwise.
- **Section 421.4.A.6.** Mixture designs must utilize one of the eight following options.
 - **Option 1.** Replace 20 to 35% of the cement with Class F fly ash.
 - **Option 2.** Replace 35 to 50% of the cement with GGBFS.
 - **Option 3.** Replace 35 to 50% of the cement with a combination of Class F fly ash, GGBFS, or silica fume. However, no more than 35% may be fly ash, and no more than 10% may be silica fume.
 - **Option 4.** Use Type IP or Type IS cement. (Up to 10% of a Type IP or Type IS cement may be replaced with Class F fly ash, GGBFS, or silica fume.)
 - **Option 5.** Replace 35 to 50% of the cement with a combination of Class C fly ash and at least 6% silica fume, UFFA, or metakaolin. However, no more than 35% may be Class C fly ash and no more than 10% may be silica fume.
 - **Option 6.** Use a lithium nitrate admixture at a minimum dosage of 0.55 gal. of 30% lithium nitrate solution per pound of alkalis present in the hydraulic cement.
 - **Option 7.** When using hydraulic cement only, ensure that the total alkali contribution from the cement in the concrete does not exceed 4.00 lb. per cubic yard.
 - **Option 8.** For any deviations from Options 1-7, test both coarse and fine aggregate separately in accordance with ASTM C 1260, using 440 g of the proposed cementitious material in the same proportions of hydraulic cement to supplementary cementing material to be used in the mix. The maximum expansion for each aggregate must not exceed 0.10%.
- **Section 424.3.B.4.** Air-entrained concrete is not required for precast concrete members unless otherwise shown on the plans. High-strength concrete ($f'_c > 9,000$ psi) is accepted on the basis of 56-day compressive strength testing.
- **Section 424.3.B.5.** Concrete should be placed “as near as possible to its final position in the forms”. Additionally, it is not acceptable to “deposit large quantities of concrete at one location and run or work [the concrete] along the forms to other locations”. The maximum free-fall height for fresh concrete is 5 feet unless otherwise approved.
- **Section 424.3.B.7.** Concrete must be cured continuously—except during form removal—until the release-of-tension compressive strength has been obtained and detensioning has been performed.

- During the curing period, the minimum concrete temperature must be at least 50°F and the maximum temperature must not exceed 150°F for mixture design options 6-8 and 170°F for mixture design options 1-5.

4. Development of Mixture Proportions

A laboratory testing program was conducted to relate material properties and mixture proportions to workability and hardened properties and to develop suitable mixture proportions for use in prestressed concrete bridge beams. A series of final mixture proportions was developed for each aggregate set. These final mixture proportions, which were used in subsequent laboratory and field testing, were developed to cover a wide portion of the range of mixture proportions likely to be used for prestressed bridge beams in Texas and to allow for a systematic evaluation of the effects of specific mixture proportioning factors on hardened properties. This chapter summarizes the laboratory testing program and the development of the final mixture proportions.

The factors varied in the final mixture proportions are shown in Figure 4.1. For each aggregate set (river gravel/natural sand; crushed limestone coarse aggregate/natural sand), 5 mixtures were developed for a nominal 16-hour compressive strength level of 5,000 psi and 3 mixtures for a nominal 16-hour compressive strength level of 7,000 psi. For the 5,000 psi mixtures, the S/A varied from 0.40 to 0.50. Three mixtures varied only in S/A (0.40, 0.45, and 0.50) to allow an independent comparison of the effects of S/A. Maintaining a constant paste volume for these three mixtures, however, resulted in paste volumes that were higher than necessary for the mixtures with S/As of 0.45 and 0.50 because the minimum necessary paste volume was found to decrease as the S/A was increased from 0.40 to 0.50. Therefore, separate mixtures were developed with S/As of 0.45 and 0.50 with optimized paste volumes. For the 7,000 psi nominal strength level, 3 mixtures were developed for each aggregate set varying only in S/A (0.42, 0.46, and 0.50). The SCC mixtures were compared to 4 conventionally placed concrete mixtures (one mixture at each nominal strength level for each aggregate set). The conventionally placed mixtures were developed by the TTI research team.

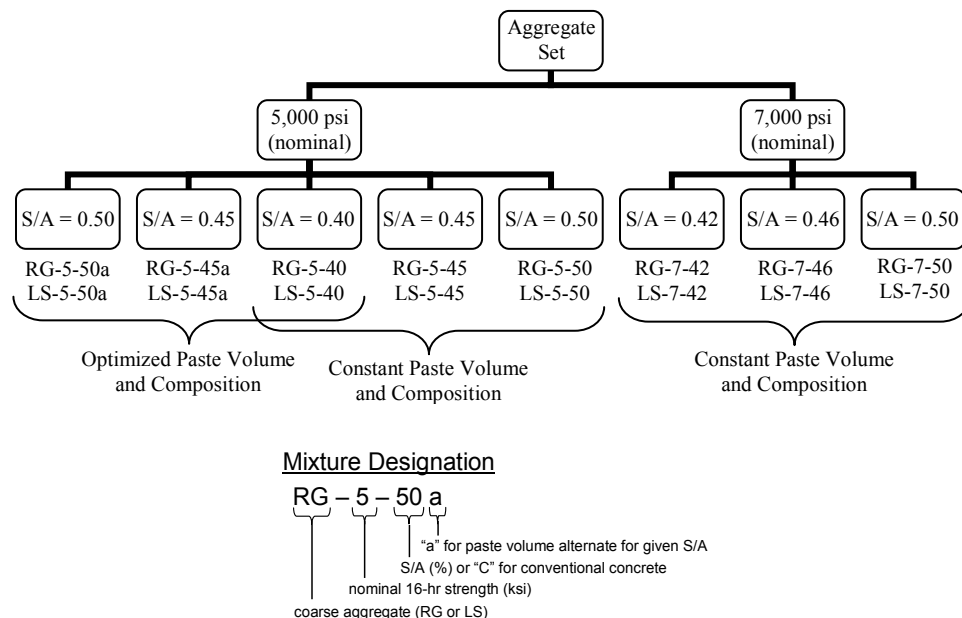


Figure 4.1 Final SCC Mixture Proportions

The main challenge in developing the SCC mixture proportions was to achieve simultaneously adequate passing ability, adequate release-of-tension compressive strength, and a cementitious materials content of less than 700 lb/yd³. Achieving any two of these parameters was less challenging. A minimum paste volume is needed to ensure passing ability. To achieve adequate stability, the high paste volume must (1) be composed of a high powder content or (2) incorporate a VMA with lower powder content. If a high powder content is used and this powder is composed entirely of cementitious materials, the TxDOT-specified limit on cementitious materials content of 700 lb/yd³ is exceeded. To provide enough cement and the proper w/c for early age strength while also achieving the required workability, it was necessary to use a high powder content rather than VMA for the development of the final mixture proportions. Numerous options were considered in the laboratory testing program to explore the trade-offs associated with achieving adequate passing ability, adequate release-of-tension compressive strength, and limited cementitious materials content simultaneously. Cement contents were minimized, but the total cementitious contents were above 700 lb/yd³ in the final mixtures. Only cementitious powders were evaluated.

4.2 Laboratory Testing Program

4.2.1 Multivariate Regression Models

Ninety-eight mixtures were evaluated with the river gravel aggregate set prior to the selection of the final mixture proportions. To simplify the evaluation of these test results, multivariate regression models were developed to relate workability and compressive strength to mixture proportions. The models are based on the assumption that SCC is a suspension of aggregates in paste. Therefore, the properties of the concrete are assumed to depend on the volumes and properties of the paste, aggregates, and transition zones between aggregates and paste. The multivariate models were developed with parameters describing the aggregate grading (sand-aggregate ratio), the paste volume, and the paste composition (w/cm, w/c, fly ash rate). Normally, the water-powder ratio is associated with workability; however, w/cm was used in all models because w/cm was equal to w/p in the mixtures considered. Additionally, the slump flow varied and was included in the models. The range of variables in the mixtures used for the regression models are shown in Table 4.1; all data is provided in Appendix B.

The multivariate regression models are shown in Table 4.2. The nominal 16-hour compressive strength model did not include retarder dosage as an independent variable because all mixtures used to develop the model for 16-hour compressive strength included retarder. The results of the regression models were consistent with expectations from the literature.

Table 4.1 Range of Parameters for River Gravel Aggregate Data Used for Regression Models

Parameter	Minimum	Maximum	Average	Independent Variable for Regression
Paste Volume (V_p)	0.289	0.404	0.339	Yes
w/cm (= w/p)	0.27	0.43	0.343	Yes
w/c	0.27	0.614	0.40	Yes
Sand/Aggregate (S/A)	0.35	0.50	0.451	Yes
Fly Ash Rate (FA)	0.0	0.32	0.129	Yes
Slump Flow (SF), inches	23.5	31	28.1	Yes
Retarder (RET), oz/cwt of cement	0.0	4.0	2.2	Yes
HRWRA, oz/cwt of cementitious	5.0	16.0	9.6	No
Cementitious Materials, lb/yd ³	700	1034	802	No
Coarse Aggregate Volume	0.317	0.443	0.363	No
Water, lb/yd ³	210	340	272	No
Materials: Cement: PC-A; Fly Ash: FA-A; Coarse: RG; Fine: NS-A; HRWRA: HR-A, Retarder: RET-A				

Table 4.2 Multivariate Regression Models for River Gravel Aggregate Set

Equation	Independent Variables Considered	R ²
HRWRA (oz/cwt) = $1/[0.00322 + 1.872(V_p)(w/cm) - 0.00726(w/cm)(SF) - 0.304(S/A)(w/cm) + 0.545(V_p)(FA) - 0.323(S/A)(FA)]$	V_p , w/cm, S/A, FA, SF	0.91
$T_{50} (s) = \exp[5.101 - 27.18(V_p)(w/cm) - 0.0827(SF) + 4.958(V_p)(S/A)]$	V_p , w/cm, S/A, FA, SF	0.78
J-Ring Δh (in.) = $[3.337 - 13.29(V_p)(S/A) + 3.134(S/A)(w/cm) - 0.0363(SF)]^2$	V_p , w/cm, S/A, FA, SF	0.53
L-Box Blk. Ratio = $-1.039 - 7.131(w/cm)^2 + 0.115(S/A)(SF) + 9.078(FA)^2 + 8.631(V_p)(w/cm) - 5.597(V_p)(FA)$	V_p , w/cm, S/A, FA, SF	0.70
f_{ci} (16-hr, nominal, psi) = $1/[-6.675 \times 10^{-5} + 6.72 \times 10^{-4}(w/c) - 9.11 \times 10^{-4}(w/c)(FA) + 2.83 \times 10^{-4}(FA)]$	V_p , w/c, S/A, FA	0.96
f_{ci} (16-hr, 72°F ambient cure, psi) = $\exp[9.951 - 3.634(w/c) - 0.00842(RET)^2]$	V_p , w/c, S/A, FA, RET	0.95
f_c (28-d, psi) = $1/[1.047 \times 10^{-5} + 7.09 \times 10^{-4}(V_p)(w/cm) - 5.940 \times 10^{-5}(RET)(V_p) + 1.04 \times 10^{-4}(w/cm)(FA) + 1.817 \times 10^{-5}(RET)]$	V_p , w/cm, S/A, FA, RET	0.97
Regression details: full quadratic model; stepwise regression procedure with p-value = 0.05; transformations of dependent variables considered: y, 1/y, ln(y), sqrt(y), 1/sqrt(y); transformation with highest R ² selected		

Figure 4.2 indicates that the dosage of HRWRA required for a constant slump flow was mainly affected by the paste volume and w/cm. Increasing the HRWRA dosage mainly increases the slump flow by reducing the yield stress of the concrete to ensure self flow. Increasing the paste volume increases the spacing between aggregates, which reduces

interparticle friction between aggregates. Similarly, increasing the w/cm increases the spacing between cement particles, which reduces both interparticle friction between cementitious particles and the extent of agglomeration. As a result, the HRWRA demand decreased with increasing paste volume and w/cm. In contrast, increasing the fly ash rate and reducing the S/A only slightly reduced HRWRA demand. Fly ash is generally known to reduce HRWRA demand due to its spherical shape; however, the extent of reduction in HRWRA demand depends on the characteristics of the specific fly ash. Decreasing the S/A reflects the grading and the relative shape characteristics of the sand and coarse aggregate. Increasing the coarseness of a grading (lower S/A) would be expected to decrease interparticle friction between aggregates because of the reduced specific surface area and, in this case, to reduce the aggregate packing density. The regression model for HRWRA demand indicates that the slump flow is highly sensitivity to changes in HRWRA dosage. A change in slump flow from 24 to 30 inches, which reflects most of the range of SCC flow properties, was associated with a difference in HRWRA dosage of less than 2 oz/cwt for all values of paste volume and w/cm considered. Increasing the paste volume and w/cm increased the sensitivity of slump flow to HRWRA demand. Because only a small range of HRWRA dosages is associated with SCC flow properties, the slump flow is used as an independent variable in the regression models (not HRWRA dosage). It is also advised that when evaluating SCC, the slump flow be held constant and the HRWRA demand and other workability properties associated with that slump flow be compared. Because the differences in HRWRA dosage—and the associated material costs—are small for a significant change in slump flow, it may be advisable to use a higher slump flow than needed to offset any workability loss and accommodate delays in placement. Such benefits must be weighed against the potential increase in susceptibility to segregation and any changes in mixture proportions required to mitigate the susceptibility to segregation.

Figure 4.3 indicates that T_{50} , which is correlated to plastic viscosity, was mainly affected by paste volume and w/cm. As with HRWRA demand, increases in spacing between aggregates and cementitious particles associated with increases in paste volume and w/cm, respectively, results in reduced interparticle friction and reduced concrete plastic viscosity—as measured indirectly with T_{50} . Reducing the S/A reduced the T_{50} slightly due to the lower specific surface area and reduced interparticle friction. The fly ash rate did not have a statistically significant effect on T_{50} . Fly ash is known to increase or decrease plastic viscosity depending on the characteristics of the specific fly ash. Increasing the slump flow also reduced the plastic viscosity (T_{50}) due to the increased dispersion of cementitious particles.

The blocking resistance, as measured with the j-ring test, was mainly affected by the paste volume and S/A, as indicated in Figure 4.4. Flow is resisted within the concrete by friction between particles and by external friction, such as from obstacles. Decreasing the internal friction increases the extent to which concrete can flow and improves passing ability. Increasing the S/A increased j-ring blocking because it resulted in an increased fraction of larger particles that must pass through the j-ring openings. Increasing the paste volume decreased j-ring blocking because it reduced the volume of aggregates that must pass through the j-ring and decreased the interparticle friction between aggregates. In addition, increasing the w/cm and slump flow decreased j-ring blocking because more flowable concretes are better able to flow around obstacles (provided the viscosity is not too low, resulting in segregation). These results indicate that for a given aggregate and grading, providing sufficient paste volume is crucial to ensuring passing ability. Although increasing the fluidity of the paste can improve j-ring

blocking, the range over which the fluidity can be varied at a given paste volume can be restricted by limits on other workability properties or hardened properties.

As expected, the nominal 16-hour compressive strength was primarily affected by the water-cement ratio (Figure 4.5). At early ages, low calcium fly ashes exhibit minimal pozzolanic reaction and associated contribution to strength development. All mixtures used to develop the regression model plotted in Figure 4.5 incorporated retarder. Figure 4.6 indicates that the use of retarder reduced 16-hour compressive strength slightly for the concrete cured at 72°F ambient conditions.

Figure 4.7 shows that the 28-day compressive strength was primarily affected by w/cm. The use of fly ash decreased 28-day compressive strength slightly because the pozzolanic reaction and associated contribution to compressive strength was likely incomplete by 28 days. Increasing the paste volume increased or decreased 28-day compressive strength slightly. The use of retarder also increased the 28-day compressive strength, likely reflecting the more favorable rate of microstructure development.

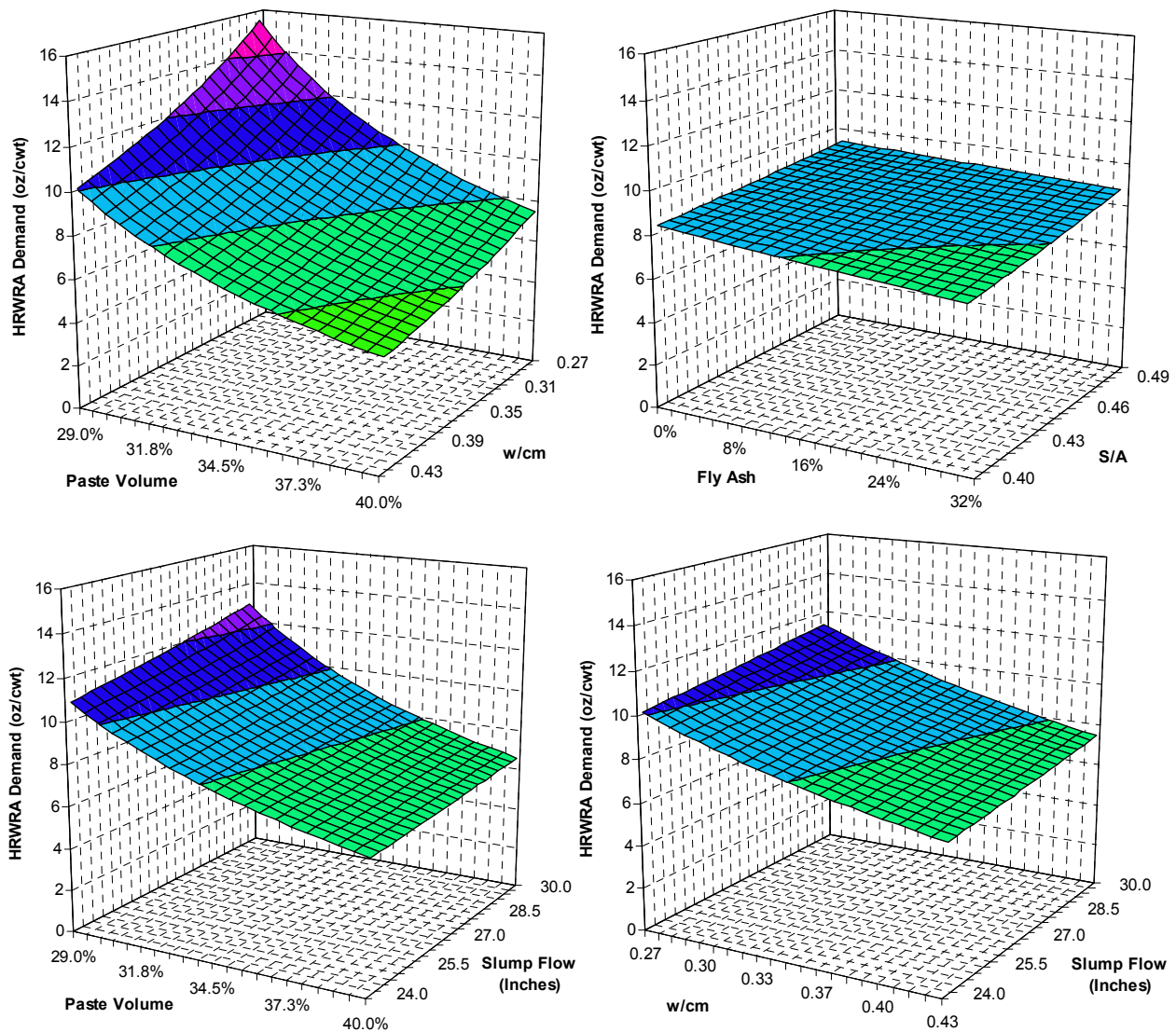


Figure 4.2 Multivariate Regression Results for HRWRA Demand (26-Inch Slump Flow Unless Noted Otherwise)

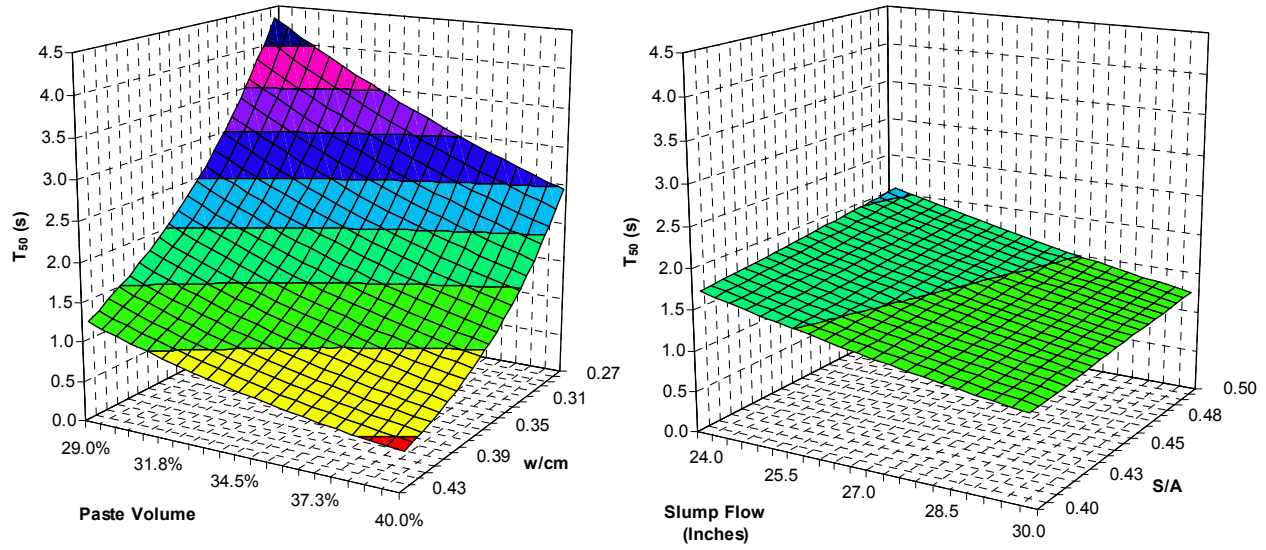


Figure 4.3 Multivariate Regression Results for T_{50} (26-Inch Slump Flow Unless Noted Otherwise)

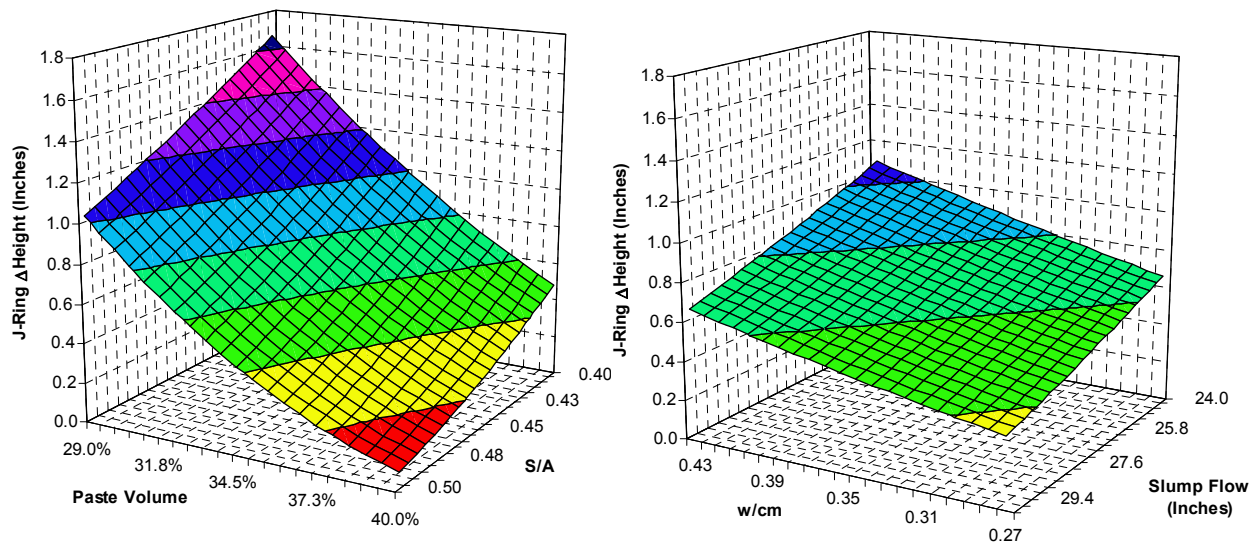


Figure 4.4 Multivariate Regression Results for J-Ring Blocking (26-Inch Slump Flow Unless Noted Otherwise)

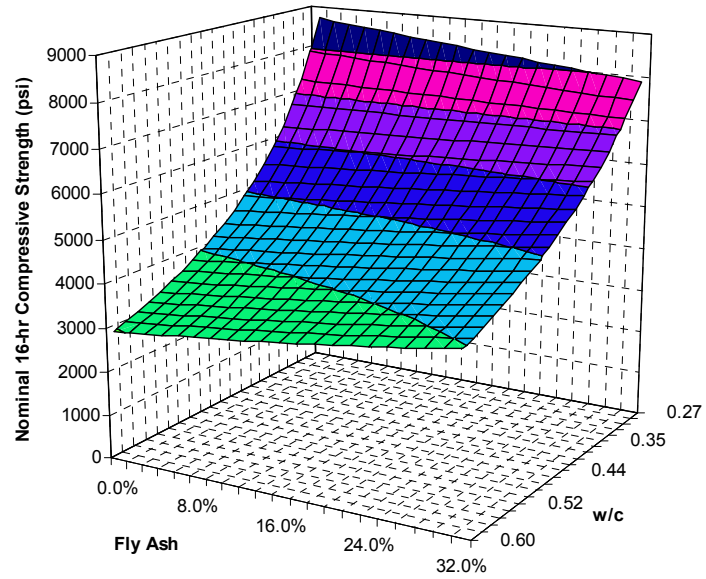


Figure 4.5 Multivariate Regression Results for Nominal 16-Hour Compressive Strength

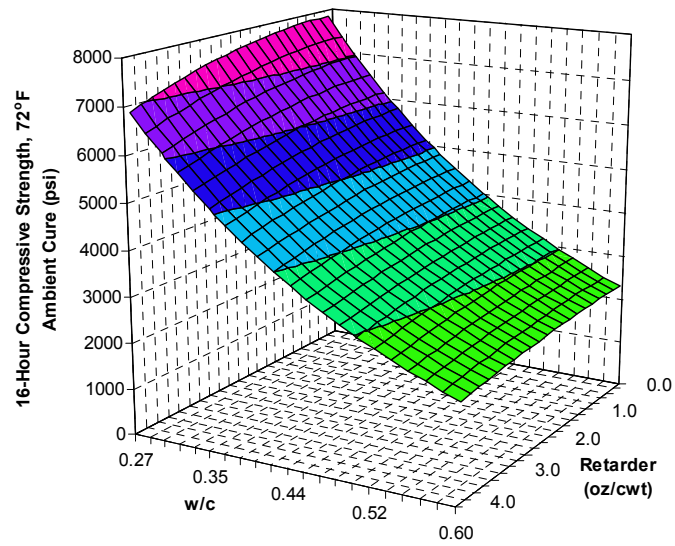


Figure 4.6 Multivariate Regression Results for 16-Hour Compressive Strength (72°F Ambient Cure)

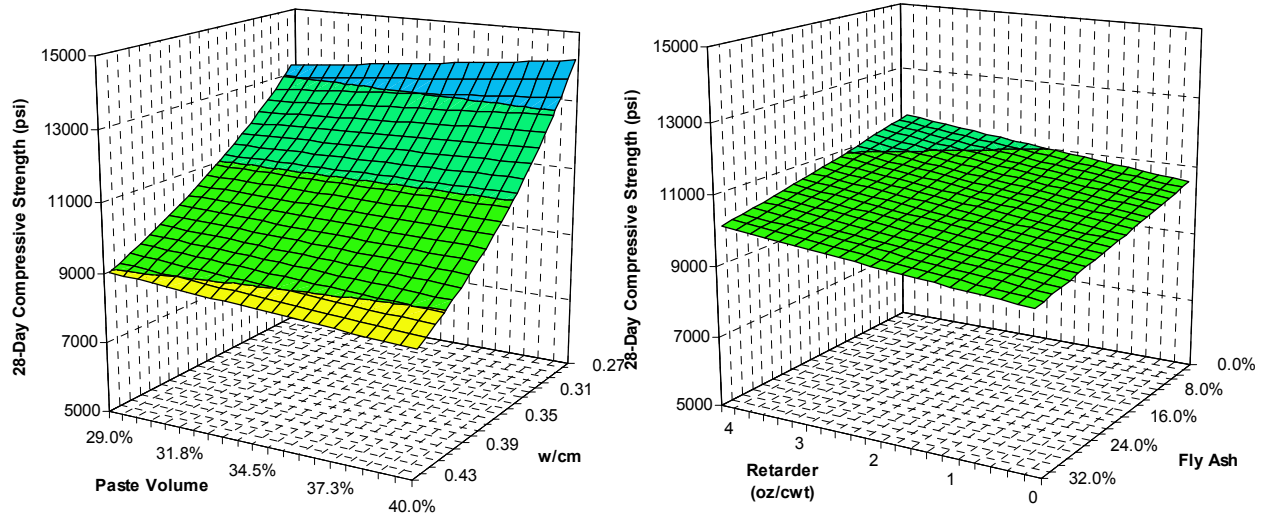


Figure 4.7 Multivariate Regression Results for 28-Day Compressive Strength (72°F Ambient Cure)

In evaluating compressive strength, it should also be noted that increasing the HRWRA dosage can increase compressive strength. For instance, Figure 4.8 indicates that in a given mixture, one ounce of HRWRA per 100 lb. of cementitious materials increased the nominal 16-hour compressive strength by 110 psi on average and the 28-day compressive strength by 345 psi on average. The use of polycarboxylate-based HRWRA provides increased dispersion of cementitious particles and can supply water to cement in a way that promotes more efficient hydration (Jeknavorian et al. 2003). For a given concrete mixture, increasing the strength by increasing the HRWRA dosage is not generally feasible because the range of HRWRA dosages associated with the possible range of SCC flow properties is small. However, when comparing different mixtures that require different HRWRA dosages for SCC flow properties or when comparing SCC to conventionally placed concrete, the difference in compressive strength due to HRWRA dosage may be consequential.

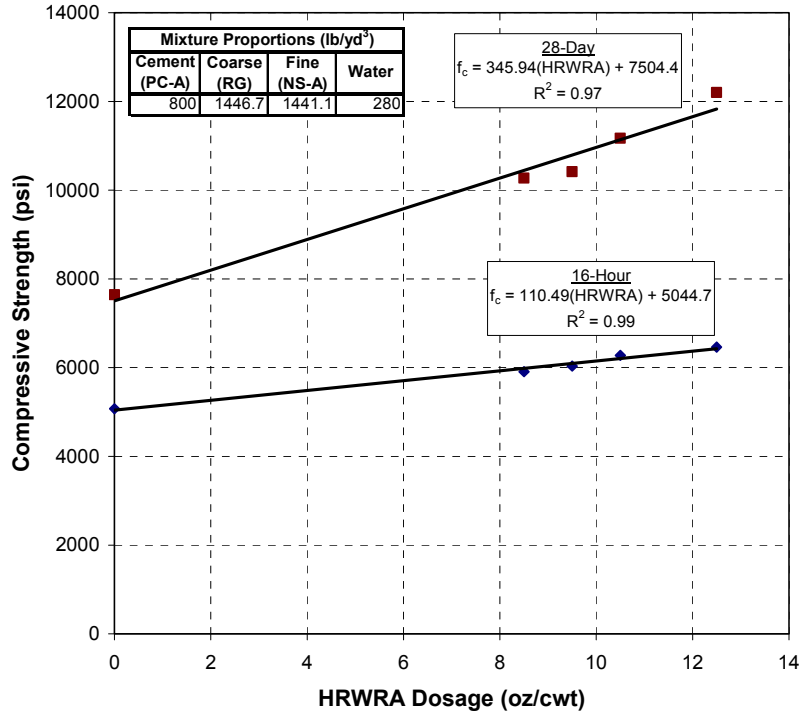


Figure 4.8 Effect of HRWRA Dosage on Nominal 16-Hour and 28-Day Compressive Strength

4.2.2 Other Factors Evaluated

4.2.2.1 Workability Retention

The effects of HRWRA type and retarder type and dosage on workability retention were evaluated. Although the need for workability retention is typically much less in a precast plant than in a ready mixed concrete setting, the potential for rapid workability loss in SCC should be considered. The use of Type III cements and certain HRWRA formulations that are not intended for long workability retention can result in rapid workability loss.

Workability retention is also known to depend on the mixture proportions, HRWRA dosage, weather conditions, and degree of agitation. Further, only two HRWRAs were compared in the test results described here. Workability retention can be strongly dependent on the structure of the HRWRA. HRWRA formulations will change over time as new commercial products are introduced and others are removed from the market. Therefore, in implementing SCC in a precast plant, it is important to perform an analysis of all of these relevant factors.

The mixture proportions used for the evaluation of workability retention are shown in Table 4.3. The HRWRA dosage was adjusted in each mixture to reach a slump flow of 29 to 32 inches. Two HRWRAs (HR-A and HR-B) and three retarders (RET-A, RET-B, and RET-C) were compared.

When HR-A was used without retarder, the slump flow dropped from 30 to 20 inches after 30 minutes. When 4 oz/cwt of RET-A was used with HR-A, the slump flow only decreased from 30 to 26.5 inches after 30 minutes. In contrast, the addition of RET-B with HR-A accelerated the loss of slump flow. The loss of workability was less with HR-B than with HR-A;

however, the higher initial slump flow with HR-B may have contributed partially to the improved workability retention with HR-B. The addition of RET-C with HR-B accelerated the loss of workability.

Table 4.3 Mixture Proportions for Comparison Effects of Admixtures on Workability Retention

Cement	Fly Ash	Coarse Aggregate	Fine Aggregate	Water
640	160	1424	1418	280
S/A = 0.50, w/cm = 0.35, $V_p = 34.8\%$, 20% fly ash				

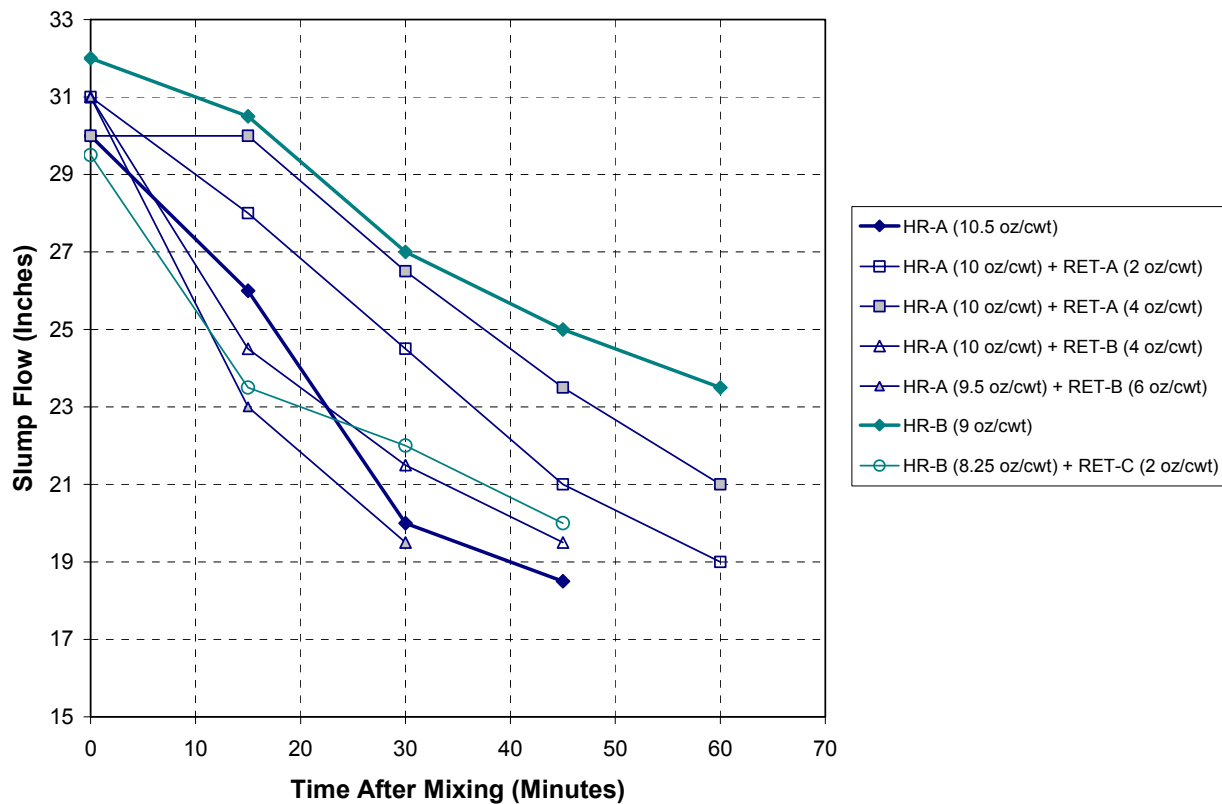


Figure 4.9 Effects of Admixtures on Workability Retention

The effects of retarders on setting time and compressive strength development must also be considered. Figure 4.10 indicates that the use of RET-A and RET-B delayed both the initial and final setting times in the mixture shown in Table 4.3. Even though RET-B reduced workability retention while RET-A increased workability retention, RET-B delayed the time of set to a greater extent than RET-A. This results illustrates that setting time should not be associated with workability retention. Despite the delay in initial and final setting times, the reduction in 16-hour compressive strength due to the addition of RET-A was minimal, as shown previously in Figure 4.6.

Based on these results, the use of RET-A with HR-A was selected because RET-A increased workability retention and had little effect on compressive strength.

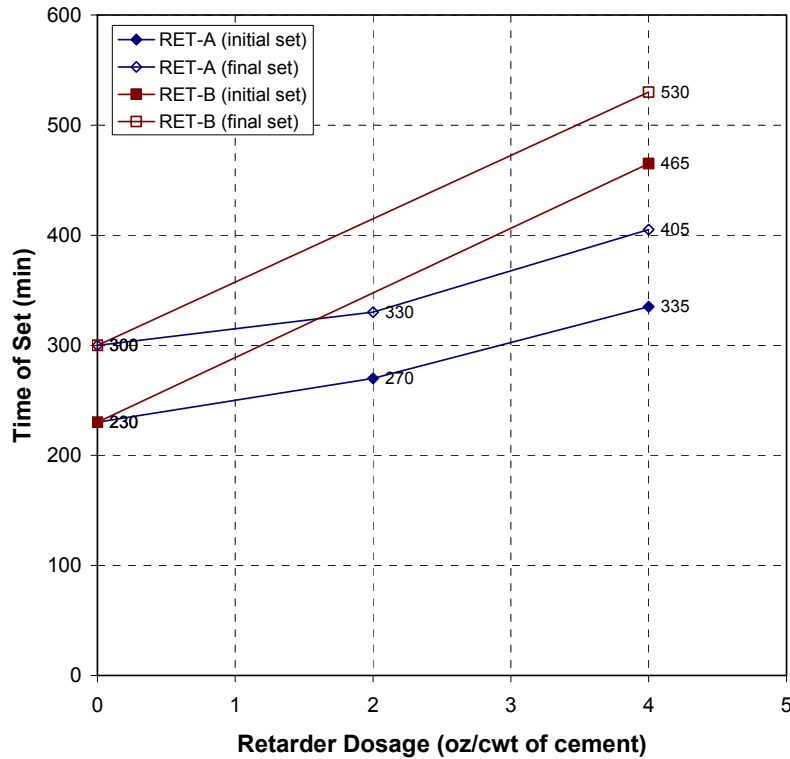


Figure 4.10 Effects of Admixtures on Setting Time

4.2.2.2 Ultra-Fine Fly Ash

The effects of ultra-fine fly ash (UFFA) on workability and compressive strength were evaluated to determine whether UFFA could improve workability sufficiently to lower the w/cm and w/c and whether UFFA was sufficiently reactive to contribute to strength at early ages. The UFFA as supplied was taken from the FA-A parent fly ash.

Four mixtures were evaluated, as shown in Table 4.4. UFFA was used to replace cement at 12 and 18% of the cement mass. The w/cm was reduced in the fourth mixture to take advantage of any improvements in workability due to UFFA.

Figure 4.11 indicates that UFFA improved workability significantly. The use of 18% UFFA reduced the HRWRA demand for a 28-inch slump flow by 20% and reduced T_{50} by nearly 50%. In contrast, the multivariate regressions based on the parent fly ash suggested an 8% reduction in HRWRA demand and a 4% reduction in T_{50} for the same dosage of FA-A. The smaller size and improved shape of the UFFA likely contributed to this improvement in workability.

The ultra-fine fly ash did not appear to contribute to the nominal 16-hour compressive strength, as indicated in Figure 4.12. The use of 18% UFFA resulted in a reduction in nominal 16-hour compressive strength of 25%. The multivariate regressions for the mixtures with FA-A suggested an 18% reduction in nominal 16-hour compressive strength with the same dosage of FA-A. In addition, the mixture with reduced w/cm had lower nominal 16-hour compressive

strength than the control mixture with no fly ash, despite having a lower w/c. At 28 days, the use of 18% UFFA resulted in a 7% reduction in compressive strength. The multivariate regression suggested a reduction of 2% in 28-day compressive strength for the same dosage of FA-A.

Based on these results, UFFA can be used to reduce the w/cm for the same workability, which can in turn allow for a lower w/c and potentially a higher 16-hour compressive strength. The UFFA rate can be lower than that of the parent fly ash. However, in order to maintain a constant paste volume, the volume of cementitious materials would need to be increased as the w/cm is reduced. In addition, the use of a lower volume of UFFA than the parent fly ash would require an increase in cement content.

Table 4.4 Mixture Proportions for Evaluation of Ultra-Fine Fly Ash

ID	Indices				Mixture Proportions (lb/yd ³)				
	UFFA (%)	w/cm	w/c	V _p	Cement	UFFA	Coarse Agg.	Fine Agg.	Water
1	0	0.33	0.33	0.327	800	0	1467	1462	264
2	12	0.33	0.375	0.332	704	96	1459	1453	264
3	18	0.33	0.402	0.334	656	144	1454	1448	264
4	12	0.28	0.318	0.308	704	96	1510	1504	224
All mixtures include RET-A at 4 oz/cwt of cement									

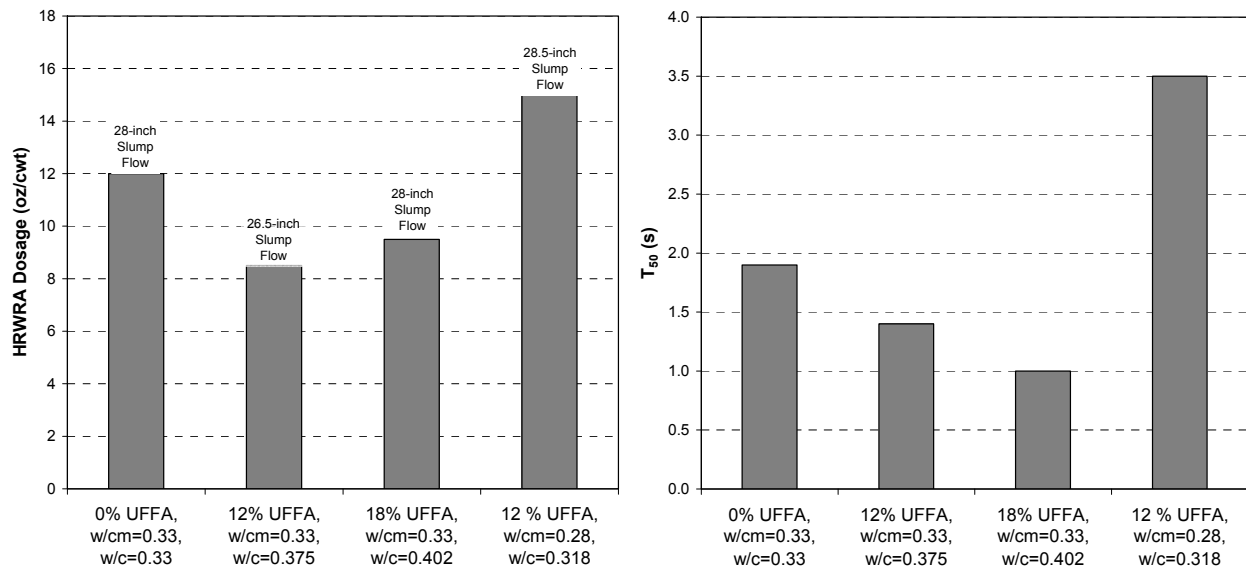


Figure 4.11 Effect of UFFA on Workability

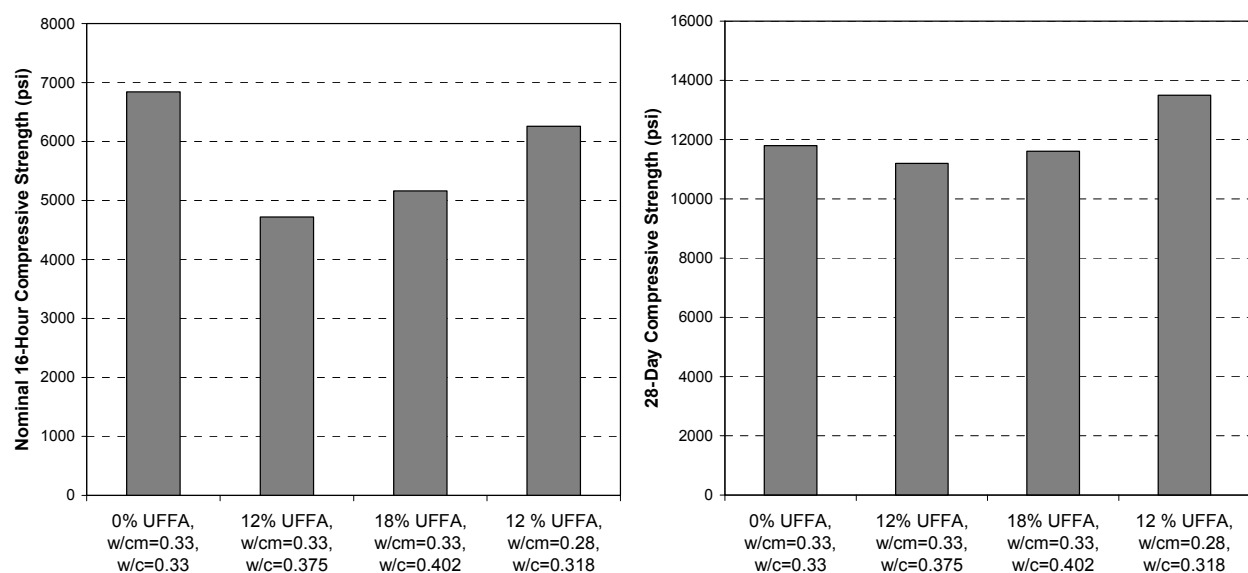


Figure 4.12 Effect of UFFA on Compressive Strength

4.2.2.3 Accelerator

The effects of accelerator on nominal 16-hour compressive strength were evaluated to determine whether the use of accelerator could allow an increase in water-cement ratio and a reduction in cement content. Three accelerators were considered. The mixture proportions for the evaluation of each accelerator are shown in Table 4.5.

Table 4.5 Mixture Proportions for Evaluation of Accelerators

ID	Indices					Mixture Proportions (lb/yd ³)				
	Accelerator	w/cm	w/c	V _p	Fly Ash	Cement	Fly Ash	Coarse Agg.	Fine Agg.	Water
1	ACC-A @ 15, 30 oz/cwt	0.35	0.35	0.337	0%	800	0	1447	1441	280
2	ACC-A @ 15, 30 oz/cwt	0.35	0.438	0.348	20%	640	160	1424	1418	280
3	ACC-B @ 10, 27.5, 45 oz/cwt	0.28	0.359	0.37	22%	739.4	208.6	1650	1095	265.4
4	ACC-C @ 20, 45 oz/cwt									
Mixtures 1 and 2 incorporate HR-B										
Mixtures 3 and 4 incorporate RET-A @ 3 oz/cwt of cement and HR-A										
Accelerators: expressed in oz/cwt of cement only										

The use of accelerator ACC-A, shown in Figure 4.13, increased the nominal 16-hour compressive strength only slightly when used at 15 oz/cwt in both mixtures. At higher dosages, ACC-A provided no increase in nominal 16-hour compressive strength. Figure 4.14 indicates that the use of accelerator ACC-B did increase the nominal 16-hour compressive strength slightly, but only at the maximum dosage of 45 oz/cwt and by less than 10%. The use of accelerator ACC-C had no effect on 16-hour compressive strength.

Based on these results, it was determined that the use of these three accelerators to reduce w/c and cement content was not feasible. It is possible that the accelerators could have a greater impact in other mixtures. Additionally, the use of other curing histories could affect the nominal 16-hour strength development. If the mixtures with the accelerators evolve more heat, their effect on strength development would likely be greater than in the results presented here, where all mixtures were cured at the same temperature history. The accelerators did allow for a slight decrease in HRWRA dosage for a given slump flow; however, this effect was not large enough to permit any other change in mixture proportions.

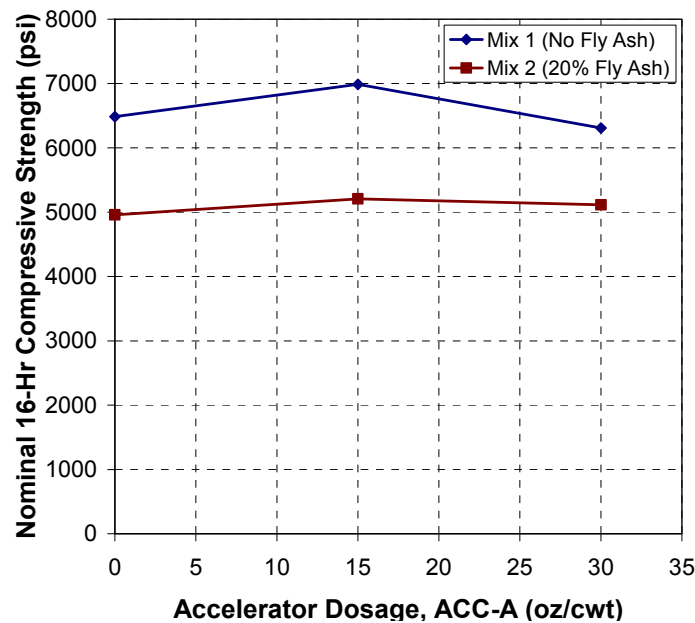


Figure 4.13 Effects of Accelerator ACC-A on Nominal 16-Hour Compressive Strength

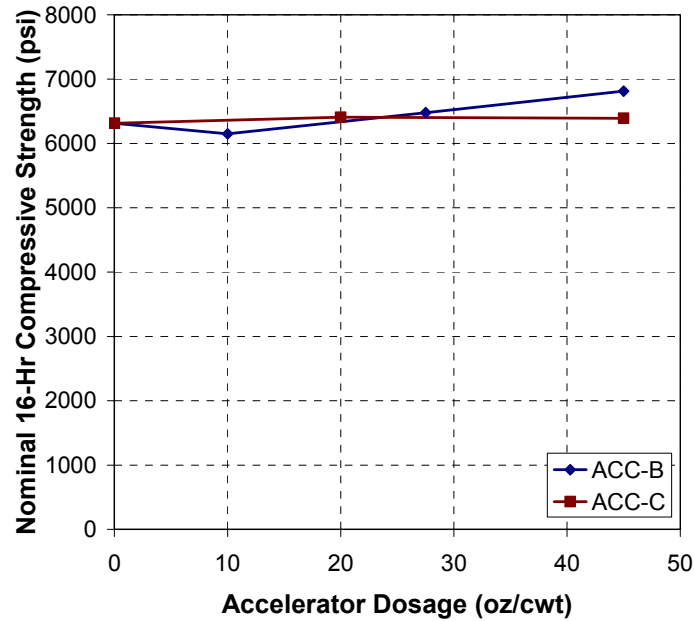


Figure 4.14 Effect of Accelerators ACC-B and ACC-C on Nominal 16-hour Compressive Strength

4.2.2.4 Alternate HRWRA

An alternate HRWRA (HR-B) was compared to HR-A. Mixtures allowing a direct comparison of the two HRWRAs are shown in Table 4.6; all other mixtures with HR-B are shown in Appendix B and a further comparison of HR-A, HR-B, and HR-D is made in Section 4.4. The effects of each HRWRA on workability and compressive strength development were determined to be similar, such that the use of one HRWRA in place of the other would not permit other changes in mixture proportions.

Table 4.6 Mixtures Showing Direct Comparison of HR-A and HR-B

S/A	w/cm	Fly Ash	V _p	ID	HRWRA	Dosage	Slump Flow	T ₅₀	VSI	J-Ring	L-Box	Nom. 16-h f _{ci}	28-d f _c
		%				oz/cwt	inches	s		inches		psi	psi
0.50	0.30	20	0.324	27	HR-A	15.0	29.0	2.2	0.5	0.25	1.00		
				38	HR-B	14.0	30.0	1.8	1.0	0.50	1.00	5654	11061
0.50	0.35	20	0.348	28	HR-A	9.0	29.0	1.0	0.5	0.31	0.71	4897	10629
				39	HR-B	7.0	26.5	1.5	0.0	0.56	0.71	4960	9929
0.50	0.35	0	0.337	36	HR-A	9.5	26.0	2.0	0.0	1.19	0.47	6373	11329
				46	HR-B	9.0	25.0	1.5	0.0	1.00	0.44	6485	10848

Mixtures 38 and 39 include 3 oz/cwt RET

4.3 Development of Final Mixture Proportions

The ICAR SCC mixture proportioning procedure (Koehler and Fowler 2007) was used to develop the final mixture proportions. In this procedure, concrete is represented as a suspension of aggregates in paste, as depicted schematically in Figure 4.15. This representation provides a consistent, fundamental framework for evaluating mixtures. To proportion mixtures, the combined aggregates, paste volume, and paste composition are selected to achieve the desired workability and hardened properties. The role of each of these factors is summarized in Table 4.7. The test data presented in Section 4.1 are consistent with the concepts of the ICAR SCC mixture proportioning procedure.

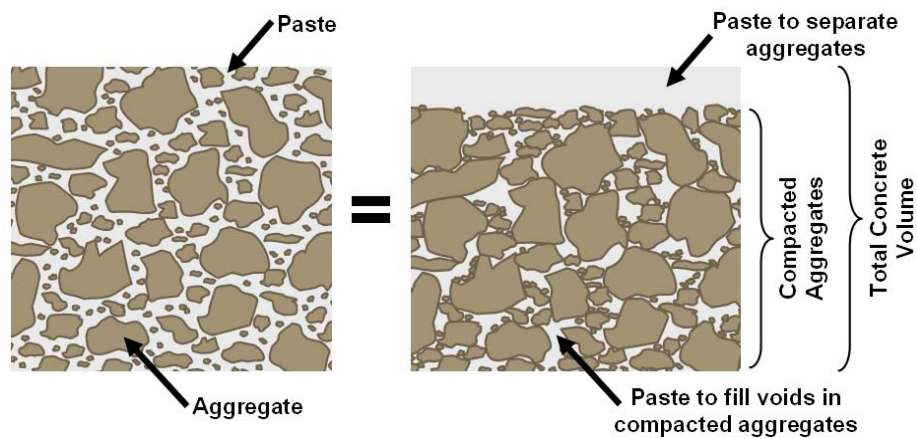


Figure 4.15 Representation of Concrete as a Suspension of Aggregates in Paste

Table 4.7 Summary of Factors Considered in the ICAR SCC Mixture Proportioning Procedure (Koehler and Fowler 2007)

Factor	Objective	Sub-Factors	Target	Typical Values
Aggregates	Minimize voids content (increase packing density) and reduce interparticle friction; limit grading as needed for passing ability and segregation resistance	Maximum Size	Reduce for passing ability or segregation resistance	¾ or 1 inch for most applications; reduce to as low as 3/8 inch for challenging passing ability
		Grading	None universally optimal, best depends on aggregate and application	Uniform gradings with high packing density preferred, 0.45 power curve or finer, S/A=0.40-0.50
		Shape, Angularity, Texture	Reduce interparticle friction	Equidimensional, rounded aggregates preferred but any can be accommodated
Paste Volume	Ensure filling and passing ability by filling voids in compacted aggregates and separating aggregates (lubrication), provide additional paste for robustness	Filling Ability	Fill voids and lubricate aggregates	Total paste volume = 28-40%
		Passing Ability	Reduce aggregate volume and interparticle friction	
		Robustness	Minimize effects of changes in materials and proportions	
Paste Composition	Ensure adequate concrete rheology (yield stress, plastic viscosity, thixotropy) and hardened properties (strength, stiffness, durability), optimize economy	Water	w/p for rheology, w/c for early-age hardened properties, w/cm for long-term hardened properties	w/p = 0.30-0.45, higher with VMA
		Powder	Relative amounts of cement, SCMs, and mineral fillers for economy, strength, durability, and to fill paste volume	Fly ash, slag, silica fume, ground limestone filler, dust-of-fracture aggregate microfines
		Air	As needed for durability	Same requirements as for conventionally placed concrete
Adjust HRWRA to reach desired slump flow (yield stress for self-flow)				

The ICAR mixture proportioning procedure consists of three steps, as summarized in Table 4.8. The steps are conducted in the order shown because the paste volume depends on the aggregates and the paste composition depends on the paste volume and aggregates.

First, the combined aggregates are selected on the basis of maximum size; grading; and shape, angularity, and texture. These factors are selected to minimize the voids content between aggregates and reduce interparticle friction between aggregates. The maximum size and grading are limited for segregation resistance and passing ability. Although a wide range of aggregates can be used in SCC, minimizing the voids content between aggregates and reducing interparticle friction by improving the shape and angularity reduces the required paste volume, resulting in greater economy and improved hardened properties.

Second, the required minimum paste volume is selected separately for filling ability and passing ability, with additionally paste used for robustness with respect to aggregate properties.

The minimum paste volume is primarily a function of the aggregate characteristics. Without the minimum paste volume, concrete would not exhibit adequate SCC workability regardless of the paste composition. For filling ability, sufficient paste volume must be provided to fill the voids within compacted aggregates and to lubricate aggregates by providing spacing between aggregates (Figure 4.15). If only sufficient paste volume is provided to fill the voids between aggregates, the substantial interparticle friction between aggregates would prohibit flow. An estimate of the paste volume for filling ability ($V_{paste-filling\_ability}$, expressed as a percentage of concrete volume) can be calculated as a function of the voids in compacted aggregates and the aggregate shape and angularity, as shown in Equation (4.1).

$$V_{paste-filling\_ability} = 100 - \frac{(100 - V_{paste-spacing})(100 - \%voids_{compacted\_agg})}{100} \quad (4.1)$$

where $V_{paste-spacing}$ is the paste volume for spacing (expressed as a percentage of concrete volume, varies from 8-16% depending on aggregate shape characteristics) and $\%voids_{compacted\_agg}$ is the percentage of voids in compacted aggregate (expressed as a percentage of the bulk aggregate volume). Selecting well-shaped and well-graded aggregates reduces the voids in compacted aggregates (increased packing density), which reduces the needed paste volume. Additionally, aggregates that are equidimensional and well rounded reduce interparticle friction, which reduces the paste needed to separate aggregates. For passing ability, increasing the paste volume reduces the volume of aggregates that must pass through congested reinforcement and reduces the interparticle friction between aggregates. The amount of paste volume for passing ability can be reduced by using a smaller maximum aggregate size, reducing the coarseness of the grading, or selected equidimensional, well-rounded aggregates. The minimum paste volume is selected as the greater of the minimum paste volume for filling ability or passing ability. This amount of paste volume can be increased for robustness with respect to aggregate properties. It is possible to increase the paste volume above the minimum paste volume, but paste volumes lower than the minimum are not possible.

Third, the composition of the paste—namely the blend of powders and the relative amounts of powder, water, and air—is selected to achieve proper workability and hardened properties. The paste composition step is where the distinction in the continuum between powder-type and VMA-type SCC is made. Powder-type SCC consists of high powder contents and low water-powder ratios (typically <0.40) whereas VMA-type SCC consists of lower powder contents, higher w/p (typically >0.45), and VMA. In selecting the paste composition, the powder blend and water-powder ratio are used to achieve the desired concrete rheology. The water-cement ratio and water-cementitious materials ratios are used to achieve the desired the early-age and long-term hardened properties, respectively. Depending on the reactivity of the SCMs, the use of w/c may be conservative. With the w/c and w/cm set, the cement content should be minimized to achieve hardened properties and other powders added to fill the paste volume and achieve the desired rheology. To maintain constant concrete rheology as the paste volume or aggregates are changed, the paste composition must be adjusted. For all mixtures, the HRWRA is adjusted to achieve the required slump flow, which is related to yield stress. The near-zero yield stress for SCC is the main workability difference between SCC and conventionally placed concrete. The plastic viscosity—which is a function of aggregates, paste

volume, and paste composition—must not be too low (poor stability) or too high (poor placeability).

Table 4.8 Summary of Steps in the ICAR Mixture Proportioning Procedure (Koehler and Fowler 2007)

STEP 1: Aggregates	1. Select individual aggregate sources (fine, intermediate, coarse sizes) 2. Evaluate various aggregate blends. a. Maximum aggregate size b. Grading (0.45 power curve, percent retained on each sieve) c. Shape and angularity (visually rate on scale of 1 to 5) 3. Determine compacted voids content of each blend.
STEP 2: Paste Volume	1. Determine minimum paste volumes for filling and passing ability. Select the larger. a. Paste volume for filling ability (Calculate from compacted voids content and visual rating of shape and angularity. Confirm with tests with various paste volumes and constant paste composition. Concrete should be able to achieve target slump flow without bleeding and segregation.) b. Paste volume for passing ability (Test various paste volumes with constant paste composition.) 2. Add paste volume for robustness.
STEP 3: Paste Composition	1. Select cement, SCMs, and mineral fillers. 2. Select maximum w/c and w/cm and maximum and minimum SCM rates for early-age and long-term hardened properties. If mineral fillers affect hardened properties, specify maximum and minimum rates. 3. Select air content for durability (assume 2% if not air entrained). 4. Select w/p for rheology (typically 0.30-0.45, may be higher with VMA) 5. Calculate paste composition. 6. Evaluate trial mixtures and adjust paste composition.

4.3.2 Nominal 5,000 psi 16-Hour Compressive Strength

The ICAR mixture proportioning procedure is demonstrated in the following subsections for the nominal 5,000 psi 16-hour compressive strength mixtures.

4.3.2.1 Selection of Aggregates

The maximum size of each aggregate set was $\frac{3}{4}$ inches. Each aggregate set was used at S/As of 0.40 to 0.50. The gradings at these S/As, shown in Figure 4.16, were considered suitable for SCC. The low amount of material retained on the #8 and #16 sieves for the crushed limestone aggregate set could increase the susceptibility to segregation but could also result in increased packing density and lower viscosity due to the reduced particle interaction in this size range. The river gravel set was more uniformly graded. Both aggregate sets, when used at an S/A of 0.50 were finer than the 0.45 power curve, which is favorable based on the ICAR mixture proportioning procedure. When used at an S/A of 0.40, both aggregate sets were coarser than the 0.45 power curve at sizes larger than the #8 sieve. Nonetheless, the deviations from the 0.45 power curves were minimal. (The 0.45 power curve was drawn through the #200 sieve—instead of the origin—because material finer than the #200 sieve is more appropriately treated as powder, not aggregate.)

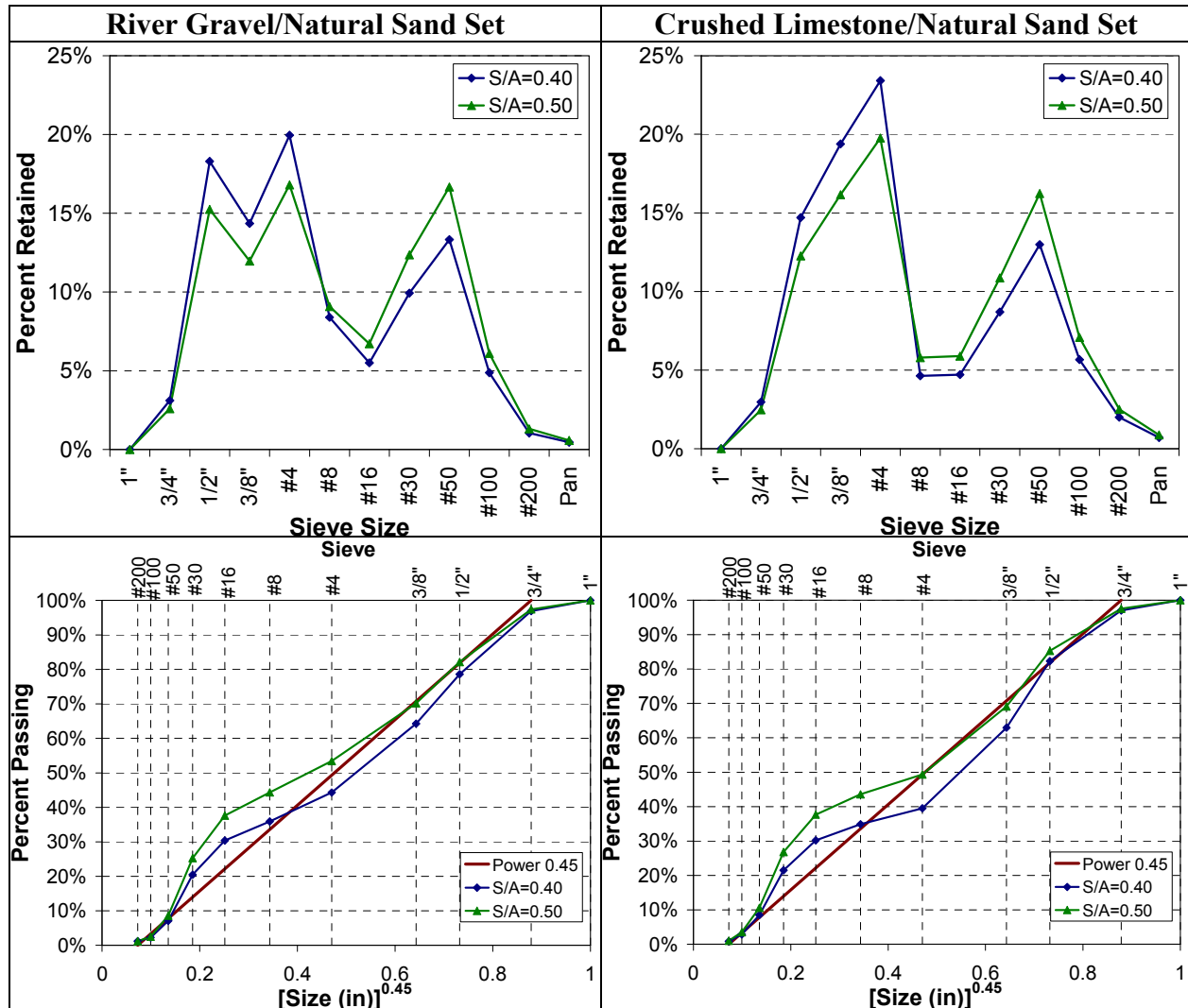


Figure 4.16 Aggregate Gradings

The differences in shape and angularity between the coarse aggregate sources are shown in Figure 4.17. Both coarse aggregates consisted of generally well-shaped particles with very small fractions of flat or elongated particles. The main difference between the coarse aggregate sources was their angularity. The river gravel was mostly well-rounded with a small number of particles containing a single fractured face. The crushed limestone coarse aggregate, in contrast, was highly angular with sharp corners. The sands were both well shaped and well rounded.



Figure 4.17 River Gravel (Left) and Crushed Limestone Coarse Aggregates

The voids contents between compacted aggregates, which are shown in Table 4.9, were similar for the two aggregate sets. Increasing the S/A from 0.40 to 0.50 reduced the voids content, which would result in less paste volume required to fill the space between voids.

Table 4.9 Minimum Paste Volumes for Filling Ability and Passing Ability

S/A	Compacted Voids Content	
	River Gravel Set	Crushed Limestone Set
0.00	34.2	41.4
0.40	23.9	23.9
0.45	23.6	23.3
0.50	23.2	22.7
1.00	33.6	32.4

4.3.2.2 Selection of Paste Volume

The minimum paste volumes for each S/A were determined experimentally by evaluating filling ability and passing ability at various paste volumes. The paste composition was held constant as the paste volume was varied. An estimate of the minimum paste volume for filling ability can also be calculated based on the aggregate voids content and the shape and angularity.

For filling ability, sufficient paste volume is needed to fill the voids between compacted aggregates and to reduce the interparticle friction between aggregates by providing spacing between aggregates. The minimum paste volume for filling ability was evaluated qualitatively. Concrete without the minimum paste volume for filling ability may not achieve the desired slump flow regardless of the HRWRA dosage, may be highly viscous, may exhibit severe bleeding and segregation, and may appear harsh. As shown in Table 4.10, the amount of paste for filling ability increased with decreasing S/A because of the increasing aggregate voids content. The crushed limestone coarse aggregate set required greater paste volumes for filling ability due to its greater angularity, even though its voids content was similar.

For passing ability, sufficient paste volume is needed to limit the volume of coarser particles and reduce the interparticle friction between aggregates. The effect of paste volume on passing ability is illustrated in Figure 4.18. Decreasing the S/A required more paste volume due to the increased amount of coarse particles. The crushed limestone coarse aggregate set required greater paste volumes for passing ability due to its greater angularity.

Due to the congested reinforcement in the prestressed beams, passing ability controlled the selection of minimum paste volume (Table 4.10). The minimum paste volumes determined for passing ability were increased by 1% to ensure robustness with respect to aggregate properties.

Table 4.10 Minimum Paste Volumes for Filling Ability and Passing Ability

S/A	River Gravel			Crushed Limestone		
	Voids Content	Req'd Paste Volume		Voids Content	Req'd Paste Volume	
		Filling	Passing		Filling	Passing
0.40	23.9	32	36	23.9	38	41
0.45	23.6	30	34	23.3	36	38
0.50	23.2	28	32	22.7	35	36
Larger minimum paste volume for filling or passing selected						

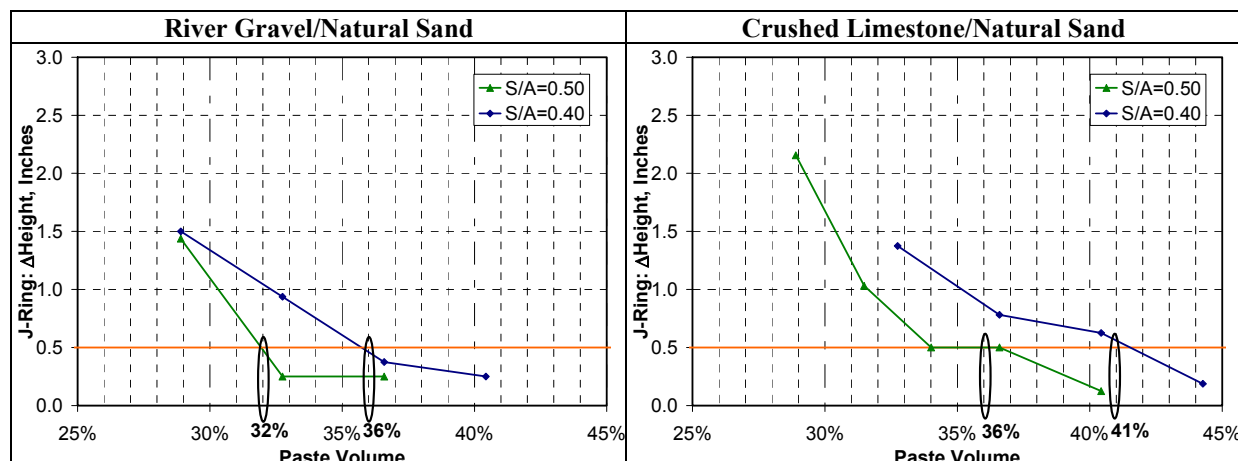


Figure 4.18 Effects of Paste Volume of Passing Ability

4.3.2.3 Selection of Paste Composition

The paste composition was optimized for each aggregate set and paste volume. A retarder was used in all mixtures to ensure workability retention. The w/c for a nominal 5,000 psi compressive strength, with retarder, was determined to be 0.41 for the river gravel aggregate set and 0.45 for the crushed limestone aggregate set (Figure 4.19). With the w/c established, the w/p (equal to w/cm in this case) and fly ash rate were adjusted for each paste volume to achieve proper concrete rheology and fill the required paste volume. Because the particular fly ash utilized had minimal influence on workability, the fly ash rate was simply calculated to fill the paste volume once the w/c and w/p were set for 16-hour strength and workability, respectively. For instance, with a w/c of 0.40 and a w/p of 0.30, the fly ash rate would be 25.2%. (In cases where the fly ash rate does affect workability, the w/p would need to be adjusted as the fly ash rate is changed to maintain a constant rheology.) Trial batches were tested to evaluate workability. In all cases, the HRWRA dosage was adjusted to reach the desired slump flow. For instance, Table 4.11 shows the trial batches to select the paste composition for a S/A of 0.50. Similar trial batches were tested for the other S/As for both aggregate sets.

The final mixture proportions for both aggregate sets are shown in Table 4.12. The conventionally placed concrete mixtures shown in Table 4.12 were developed by the TTI researchers. Although it was not possible to keep the cementitious materials content below 700 lb/yd³ in the SCC mixtures, the cement content is below 700 lb/yd³ in all mixtures. The consequences of the cementitious materials contents higher than 700 lb/yd³ will be evaluated in subsequent parts of this report.

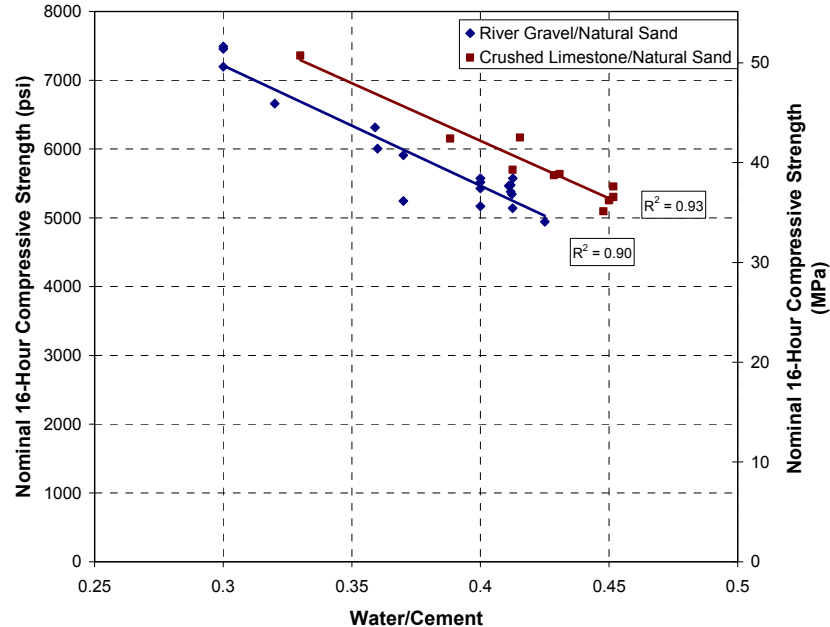


Figure 4.19 Effects of Water/Cement on Nominal 16-Hour Compressive Strength (RET-A in All Mixtures)

Table 4.11 Trial Batches to Select Paste Composition (River Gravel Aggregate Set; S/A=0.50)

ID	Mixture Proportions (lb/yd ³)						Indices				Properties					
	Cement	Fly Ash	Coarse Agg.	Fine Agg.	Water	HR-A	V _p	w/p w/cm	w/c	Fly Ash	Slump Flow	T ₅₀	VSI	J-Ring	L-Box	16-h f _{ci}
						oz/cwt	%			%	inches	s		inches		psi
75	759.7	0.0	1461.8	1456.2	281.1	9.0	33.0	0.37	0.37	0	27.5	1.0	1.0	0.31	0.77	5245
76	665.6	166.4	1461.8	1456.2	246.3	11.0	33.0	0.296	0.37	20	26.5	1.8	0.5	0.50	0.64	5909
78	622.0	155.5	1461.8	1456.1	264.4	9.0	33.0	0.34	0.425	20	29.0	1.8	0.5	0.25	0.79	4941
96	623.8	156.0	1461.8	1456.2	257.3	9.5	33.0	0.33	0.413	20	27.0	2.2	1.0	0.40	0.71	5139

Note: All mixtures include RET-A

Mixture 75: The preliminary mixture with w/p = 0.37 for workability; viscosity too low, as reflected with T₅₀

Mixture 76: Fly ash is added to reduce w/p; the w/c is too high, resulting in strength too high

Mixture 78: The w/p is increased for better workability

Mixture 96: The w/p is reduced slightly for better workability (T₅₀=2.2s) and to ensure 16-hr strength

Table 4.12 Final Mixture Proportions for Nominal 5,000 psi Compressive Strength

		River Gravel/Natural Sand						Crushed Limestone/Natural Sand					
		RG-5-C	RG-5-50	RG-5-45	RG-5-40	RG-5-50a	RG-5-45a	LS-5-C	LS-5-50	LS-5-45	LS-5-40	LS-5-50a	LS-5-45a
Proportions													
Cement	lb/yd ³	625.0	633.3	633.3	633.3	623.8	645.7	600.0	639.7	639.7	639.7	603.8	604.0
Fly Ash	lb/yd ³		298.0	298.0	298.0	156.0	238.8	0.0	426.4	426.4	426.4	297.4	370.2
Coarse Aggregate	lb/yd ³	1937.3	1371.5	1508.7	1646.0	1458.8	1532.4	1751.6	1262.6	1389.0	1515.3	1371.9	1461.3
Fine Aggregate	lb/yd ³	1233.8	1366.2	1229.7	1093.1	1453.2	1248.9	1381.5	1267.5	1140.8	1014.1	1377.2	1200.2
Water	lb/yd ³	225.0	260.8	260.8	260.8	257.3	265.4	252.0	287.8	287.8	287.8	270.4	272.8
HRWRA (HR-E)	oz/yd ³	68						44					
HRWRA (HR-A)	oz/yd ³		91	83	78	83	89		84	70	65	75	64
Retarder (RET-A)	oz/yd ³		25	25	25	25	26		26	26	26	24	24
Indices													
S/A		0.39	0.5	0.45	0.4	0.5	0.45	0.44	0.5	0.45	0.4	0.5	0.45
Coarse Agg. Vol.	%	44.4	31.4	34.6	37.7	33.4	35.1	40.1	28.9	31.8	34.7	31.4	33.5
Paste Volume	%	27.2	37.1	37.1	37.1	33.1	36.1	28.3	42.1	42.1	42.1	37.1	39.1
w/cm		0.36	0.28	0.28	0.28	0.33	0.3	0.42	0.27	0.27	0.27	0.3	0.28
w/c		0.36	0.412	0.412	0.412	0.413	0.411	0.42	0.450	0.450	0.450	0.448	0.452
Fly Ash Rate	%	0	32	32	32	20	27	0	40	40	40	33	38
Total Cementitious	lb/yd ³	625	931.3	931.3	931.3	779.8	884.5	600	1066.1	1066.1	1066.1	901.2	974.2
Typical Fresh Properties													
Yield Stress	Pa	294	6	44	16	11		177	2	15	18	6	
Plastic Viscosity	Pa.s	42	34	37	24	23		47	34	33	27	23	
Slump	In.	7.5						8.25					
Slump Flow	In.		30	28	30	28			29	28.5	28	29.5	
T ₅₀	s		3.8	4.2	3.8	4.9			6	4.4	5.1	4.9	
VSI			0	0	0.5	0			0	0.5	0.5	1	
J-Ring ΔH	In.		0.13	0.34	0.19	0.00			0.25	0.28	0.27	0.22	
Column Seg.	% Seg		5	0	3	13			1	8	13	20	
Typical Hardened Properties													
Nominal 16-hr f _{ci}	psi	5784	5622	6021	5501	5521		5260	6112	5850	5641	5603	
28-day (72°F) f _c	psi	10854	11733	12062	11774	11223		11135	13713		13066	13075	

4.3.3 Nominal 7,000 psi 16-Hour Compressive Strength

4.3.3.1 Preliminary Considerations

For mixtures at a nominal 7,000 psi (48 MPa) 16-hour compressive strength level, the challenge of simultaneously achieving passing ability, early-age compressive strength, and maximum cementitious materials content was increased. The paste volume requirements for passing ability were unchanged. However, to achieve the nominal 7,000 psi compressive strength level, the maximum w/c based on the data in Figure 4.19 was 0.31 for the river gravel aggregate set and 0.34 for the crushed limestone aggregate set, with use of retarder for workability retention.

Table 4.13 shows several options that were considered for S/As of 0.42 and 0.50 for the river gravel aggregate set. For an S/A of 0.42, the first option limited the cement content to 700 lb/yd³; however, the w/c was too high. In the second option, the w/c was reduced to 0.31; however, the cement content was too high. In the third option, 20% fly ash was added to reduce the cement content and the water content was reduced to ensure the proper w/c; however, the w/p was too low for workability, resulting in extremely viscous concrete. For an S/A of 0.50, the same three options were considered. Reducing the cement content to 700 lb resulted in a w/c that was too high. Using a w/c of 0.31 resulted in a cement content that was too high. Adding fly ash while maintaining a w/c of 0.31 resulted in a w/p that was too low.

Table 4.13 Options Considered for 7,000 psi Mixtures (River Gravel Aggregate Set)

S/A	Option	Paste Volume	w/p w/c m	w/c	Fly Ash	Cement	Cementitious
		%			%	lb/yd ³	lb/yd ³
0.42	1) 700 lb cement	36.0	0.498	0.498	0	700.0	700.0
	2) w/c=0.31, no fly ash	36.0	0.31	0.31	0	910.3	910.3
	3) add fly ash, reduce w/cm	36.0	0.248	0.31	20	777.3	971.7
0.50	4) 700 lb cement	33.0	0.429	0.429	0	700.0	700.0
	5) w/c=0.31, no fly ash	33.0	0.31	0.31	0	832.4	832.4
	6) add fly ash, reduce w/cm	33.0	0.248	0.31	20	710.8	888.5

Notes: mixtures are shown for illustrative purposes and were not tested independently; minimum paste volume for passing ability is 33% for S/A of 0.50 and 36% for passing ability for S/A of 0.42, w/c=0.31 for strength with retarder; retarder included in mixes 1-6 for workability retention; VMA only used in option 7 for mixture with S/A of 0.42

4.3.3.2 Selection of Final Mixture Proportions

Because none of options in Table 4.13 was viable, external technical assistance was sought. BASF Construction Chemicals, LLC, responded to this request by providing a new admixture product and assisting in the selection of final mixture proportions. The new admixture product, PT-1482, is intended to extend workability retention and accelerate early strength gain. The Admixture Systems Group of BASF Construction Chemicals utilized proprietary software to generate initial trial mixture proportions. Laboratory trial batches were then evaluated to

optimize the final mixture proportions. The Admixture Systems Group first tested a preliminary series of trial batches in their laboratory and then provided a series of mixture proportions to be evaluated at the authors' laboratory. Sixty-two mixtures (47 with the river gravel aggregate set and 15 with the crushed limestone aggregate set) were evaluated in the authors' laboratory. Each trial mixture was set by the Admixture Systems Group based in part on feedback from the authors.

Mixtures were optimized in terms of the PT-1482 admixture formulation (in reference to workability retention and strength development) and the mixture proportions (in reference to workability retention, strength development, cement and cementitious materials content, and compliance with 2004 TxDOT Specification Section 421.4.A.6). To optimize the PT-1482 product formulation, various experimental admixture combinations were evaluated at different dosages. To optimize the mixture proportions, the cement, fly ash, and water contents, and the dosages of HRWRA and VMA were varied. The optimization of the PT-1482 product formulation and of the mixture proportions were interdependent. All trial mixtures tested in the authors' laboratory are shown in Appendix B.

The final mixture proportions for the nominal 7,000 psi 16-hour compressive strength level are shown in Table 4.14. The sand-aggregate ratio is varied from 0.42 to 0.50 and the paste volume and paste composition are held constant. Low dosages of VMA are used in the mixtures with an S/A of 0.42 to ensure stability. The cementitious materials contents are greater than 700 lb/yd³; however, the cement contents were minimized during the optimization process. The conventionally placed concrete mixtures shown in Table 4.14 were developed by the TTI researchers.

Although the PT-1482 admixture product formulation and the mixture proportioning software used to develop the nominal 7,000 psi compressive strength mixtures are proprietary to BASF Construction Chemicals, LLC, the use of PT-1482 admixture and the mixture proportions shown in Table 4.14 does not necessarily represent an exclusive means of achieving the mixture proportioning criteria established in Chapter 3.

Table 4.14 Final Mixture Proportions—Nominal 7,000 psi 16-Hour Compressive Strength

		River Gravel/Natural Sand				Crushed Limestone/Natural Sand			
		RG-7-C	RG-7-50	RG-7-46	RG-7-42	LS-7-C	LS-7-50	LS-7-46	LS-7-42
Proportions									
Cement	lb/yd ³	700.0	720.0	720.0	720.0	680.0	720.0	720.0	720.0
Fly Ash	lb/yd ³		180.0	180.0	180.0	0.0	180.0	180.0	180.0
Coarse Aggregate	lb/yd ³	1938.5	1400.5	1512.6	1624.7	1753.6	1383.3	1494.0	1604.8
Fine Aggregate	lb/yd ³	1234.6	1395.1	1283.5	1171.9	1383.1	1388.6	1277.6	1166.6
Water	lb/yd ³	199.5	243.0	243.0	243.0	224.4	256.5	256.5	256.5
HRWRA (HR-E)	oz/yd ³	117				78			
HRWRA (HR-A)	oz/yd ³		118	119	115		110	88	88
PT-1482	oz/yd ³		578	578	578		578	578	578
VMA-A	oz/yd ³				4				3
Indicies									
S/A		0.39	0.5	0.46	0.42	0.44	0.5	0.46	0.42
Coarse Agg. Vol.	%	44.4	32.1	34.7	37.2	40.2	31.7	34.2	36.8
Paste Volume	%	27.2	35.8	35.8	35.8	28.2	36.6	36.6	36.6
w/cm		0.285	0.27	0.27	0.27	0.33	0.285	0.285	0.285
w/c		0.29	0.338	0.338	0.338	0.33	0.356	0.356	0.356
Fly Ash Rate	%	0	20	20	20	0	20	20	20
Total Cementitious	lb/yd ³	700	900.0	900.0	900.0	680	900.0	900.0	900.0
Typical Fresh Properties									
Yield Stress	Pa	195	0	0	0	121	0	0	0
Plastic Viscosity	Pa.s	263	56	66	51	133	30	32	25
Slump	In.	7.25				7			
Slump Flow	In.		28	28	27		29	29	30
T ₅₀	s		8	7.4	7.4		6.3	5.5	4
VSI			0	0	0		0	0	1
J-Ring ΔH	In.		0.25	0.25	0.44		0.16	0.44	0.31
Column Seg.	% Seg		7	0	7		8	0	4
Typical Hardened Properties									
Nominal 16-hr f _{ci}	psi	6871	7108	6986	7296	7165	7422	7187	7657
28-day (72°F) f _c	psi	12706	13023	13970	14042	13146	14802		14931

4.4 Discussion of Final Mixture Proportions

SCC is defined in terms of its workability. Changes in hardened properties associated with SCC are primarily attributable to specific changes in mixture proportions required to achieve workability characteristics, not to the workability characteristics themselves. These changes in mixture proportions can vary depending on the application and available materials. Therefore, hardened properties should be evaluated on a case-by-case basis in terms of specific changes to mixture proportions instead of associating certain hardened properties with SCC in general. The direct and indirect effects of changes in mixture proportions should be considered separately. For instance, reducing the maximum aggregate size typically results in increased required paste volume. Changes in shrinkage and modulus of elasticity due to this higher paste volume should be considered as an indirect effect of reducing the maximum aggregate size. In addition, the improved consolidation and lack of vibration associated with SCC and the improved cement dispersion associated with high HRWRA dosages may result in improved hardened properties.

4.4.1 Mixture Indices

For the SCC mixtures developed in this project, the major changes in mixture proportions were increased paste volume, reduced S/A, and use of fly ash. The maximum aggregate size was unchanged.

Paste volume—SCC must have higher paste volume than conventionally placed concrete to ensure filling ability, passing ability, and robustness. For prestressed concrete bridge beams and other highly congested members, the minimum paste volume for passing ability typically controls the selection of the minimum paste volume. The actual paste volume may be set higher than the minimum paste volume; however, paste volumes lower than the minimum paste volume are not possible. Depending on the aggregate, paste volumes of 28-40% are typical for SCC in general (Table 4.7). The increased paste volume can be comprised of water, cement, cementitious materials, or other powder materials and does not necessarily require higher cement or cementitious materials contents. The consequence of higher powder contents will be explored in greater detail in subsequent parts of this report. In the literature, higher paste volume has been associated with increased shrinkage (EFNARC 2005; Koehler and Fowler 2007; Bissonnette, Pascale, and Pigeon 1999; Hammer 2003; Roziere, Turcry, Loukili, and Cussigh 2005), reduced modulus of elasticity (EFNARC 2005; Koehler and Fowler 2007; Ahmad and Shah 1985), and increased cost. Although increasing the paste volume may reduce modulus of elasticity, changing the stiffness of the aggregates can have a greater effect (Koehler and Fowler 2007). The paste volume required for passing ability is primarily a function of the aggregate characteristics and can be reduced by reducing the maximum size, reducing the coarseness of the combined grading, and improving the shape and reducing the angularity of both the fine and coarse aggregates (Koehler and Fowler 2007). These changes also can also affect hardened properties themselves and should be evaluated accordingly. To a lesser extent, passing ability can be increased by reducing the paste yield stress and plastic viscosity. Further, improving the level of quality control can reduce the amount of paste volume required for robustness.

S/A—The sand-aggregate ratio relates to aggregate grading and the relative shape and mechanical characteristics of the coarse and fine aggregates. In terms of aggregate grading, increasing the S/A can improve passing ability and segregation resistance while decreasing the S/A can often result in improved filling ability. When changing the S/A, it is important to consider the individual properties of the sand and coarse aggregates. For instance, reducing the coarseness of a grading (higher S/A) is known to reduce modulus of elasticity (EFNARC 2005; Ahmad and Shah 1985). If the coarse aggregate is of much lower stiffness than the sand, an increase in S/A may result in increased modulus of elasticity. There are typically optimal S/As associated with packing density and filling ability. The actual optimal S/A for a given aggregate set depends on the grading, shape, and angularity of the individual fine and coarse aggregates.

Maximum aggregate size—The maximum aggregate size in SCC is limited by passing ability and segregation resistance. Increasing the maximum size improves filling ability to the extent it improves the combined aggregate grading. Although decreasing the maximum aggregate size may decrease the paste volume required for passing ability, it may increase the paste volume required for filling ability. Therefore, the optimum maximum aggregate size for minimum paste volume can vary depending on the requirements for filling and passing abilities. In general, the largest maximum aggregate size, subject to limits on passing ability and segregation resistance, should be used. Increasing the maximum aggregate size is known to increase modulus of elasticity and reduce compressive strength.

Fly ash—SCC often incorporates fly ash and other SCMs to achieve adequate paste volume and w/p for workability. Fly ash also reduces cost, reduces heat generation, and improves durability. In the SCC mixtures developed in this project, the cement contents were approximately the same or slightly increased relative to the comparable conventional control

mixtures; however, the water contents and fly ash contents were increased to achieve adequate paste volume and w/p for workability.

4.4.2 Constituent Volume Comparison

The tradeoffs associated with the development of SCC mixture proportions are further illustrated in Figure 4.20. The mixtures shown in this figure were optimized for economy, workability, and hardened properties for each S/A and aggregate set. In the conventional mixture for the river gravel aggregate set, the paste volume is 27.2% because vibration is available to ensure filling ability and passing ability. For the SCC mixture with the river gravel aggregate set and S/A of 0.50, the paste volume must be increased to 33% to ensure filling ability and passing ability. To increase the paste volume, the cement content is held constant, the water content is increased slightly, and fly ash is added. Despite the higher w/c in the SCC mixture, the same nominal strength level is attained. In addition, the volume of coarse aggregate is decreased and the volume of fine aggregate increased to ensure passing ability and segregation resistance. Decreasing the S/A to 0.40 in mixture RG-5-40 does allow an increase in coarse aggregate volume; however, the paste volume must be increased to ensure passing ability such that the total aggregate volume is reduced. Most of the increase in paste volume required to accommodate the higher S/A is accomplished by increasing the fly ash content; the water and cement volumes are increased only slightly. Increasing the nominal compressive strength level to 7,000 psi for mixture RG-7-42 only requires an adjustment in paste composition relative to the 5,000 psi nominal strength level. Relative to RG-5-40, the paste volume is slightly less (due to the slightly higher S/A) but the cement content is increased and the water content is reduced. In addition, the PT-1482 admixture is used. When the crushed limestone aggregate set is used at an S/A of 0.50, the paste volume must be increased relative to the comparable mixture in the river gravel aggregate set (RG-5-50a) to ensure adequate filling and passing abilities. Most of this increase in paste volume is comprised of fly ash and, to a lesser extent, water. The cement content can be reduced because of the improved strength associated with concrete mixtures composed of the crushed limestone coarse aggregate, which was likely due to the improved paste-aggregate bond.

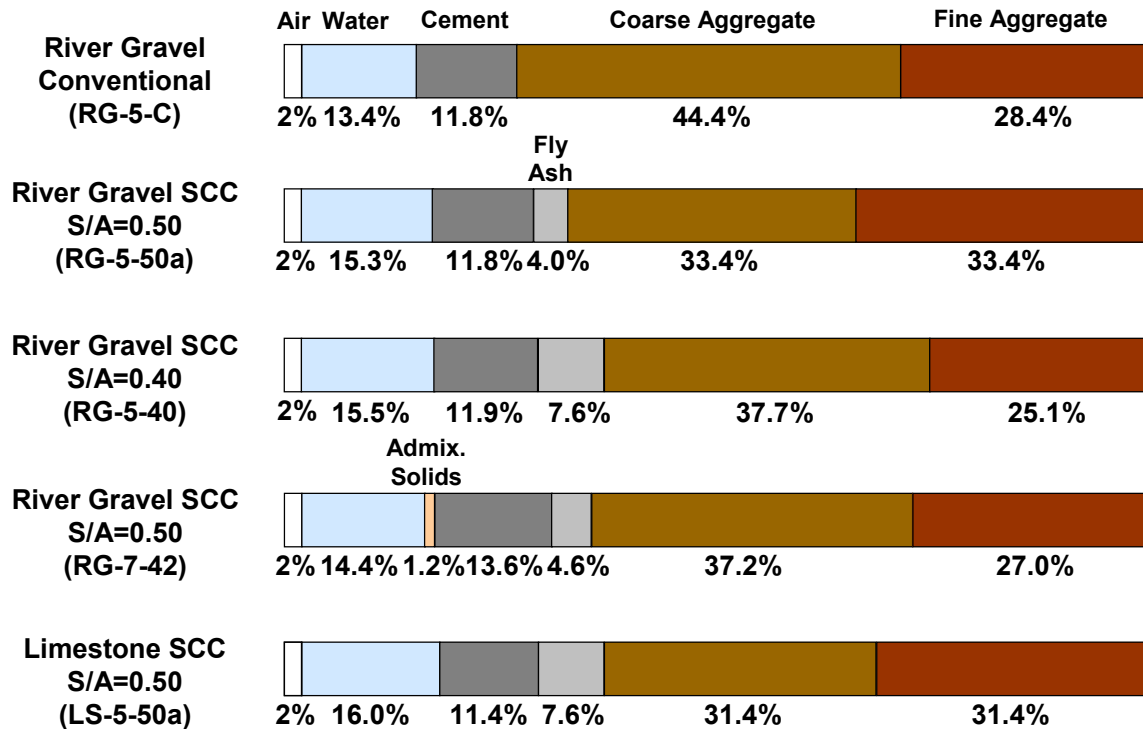


Figure 4.20 Comparison of Constituent Volumes for Final Mixture Proportions (Some Volumes May Not Add to 100% Due to Rounding)

The tradeoffs illustrated in Figure 4.20 can be summarized as follows:

- Decreasing the S/A allows an increase in coarse aggregate volume; however, the paste volume must also be increased to ensure passing ability, resulting in lower total aggregate content.
- Improving the aggregate shape characteristics reduces the paste volume needed for filling and passing abilities, allowing greater aggregate volume.
- The increase in paste volume required for SCC workability can be attained mainly by adding fly ash (for a constant release-of-tension compressive strength). The cement content should be minimized for the required strength. It may be possible to keep the cement content constant while increasing the fly ash because early-age compressive strength depends mainly on w/c.
- Increasing the strength level requires an adjustment in the paste composition, but not the S/A or paste volume.

4.4.3 Potential Changes in Final Mixture Proportions

The final mixture proportions developed in this project are based on the materials and requirements enumerated in Chapter 3. They are intended to represent a wide portion of the range of SCC mixtures likely to be used in Texas; however, they do not necessarily represent the full range. SCC mixtures developed with different materials and different requirements may not be subject to the same limitations described in this report. For instance, it may be possible to increase the w/c and reduce the cement content for applications where the release-of-tension

compressive strength requirements are lower, the release-of-tension time is later, or the concrete curing temperatures are higher. During the summer months in Texas, precast producers may find that the actual compressive strengths of the nominal 5,000 psi mixtures exceed the required release-of-tension compressive strengths, thus allowing mixture proportions with higher w/c, lower cement content, or both. Although more than 180 mixtures were evaluated in the University of Texas laboratory as part of the development of the final mixture proportions, it was not possible to explore every possible combination of materials and placement conditions. None of the options considered for achieving the mixture proportioning objectives was explored exhaustively. As the state of the art in SCC advances and new materials become available, the SCC mixture proportions developed in this project can be improved.

A key limitation in the development of mixture proportions in this project was the minimum necessary paste volume. While it is easier to establish the minimum necessary paste volumes for filling ability in the laboratory, additional field testing is needed to aid in establishing minimum paste volumes for passing ability. In the research described in this report, the minimum paste volume required for passing ability was determined with the j-ring. At the time of this writing, published data relating j-ring test results to passing ability in actual concrete members is limited. If the paste volumes required for passing ability can be reduced, based on field experience, the powder contents can be reduced and potential mixtures that were not feasible in this research may be possible.

In the SCC literature, a distinction is commonly made between powder-type, VMA-type, and combination-type SCC. The required minimum paste volume for a given aggregate and application is approximately the same for each type—the difference is in the paste composition. That is, powder-type SCC has more powder and less water whereas VMA-type has more water and less powder but both have the same total volume of water, powder, and air. The SCC mixtures developed in this research project were all powder-type SCC. The potential disadvantages of using high powder contents include the costs of the powder materials and admixtures and the potential for increased autogenous shrinkage associated with water-cement ratios below 0.40. In addition, when cementitious materials are used to comprise the required powder content, the total cementitious materials content typically exceeds the TxDOT limit of 700 lb/yd³. These problems could be mitigated by increasing the w/p and reducing the powder content—that is, using combination-type or VMA-type SCC.

Although the use of higher w/p would allow a reduction in powder content, the associated w/c would likely be too high to obtain high early-age compressive strengths. As shown in Table 4.15, the most likely candidate for the use of higher w/p is mixture RG-5-50a. To achieve the needed paste volume of 33% with a cement content of 700 lb/yd³, the resulting w/c would be 0.429, which is only slightly higher than the w/c of 0.41 determined separately for achieving a 5,000 psi nominal strength. The use of an accelerator may be adequate to achieve the target nominal strength. Alternatively, the cement content could be increased to 718 lb/yd³ to achieve a w/c of 0.41. The use of higher paste volumes—necessary due to either lower S/A or the use of the crushed limestone coarse aggregate set—would result in even higher w/c or cement content. It is possible that not all of the mixtures shown in Table 4.15 may require VMA due to the higher w/p. VMA would be required when the viscosity or stability are too low. The need for VMA increases as the w/p and paste volume are increased. If it can be shown with field testing that paste volumes can be reduced while still maintaining adequate filling ability, passing ability, and robustness, then VMA-type or combination-type SCC may be more feasible.

Although it is more likely that powder-type SCC will be used in prestressed concrete bridge beams, the use of higher w/p or VMA-type SCC should not necessarily be excluded. It is possible that higher w/p or VMA-type SCC could be used in cases such as where the early-age strength requirements are less—such as for lower required release-of-tension strength, higher curing temperatures, or later release-of-tension times—or when accelerator is added. However, the addition of VMA introduces an additional cost and another variable to control. In addition, the unit water content is higher and the beneficial pozzolanic activity associated with the use of SCMs is not obtained. It has been recommended that SCC be proportioned without VMA if possible (Berke et al. 2002). The three admixtures manufacturers providing materials for this research project were consulted regarding the suitability of their VMAs for this application; all three recommended against using VMA for this application.

Table 4.15 Potential Higher w/p or VMA-Type SCC Mixtures (Nominal 5,000 psi Compressive Strength)

Option		Paste Volume	w/p w/cm w/c	Fly Ash	Cement
		%		%	lb/yd ³
700 lb Cement	1) RG @ S/A=0.50	33.0	0.429	0	700.0
	2) RG @ S/A=0.40; LS @ S/A=0.50	37.0	0.525	0	700.0
Higher Cement	3) RG @ S/A=0.50 (max w/c=0.41)	33.0	0.41	0	718.0
	4) RG @ S/A=0.40 (max w/c=0.41)	37.0	0.41	0	810.6
	5) LS @ S/A=0.50 (max w/c=0.45)	37.0	0.45	0	768.4
Notes: Mixtures are illustrative and were not tested; maximum w/c for RG (0.41) and LS (0.45) assumes use of RET-A. Not all mixtures may require VMA.					

4.5 Material Sensitivity Analysis

A sensitivity analysis was conducted to evaluate the relative effects of using alternate cement, fly ash, and HRWRAs.

4.5.1 Alternate Cement

The primary Type III cement (PC-A) was compared to the alternate Type III cement (PC-B) in mixtures RG-5-50a and RG-7-50. Table 4.16 indicates the workability—measured in terms of slump flow and rheology initially and at 15 minutes, passing ability, and segregation resistance—was similar with both cements. Table 4.17 indicates that PC-B gained less strength at 16-hours at each curing temperature in both mixtures; however, the 28-day strengths were similar. In mixture RG-5-50a, the reduction in 16-hour compressive strength was approximately 10% in the specimens with the 120°F and 170°F maximum temperature. In mixture RG-7-50a, the 16-hour compressive strength was only 4 to 7% lower. Despite these small differences, the two cements could be used interchangeably with minimal modifications to mixture proportions.

Table 4.16 Evaluation of Alternate Cement: Workability

Mixture	Cement	HR-A	Initial						15 Minutes						J-ring		Col. Seg.
			Slump Flow			Rheology			Slump Flow			Rheology			Δh	r-u	
			Flow	T ₅₀	VSI	τ ₀	μ	Thix	Flow	T ₅₀	VSI	τ ₀	μ	Thix			
		oz/yd ³	in.	s		Pa	Pa.s	Nm/s	in.	s		Pa	Pa.s	Nm/s	in.	in.	%
RG-5-50a	Primary (PC-A)	83	28	4.9	0	10.7	22.9	0.03	26.0	4.4	0	14.3	35.9	0.23	0.00	0.00	13.0
	Alternate (PC-B)	88	29	4.7	0.5	0.0	24.4	0.05	24.0	7.4	0	40.1	50.7	0.32	0.25	-0.50	8.4
RG-7-50	Primary (PC-A)	118	28	8.0	0	0.0	55.8	0.12	26.5	10.8	0	0.0	94.1	0.84	0.25	0.00	6.6
	Alternate (PC-B)	118	29	7.1	0	3.4	62.3	0.12	29.0	9.8	0	0.0	89.3	0.54	0.25	0.00	12.2
Notes: thixotropy expressed in terms of breakdown area; concrete not agitated between initial and 15-minute test																	

Table 4.17 Evaluation of Alternate Cement: Compressive Strength

Mixture	Cement	16-hour			28-day
		--	8-hr	4-hr	
		72°F	120°F	170°F	72°F
		psi	psi	psi	psi
RG-5-50a	Primary (PC-A)	4499	5521	8094	11223
	Alternate (PC-B)	3529	5031	7304	11773
RG-7-50	Primary (PC-A)	4926	7108	8767	13023
	Alternate (PC-B)	4716	6617	8462	12954

Notes: 16-hour strengths identified in terms of pre-set time and maximum temperature

4.5.2 Alternate Fly Ash

The primary Class F fly ash (FA-A) was compared to the alternate Class F fly ash (FA-B) in mixtures RG-5-50a and RG-7-50. Table 4.18 indicates that the main difference in workability between the two fly ashes was the lower viscosity and higher segregation in the mixtures with FA-B. The lower viscosity associated with FA-B was most pronounced in mixture RG-7-50. Table 4.19 shows that the 16-hour compressive strengths were similar between the two fly ashes. The 28-day strengths, however, were 6 to 7% higher for the mixtures with FA-B. Both fly ashes could be used interchangeably with minimal modifications to mixtures proportions. It may be possible to take advantage of the lower viscosity by decreasing the water-powder ratio, which would likely decrease the segregation. Decreasing the water-powder ratio would also increase the total powder content because the total paste volume would remain unchanged to ensure passing ability.

Table 4.18 Evaluation of Alternate Fly Ash: Workability

Mixture	Fly Ash	HR-A	Initial						15 Minutes						J-ring		Col. Seg.
			Slump Flow			Rheology			Slump Flow			Rheology			Δh	r-u	
			Flow	T ₅₀	VSI	τ ₀	μ	Thix	Flow	T ₅₀	VSI	τ ₀	μ	Thix			
		oz/yd ³	in.	s		Pa	Pa.s	Nm/s	in.	s		Pa	Pa.s	Nm/s	in.	in.	%
RG-5-50a	Primary (FA-A)	83	28	4.9	0	10.7	22.9	0.03	26.0	4.4	0	14.3	35.9	0.23	0.00	0.00	13.0
	Alternate (FA-B)	86	29	2.2	1.5	0.0	18.1	0.03	25.5	3.4	0	24.2	35.0	0.24	0.19	0.00	19.6
RG-7-50	Primary (FA-A)	118	28	8.0	0	0.0	55.8	0.12	26.5	10.8	0	0.0	94.1	0.84	0.25	0.00	6.6
	Alternate (FA-B)	115	28	4.4	0	0.0	32.4	0.10	26.0	8.38	0	0.0	67.2	0.77	0.13	0.00	18.0
Notes: thixotropy expressed in terms of breakdown area; concrete not agitated between initial and 15-minute test																	

Table 4.19 Evaluation of Alternate Fly Ash: Compressive Strength

Mixture	Fly Ash	16-hour			28-day
		--	8-hr	4-hr	
		72°F	120°F	170°F	72°F
		psi	psi	psi	psi
RG-5-50a	Primary (FA-A)	4499	5521	8094	11223
	Alternate (FA-B)	4342	5406	7703	11889
RG-7-50	Primary (FA-A)	4926	7108	8767	13023
	Alternate (FA-B)	5613	7192	8707	13980

Notes: 16-hour strengths identified in terms of pre-set time and maximum temperature

4.5.3 Alternate HRWRA

The effects of three different HRWRAs on slump flow and rheology were evaluated in terms of dosage response and workability retention in mixture RG-5-50a (with no RET-A). To evaluate the dosage response, a single batch of concrete was used for each HRWRA and the dosage in each batch was increased in increments. To evaluate workability retention, separate concrete batches were prepared. The HRWRA dosages in each batch were adjusted to achieve an initial slump flow of 27 inches and workability was measured at 10 minute intervals. The concrete batches were continuously mixed between tests.

The results are compared in Figure 4.21. The HRWRA dosages were expressed in terms of HRWRA solids mass per cementitious materials mass (% cm mass) to allow a consistent comparison between admixtures. For a given dosage, HR-D consistently resulted in the highest slump flow and lowest yield stress and plastic viscosity while HR-A resulted in the lowest slump flow and highest yield stress and plastic viscosity. Nonetheless, it was possible to adjust the dosages to achieve very similar rheology with each HRWRA. As the dosage was increased for each admixture, the rate of increase in slump flow and rate of decrease in yield stress decreased. The practical maximum slump flow for this particular mixture was 28 to 30 inches. Further increases in slump flow would have resulted in severe segregation. It was possible to reduce the yield stress to near zero. In contrast, the plastic viscosity decreased by a relatively smaller amount.

HR-B exhibited the best workability retention while there was little difference between HR-A and HR-D. The workability retention was relatively poor in these three mixtures, reflecting the lack of retarder and use of Type III cement. The slump flow decreased to less than

24 inches in several minutes for HR-A and HR-D and after slightly more than 10 minutes for HR-B. The increase in yield stress with time generally mirrored the decrease in slump flow. In contrast, the plastic viscosity initially declined slightly and then remained unchanged.

These results reflect the fact that the yield stress is the main fundamental difference between the workability of self-consolidating and conventionally placed concrete. HRWRA dosage most directly influences yield stress. Figure 4.22 indicates the relationship between yield stress and slump flow for all test points plotted in Figure 4.21. In order to achieve SCC levels of slump flow—namely 23 to 24 inches or greater—the yield stress for this particular mixture was near zero. Further increases in HRWRA dosage increased slump flow, though at a decreasing rate, and had minimal additional effect on concrete yield stress. Although the plastic viscosity decreased with further HRWRA dosage, it remained sufficiently high to ensure stability. As SCC flow was lost over time, the yield stress increased but the plastic viscosity did not. Therefore, it is the yield stress that must be changed to achieve SCC flow properties. Once the yield stress is near zero, the plastic viscosity reflects differences in workability between mixtures. Plastic viscosity can vary over a much wider range than yield stress for SCC mixtures. Although the main fundamental difference between the workability of self-consolidating and conventionally placed concrete mixtures is the yield stress, the plastic viscosity associated with the near zero yield stresses must not be too high nor too low for adequate SCC workability.

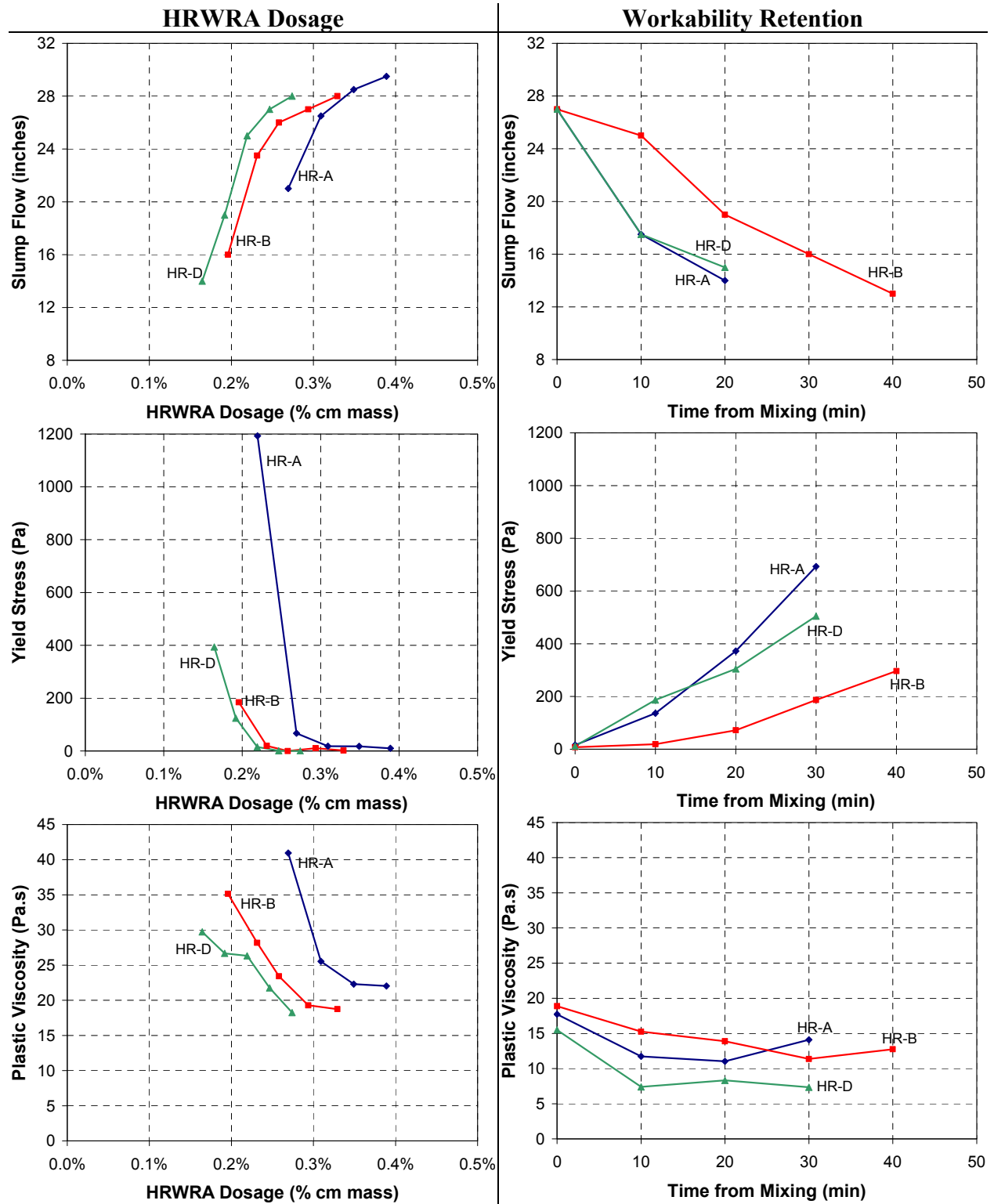


Figure 4.21 Effect of HRWRA Type on Dosage Response and Workability Retention

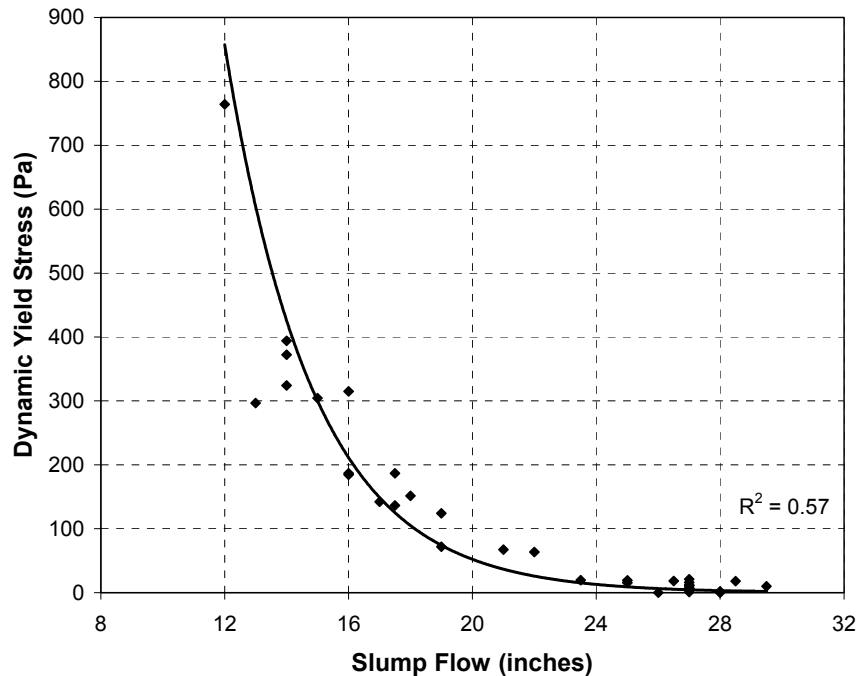


Figure 4.22 Relationship Between Dynamic Yield Stress and Slump Flow (Mixture RG-5-50a; Data From Figure 4.21)

Due to the significance of yield stress in affecting workability—including slump flow (filling ability), formwork pressure, and segregation resistance (Chapter 8)—and the close relationship between admixture dosage and yield stress, variations in admixture performance are crucial to the workability performance of SCC. Admixture performance can vary significantly between different products; therefore, admixtures must be carefully selected for the application. For instance, the combination of retarder and HRWRA influence the change in yield stress with time (workability retention). The rate of change must be balanced between having sufficiently low yield stress at the time of placement to ensure self-flow and having the yield stress increase quickly after placement to reduce formwork pressure and prevent segregation. Workability retention may range from 10 or 20 minutes to several hours depending on the admixtures, mixture proportions, weather conditions, and extent of agitation. Greater workability retention than needed may not be desirable because of the increased risk of segregation and higher formwork pressures.

4.6 Guidelines for Modifying Mixtures

In modifying mixture proportions, the unique workability of SCC must be taken into consideration. SCC mixtures are highly sensitivity to changes in materials and mixture proportions. For each SCC mixture, the HRWRA dosage must be set to reach a certain yield stress and slump flow; however, the range of HRWRA dosages corresponding to SCC slump flows is typically small. Therefore, the dosage of HRWRA must be set precisely. Further, any changes in mixture proportions—such as from variations in aggregate moisture content, change the HRWRA demand for a given slump flow.

The consequences of inadequate workability are much greater for SCC than for conventionally placed concrete. If the slump flow of SCC is too low, the concrete will not consolidate. If the slump flow is too high, severe segregation may result. If conventionally placed concrete is too stiff, it can be vibrated longer to achieve sufficient consolidation. In addition, any segregation resulting from excess water or HRWRA dosage is likely to be modest compared to the potential segregation in SCC.

Figure 4.23 shows the narrow ranges of HRWRA dosages corresponding to SCC flow properties for four different concrete mixtures. The sensitivity to changes in HRWRA shown in these plots is magnified because the HRWRA dosages were increased in increments and some loss of workability occurred between measurements of each increment. The range of HRWRA dosages corresponding to SCC slump flows ranged from 1 to 2 oz/cwt. In contrast, the T_{50} measurements, which are correlated to plastic viscosity, varied to a relatively small extent once a slump flow of approximately 24 to 25 inches was obtained (Figure 4.24).

The variation in slump flow with HRWRA dosage is further illustrated in Figure 4.25 based on the results of the multivariate regression analysis presented in Section 4.1.1. These results are not affected by workability loss as in Figure 4.23. For these mixtures, which vary in paste volume (V_p) and w/cm, the dosage corresponding to slump flows between 24 and 30 inches ranges from less than 1 oz/cwt to over 2.5 oz/cwt.

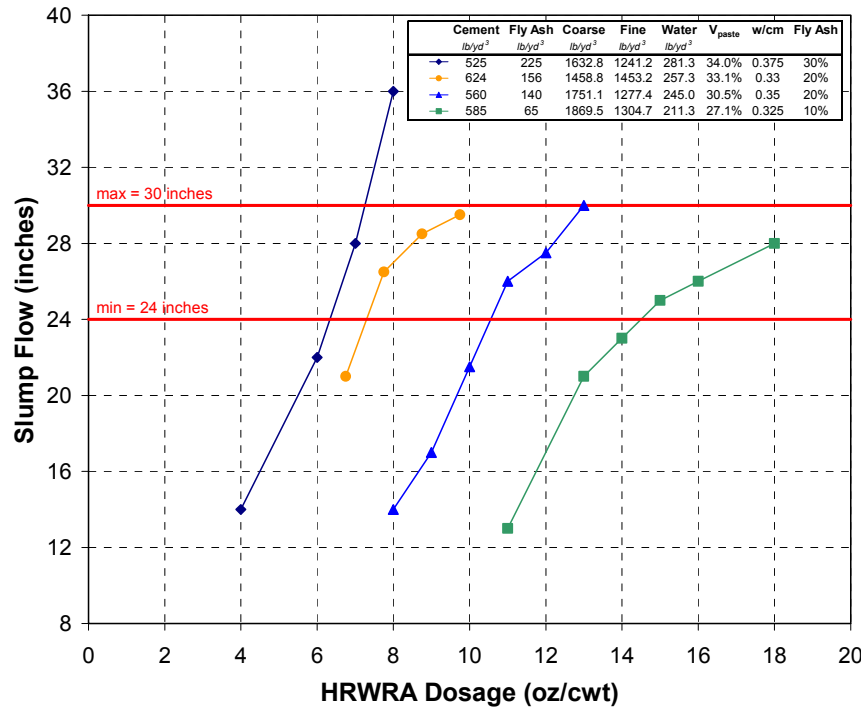


Figure 4.23 Effect of HRWRA Dosage (HR-A) on Slump Flow (Test Data)

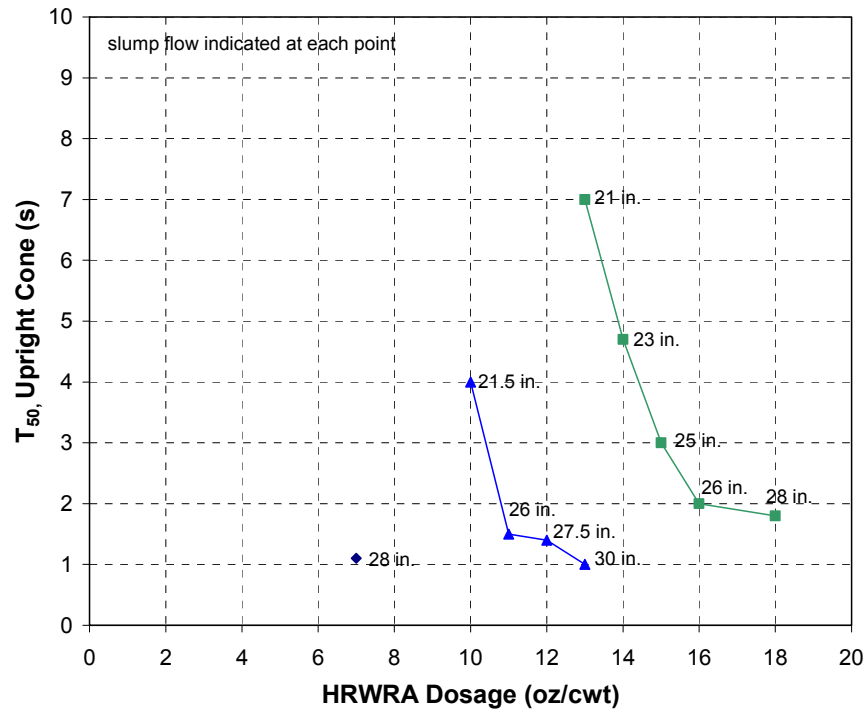


Figure 4.24 Effects of HRWRA Dosage (HR-A) on T_{50} (Test Data)

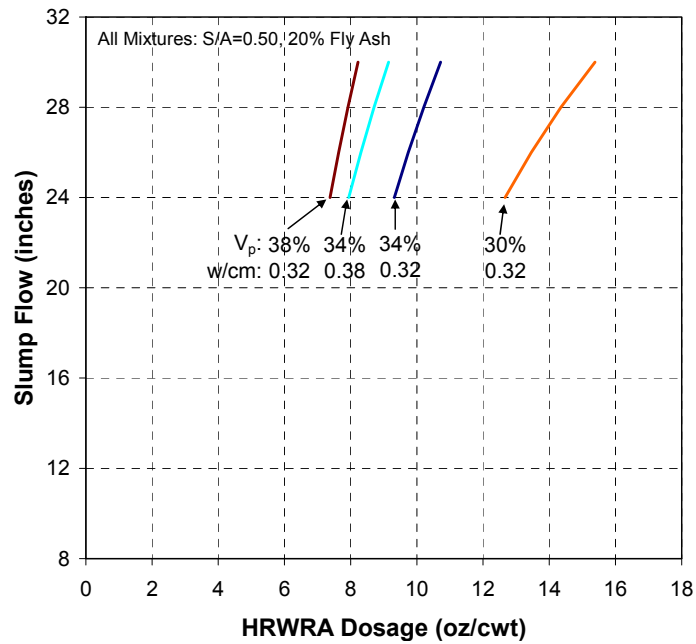


Figure 4.25 Effect of HRWRA Dosage on Slump Flow for Four Mixtures (Multivariate Regression Results)

In addition to HRWRA dosage, mixtures are sensitive to other changes in mixture proportions. If other changes occur in mixture proportions—such as from material variations, production variations, and changes in weather conditions—and the HRWRA dosage is not adjusted accordingly, the slump flow may be significantly different than expected and the mixture may not exhibit SCC flow properties. Figure 4.26 utilizes the multivariate regression results from Section 4.1.1 to illustrate the effects of fine aggregate moisture content variation. If the HRWRA dosage is held constant at 10 oz/cwt and the fine aggregate moisture content is increased by one percentage point without appropriate correction, the slump flow would be greater than 30 inches and segregation would likely be severe. If the HRWRA dosage is held constant at 10 oz/cwt and the fine aggregate moisture content is decreased by one percentage point without appropriate correction, the slump flow would be less than 24 inches and the mixture may not adequately consolidate. The effects are even more severe when the aggregate moisture content varies by 2% and appropriate corrections are not made. Figure 4.26 also shows the corresponding changes in T_{50} . If the HRWRA dosage is held constant and the aggregate moisture content is varied by one percentage point without appropriate correction, the changes in T_{50} are much greater than if the slump flow is varied from 24-30 inches by adjusting the HRWRA dosage. If the HRWRA dosage is correctly adjusted to reach a target slump flow within several inches, any deviations in T_{50} will reflect other changes in mixture proportions, such as aggregate moisture variation. Therefore, T_{50} can be used to identify changes in materials or mixture proportions.

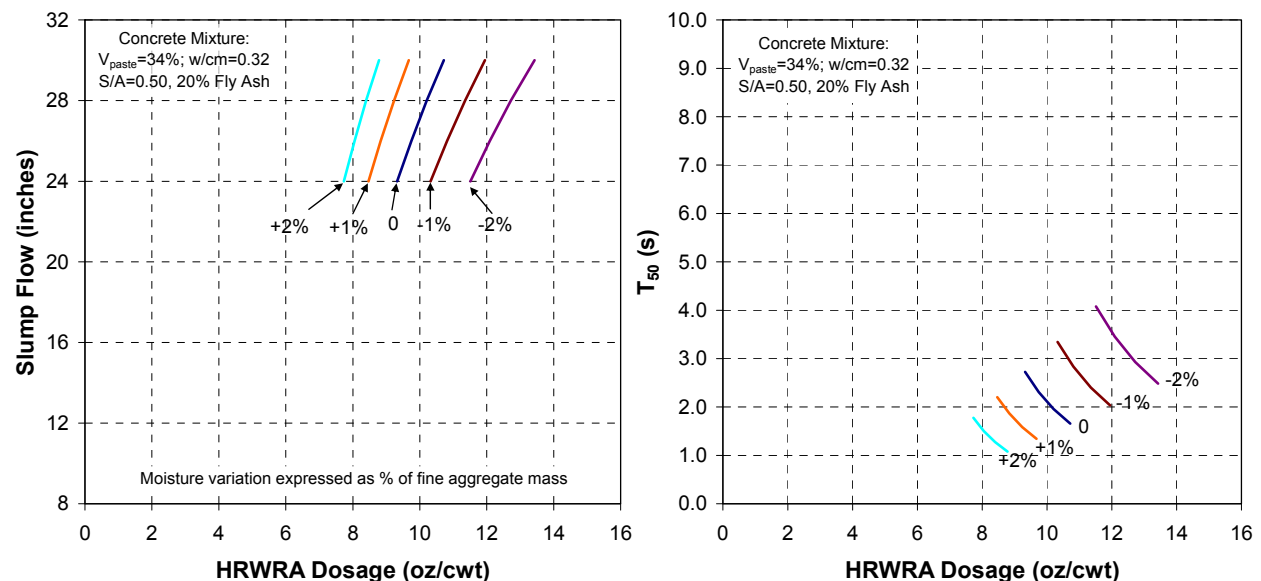


Figure 4.26 Effect of Uncorrected Fine Aggregate Moisture Variation on Slump Flow and T_{50} (Multivariate Regression Results)

During production, the options available to rectify a SCC mixture due to changes in materials, mixture proportions, and weather conditions are limited. Once mixing has started, the three main options are to increase water content, adjust HRWRA dosage, or adjust VMA dosage. If trends in performance are identified in several mixtures, additional options are available for subsequent mixtures. Changing the water content should be avoided because of its effect on hardened properties. In contrast, changing the HRWRA dosage slightly and adding VMA will

likely have minimal effects on hardened properties. Adjusting the HRWRA dosage is likely the most effective measure to accommodate batch-to-batch variations; however, it is preferable not to have to adjust the HRWRA dosage for every mixture. If the slump flow is too low, the HRWRA dosage can be increased to ensure that the concrete will consolidate adequately. If the HRWRA dosage in a given mixture has been set too high, it is possible to pause until effect of HRWRA declines sufficiently and then thoroughly mix the concrete to re-homogenize it just prior to placement. It is also possible to add VMA to offset HRWRA over-dosage; however, VMAs may not be effective enough to offset severe over-dosages. If the HRWRA dosage must be varied in a mixture, it is important to consider T_{50} . As indicated previously, T_{50} measurements can indicate changes in materials or mixture proportions—such as inaccurate aggregate moisture compensation. In addition, the viscosity must be suitable for the placement conditions. If the viscosity is too low, it may be necessary to add VMA even if the slump flow is acceptable. Due to the limited options available and the limited effectiveness of these options, it may be necessary to discard mixtures that cannot be rectified.

During the mixture proportioning process, the options to compensate for variations in material properties are greater than during production. Table 4.20, which summarized the effects of materials and mixture proportions on SCC workability, can be used to modify and optimize mixtures. Producers are likely to have multiple mixtures for use depending on expected weather conditions. In addition to the development of mechanical properties, the retarder and HRWRA type and dosage must be controlled for adequate workability retention in the expected weather conditions.

Table 4.20 Effects of Mixture Proportions on SCC Workability (Koehler and Fowler 2007)

		Slump Flow	Viscosity	Filling Ability	Passing Ability	Segregation Resistance
Aggregates	↑ Maximum Size	↑	↓	↑↓	↓	↓
	Grading	Higher pkg. density; coarser or gap grading: ↑	Higher pkg. density or gap grading: ↓	↑↓	Finer grading: ↑	Uniform or finer grading: ↑
	Improved Shape	↑	↓	↑	↑	↓
	Increased Angularity	↓	↑	↓	↓	↑
↑ Paste Volume		↑	↓	↑	↑	↑
Paste Composition	↑ Water/Powder	↑	↓	↑	↑	Not too high or too low: ↑
	Fly Ash	↑	↓	↑	↑	↑↓
	Slag	↑↓	↑↓	↑↓	↑↓	↑↓
	Silica Fume (Low %)	↑↓	↓	↑	↑	↑↓
	Silica Fume (High %)	↓	↑	↑↓	↓	↑↓
	VMA	↓	↑	↑	↑↓	↑
	HRWRA	↑	↓	↑	↑	↓
	Air	↑↓	↓	↑	↑	↑↓
Notes:						
- There are exceptions for every case. - Slump flow is inversely proportional to yield stress. Viscosity is proportional to T_{50} or v-funnel time. - This table reflects trends over the range of values typical for SCC and may not apply for extreme values. For instance, increasing water/powder to extremely high values will not improve filling or passing abilities. Stated effects assume mixtures are adjusted to achieve SCC slump flow before and after change.						

Due to the potentially severe consequences of any variations during production, the monitoring of SCC workability is more critical than for conventionally placed concrete mixtures and should be done with equal or greater frequency. The frequency of testing for slump flow and T_{50} during production should be based on typical variations in a given plant. Monitoring workability continuously with visual observations and measurements of the power drawn by mixers may be advantageous. Additionally, it may be prudent to monitor air content more regularly because of the high admixture dosages needed for SCC.

5. Early-Age Engineering Properties

The early-age engineering properties of the final concrete mixture proportions developed in Chapter 4 were evaluated in terms of setting time, isothermal and semi-adiabatic calorimetry, and compressive strength and modulus of elasticity development. This chapter presents the results of this testing.

The testing was performed based on Figure 5.1. First, isothermal calorimetry measurements were conducted with pastes to compare the heat generation and hydration rates of different combinations of admixtures and cementitious materials. The isothermal calorimetry results were also used to compute activation energy, which quantifies the temperature sensitivity of hydration. Next, semi-adiabatic calorimetry measurements were conducted to evaluate heat generation and hydration rates for each concrete mixture. Additionally, compressive strength was measured at various times and temperatures to develop maturity relationships for each mixture. Modulus of elasticity was measured over time at a constant temperature history. Lastly, computer simulations were performed based on these results to predict heat generation and strength development in actual AASHTO Type IV beams for various weather conditions. The amount of heat generated is important because the curing temperature history significantly affects 16-hour compressive strength. In addition, the total heat generated in a concrete element should be limited to prevent thermal cracking and delayed ettringite formation.

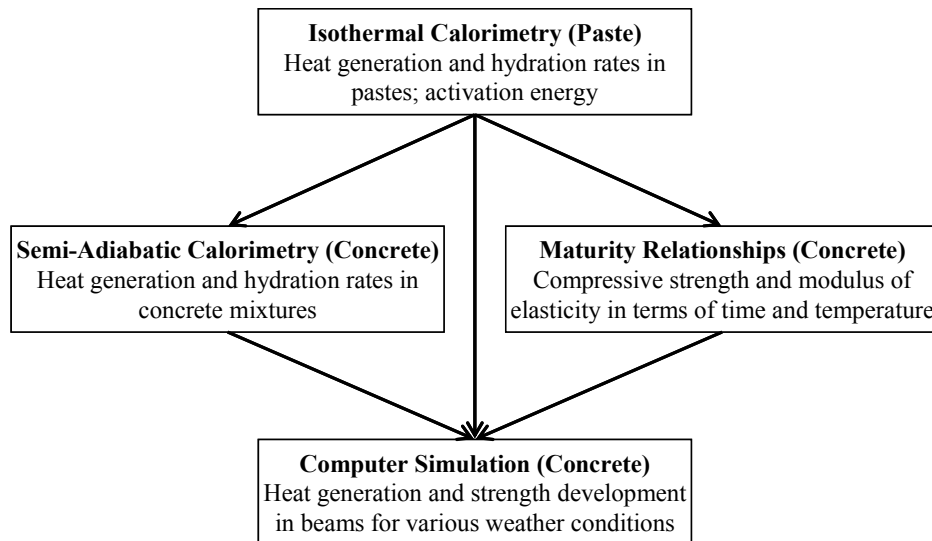


Figure 5.1 Evaluation of Early Age Properties

To evaluate the effects of hot and cold weather placement and curing conditions, concrete mixtures were tested with the cold, mild, and hot temperature scenarios shown in Table 5.1. All mixtures evaluated in this chapter were tested with the mild temperature scenario. Mixtures RG-5-50a, RG-7-50, and RG-5-C were also tested with the cold and hot temperature scenarios.

Table 5.1 Cold, Mild, and Hot Temperature Scenarios for Laboratory Testing (°F)

	Cold	Mild	Hot
Precondition Materials	45	72	90
Ambient Temperature at Mixing	72	72	72
Concrete Temperature at Mixing	57-58	77-81	87-90
Ambient Temperature After Mixing	50	72	95

The temperature scenarios shown in Table 5.1 are based on typical weather conditions in Texas and the 2004 TxDOT Specifications, which require the following:

- **Section 424.3.B.5.** The concrete temperature at the time of placement must be between 50 and 95°F. The initial time of setting should be determined on trial batches tested at a temperature representative of placement conditions.
- **Section 424.3.B.5.a.** Concrete can be placed if the ambient temperature in the shade is at least 35°F and rising or above 40°F. Adequate cold weather protection provisions must be provided when weather conditions indicate the possible need for such provisions. It is permissible to heat the aggregate and water and to use steam curing.
- **Section 424.3.B.5.b.** A retarder should be used if necessary when the air temperature is greater than 85°F. It is permissible to cool the ingredients or the concrete.
- **Section 424.3.B.7.** During the curing period, the minimum concrete temperature must be at least 50°F and the maximum temperature must not exceed 150°F for mixture design options 6-8 (Section 421.4.A.6) and 170°F for mixture design options 1-5.

5.2 Setting Time and Calorimetry

5.2.1 Setting Time

Setting time was measured in accordance with ASTM C 403 on the sieved mortar fractions from concrete mixtures. For the specimens tested with the mild weather scenario, the sieved mortar fractions were obtained from the same batches used for the semi-adiabatic calorimetry measurements. The HRWRA dosage was adjusted for each mixture to obtain a slump flow of 24 to 26 inches for the SCC mixtures and a slump of 7 to 8 inches for the conventionally placed concrete mixtures.

The setting times of the SCC and conventionally placed concrete mixtures are shown in Figure 5.2 and Figure 5.3 for the river gravel and crushed limestone mixtures, respectively (mild temperature scenario). For the nominal 5,000 psi strength level, the initial and final setting times of the SCC mixtures increased slightly relative to the conventionally placed mixtures. For the nominal 7,000 psi strength level, the initial and final setting times of the SCC mixtures were generally equal to or less than those of the conventionally placed mixtures. The nominal 7,000 psi mixtures exhibited longer initial and final setting times than the nominal 5,000 psi mixtures for both SCC and conventionally placed concrete mixtures. For the SCC mixtures, the increases in setting times were likely due to the PT-1482 admixture and increased HRWRA dosage. For

the conventionally placed concrete mixtures, the increases were likely mainly attributable to the increased dosages of HR-E, which includes a retarder.

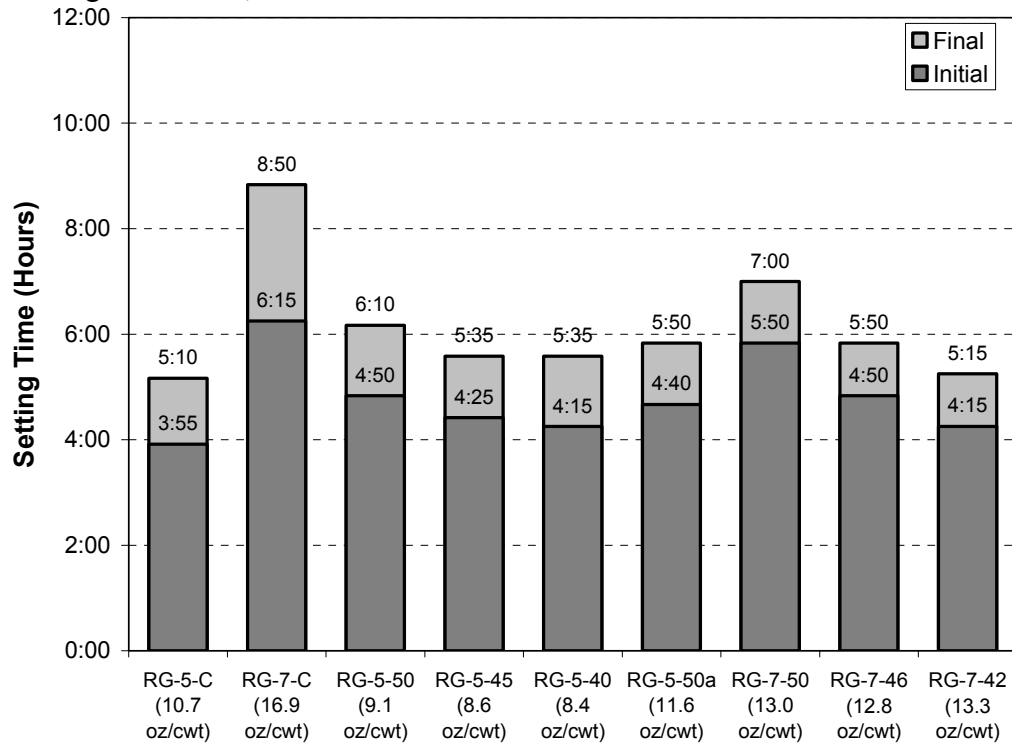


Figure 5.2 Setting Times for River Gravel Concrete Mixtures (Mild Temperature Scenario), HRWRA Dosage Indicated in Parenthesis beneath Each Column

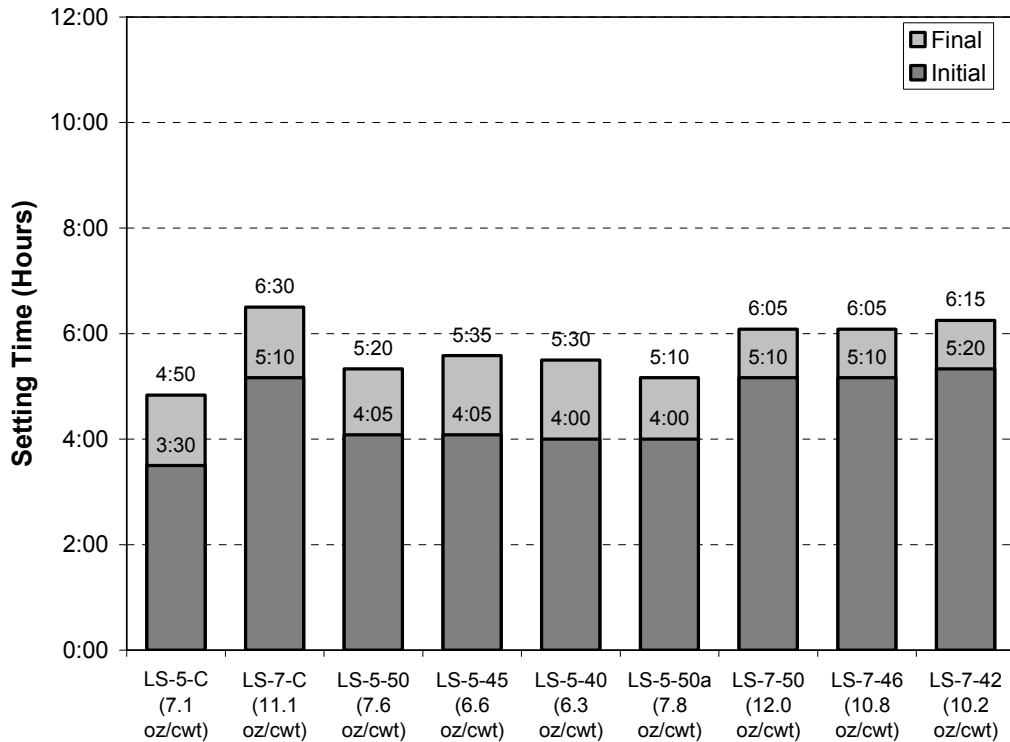


Figure 5.3 Setting Times for Crushed Limestone Concrete Mixtures (Mild Temperature Scenario), HRWRA Dosage Indicated in Parenthesis beneath Each Column

Figure 5.4 indicates that increasing the curing temperature reduced initial and final setting times in the one conventionally placed concrete and two SCC mixtures evaluated. The effect of temperature on setting time was greatest for RG-5-C and least for RG-7-50. For RG-5-50a, a constant dosage of RET-A was used for each temperature to allow for a consistent comparison between mixtures. In practice, it is possible that a retarder would not be used in cold weather conditions, provided adequate workability retention could be achieved, and that the dosage of retarder would be increased in hot weather conditions. By varying the retarder dosage based on the expected weather conditions, the variations in setting times with temperature would be reduced in this mixture. For the conventional mixture, the retarder present in HR-E was likely the main cause of the variation in setting time. The use of a different HRWRA without retarder, such as HR-A, would have likely caused less of a delay in setting time at the lower temperatures.

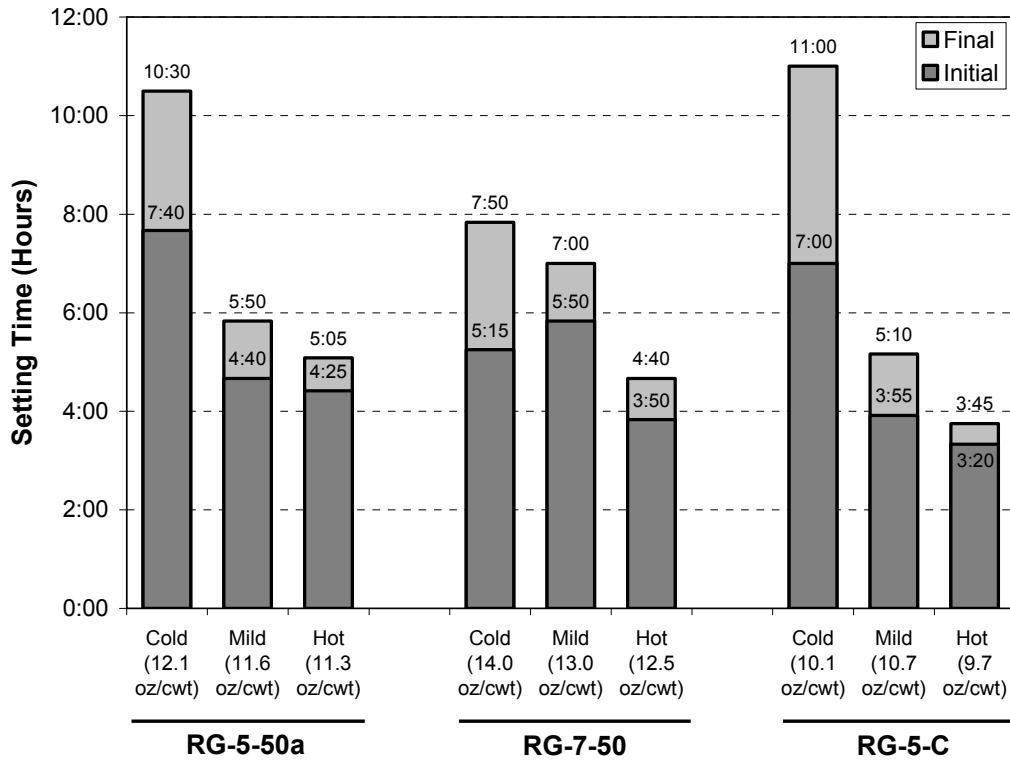


Figure 5.4 Effects of Temperature on Setting Time, HRWRA Dosage Indicated in Parenthesis beneath Each Column

The effects of the PT-1482 and RET-A admixtures on HRWRA demand, setting time, and measured nominal 16-hour compressive strength are shown in Figure 5.5. Neither admixture affected HRWRA demand; however, both caused delays in initial and final setting. PT-1482 delayed initial set by 2 hours and 40 minutes while RET-A delayed initial set by 1 hour and 20 minutes. Despite these delays in setting times, PT-1482 and RET-A did not affect the measured nominal 16-hour compressive strengths, suggesting that once initial set occurs, mixtures with PT-1482 and RET-A gain compressive strength at a faster rate than mixtures without these admixtures for a given curing temperature history. Further, the use of PT-1482 and RET-A had minimal effects on 28-day compressive strength.

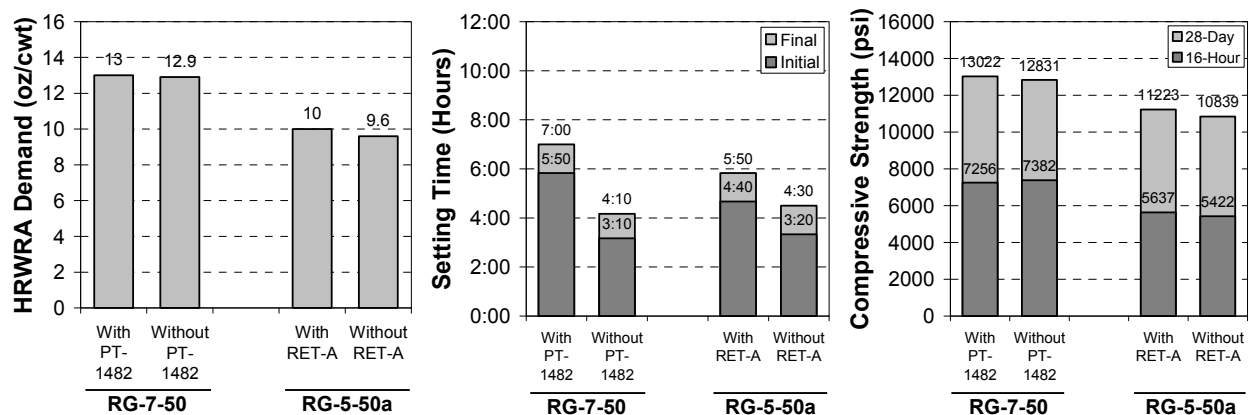


Figure 5.5 Effects of Set-Modifying Admixtures on HRWRA Demand, Setting Time, and Measured Nominal Compressive Strength (Mild Temperature Scenario)

5.2.2 Isothermal Calorimetry

In isothermal calorimetry, hydrating paste specimens are maintained at a constant temperature and the heat generated by the paste specimens is monitored continuously. The results can be used to evaluate the rate of heat evolution and the total amount of heat evolved. By measuring specimens over a range of different temperatures, the temperature sensitivity of hydration can be quantified in terms of the activation energy. By definition, activation energy is the amount of energy a molecule must acquire to take part in a reaction.

Isothermal calorimetry measurements were made on 11 paste specimens (Table 5.2) representing the range of paste compositions used in the final SCC and conventionally placed concrete mixtures presented in Chapter 4. A paste specimen mixed and measured separately from concrete may not fully reflect the properties of the paste in concrete; however, tests on paste do allow a general comparison of different admixture and cementitious materials combinations. The measurements were made with an 8-channel isothermal heat conduction calorimeter with 20 ml maximum specimen size (Thermometric 3114 TAM Air). Pastes were mixed in a blender and measured in the calorimeter at constant temperatures ranging from 5 to 60°C (41 to 140°F). Because the calorimeter requires approximately one hour to equilibrate after samples are inserted, the data from this time cannot be used and must be discarded. Therefore, the initial heat of solution is not analyzed and the total heat evolved is underestimated slightly.

Table 5.2 Paste Compositions for Isothermal Calorimetry Measurements

ID	w/cm	Fly Ash	HR-A	HR-E	RET-A	PT-1482
		%	% <i>cm mass</i>	% <i>cm mass</i>	% <i>c mass</i>	% <i>c mass</i>
1	0.27	20	0.33%			2.43%
2	0.27	20	0.33%			
3	0.28	32	0.25%		0.04%	
4	0.28	32	0.25%			
5	0.27	40	0.19%		0.04%	
6	0.33	20	0.29%		0.04%	
7	0.285	0		0.49%		
8	0.42	0		0.20%		
9	0.28	0				
10	0.28	32				
11	0.28	0	0.25%			
Notes:						
1.) Pastes 1-6 and 11 represent SCC mixtures; pastes 7-10 represent conventional placed concrete mixtures						
2.) Admixture dosages expressed as % admixture solids mass per cementitious materials mass (HRWRAs) or per cement mass (retarder and PT-1482). HRWRA dosages equivalent to those used in concrete mixtures.						

5.2.2.2 Heat Evolution at 23°C (73.4°F)

Paste specimens measured at 23°C (73.4°F) were compared in terms of the timing, rate, and amount of heat evolved. Other aspects of hydration—such as the reaction of individual phases and the role of sulfate—were not evaluated.

The effects of fly ash, HRWRA, and retarder on the rate of heat evolution are shown in Figure 5.6 for 5 pastes with a w/cm of 0.28 and tested at 23°C (73.4°F). The addition of fly ash delayed the start of heat evolution and reduced the peak rate of heat evolution. This reduction was expected due to the lack of reactivity of low calcium fly ashes at early ages. HRWRA delayed the start of heat evolution but increased the maximum rate of heat evolution. This behavior was expected because HRWRAs delay initial setting but result in improved dispersion and more efficient hydration (Jeknavorian et al. 2003; Bury and Christensen 2002). When combined, fly ash and HRWRA resulted in a delayed start of heat evolution and increased peak rate of heat evolution relative to the cement only paste with no HRWRA. The addition of retarder to the paste with fly ash and HRWRA further delayed the start of heat evolution and slightly reduced the peak rate of heat evolution. In terms of total heat evolved, the mixtures with fly ash evolved less heat per mass of cementitious materials at all ages measured. However, when compared per mass of cement only, as shown in Figure 5.7, the mixtures with fly ash evolved heat at a faster rate per cement mass after a certain time (6-8 hours) and generated more total heat per cement mass, reflecting the eventual contribution of fly ash to hydration. These results help to justify the use of w/c for early-age properties and w/cm for long-term properties

when fly ash with low initial reactivity is used. The final SCC mixtures developed in Chapter 4 had similar or only slightly higher total cement contents as the comparable conventional mixtures; therefore, Figure 5.7 is a relevant comparison.

The effects of fly ash are further indicated in Figure 5.8. Increasing the fly ash rate from 20 to 40% reduced the rate and total amount of heat evolved expressed per mass of cementitious materials. When expressed per mass of cement only, as shown in Figure 5.9, the rate and total amount of heat evolved were essentially the same. In addition to changing the fly ash rate, the w/cm and HR-A dosage were varied to achieve constant strength and workability characteristics in the corresponding concrete mixtures.

The isothermal calorimetry results for the conventional mixtures are shown in Figure 5.10. Compared to an SCC mixture with only HR-A (no fly ash or retarder), the use of HR-E in the conventional mixtures delayed the start of heat evolution, but did not reduce the maximum rate of heat evolution. The delay in the start of heat evolution was much longer for the conventional mixture with the w/cm of 0.285, which was likely mainly attributable to the much higher dosage of HR-E. As indicated previously, HR-E includes a retarder whereas HR-A does not. The SCC paste with a w/cm of 0.28, 32% fly ash, HR-A, and retarder is shown for further reference. Despite the delayed setting time associated with HR-E, the total heat evolved was greater after 40 hours for the conventional mixture with a w/cm of 0.42 and approximately the same for the conventional mixture with w/cm of 0.285, as compared to the SCC mixture with a w/cm of 0.28 and HR-A. It should be noted that the total mass of cementitious materials is greater in all of the final SCC mixtures presented in Chapter 4 than in the conventional mixtures; however, the cement contents for a given compressive strength level were similar. Therefore, a further distinction should be made between heat evolved per mass of cementitious materials and per unit volume of concrete.

Figure 5.11 indicates that PT-1482 only slightly delayed the start of heat evolution but significantly reduced the maximum heat evolved. As a result, the total heat evolved was consistently less for the paste with PT-1482, especially between 8 and 20 hours.

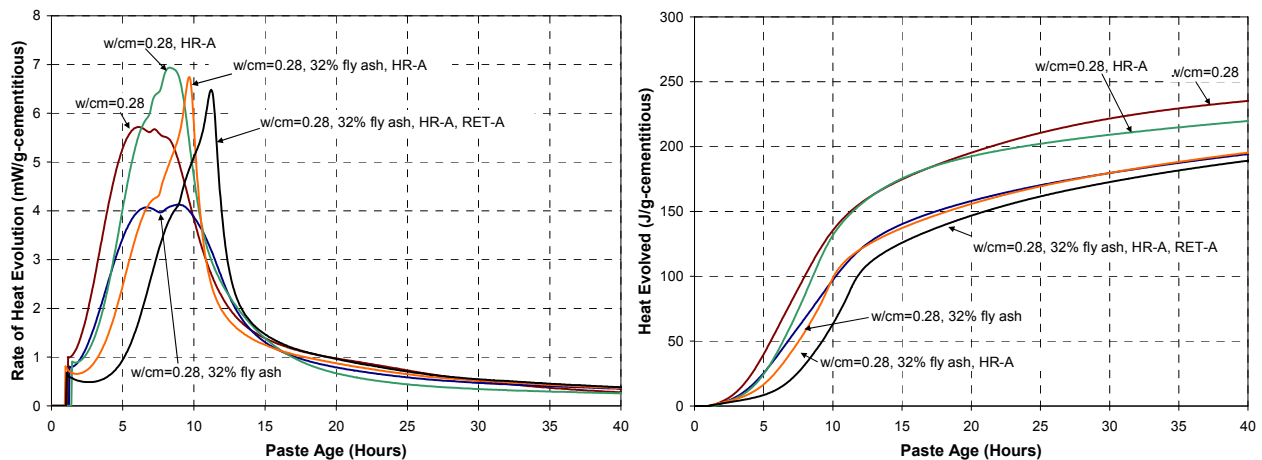


Figure 5.6 Isothermal Calorimetry Results (23°C, per cementitious materials mass)

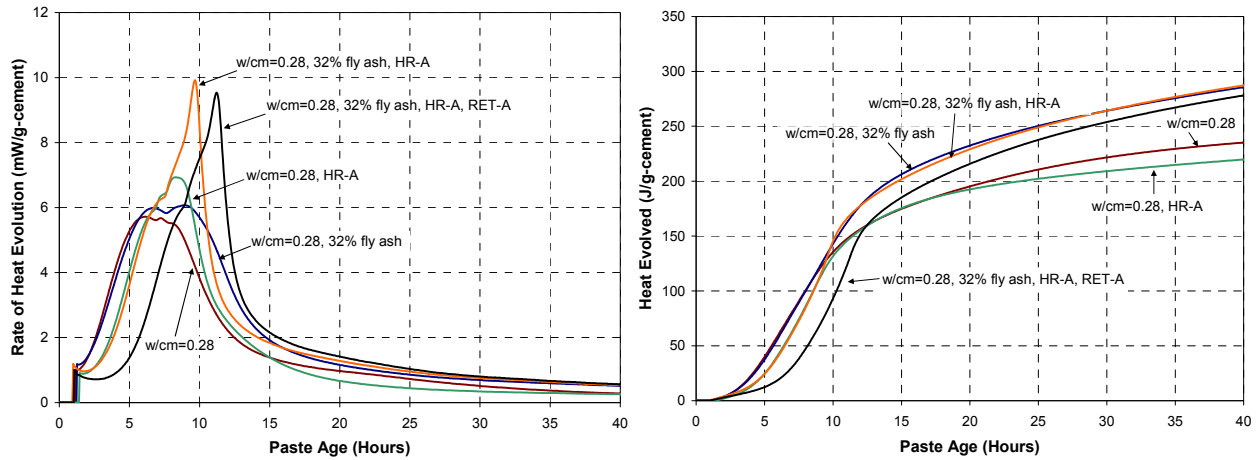


Figure 5.7 Isothermal Calorimetry Results (23°C, per cement mass)

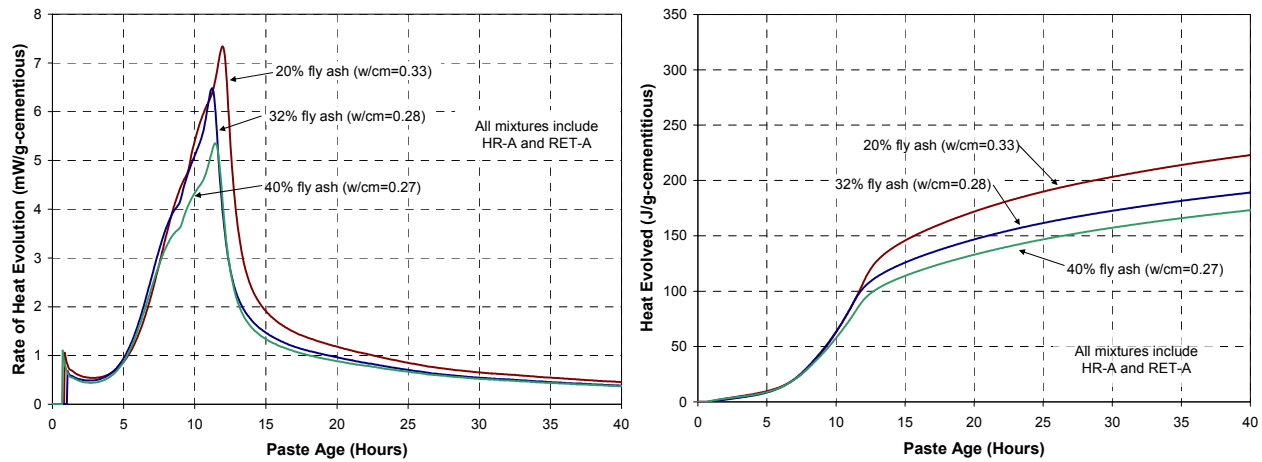


Figure 5.8 Isothermal Calorimetry Results: Effects of Fly Ash (23°C, per cementitious materials mass)

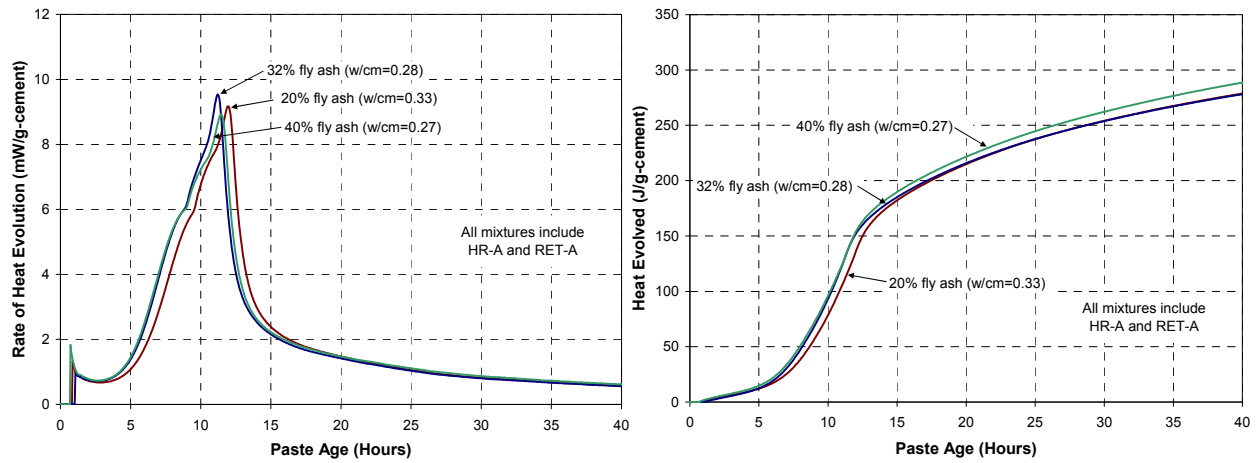


Figure 5.9 Isothermal Calorimetry Results: Effects of Fly Ash (23°C, per cement materials mass)

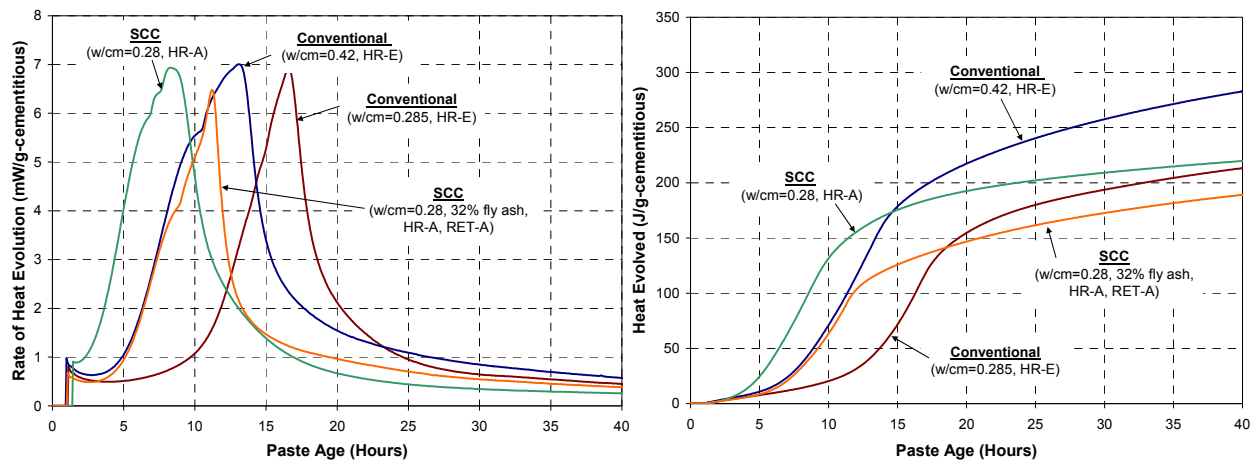


Figure 5.10 Isothermal Calorimetry Results: Pastes Representing Conventional Placed Concrete (23°C, per cementitious materials mass)

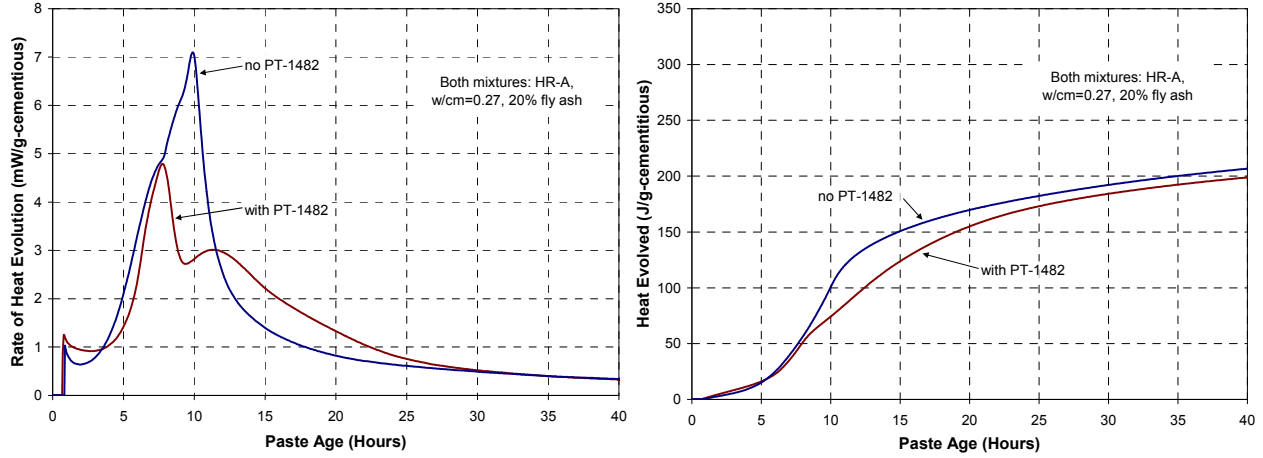


Figure 5.11 Isothermal Calorimetry Results: Effects of PT-1482 (23°C, per cementitious materials mass)

5.2.2.3 Activation Energy

Activation energy was determined from the isothermal calorimetry measurements with the modified ASTM C 1074 method described by Poole et al. (2007). In this procedure the activation energy is assumed to be independent of temperature. The procedure is described as follows:

1. Measure the total heat generated over time ($H(t)$) at 5, 15, 23, 38, and 60°C.
2. Calculate the ultimate heat of hydration (H_u , J/gram) based on Equation (5.1), which was developed by Schindler (2002):

$$H_u = H_{cem} \cdot p_{cement} + 1800 \cdot p_{flyash\_CaO} \cdot p_{flyash} \quad (5.1)$$

where H_{cem} is the heat of hydration of cement at 100% degree of hydration (J/gram), p_{cement} is the cement mass to total cementitious materials mass ratio, p_{flyash_CaO} is the fly ash CaO mass to total fly ash mass ratio, and p_{flyash} is the fly ash mass to total cementitious materials mass ratio. The value of H_{cem} is calculated with Equation (5.2), which was also developed by Schindler (2002):

$$H_{cem} = 500 \cdot p_{C_3S} + 260 \cdot p_{C_2S} + 866 \cdot p_{C_3A} + 420 \cdot p_{C_4AF} + 624 \cdot p_{SO_3} + 1186 \cdot p_{FreeCa} + 850 \cdot p_{MgO} \quad (5.2)$$

where p_i is the mass fraction of the i -th component to the total mass of cement ratio.

3. Calculate the degree of hydration with time ($\alpha(t)$) at each temperature, based on (5.3):

$$\alpha(t) = \frac{H(t)}{H_u} \quad (5.3)$$

4. Fit the three-parameter model shown in Equation (5.4) to the degree of hydration data:

$$\alpha(t) = \alpha_u \cdot e^{-\left[\frac{\tau}{t}\right]^\beta} \quad (5.4)$$

where α_u , β , and τ are empirical constants (α_u and β assumed to be independent of temperature).

5. Fit a straight line to the plot of $\ln(\tau)$ versus $1/T_c$, where T_c is the temperature in Kelvin. The slope of the resulting line is used to calculate activation energy (E_a) with Equation (5.5):

$$E_a = -\frac{\ln\left(\frac{\tau_{ref}}{\tau_c}\right)}{\left(\frac{1}{T_{ref}} - \frac{1}{T_c}\right)} \cdot R \quad (5.5)$$

where R is ideal gas constant (8.314 J/(K mol)).

The results of this analysis, which are shown in Table 5.3, indicate that the activation energies of the pastes representing the SCC mixtures varied from 34 to 37 kJ/mol. These activation energies were similar to the 35 kJ/mol determined for the two pastes representing the conventionally placed concrete mixtures. The activation energy of the paste with PT-1482, however, was significantly reduced to 24.6 kJ/mol.

Table 5.3 Activation Energies Determined from Isothermal Calorimetry

ID	w/cm	Fly Ash	HR-A	HR-E	RET-A	PT-1482	Activation Energy
		%	% <i>cm mass</i>	% <i>cm mass</i>	% <i>c mass</i>	% <i>c mass</i>	<i>kJ/mol</i>
1	0.27	20	0.33%			2.43%	24.6
2	0.27	20	0.33%				34.0
3	0.28	32	0.25%		0.04%		36.3
4	0.28	32	0.25%				37.2
5	0.27	40	0.19%		0.04%		35.7
6	0.33	20	0.29%		0.04%		34.7
7	0.285	0		0.49%			35.2
8	0.42	0		0.20%			35.3
Notes:							
1.) Pastes 1-6 represent SCC mixtures, 7-8 conventional placed concrete mixtures							
2.) Admixture dosages expressed as % admixture solids mass per cementitious materials mass (HRWRAs) or per cement mass (RET-A and PT-1482). HRWRA dosages equivalent to those used in concrete mixtures.							

5.2.3 Semi-Adiabatic Calorimetry

Adiabatic temperature development was calculated from semi-adiabatic calorimeter measurements of 16 SCC and 4 conventionally placed concrete mixtures. In adiabatic calorimetry, a concrete specimen is stored so that no heat is lost and the total heat generated is monitored over time. In semi-adiabatic calorimetry, some heat loss from the specimen occurs. This heat loss is accounted for and used to calculate the adiabatic temperature rise.

The semi-adiabatic calorimeter consisted of an insulated, 55-gallon steel drum with a 6x12-inch cylindrical concrete specimen positioned inside. Measurements were made of the concrete temperature, heat loss through the calorimeter walls, and ambient temperature surrounding the calorimeter. These measurements were used to calculate the adiabatic temperature rise based on the procedure described by Schindler (2002). The concrete mixtures tested had fresh temperatures of 75 to 81°F and were measured in the semi-adiabatic calorimeter for 7 days.

For each mixture, the three-parameter hydration model in Equation (5.4) was fit to the results and the adiabatic temperature rise was computed, as summarized in Table 5.4. For both aggregate sets and strength levels, the SCC mixtures generated greater adiabatic temperature rise than the comparable conventional mixtures, which was likely due to the reduced w/cm, higher cementitious materials content, and improved dispersion and hydration from the HRWRA. Figure 5.12 indicates that the conventionally placed concrete mixtures at the 5,000 psi nominal strength level initially generated adiabatic temperature rise similar to the SCC mixtures, but generated less adiabatic temperature rise after approximately 10 hours. The initial starts of adiabatic temperature rise in the 7,000 psi conventionally placed concrete mixtures were delayed significantly for both aggregate sets. These delays were related to the high dosage of HR-E

required. The 7,000 psi SCC mixtures generated less heat than the 5,000 psi SCC mixtures prior to 24 hours. Figure 5.13 re-plots data from Figure 5.12 to present a direct comparison between mixtures from the river gravel and crushed limestone aggregate sets. The timing, rate, and total amount of adiabatic temperature rise were similar for the two aggregate sets because the paste composition was established for the same nominal 16-hour compressive strengths.

Figure 5.14 indicates that the use of RET-A delayed the start and reduced the total amount of adiabatic temperature rise. In contrast, the use of PT-1482 delayed the start of adiabatic temperature rise but only slightly reduced the total amount of adiabatic temperature rise at 100 hours. In Figure 5.5, the use of PT-1482 or RET-A was shown to affect the nominal 16-hour compressive strength only slightly; however, the comparison of nominal strengths may not be fully relevant to actual concrete members. It is possible that the strength of concrete mixtures with PT-1482 or RET-A would be further reduced in actual concrete members due to the lower amount of heat generated by mixtures with PT-1482 or RET-A and the associated lower member temperatures. Such a scenario was not tested.

Table 5.4 Summary of Semi-Adiabatic Calorimetry Results

Mixture	E_a	H_u	Hydration Parameters (Eq. (5.4))			Adiabatic Temp. Rise	
			α_u	τ	β	16 hr	100 hr
	<i>kJ/mol</i>	<i>J/kg</i>				<i>°F</i>	<i>°F</i>
RG-5-C	35	466	0.769	11.740	1.135	83.6	97.7
RG-5-50	35	398	0.690	12.970	1.100	98.5	116.1
RG-5-45	35	398	0.681	12.718	1.077	99.3	114.5
RG-5-40	35	398	0.670	12.857	1.060	96.3	113.2
RG-5-50a	35	423	0.727	12.722	1.108	91.9	107.7
RG-7-C	35	466	0.695	16.345	1.623	75.0	99.6
RG-7-50	25	423	0.701	13.461	1.120	86.3	114.1
RG-7-46	25	423	0.699	12.663	1.060	90.0	116.6
RG-7-42	25	423	0.684	12.632	1.014	85.3	112.8
LS-5-C	35	466	0.775	10.019	1.256	78.7	88.5
LS-5-50	35	381	0.692	14.543	0.905	97.7	122.1
LS-5-45	35	381	0.681	14.611	0.944	95.2	120.3
LS-5-40	35	381	0.688	13.878	0.955	97.0	120.4
LS-5-50a	35	396	0.736	13.646	0.934	91.8	112.8
LS-7-C	35	466	0.732	12.039	1.558	86.9	95.8
LS-7-50	25	423	0.700	12.454	1.132	87.2	111.2
LS-7-46	25	423	0.700	12.163	1.160	87.8	110.8
LS-7-42	25	423	0.717	12.152	1.176	88.7	112.4
RG-5-50a (no RET-A)	35	423	0.821	12.519	0.876	100.9	120.7
RG-7-50 (no PT-1482)	35	423	0.684	10.934	1.083	104.9	116.4

E_a determined from isothermal measurements; H_u computed with Equation (5.1)

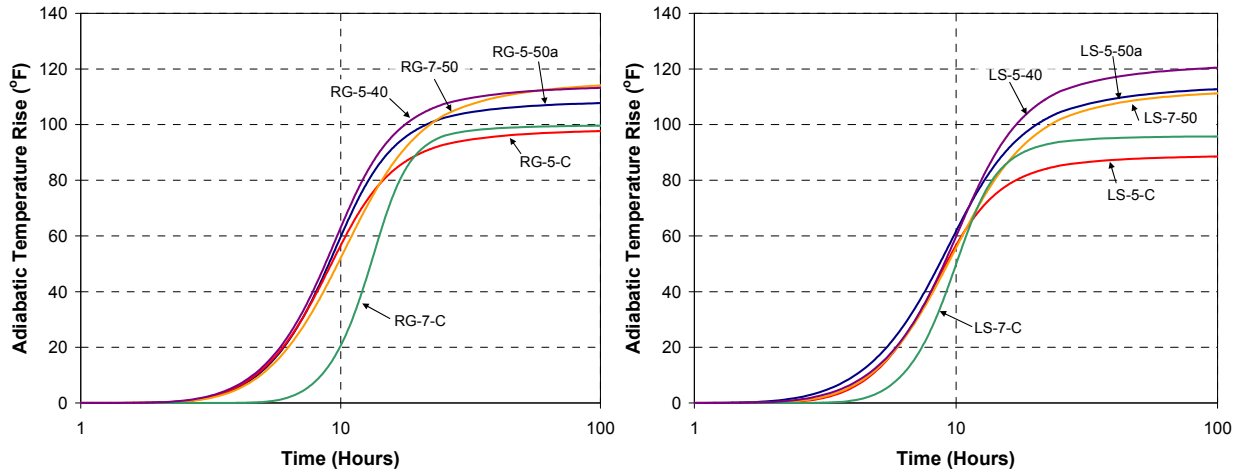


Figure 5.12 Calculated Adiabatic Temperature Rise (Semi-Adiabatic Calorimetry Results)

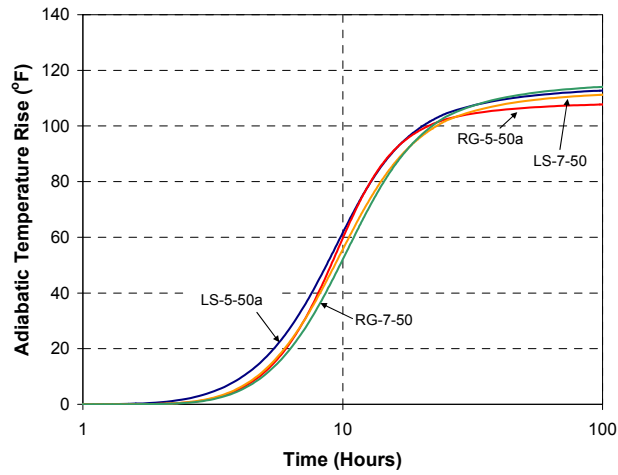


Figure 5.13 Calculated Adiabatic Temperature Rise: Effects of Aggregate Type and Associated Differences in Mixture Proportions (Semi-Adiabatic Calorimetry)

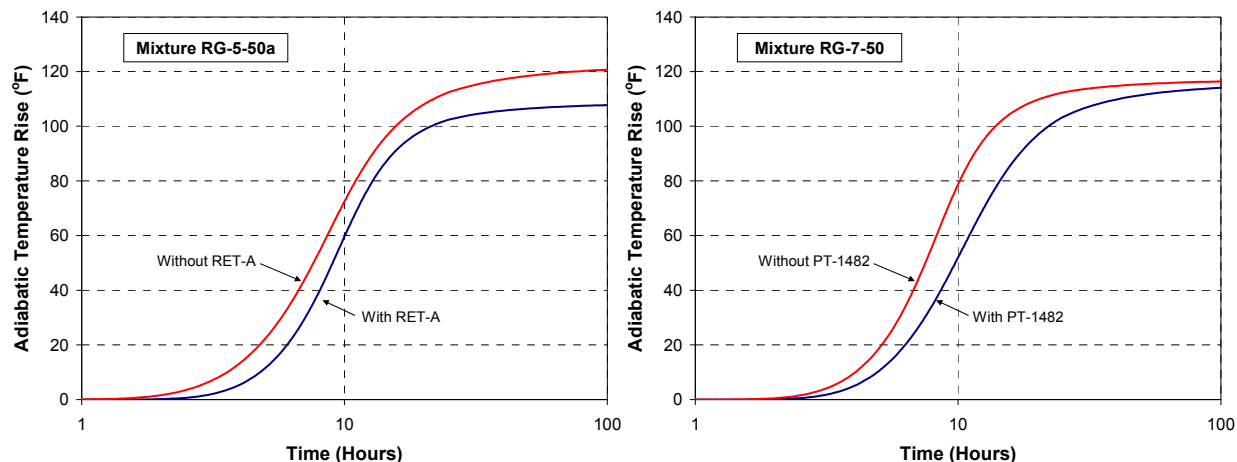


Figure 5.14 Calculated Adiabatic Temperature Rise: Effects of RET-A and PT-1482 (Semi-Adiabatic Calorimetry)

5.3 Compressive Strength

In Chapter 4, each concrete mixture was defined in terms of its nominal 16-hour compressive strength, which was achieved with a specified temperature history (8-hour pre-set time, 120°F maximum temperature). Because actual early-age compressive strengths vary significantly with time and temperature, the 16-hour strengths were tested with varying curing temperature histories and compressive strength-maturity relationships were developed. Traditionally, compressive strength-maturity relationships are developed by testing specimens cured with the same temperature history but tested at different ages, such as in ASTM C 1074. In this project, however, match curing equipment was used to apply various temperature histories to concrete specimens taken from the same batch and tested at 16 hours. This approach better represents construction procedures at precast concrete plants.

For the mixtures that were batched and mixed under the mild temperature scenario shown in Table 5.1, the temperature histories shown in Figure 5.15 were used. These temperature histories vary in the pre-set period and maximum temperature. The pre-set period was 4, 6, or 8 hours and the maximum temperature was 120, 145, or 170°F. One mixture was also cured under laboratory conditions of 72°F ambient temperature.

Mixture RG-5-50a, RG-7-50, and RG-5-C were also batched and mixed under the cold and hot temperature scenarios shown in Table 5.1. Specimens from these batches were cured under constant ambient temperatures of 50°F or 95°F and were tested periodically from 16 to 48 hours for the hot temperature scenario and from 16 to 144 hours for the cold temperature scenario. Separate cylinders from each batch were match cured according to the temperature histories shown in Figure 5.16.

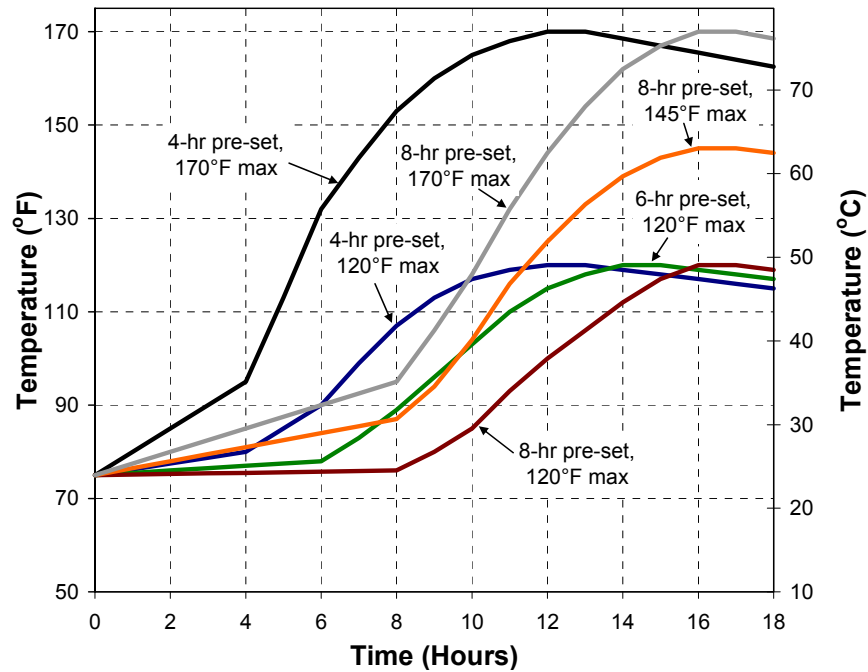


Figure 5.15 Temperature Histories for Development of Compressive Strength-Maturity Relationships (Specimens Cast at Mild Temperature Scenarios)

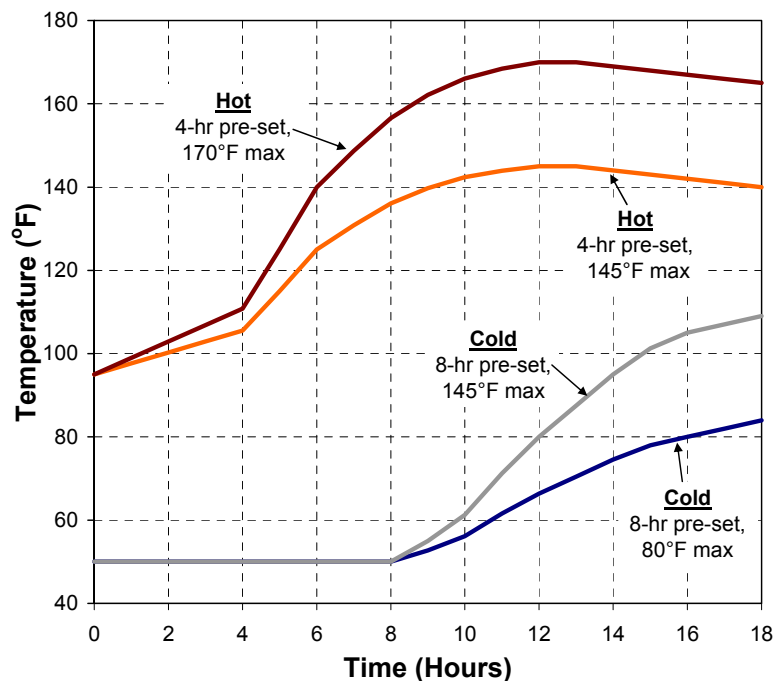


Figure 5.16 Temperature Histories for Development of Compressive Strength-Maturity Relationships (Specimens Cast at Hot and Cold Temperature Scenarios)

Figure 5.17, which shows data for selected batches mixed and cast under the mild temperature scenario, indicates that 16-hour compressive strengths varied widely as a function of the curing temperatures. For instance, the 16-hour compressive strength varied from 4,500 to 8,100 psi for mixture RG-5-50a and from 4,900 to 9,000 psi for mixture RG-7-50. The compressive strength of the nominal 5,000 psi mixtures increased linearly with increasing maximum temperature. In contrast, the nominal 7,000 psi mixtures exhibited diminishing increases in strengths as the maximum temperature was increased. Full results for all mixtures tested are provided in Appendix C. These results, along with knowledge of the amount of heat generated by the concrete mixtures, can be used to select curing conditions and mixture proportions based on expected weather conditions.

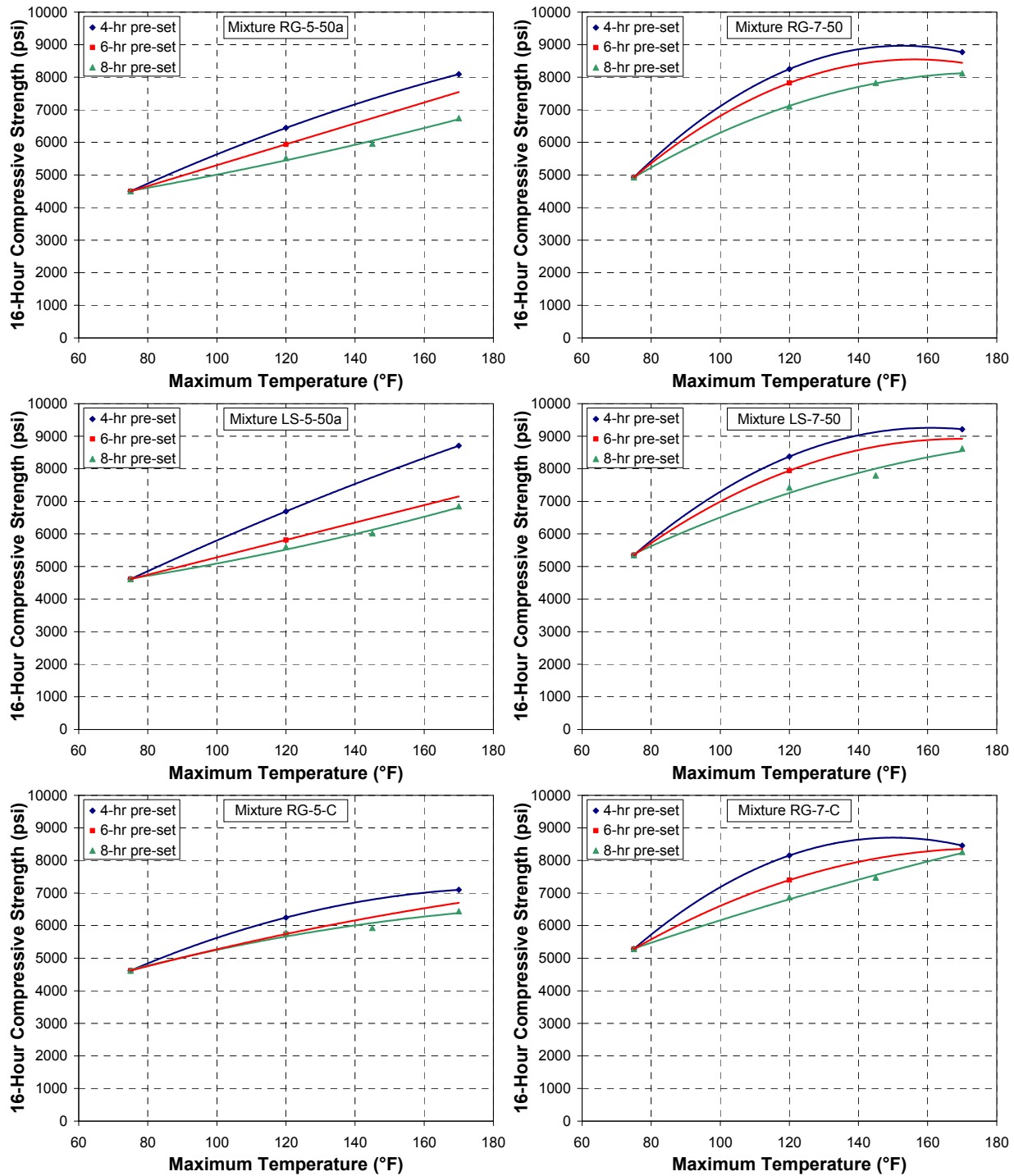


Figure 5.17 Effects of Pre-Set and Maximum Curing Temperature on 16-Hour Strength of Selected Mixtures (Specimens Mixed and Cast at Mild Temperature)

The compressive strength-maturity relationships were developed based on the equivalent-age concept. Equivalent age (t_e) was calculated with Equation (5.6), which is based on Freiesleben Hansen and Pedersen (1977):

$$t_e(T_r) = \sum_0^t \exp\left(\frac{E_a}{R} \left(\frac{1}{T_c} - \frac{1}{T_r}\right)\right) \cdot \Delta t \quad (5.6)$$

where T_r is the reference temperature in Kelvin (23°C or 296 Kelvin), T_c is the temperature of concrete in Kelvin, and t is the time. The activation energies were taken as 25 kJ/mol for the nominal 7,000 psi strength SCC mixtures and 35 kJ/mol for all other mixtures. Compressive strength was then calculated as a function of equivalent age with the three-parameter model shown in Equation (5.7):

$$f_c(t_e) = (f_c)_{ult} \cdot e^{-\left[\frac{\tau_s}{t_e}\right]^{\beta_s}} \quad (5.7)$$

where $(f_c)_{ult}$, τ_s , and β_s are empirical constants. This equation is analogous to Equation (5.4) for degree of hydration.

The resulting compressive strength-maturity relationships are listed in Table 5.5. These equations are only applicable to the conditions under which they were developed—namely, tests for release-of-tension strength conducted under the range of temperature histories considered.

The value of $(f_c)_{ult}$ does not necessarily represent the ultimate compressive strength. The results for four selected mixtures are plotted in Figure 5.18.

Table 5.5 Compressive Strength-Maturity Relationships

Mixture	Hydration Model Parameters (Equation (5.7))			
	$(f_c)_{ult}$	τ_s	β_s	R^2
<i>Mild Temperature</i>				
RG-5-C	25613	420.9	0.152	0.91
RG-5-50	32363	350.0	0.213	0.95
RG-5-40	30730	378.7	0.198	0.97
RG-5-50a	30274	354.8	0.203	0.96
RG-7-C	8655	10.8	1.856	0.95
RG-7-50	8647	13.5	3.610	0.97
RG-7-42	9070	12.1	2.637	0.95
LS-5-C	25364	335.5	0.184	0.91
LS-5-50	34815	202.3	0.277	0.91
LS-5-40	36488	327.7	0.246	0.94
LS-5-50a	33017	362.7	0.217	0.94
LS-7-C	10068	8.7	0.956	0.95
LS-7-50	9288	12.5	2.633	0.96
LS-7-42	8973	12.9	3.375	0.96
<i>Hot Temperature</i>				
RG-5-C	6859	11.8	1.800	0.75
RG-5-50a	8973	15.0	1.028	0.95
RG-7-50	17671	14.9	0.354	0.82
<i>Cold Temperature</i>				
RG-5-C	7419	13.9	2.069	0.96
RG-5-50a	8821	16.5	1.241	0.98
RG-7-50	10376	23.0	1.278	0.99

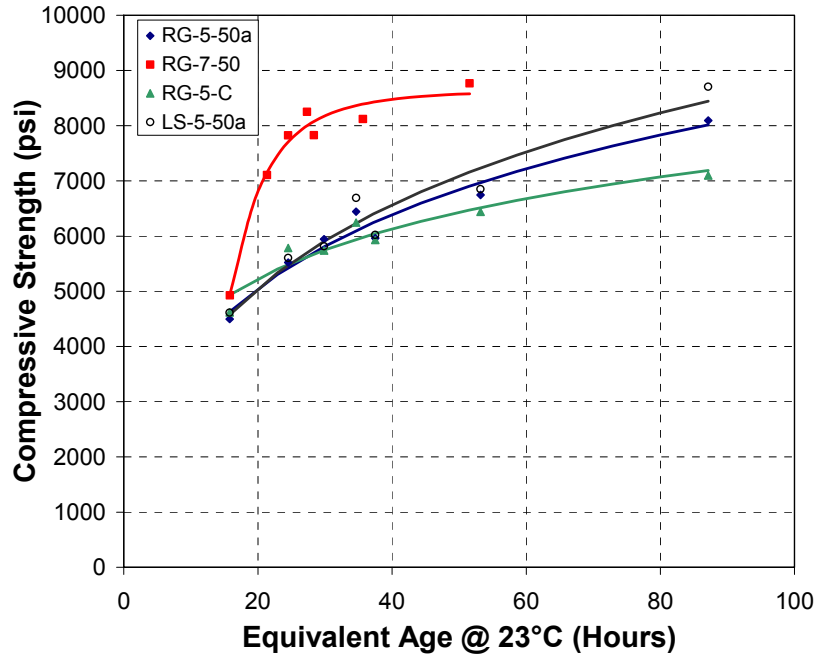


Figure 5.18 Compressive Strength-Maturity Relationships for Four Selected Mixtures (Specimens Mixed and Cast at Mild Temperature Scenario)

The compressive strength-maturity relationships are shown Figure 5.19 for specimens mixed and cast in the hot, mild, and cold temperature scenarios. For mixtures RG-5-50a and RG-5-C, the relationships were similar despite the different mixing and curing temperatures. In contrast, mixture RG-7-50 gained less strength for a given equivalent age when mixed and cured at the cold temperature scenario. Although the setting time of RG-7-50 was not delayed as significantly as the other mixtures at cold temperature scenario, it did not gain strength as quickly once set occurred.

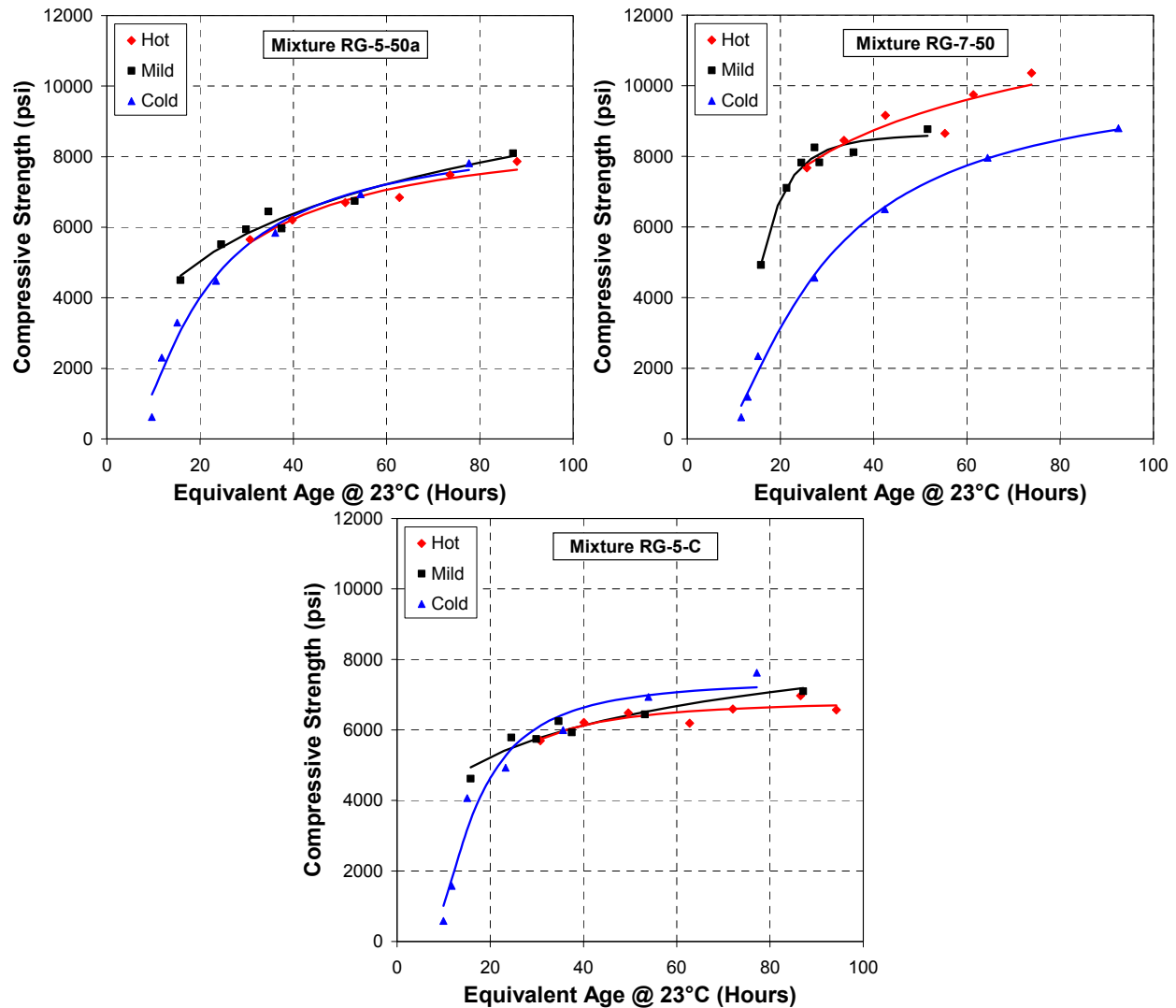


Figure 5.19 Compressive Strength-Maturity Relationships for Specimens Mixed and Cured in Hot, Mild, and Cold Temperature Scenarios

The effect of curing temperature on 28-day compressive strength is shown in Figure 5.20. Specimens cured at higher temperatures are generally known to exhibit lower long-term strength. Such a trend was evident for RG-5-50a and RG-5-C, but not for RG-7-50.

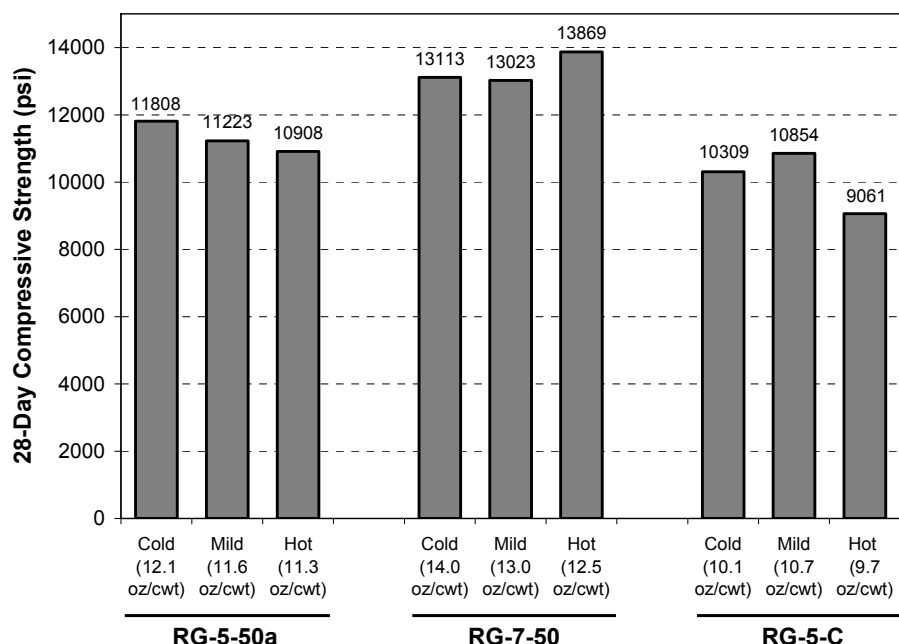


Figure 5.20 Effects of Curing Temperature on 28-Day Compressive Strength

5.4 Modulus of Elasticity

The development of static and dynamic modulus of elasticity over time was measured for 6 concrete mixtures (RG-5-50, RG-5-45, RG-5-40, RG-5-50a, LS-5-50a, RG-5-C). The static modulus of elasticity, also referred to as the chord modulus, is determined by loading a specimen to 40% of the compressive strength and measuring the axial stress and strain in the specimen at a strain of 50 μ -strain and a stress of 40% of the compressive strength. The static modulus of elasticity is the slope of the line between these two points on the stress-strain curve. The dynamic elastic modulus is determined by generating stress waves and measuring their propagation through the specimen. Due to the low strain rates associated with the stress waves, the dynamic modulus of elasticity is approximately equal to the initial tangent modulus. Multiple methods are available for measuring dynamic modulus of elasticity (ASTM C 215; ASTM C 597; ASTM C 1259; Philleo 1955; Rix, Bay, and Stokoe 1990; Lydon and Iacovou 1995; Qixian and Bungey 1996; Krstulovic-Opara, Woods, and Al-Shayea 1997; Nagy 1997; Leming, Nau, and Fukuda 1998; Kolluru, Popovics, and Shah 2000; Jin and Li 2001; Mesbah, Lachemi, and Aitcin 2002; Cho 2003; Han and Kim 2004). These methods typically involve the calculation of elastic modulus from the measurement of the velocity of stress waves (compression, shear, or Rayleigh) or the resonant frequencies of specimens.

The dynamic modulus of elasticity, which is well-suited to monitoring material quality or uniformity (Kolluru, Popovics, and Shah 2000), offers several advantages over the static modulus (Nagy 1997). First, the determination of dynamic modulus of elasticity is non-destructive and can be performed continuously on a single specimen, which may reduce variability. Second, the determination of dynamic modulus can be commenced at earlier ages because it is not affected by the plasticity of the concrete as with static modulus of elasticity determination. Third, testing artifacts associated with static modulus of elasticity

measurements—such as loading rate, specimen size, specimen end condition, and creep—do not affect dynamic modulus of elasticity measurements. The static modulus of elasticity, however, may be more relevant to prestressed concrete design because of the higher strains present in prestressed concrete members.

The static and dynamic modulus of elasticity measurements were conducted on 4 by 8-in. cylinders, which were cast in sealed plastic molds. For a given mixture, two cylinders were used for all dynamic modulus of elasticity tests and one cylinder was used for each test time for the static modulus of elasticity tests. The materials for each concrete mixture were preconditioned, mixed, and cast into cylinders at 73°F ambient temperature. At 90 minutes after mixing, the cylinders were transferred to a separate room with 90°F ambient temperature. After 24 hours, the plastic cylinder molds were removed and the specimens were transferred to moist curing conditions (73°F ambient temperature and 100% humidity). Thermocouples were embedded in one cylinder for each mixture to monitor concrete temperature.

Static modulus of elasticity and Poisson's ratio were measured in accordance with ASTM C 469. For each static modulus of elasticity measurement, compressive strength was measured in accordance with ASTM C 39 on the same specimen plus one additional specimen.

Dynamic elastic modulus, shear modulus, and Poisson's ratio were determined from measurements of compression wave (P-wave) and shear wave (S-wave) velocities. These waves were generated with a piezo-transducer at the top end of the specimen and were recorded with an accelerometer at the other end of the specimen (Figure 5.21 and Figure 5.22). Separate piezo-transducers and accelerometers were used for compression and shear wave measurements. For compression waves, a piezo-actuator was oriented to generate sinusoidal compression waves axially along the specimen and an accelerometer was oriented axially to the specimen. For shear waves, a bender element was oriented to generate sinusoidal stress waves transverse to the specimen and an accelerometer was oriented transverse to the specimen.

Both piezo-transducers were manually held in contact with a metal washer on the center of the top of each specimen. Each washer was manually held to the specimen before initial set and was attached to the specimen with epoxy after initial set. Both accelerometers were mounted with a metal washer attached to the accelerometer and a magnet on the center of the bottom of each specimen, which allowed for quick placement and removal of the accelerometers. Each magnet was secured with tape to a hole in the plastic cylinder mold, which provided adequate contact between the concrete and the magnet. When the plastic cylinder molds were removed, the magnets were attached to the concrete with epoxy. The specimens were suspended in a hanging plastic bucket to allow easy access to the specimen and to isolate the specimen from ambient noise (Figure 5.23). Preliminary testing indicated that the plastic bucket and plastic cylinder mold had negligible effect on the measured results.

The amplitude and frequency of the generated stress waves were varied to ensure sufficient precision. For each measurement, the input signal waves to the source (piezo-actuator or bender element) and the output signals from the receiver (accelerometer) were captured by a digital oscilloscope and recorded by a personal computer. These wave records were used to determine the travel time of the P- and S-waves. The P-wave travel time through the specimen varied from approximately 40 to 1900 μs and the S-wave travel time varied from approximately 60 to 105 μs , depending on the specimen. The stress wave travel times were measured to the nearest 1 μs or less.

Compression wave velocities were measured beginning at 1 hour after the start of mixing and shear wave velocities were measured beginning 8 hours after the start of mixing.

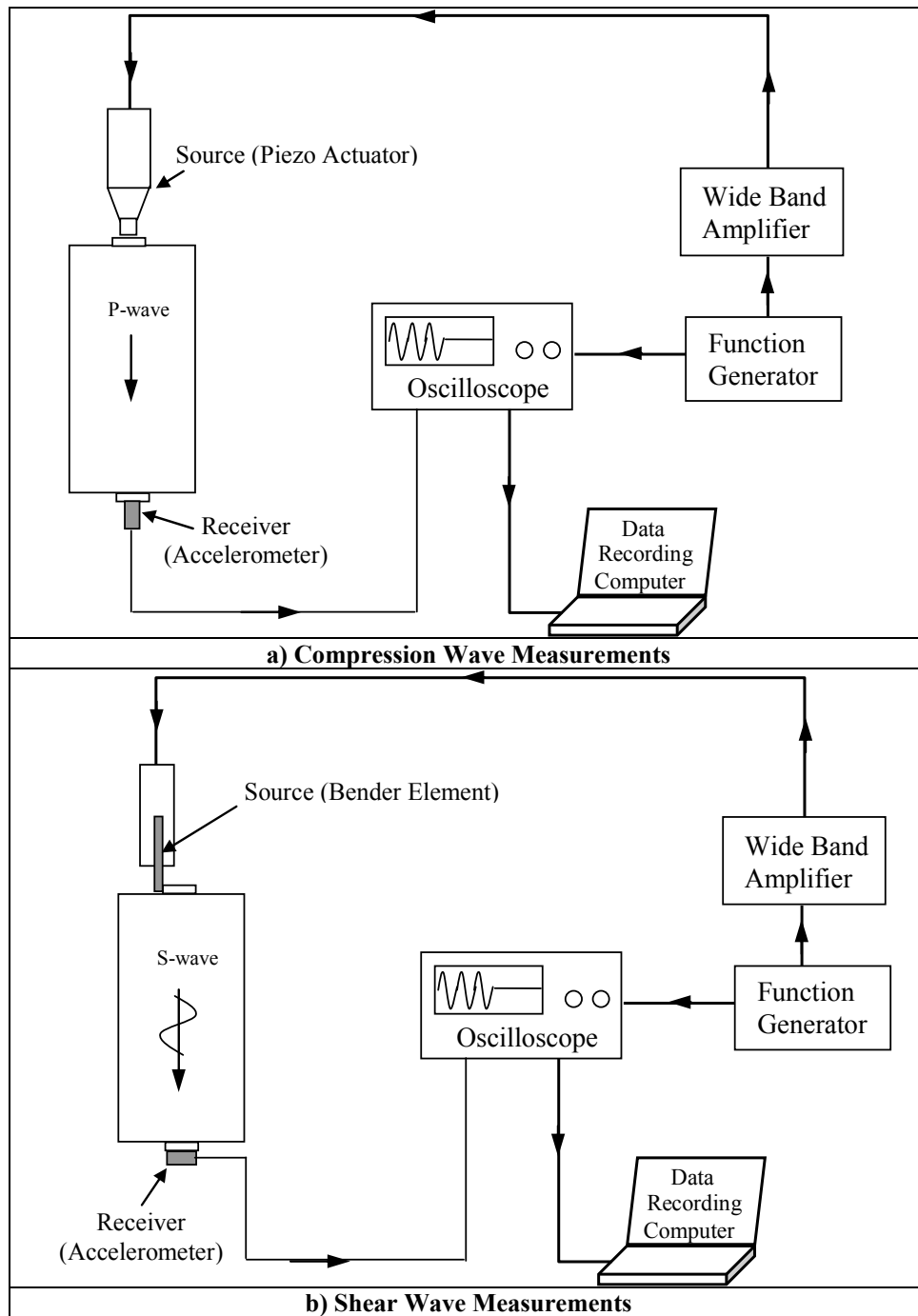


Figure 5.21 Dynamic Moduli Test Setup

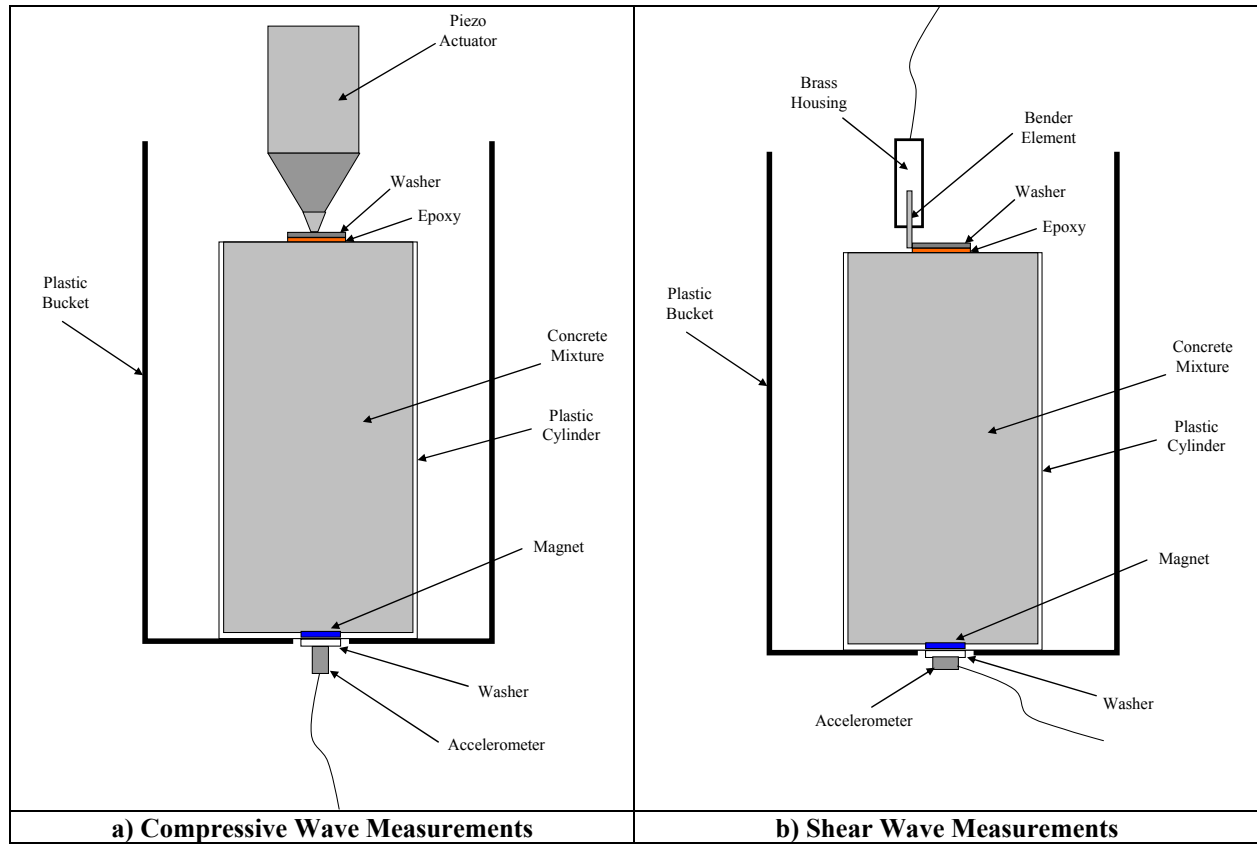


Figure 5.22 Mounting of Piezo-Transducers and Accelerometers for Dynamic Moduli Measurements



Figure 5.23 Specimen Suspended in Hanging Bucket for Testing

The dynamic elastic (Young's) modulus (E_d) is a function of the density of the material (ρ), the dynamic Poisson's ratio (μ_d), and the compression wave velocity (V_p), as given in Equation (5.8):

$$E_d = \frac{\rho(1-2\mu_d)(1+\mu_d)}{(1-\mu_d)} V_p^2 \quad (5.8)$$

The dynamic shear modulus (G_d) is a function of the density of the material and the shear wave velocity (V_s), as given in Equation (5.9):

$$G_d = \rho V_s^2 \quad (5.9)$$

The shear and elastic moduli (static or dynamic) are related to each other by the Poisson's ratio (static or dynamic) as shown in Equation (5.10):

$$\mu = \frac{E}{2G} - 1 \quad (5.10)$$

The dynamic elastic modulus, shear modulus, and Poisson's ratio can be calculated from the compression and shear wave velocity measurements by solving Equations (5.8) to (5.10) simultaneously. The theoretical unit weight of each mixture was used to compute density. Because only the P-wave velocity was measured between 1 and 8 hours, only the P-wave modulus (E_p) could be calculated during this time period, as given in Equation (5.11):

$$E_p = \rho V_p^2 \quad (5.11)$$

The static and dynamic elastic moduli are shown in Figure 5.24. For comparison purposes, all mixtures had the same nominal 16-hour compressive strength. For the static elastic modulus measurements, the conventionally placed concrete mixture exhibited the highest elastic modulus at all ages. The four SCC mixtures with the river gravel aggregate set exhibited very similar elastic modulus at all ages despite varying in S/A, paste volume, and paste composition. The mixture with the limestone coarse aggregate set exhibited the lowest elastic modulus. For the dynamic modulus measurements, the trends between individual mixtures were less clear. The modulus of elasticity of the conventionally placed concrete mixture was not always highest of all mixtures and the modulus of elasticity of the mixture with the crushed limestone aggregate set was never the lowest of all mixtures.

The dynamic modulus measurements were consistently greater than the static modulus measurements, which was consistent with other research (Nagy 1997; Mesbah, Lachemi, and Aitcin 2002; Han and Kim 2004). Figure 5.25 indicates that the ratio between dynamic and static elastic modulus decreased from 8 to 16 hours, before increasing again at later ages. The static elastic modulus was lower than the dynamic elastic modulus because the static elastic modulus is affected by micro-cracking and creep, which result in non-linearity of the stress-strain curve (Mesbah, Lachemi, and Aitcin 2002). This non-linearity reduces static elastic modulus but has little to no effect on dynamic elastic modulus, which is measured at low strains. The ratio between dynamic and static elastic modulus is generally known to decrease as strength increases because the shape of the stress-strain curve becomes more linear as strength increases (Khan, Cook, and Mitchel 1993; Mesbah, Lachemi, and Aitcin 2002; Han and Kim 2004). In higher strength concretes, the difference in modulus of elasticity between aggregate and paste is less, resulting in lower stress concentration and less micro-cracking in the transition zone (Neville

1997). The ratio of dynamic to static elastic modulus was found to vary with age from 1.1 to 1.2 by Nagy (1997) and from 1.4 to 1.3 by Mesbah, Lachemi, and Aitcin (2002). In both cases, the ratio decreased quickly from the maximum initial value and subsequently remained approximately unchanged at the minimum value. Mehta and Monteiro (1997) reported that the long-term dynamic modulus of elasticity is typically 20 to 40% higher than the static modulus of elasticity, with smaller differences for higher compressive strengths. Empirical relationships between static and dynamic modulus were proposed by Nagy (1997), Mesbah, Lachemi, and Aitcin (2002), and Han and Kim (2004). For the SCC mixtures shown in Figure 5.25, the ratio of dynamic to static modulus was similar or lower than the final long-term ratios reported by Nagy (1997) and Mesbah, Lachemi, and Aitcin (2002). The higher initial ratios were not measured, likely because static elastic modulus measurements were not commenced early enough given the rapid development of strength and elastic modulus. The ratio of dynamic to static elastic modulus was typically greater for the five SCC mixtures than the conventionally placed concrete mixture, suggesting the possibility of more micro-cracking or creep in the SCC mixtures.

Whereas the rate of increase in modulus of elasticity slowed considerably after 24 hours, the compressive strength continued to increase at a relatively faster rate, as indicated in Figure 5.26. Similarly, Mesbah, Lachemi, and Aitcin (2002) found that the modulus of elasticity increased rapidly up to 24 hours but increased at a much slower rate thereafter for mixtures with w/c varying from 0.30 to 0.45.

Figure 5.27 indicates that the static Poisson's ratio initially increased with time and remained approximately constant after 3 days. The Poisson's ratio for the mixture with the crushed limestone coarse aggregate set was consistently higher than the other mixtures. In contrast, the dynamic Poisson's ratio decreased up to 24 hours and remained approximately constant thereafter. For a given mixture, the dynamic Poisson's ratio was greater than the static Poisson's ratio, which was consistent with expectations (Mehta and Monteiro 1993; Neville 1996). Poisson's ratio has generally been found to not vary significantly with concrete strength or age (Carrasquillo, Nilson, and Slate 1981; Oluokun, Burdette, and Deatherage 1991; Mehta and Monteiro 1993; Jin and Li 2002). In contrast, Mesbah, Lachemi, and Aitcin (2002) found that Poisson's ratio decreased at early ages and subsequently increased over time.

The development of the dynamic shear modulus with time is shown in Figure 5.28. As with the dynamic elastic modulus, the trends between individual mixtures were not clear.

The development of P-wave modulus with time is shown in Figure 5.29. At one hour, the P-wave modulus of each mixture was near zero. The P-wave modulus increased slightly at the 3.5 and 4.5-hour measurements; however, the rate of increase in P-wave modulus increased significantly after the 4.5-hour measurement due to the initial setting of the concrete. By definition, the P-wave modulus is always greater than the dynamic elastic modulus.

The relationships between compressive strength and static and dynamic elastic moduli are compared in Figure 5.30. For static modulus of elasticity, the conventionally placed concrete mixture exhibited the greatest modulus of elasticity for a given compressive strength for all test ages. The higher elastic modulus of the conventionally placed concrete mixture was likely due to the lower paste volume (27% versus 33 or 37% for the SCC mixtures). The S/A of 0.39 in the conventionally placed concrete mixture was similar to the S/A of 0.40 in one of the SCC mixtures (RG-5-40). The SCC mixture with the crushed limestone coarse aggregate set exhibited the lowest modulus of elasticity for a given compressive strength for all test ages, which was likely due to the lower stiffness of the crushed limestone coarse aggregate. The paste

volume of 37% in this mixture was equal to three of the other SCC mixtures (RG-5-50, RG-5-45, and RG-5-40) but was higher than the 33% in the RG-5-50a mixture. There was no significant difference between the river gravel SCC mixtures that varied in S/A, paste volume, and paste composition. The lack of effect of S/A on static modulus of elasticity (RG-5-50, RG-5-45, and RG-5-40) was likely due to the river gravel and natural sand being from the same aggregate pit and having the same mineralogy. The reduction in paste volume in mixture RG-5-50a from 37 to 33%, which would have been expected to result in increased modulus of elasticity, may have been offset by changes in the paste composition (reduced fly ash content, increased w/cm). In contrast to the static modulus of elasticity measurements, the trends between individual mixtures were not as clear as for dynamic modulus of elasticity measurements.

The lack of clear trend between mixtures for the dynamic shear and elastic moduli measurements was likely due to the method of measuring dynamic moduli. The dynamic moduli measurements were based on stress wave direct travel time, which could vary depending on the path the waves traveled. For instance, a wave traveling in a circuitous path around aggregates and only through the paste may arrive at a different time than a wave traveling in a straight path through both aggregate and paste. A single generated wave results in multiple waves traveling separate paths through the specimen simultaneously. The dynamic moduli tests record the travel time of the wave that arrives first, which reflects only the material along the path it travels and not necessarily the entire specimen. In contrast, resonant frequency measurements would reflect the travel of all waves and may detect a greater difference in shear and elastic moduli measurements between specimens.

The ACI 318 equation relating static modulus of elasticity and compressive strength under-predicted the static modulus of elasticity for all mixtures. The ACI 318 equation was developed on a large set of concrete mixtures with wide ranges of aggregates, mixture proportions, and testing conditions. Therefore, the absolute magnitude of the elastic modulus can be expected to deviate from that predicted by the ACI 318 equation. The shape of the ACI 318 equation suitably represents the shape of the measured static and dynamic modulus of elasticity versus compressive strength relationships for all mixtures and test times.

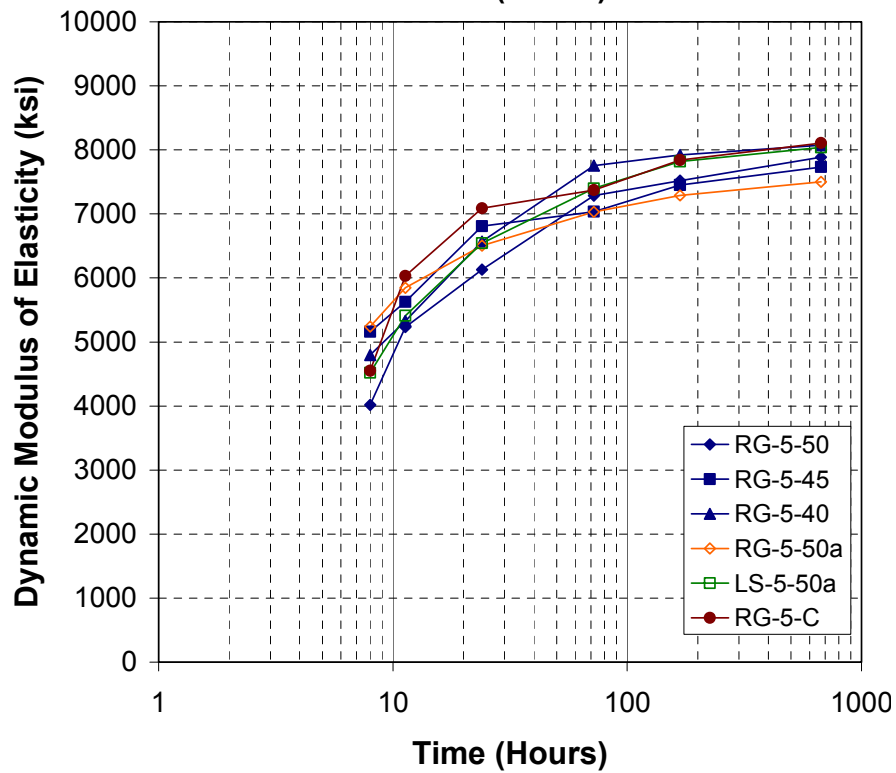
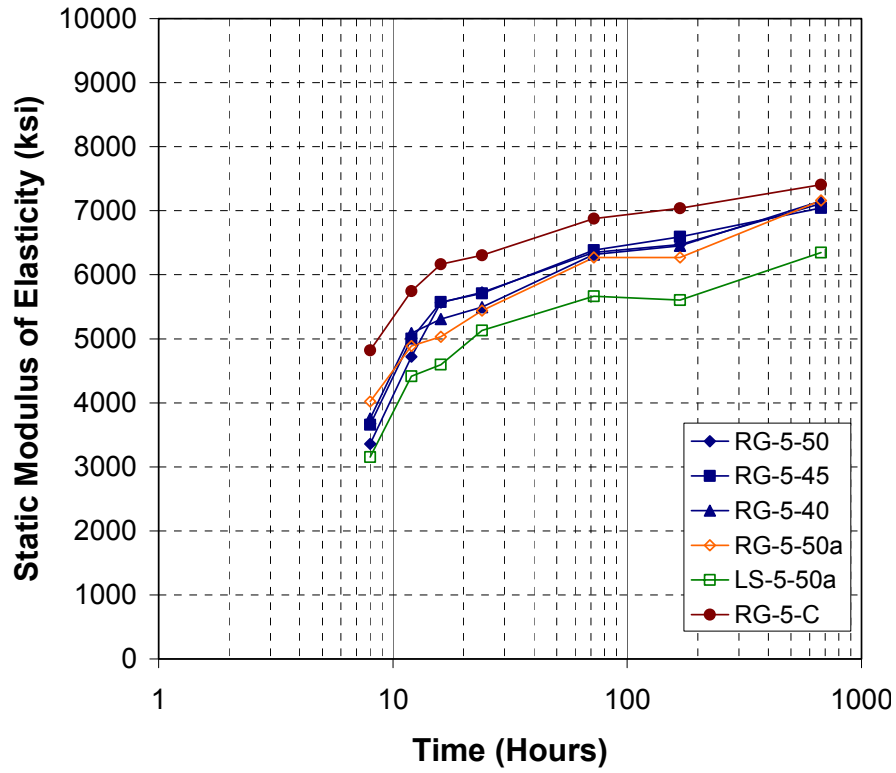


Figure 5.24 Development of Static and Dynamic Modulus of Elasticity

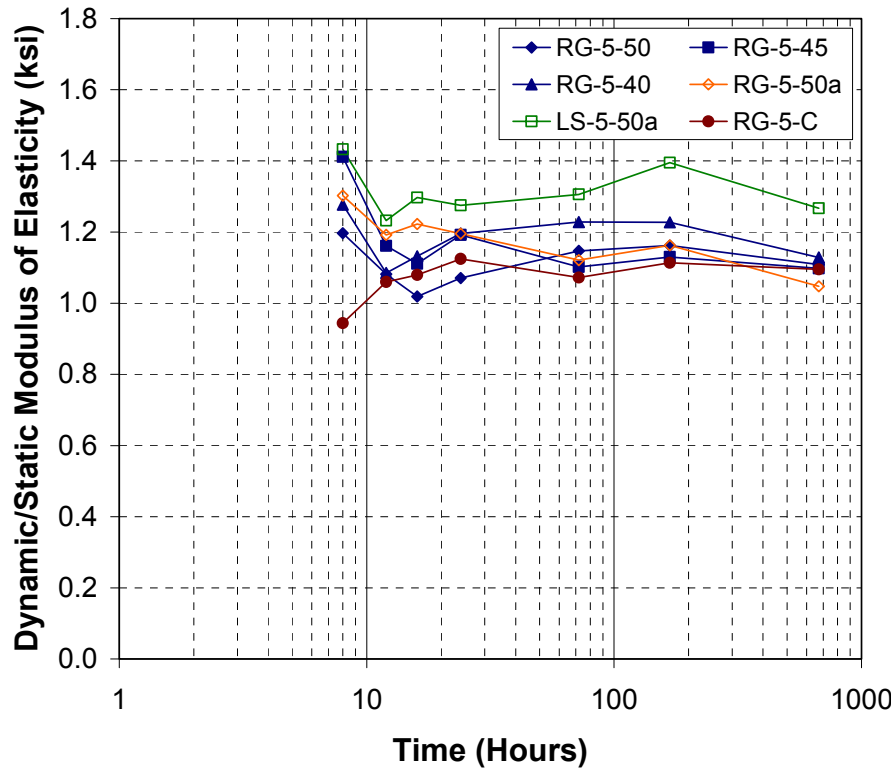


Figure 5.25 Comparison of Dynamic and Static Modulus of Elasticity

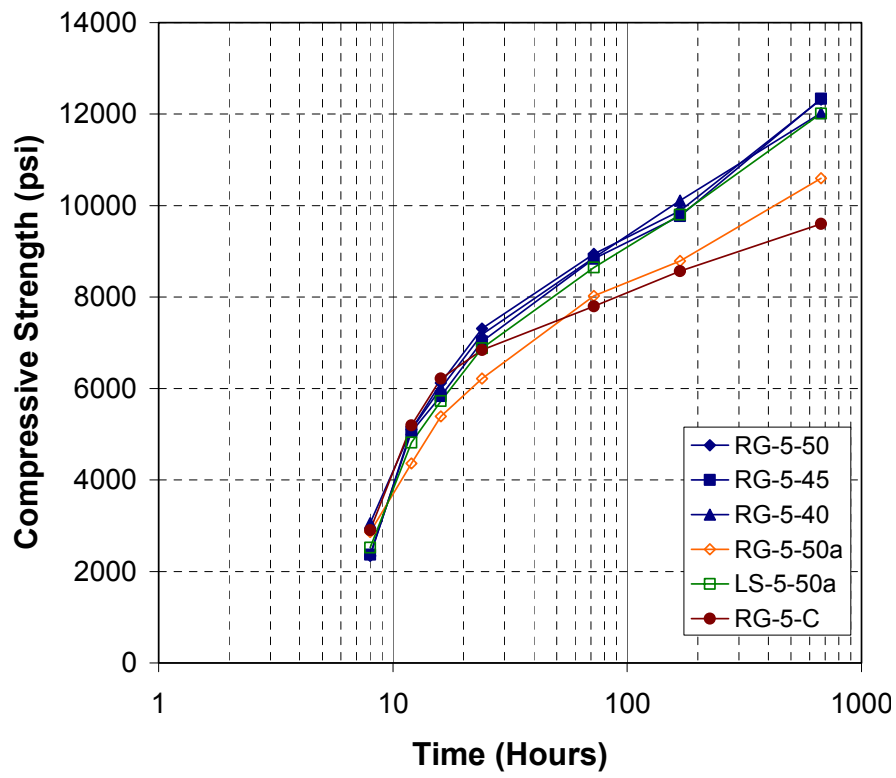


Figure 5.26 Development of Compressive Strength

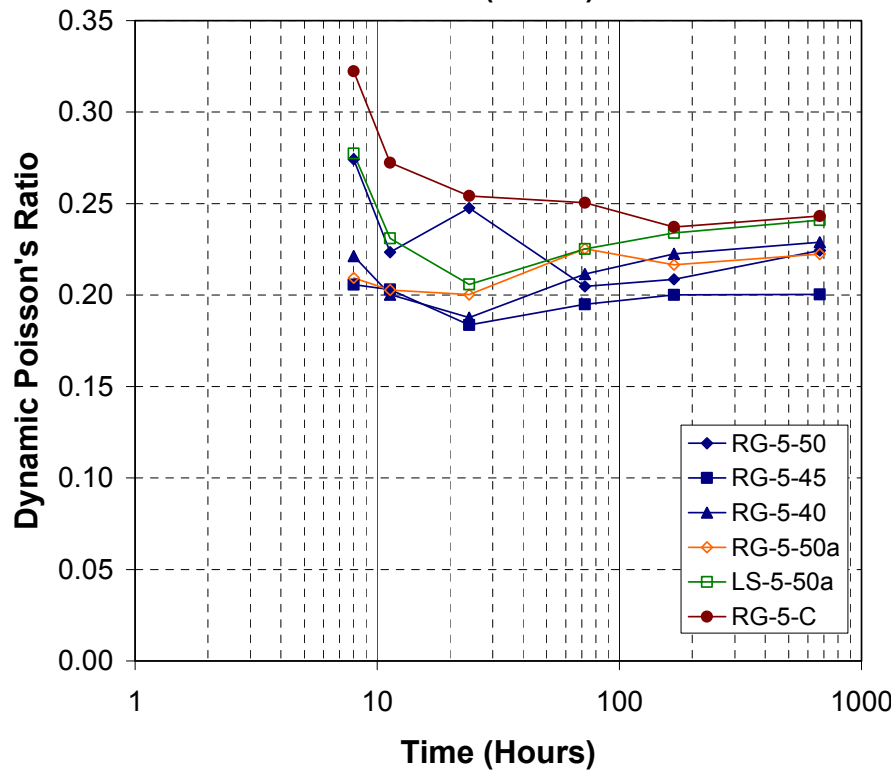
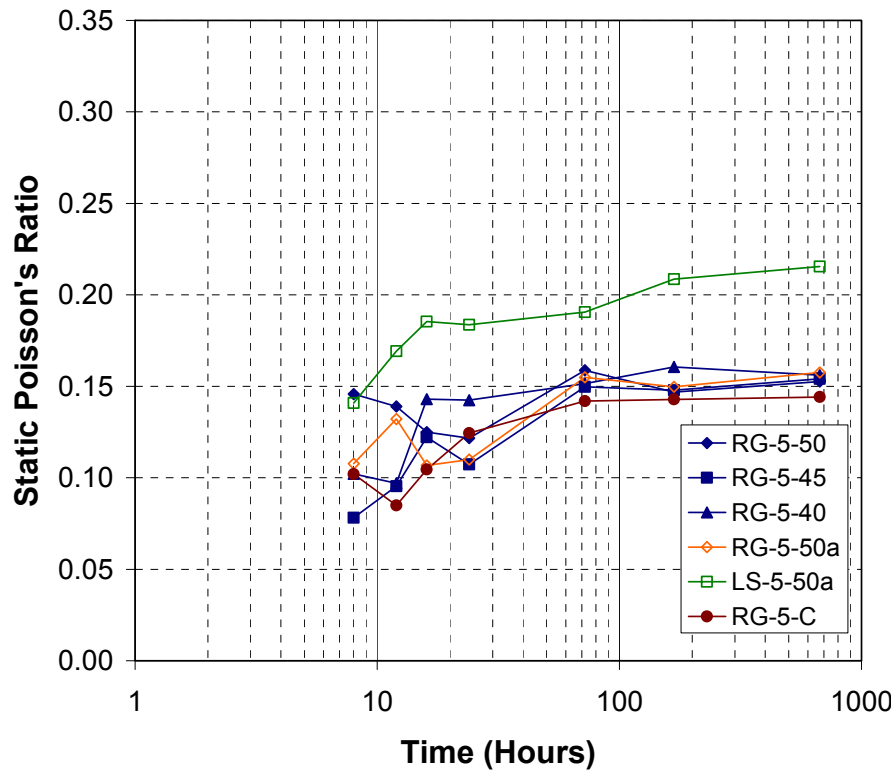


Figure 5.27 Development of Static and Dynamic Poisson's Ratio

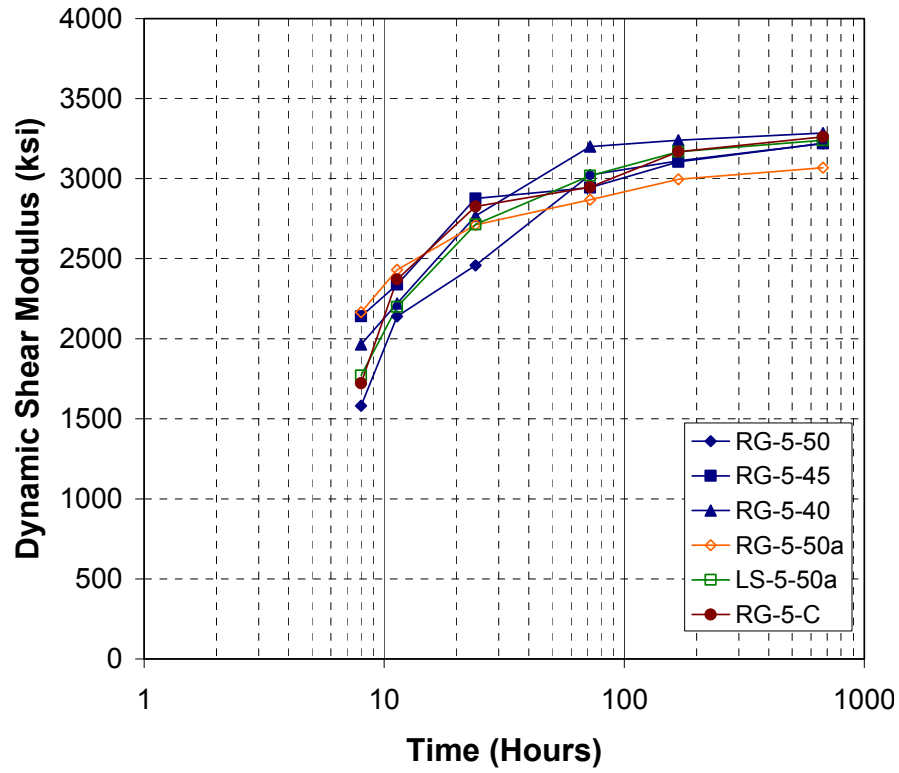


Figure 5.28 Development of Dynamic Shear Modulus

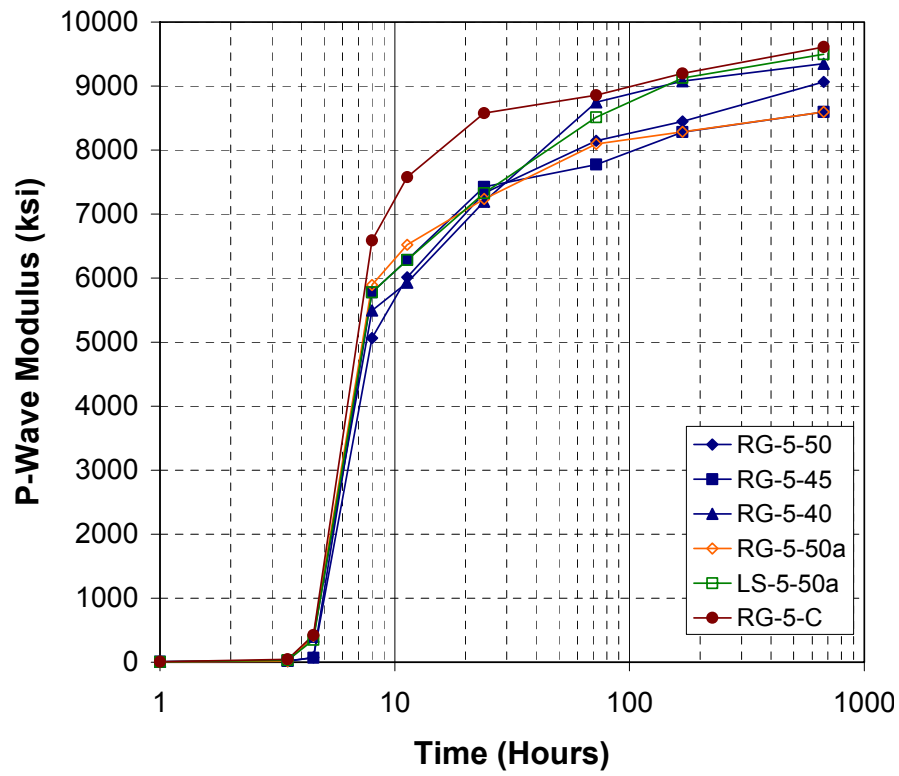


Figure 5.29 Development of P-Wave Modulus

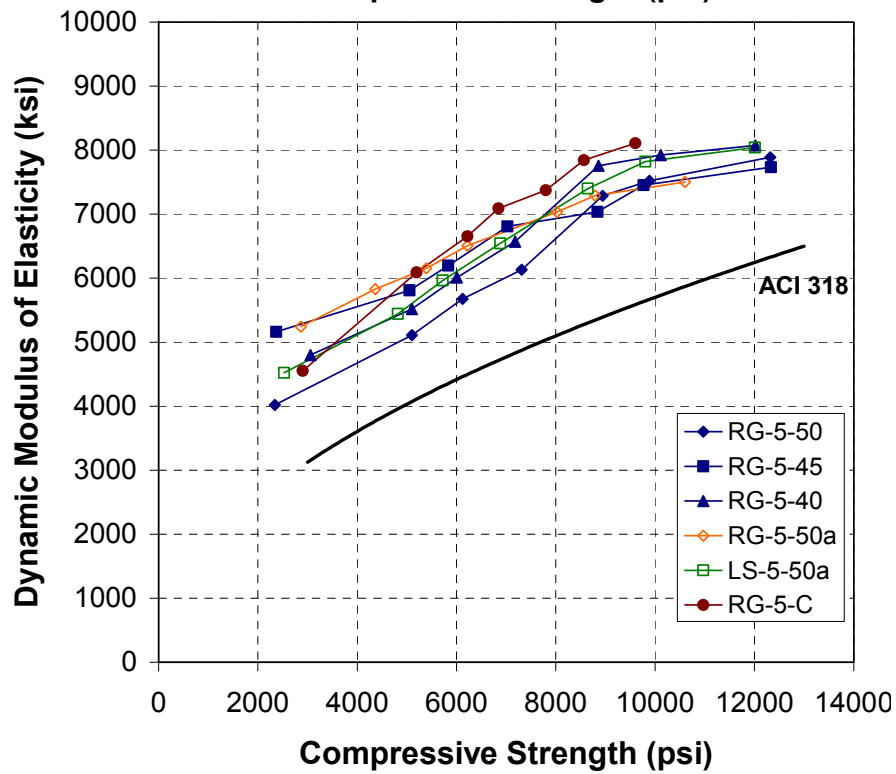
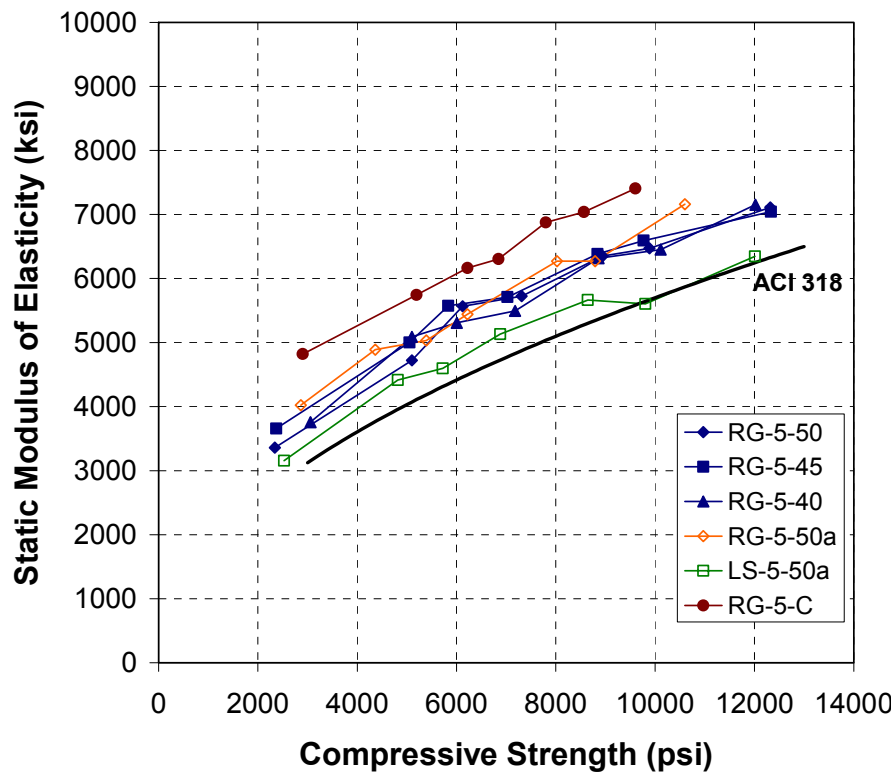


Figure 5.30 Relationships between Modulus of Elasticity and Compressive Strength

5.5 Computer Simulation

The concrete mixture properties determined in the first part of this chapter were used to simulate the temperature history and strength development in AASHTO Type IV beams. The simulations were conducted with *Concrete Works*, a software program developed as part of TxDOT Research Project 0-4563. *Concrete Works* predicts the temperature history in concrete members based on the concrete mixture proportions, concrete material properties, formwork, curing methods, and weather conditions. Compressive strength can be estimated from the temperature history with the maturity method. (To evaluate the validity of *Concrete Works* in predicting precast element temperatures, computer simulation results were compared to the field testing data in Chapter 9. The results of this comparison are presented in Appendix C.)

Simulations were performed under cold, mild, and hot weather conditions, which were intended to be similar to the scenarios shown in Table 5.1. The input parameters for the *Concrete Works* analysis are shown in Table 5.6. All simulations were for AASHTO Type IV beams, with the temperature and compressive strength development monitored in the center of the web. The actual mixture proportions, aggregate types, and cement and fly ash material properties were used for each mixture. The properties for hydration calculations were input based on the semi-adiabatic calorimetry results, activation energies, and ultimate heat of hydration values determined in the earlier part of this chapter. (The hydration parameters determined from semi-adiabatic calorimetry results were calculated from the first 30 hours of calorimetry data instead of the first 150 hours, which are shown in Table 5.4, in order to reflect the rate of early temperature development more accurately.) The formwork was assumed to be made of red steel. Blankets and tarps were applied on the sides and top of the beam one hour after placement. The combined R-value of the blanket and tarp was varied with the weather conditions. The temperature, humidity, wind speed, and cloud cover were set by *Concrete Works* based on historical data for the selected location (Victoria, TX).

Table 5.6 Input Parameters for Concrete Works Analysis

Parameter	Cold	Mild	Hot
Location	Victoria, TX	Victoria, TX	Victoria, TX
Date	January 1	May 1	August 1
Placement Time	2 pm	2 pm	2 pm
Analysis Duration	48 hours	24 hours	24 hours
Fresh Concrete Temperature	60°F	75°F	90°F
Blanket R-Value	5.5 hr-ft ² -°F/BTU	2.9 hr-ft ² -°F/BTU	1.5 hr-ft ² -°F/BTU

The simulation results for the specified mild weather conditions are shown in Figure 5.31 for six mixtures. All mixtures exhibit very similar web temperature histories. The web temperatures are characterized by an approximate 4-hour pre-set time and a maximum temperature of approximately 140°F, achieved between 14 and 18 hours. As expected, the RG-7-50 and LS-7-50 mixtures gain strength at the fastest rate and the RG-5-C and LS-5-C at the slowest rate. At 16 hours, the RG-7-50 and LS-7-50 mixtures achieve compressive strengths

greater than 8,000 psi, while RG-5-50a and LS-5-50a achieve compressive strengths of over 6,500 psi.

For the specified hot weather conditions, Figure 5.32 indicates that the pre-set times are shortened by about one hour as compared to the specified mild weather conditions. The maximum temperatures are increased by about 10°F and occur earlier. By 16 hours, the RG-7-50 mixture achieves a compressive strength of over 8,500 psi while the RG-5-50a mixture achieves a compressive strength of over 7,000 psi.

For the specified cold weather conditions, Figure 5.33 indicates that the pre-set times are increased to approximately 6 hours and the maximum temperatures are reduced to approximately 120°F and delayed until approximately 24 hours. At 16 hours, the compressive strengths are between 2,500 and 4,500 psi. The RG-5-50a mixture does not achieve its nominal strength until approximately 19 hours, while the RG-7-50 mixture achieves its nominal strength after 29 hours. Therefore, it is likely that different mixture proportions or other cold weather concrete practices—such as heavier blankets, heated materials, or steam curing—would be used.

Lastly, Figure 5.34 indicates the effects of RET-A and PT-1482 on the web temperature. The use of RET-A increases the pre-set period but only slightly reduces the maximum temperature. In contrast, the use of PT-1482 increases the pre-set period by approximately 1 hour, reduces the maximum temperature by approximately 10°F and delays the time of the maximum temperature from approximately 14 hours to 18 hours.

The simulation results are only estimates and should be verified with field testing prior to formulating final conclusions. The relative results comparing one mixture to another are likely to be more reliable than the absolute values of temperatures and compressive strengths. Therefore, the fact that the pre-set times and maximum temperatures are similar for both the SCC and conventionally placed concrete mixtures is important because it suggests that the temperature rise of the SCC mixtures should be similar to the conventionally placed concrete mixtures despite the higher cementitious materials content in the SCC mixtures. The maximum temperature limits from the 2004 TxDOT specifications are not exceeded in any of the simulations shown (in this case, 170°F for the SCC mixtures and 150°F for the conventionally placed concrete mixtures).

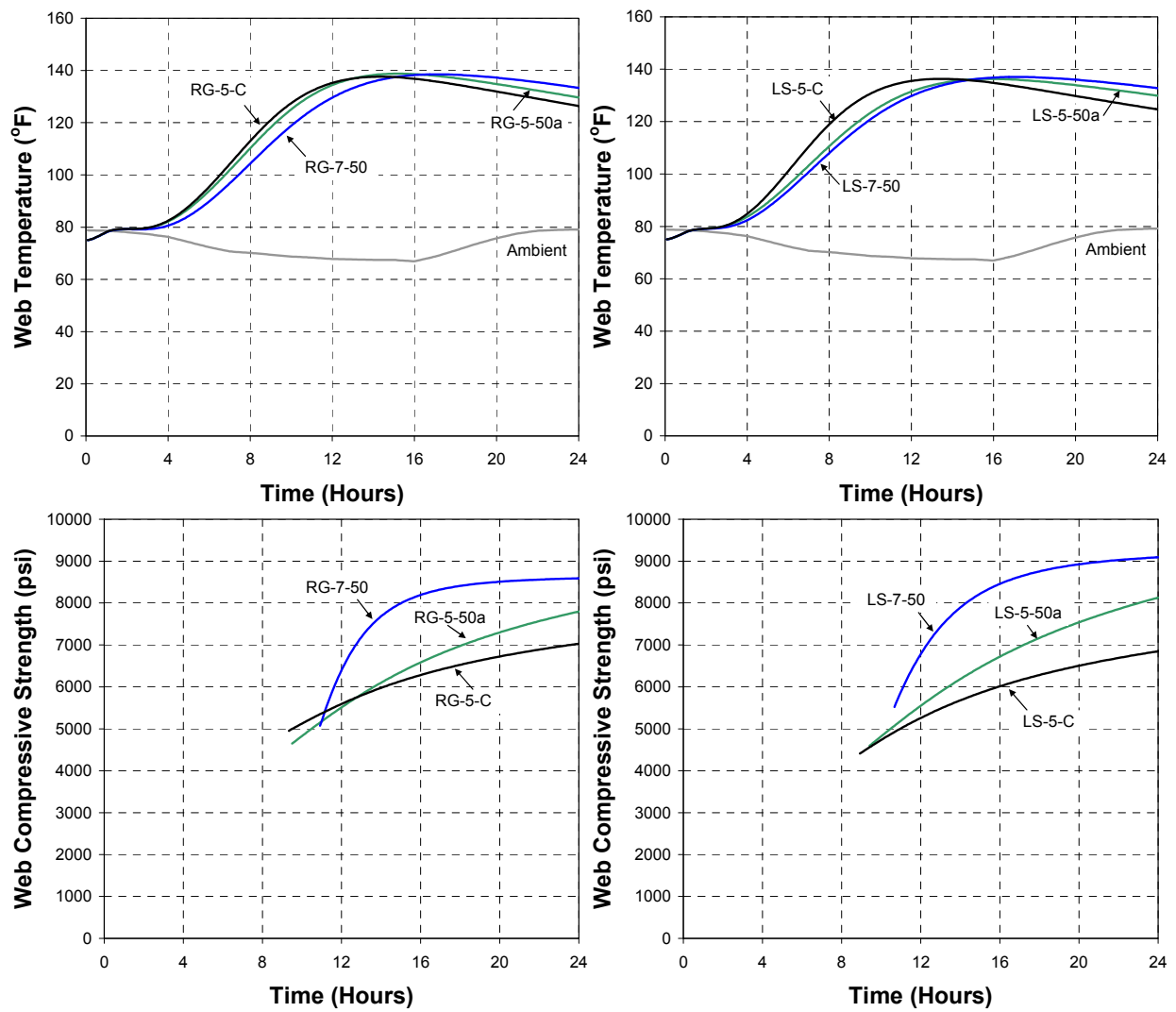


Figure 5.31 Web Temperature and Compressive Strength Development for Mild Weather Conditions

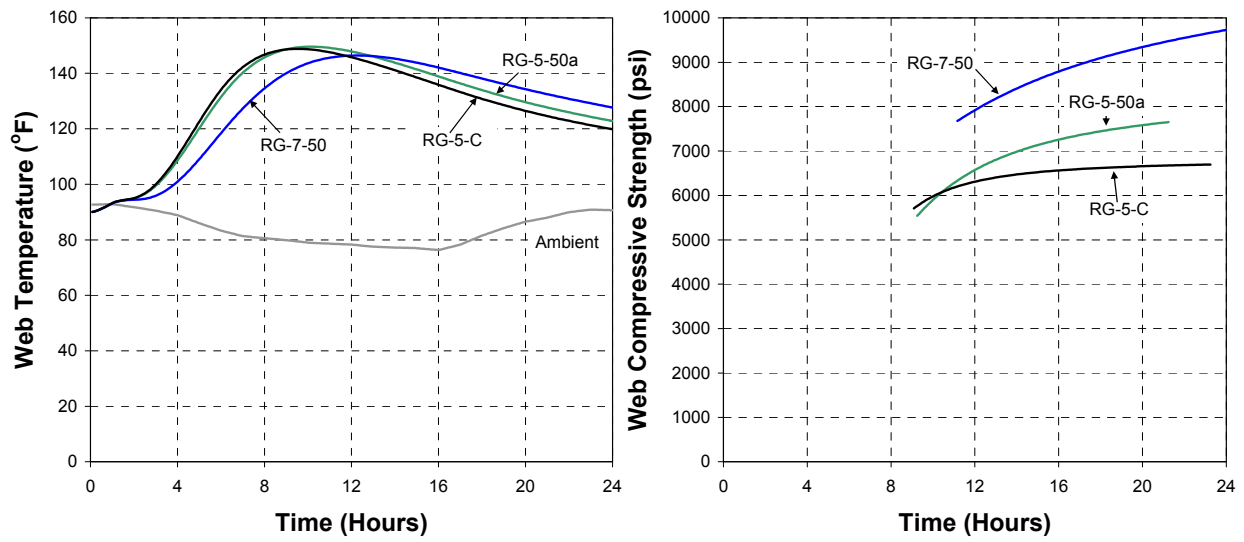


Figure 5.32 Web Temperature and Compressive Strength Development for Hot Weather Conditions

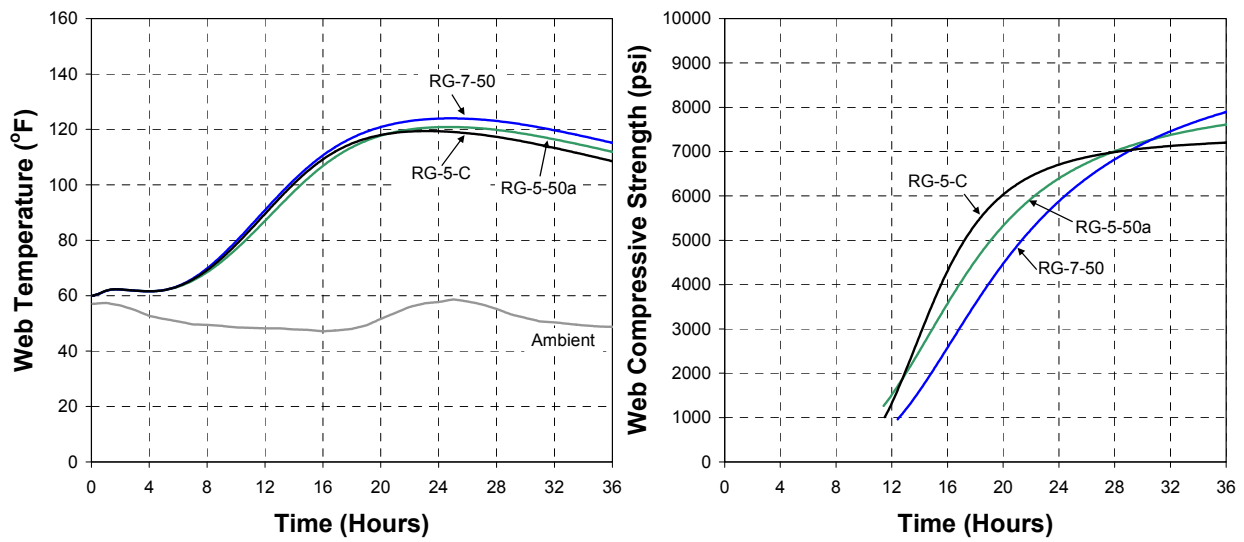


Figure 5.33 Web Temperature and Compressive Strength Development for Cold Weather Conditions

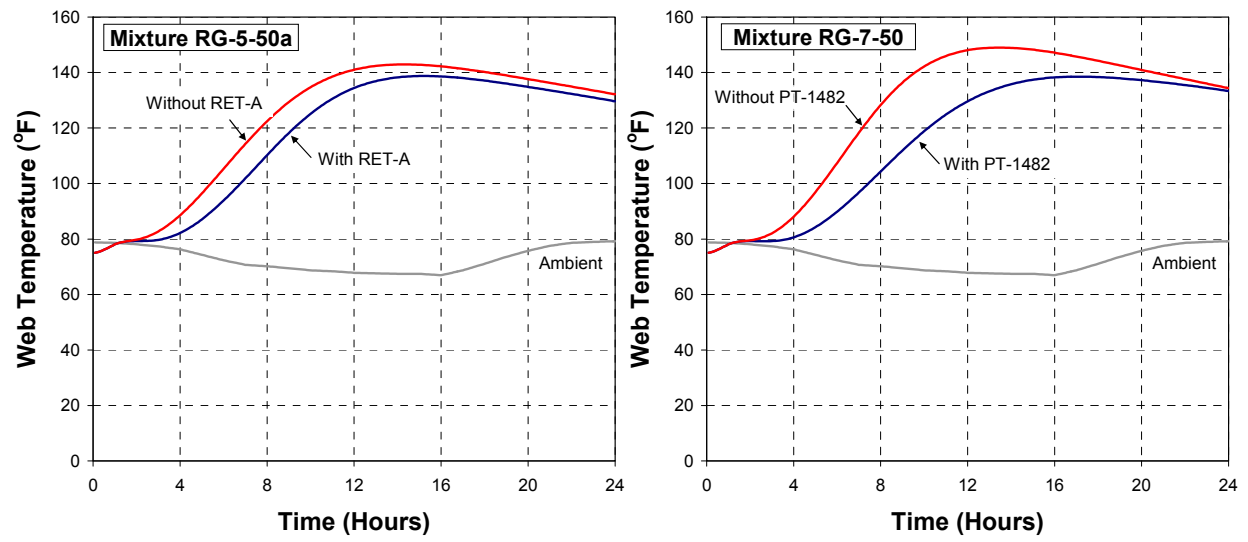


Figure 5.34 Effects of RET-A and PT-1482 on Web Temperature for Mild Weather Conditions

6. Shrinkage

Shrinkage measurements were performed beginning at 16 hours and continued through 112 days. All final SCC and conventionally placed concrete mixtures were tested for shrinkage with the exception of RG-5-45a and LS-5-45a. Shrinkage was measured in accordance with ASTM C 157 on 3 x 3 x 11.25-inch specimens with 10-inch gage length. Concrete was mixed at the mild temperature scenario. After casting, specimens were stored at 73°F and 100% relative humidity for the first 16 hours. The specimens were then demolded, measured for initial length, and stored at 73°F and 50% relative humidity for the remainder of the test. For each mixture, three specimens were obtained from a single batch.

For the 5000 psi nominal strength level, the shrinkage of the SCC mixtures was approximately unchanged for the river gravel mixtures (Figure 6.1) and slightly lower for the crushed limestone mixtures (Figure 6.2), when compared to the conventionally placed concrete mixtures. The higher paste volumes and water contents of the SCC mixtures would be expected to increase shrinkage; however, this effect was not observed. The use of a lower w/cm's and higher long-term strengths in the SCC mixtures may have offset the effects of higher paste volumes and water contents.

For the 7,000 psi nominal strength level, shrinkage increased for both aggregate sets compared to the conventional mixtures. Most of this increase in shrinkage in the 7,000 psi nominal strength mixtures was attributable to the use of PT-1482. Figure 6.3 shows the effect of PT-1482 on the shrinkage of mixture RG-7-50. Without PT-1482, mixture RG-7-50 exhibited a 112-day shrinkage strain of 363 μ -strain, which was 28% less than the mixture with PT-1482 (505 μ -strain) and 14% less than the control mixture (423 μ -strain). In contrast, Figure 6.4 shows that the use of RET-A had essentially no effect on the 112-day shrinkage of mixture RG-5-50a.

For the SCC mixtures where only the S/A was varied, reducing the S/A had no effect on the mixtures with the river gravel aggregate set and resulted in only a slightly lower shrinkage for the crushed limestone aggregate set mixture with S/A of 0.42. For the river gravel aggregate set, the stiffnesses of the fine and coarse aggregates were likely similar because they were from the same pit, resulting in no difference in restraint against shrinkage as the S/A was varied.

Decreasing the paste volume and modifying the paste composition at a given S/A in mixtures RG-5-50a (vs. RG-5-50) and LS-5-50a (vs. LS-5-50) had no effect on shrinkage. Despite the lower paste volumes in RG-5-50a and LS-5-50a, the total water contents per unit volume were approximately the same as in the SCC mixtures with higher paste volume (RG-5-50a vs. RG-5-50; LS-5-50a vs. LS-5-50).

For a given strength level and S/A, the mixtures with the crushed limestone aggregate set exhibited 10-30% higher shrinkage than the mixtures with the river gravel aggregate set. This difference in shrinkage strains could have been due to the reduced stiffness of the crushed limestone aggregates and the higher paste volumes in the mixtures with the crushed limestone aggregate sets.

Figure 6.5 and Figure 6.6 show the variation in shrinkage over time for the mixture with the river gravel aggregate set and crushed limestone aggregate set. Figure 6.7 and Figure 6.8 show the shrinkage measurements up to 7 days for these same mixtures. The average difference

in shrinkage strains between the 5,000 psi and 7,000 psi mixtures for each aggregate set increased up to 28 days, after which the rates of shrinkage were similar.

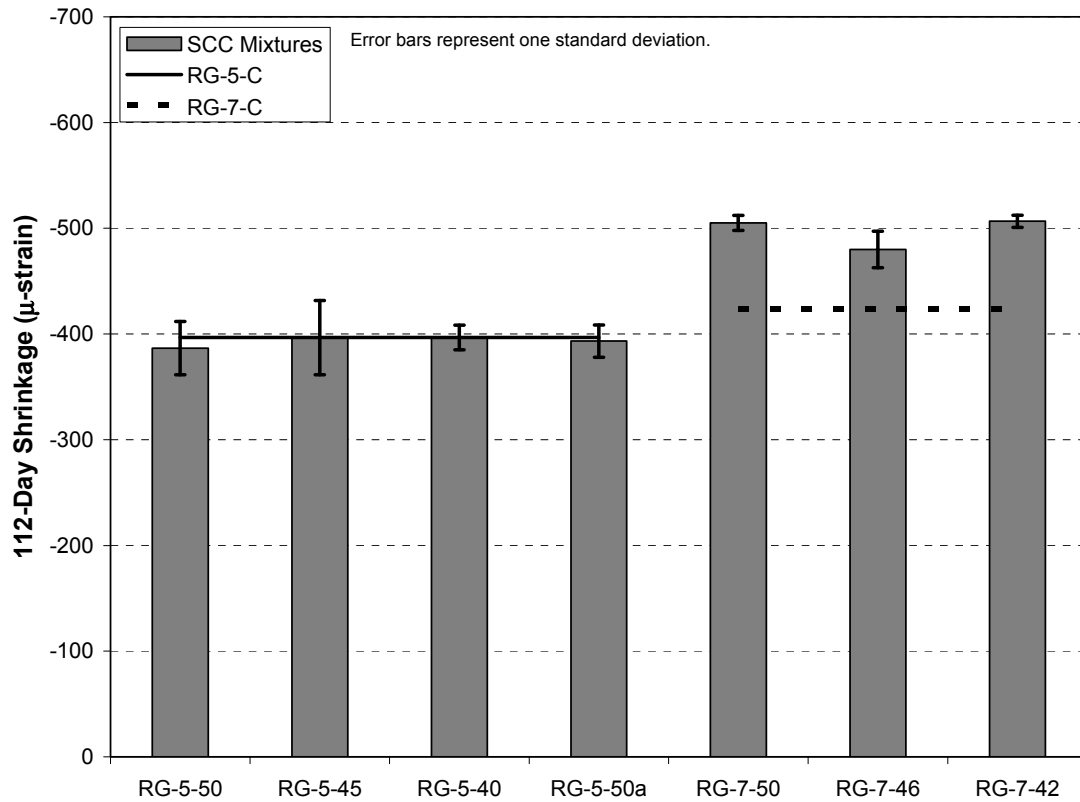


Figure 6.1 112-Day Shrinkage for Mixtures with River Gravel Aggregate Set

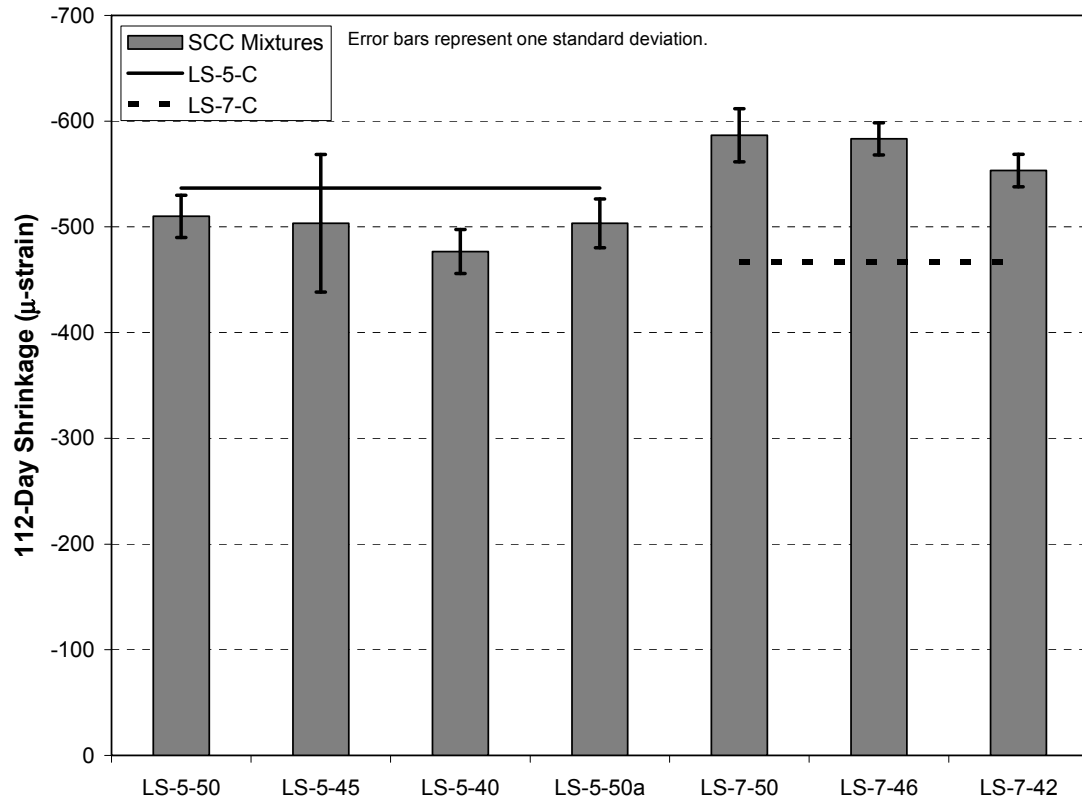


Figure 6.2 112-Day Shrinkage for Mixtures with Crushed Limestone Aggregate Set

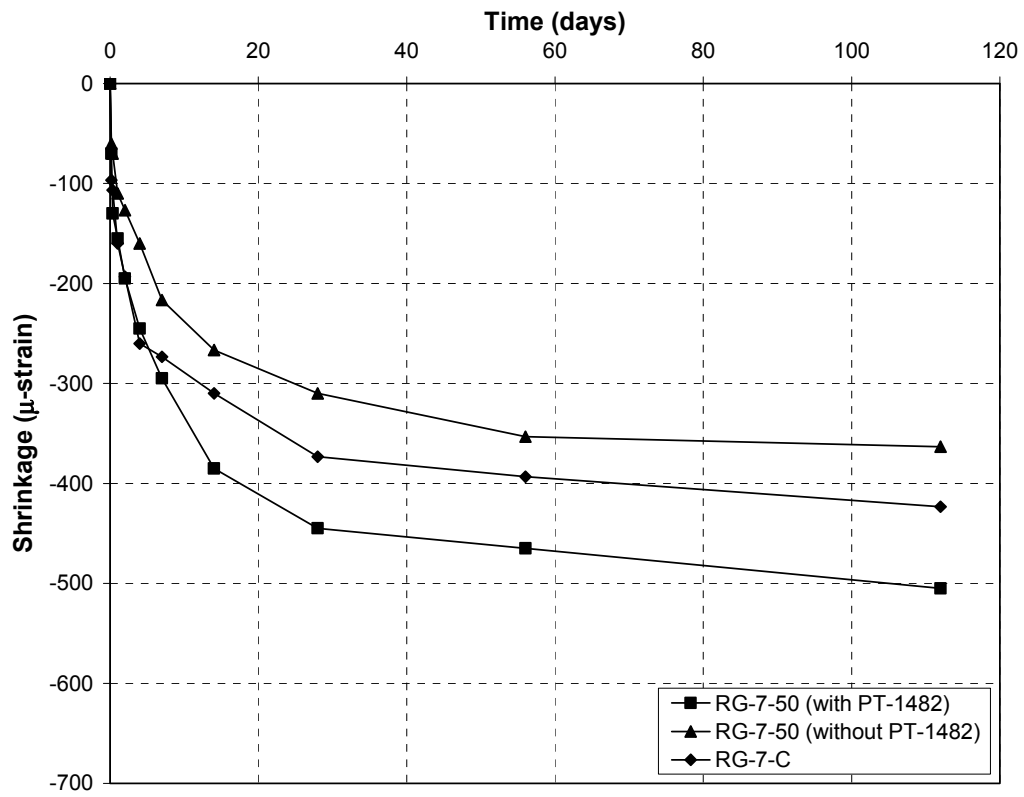


Figure 6.3 Effect of PT-1482 on Shrinkage Strains

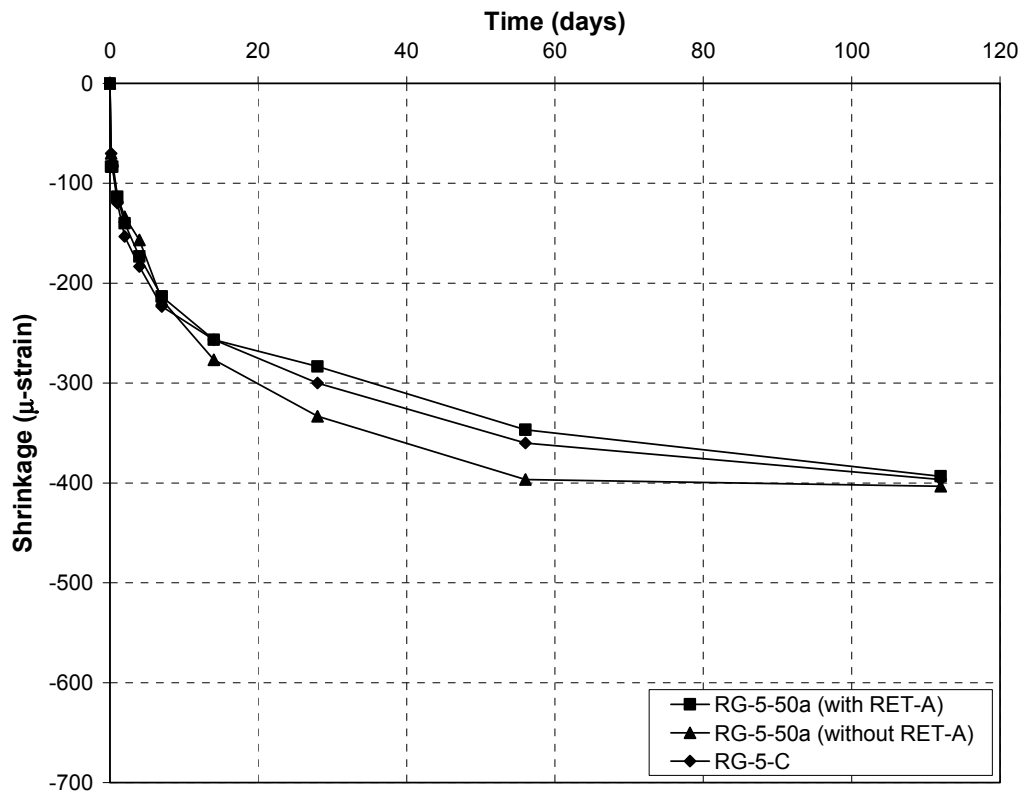


Figure 6.4 Effect of RET-A on Shrinkage Strains

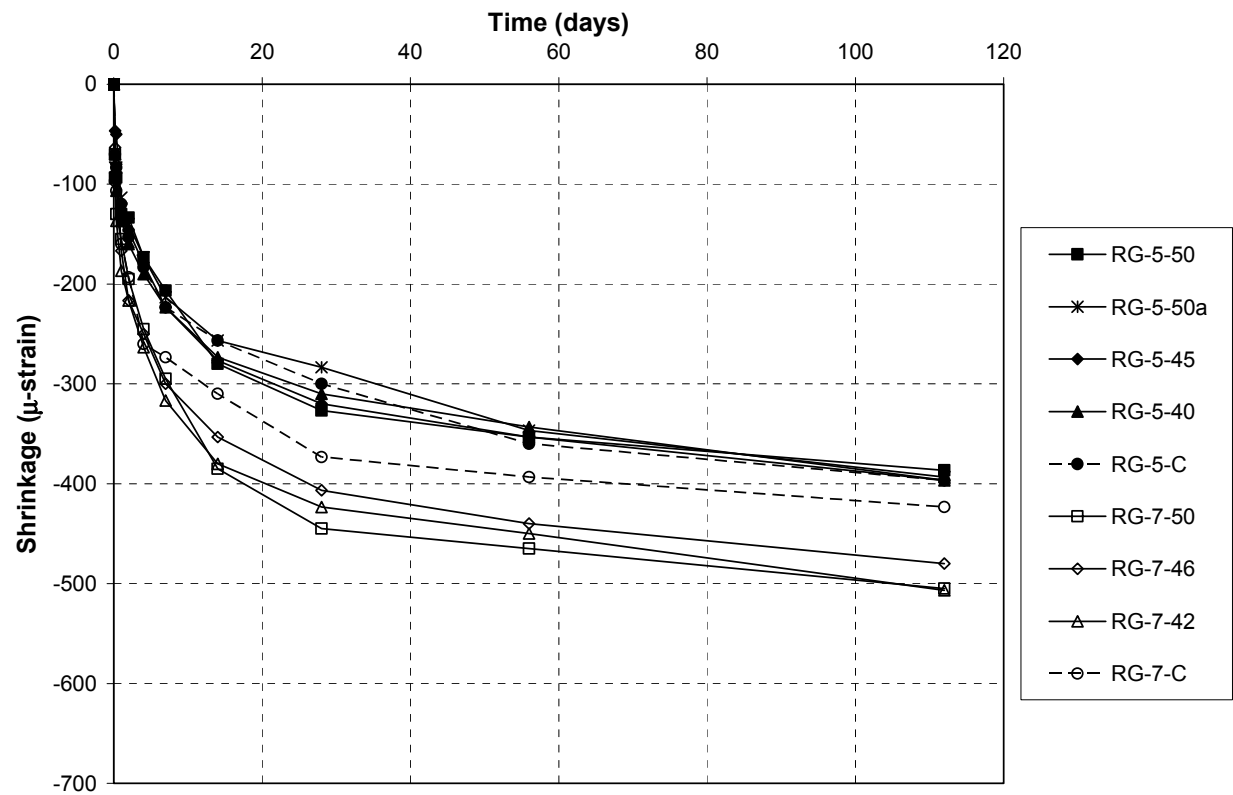


Figure 6.5 Shrinkage Strain Measurements to 112 Days (River Gravel Aggregate Set)

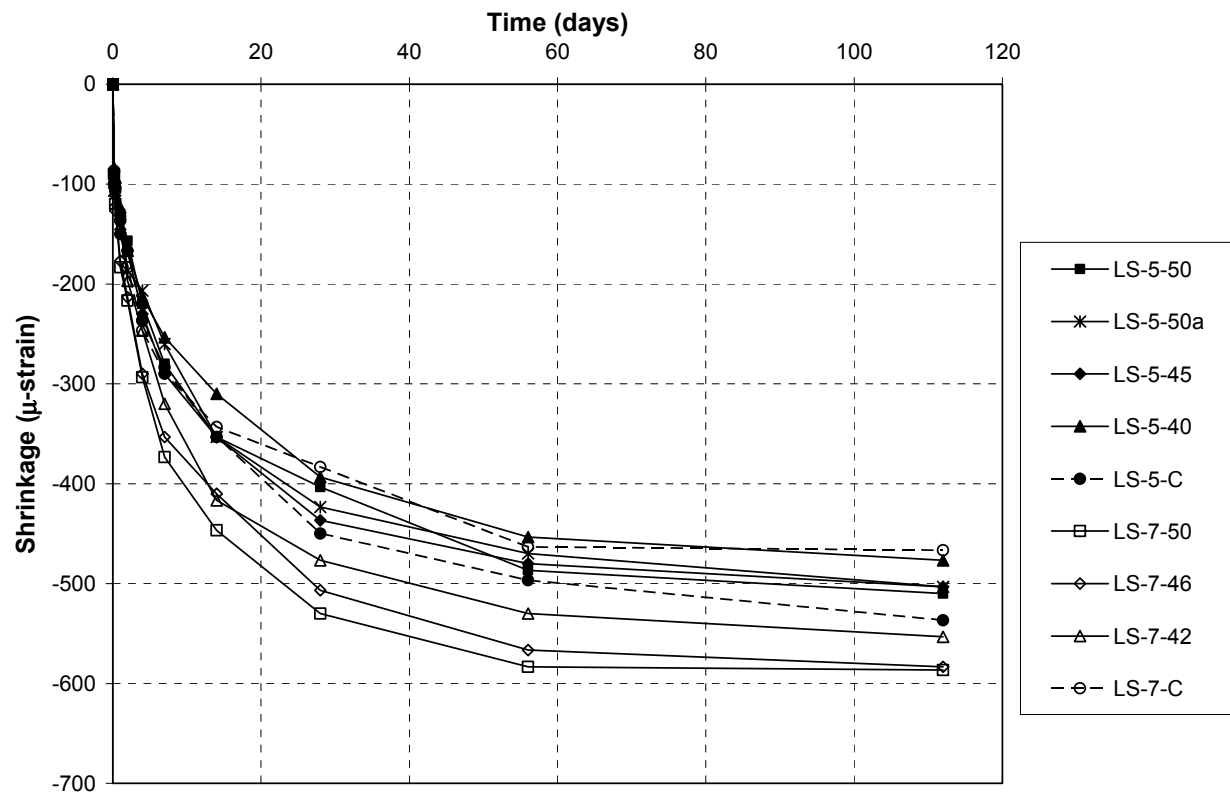


Figure 6.6 Shrinkage Strain Measurements to 112 Days (Crushed Limestone Aggregate Set)

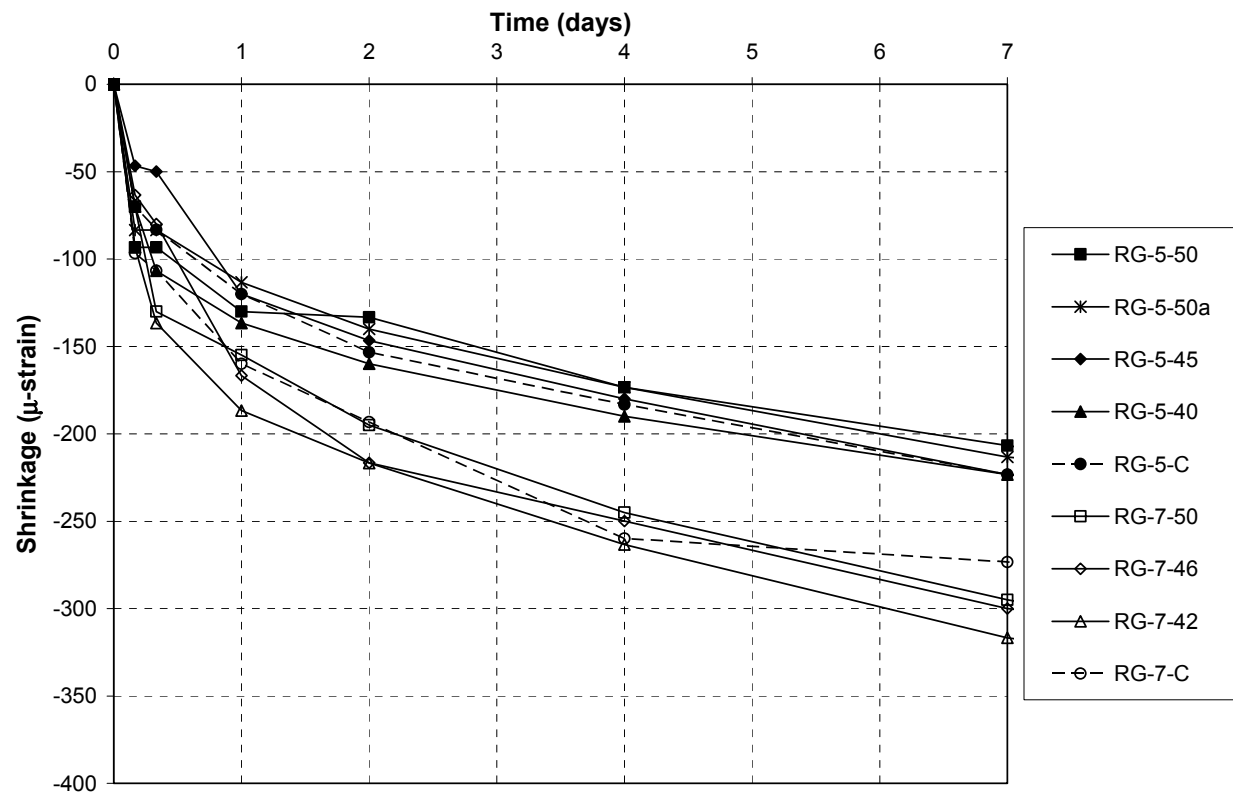


Figure 6.7 Shrinkage Strain Measurements to 7 Days (River Gravel Aggregate Set)

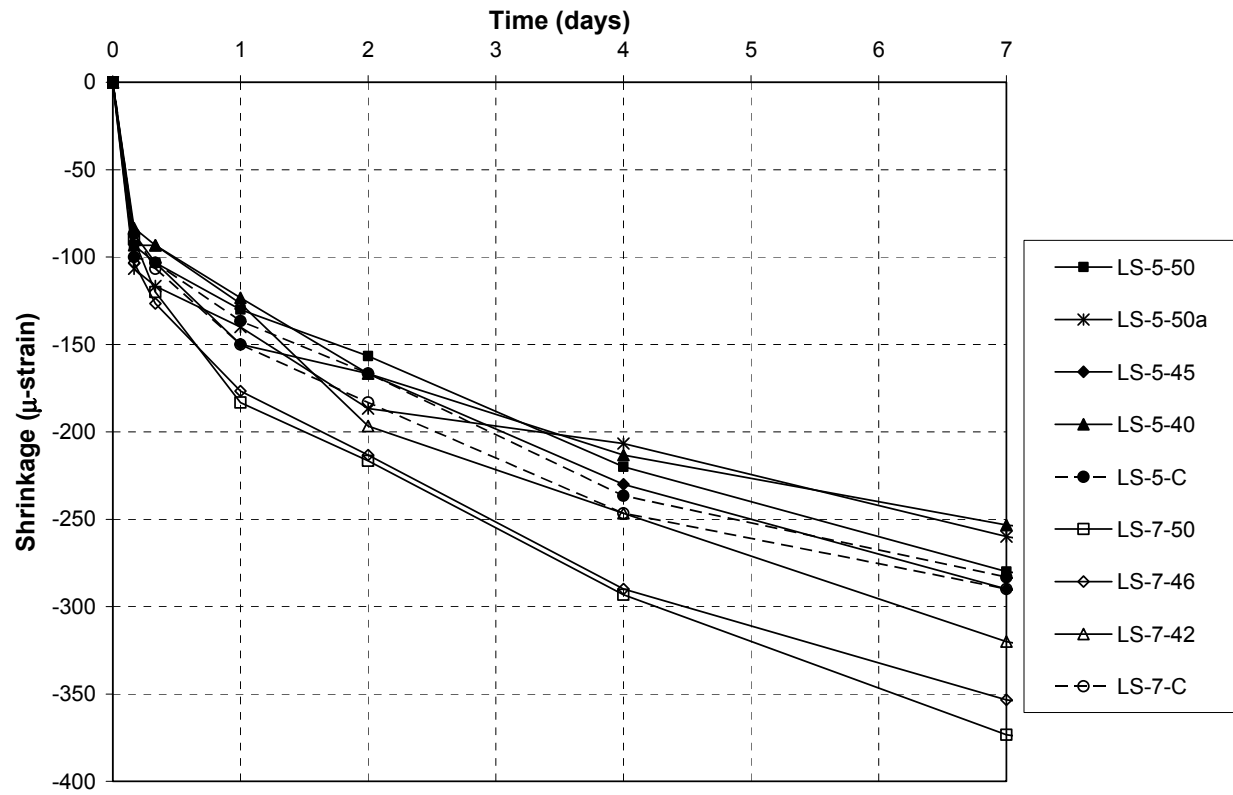


Figure 6.8 Shrinkage Strain Measurements to 7 Days (Crushed Limestone Aggregate Set)

7. Segregation Resistance

Segregation resistance is a critical property for all SCC mixtures. SCC is susceptible to severe segregation due to the low yield stresses necessary to achieve self-flow and due to the potentially low thixotropy resulting from the high degree of dispersion of powder materials. A laboratory study was conducted to evaluate the factors contributing to segregation resistance and to evaluate test methods available for measuring segregation resistance. To accomplish this, segregation resistance was related to rheology and rheology was related to mixture proportions. Five test methods were evaluated to determine their suitability for measuring segregation practically and reliably. These test methods were the column segregation test, sieve stability test, penetration apparatus test, visual stability index, and hardened concrete column test.

SCC must be proportioned to exhibit both dynamic and static stability. Dynamic stability describes segregation resistance as concrete undergoes dynamic conditions such as mixing, pumping, flowing down chutes, filling formwork, and passing through reinforcement. Static stability describes segregation resistance when the concrete is at rest. This chapter addresses static stability.

7.1 Background

Segregation resistance has been studied extensively both in the concrete industry and in other fields. Most commonly, models have been developed for the movement of a sphere or other object within a visco-plastic fluid. Attempts have been made to incorporate the complexities associated with concrete into the models of segregation resistance; however, no models take into account all relevant factors. In addition, various recommendations are available for proportioning SCC to achieve segregation resistance.

7.1.1 Modeling of Segregation

The movement of aggregates in concrete is a complex problem without a precise analytical solution. Segregation resistance depends on the rheology of the paste; the relative densities of the aggregates and paste; the aggregate shape, grading, and volume fraction; and the geometry of the concrete element and any inclusions such as reinforcing bars.

The movement of a single sphere in a Bingham material has been studied experimentally, analytically, and numerically, as summarized by Blackery and Mitsoulis (1997) and de Besses, Magnin, and Jay (2004). Such work can be extended to the case of aggregates in cement paste. For the general case of a sphere in a Newtonian fluid, gravitational and buoyant forces act on the sphere. If the density of the sphere is greater than that of the fluid, the gravitational force will exceed the buoyant force, resulting in a net downward force, F , given in Equation (7.1):

$$F = \frac{4}{3}\pi R^3(\rho_{sphere} - \rho_{fluid})g \quad (7.1)$$

where R is the sphere radius, ρ_{sphere} is the density of the sphere, ρ_{fluid} is the density of the fluid, and g is acceleration due to gravity. If the fluid exhibits a yield stress, however, an opposing

force attributable to the yield stress will offset the net downward force and—depending on the magnitude of the yield stress—may prevent the sphere from settling. If the yield stress is sufficiently high, the sphere will not settle.

Berris et al. (1985) performed finite element modeling of a sphere in an infinite Bingham material. They defined a dimensionless yield stress parameter, Y_g , given in Equation (7.2), as the ratio of the force due to yield stress to the net gravitational and buoyant forces:

$$Y_g = \frac{2\tau_0\pi R^2}{F} = \frac{2\tau_0\pi R^2}{\frac{4}{3}\pi R^3(\rho_{sphere} - \rho_{fluid})g} = \frac{3\tau_0}{2R(\rho_{sphere} - \rho_{fluid})g} \quad (7.2)$$

where τ_0 is the yield stress. The value of Y_g can vary from zero for a Newtonian fluid to 0.143, beyond which no flow around the sphere occurs. Therefore, if the value of Y_g is greater than 0.143, the sphere does not settle. For cases where Y_g is less than 0.143, the Stokes' drag coefficient, C_s , is used to compute the terminal settling velocity, V , as shown in Equation (7.3):

$$V = \frac{2(\rho_{sphere} - \rho_{fluid})R^2g}{9C_s\mu} \quad (7.3)$$

where μ is the plastic viscosity. The value of Stokes' drag coefficient can be determined from the results of various finite element simulations. Based on the simulation of Beris et al. (1985), the Stokes' drag coefficient increases from 1.0 for a Newtonian fluid to infinity for $Y_g = 0.143$. Therefore, viscosity alone is not sufficient to determine the rate of settlement for a given difference in aggregate and paste density once the yield stress is exceeded—yield stress is also important for predicting the rate of settlement.

Blackery and Mitsoulis (1997) extended the work of Beris et al. to the case of a sphere in tubes of various sizes. For this case, the value of Y_g beyond which no flow occurs remained 0.143; however, the value of Stokes' drag coefficient increased as the ratio of the tube to the sphere decreased, as shown in Figure 7.1. In concrete, the aggregates, formwork, and any inclusions such as reinforcement would increase the Stokes' drag coefficient, just as decreasing the tube to sphere diameter ratio did for the simulations of Blackery and Mitsoulis (1997).

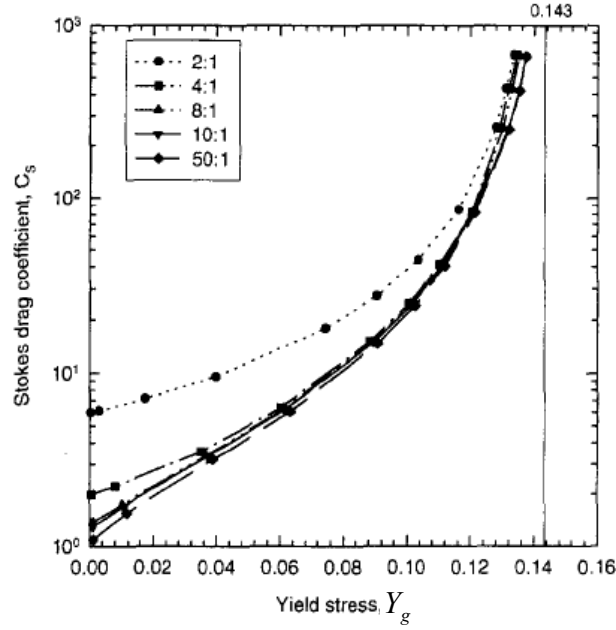


Figure 7.1 Stokes Drag Coefficient for Different Ratios of Tube to Sphere Diameter (From Blackery and Mitsoulis 1997)

Petrou et al. (2000) applied the work of Beris et al. (1985) to a vibrated concrete mixture. They verified that if the yield stress is high enough, the aggregate would not settle. When vibration was applied, however, the yield stress was reduced and the aggregate settled.

Jossic and Magnin (2001) experimentally evaluated the stability of objects of various shape, orientation, and roughness. The objects were displaced at a slow speed in a Herschel Bulkley solution and the resulting force to initiate flow was measured. The stability criterion was established in terms of Y_{max} , as shown in Equation (7.4):

$$Y_{max} = \frac{\tau_0}{gd_e \Delta \rho} \quad (7.4)$$

where d_e is the diameter of a sphere with the same volume as the object. (Similarly, Petrou et al (2000) suggested the use of d_e for irregularly shaped aggregates.) The variation in the value of Y_{max} is summarized in Figure 7.2. The value of Y_{max} was determined to be 0.088 for a smooth sphere and 0.062 for a rough sphere. Stability was found to increase as the frontal area increased.

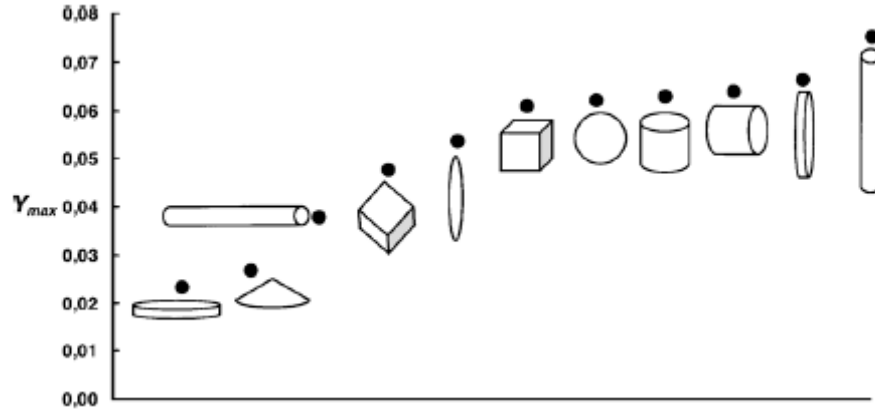


Figure 7.2 Effect of Shape and Orientation on Y_{max} for Rough Objects (From Jossic and Magnin 2001)

Saak, Jennings, and Shah (2001) developed and experimentally verified an analytical model for concrete segregation. The model consisted of a single sphere in cement paste. For segregation to be avoided, the gravitational force acting on an aggregate must be offset by the buoyant force and the restoring force, as shown in Figure 7.3.

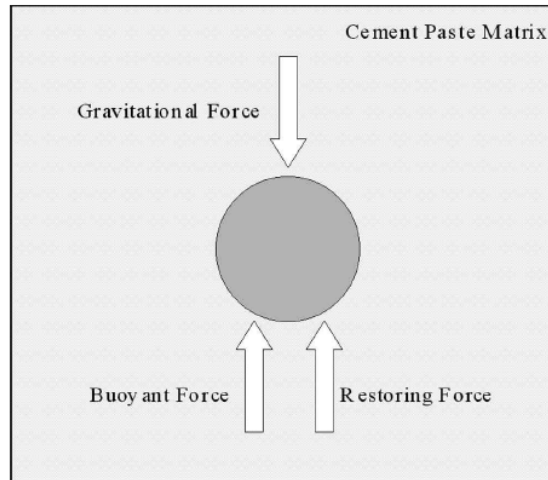


Figure 7.3 Model of Aggregate in Paste (From Saak, Jennings, and Shah 2001)

For static conditions, the restoring force is provided by the yield stress of the paste. Therefore, to prevent static segregation, Equation (7.5) must be satisfied.

$$\tau_0 \geq \frac{4}{3} g (\rho_{sphere} - \rho_{fluid}) R \quad (7.5)$$

For dynamic conditions where the yield stress is exceeded and the aggregate does settle, the velocity of aggregate must be minimized. In this case, the restoring force is provided by the drag force, given in Equation (7.6):

$$F_{drag} = \frac{1}{2} C_D \rho_{fluid} V^2 A_p \quad (7.6)$$

where C_D is the drag coefficient and A_p is the cross sectional area of the sphere. The terminal velocity of the aggregate can be expressed by combining the drag force equation with the balance of forces shown in Figure 7.3, as given in Equation (7.7).

$$V = \sqrt{\frac{8}{3} \frac{g(\rho_{sphere} - \rho_{fluid})R}{C_D \rho_{fluid}}} \quad (7.7)$$

The drag coefficient is a function of the Reynolds number (Re), with high Reynolds numbers associated with low drag coefficients. The Reynolds number is given in Equation (7.8):

$$Re = \frac{2R\rho_{fluid}V}{\eta} \quad (7.8)$$

where η is the apparent viscosity. A higher apparent viscosity corresponds to a lower Reynolds number, a higher drag coefficient, and lower terminal velocity.

Based on the equations from Saak, Jennings, and Shah, the yield stress and apparent viscosity should be high enough to prevent static and dynamic segregation, respectively. Additionally, the difference in densities between the paste and aggregates should be minimized to mitigate segregation. Yield stress and apparent viscosity, however, must also be sufficiently low for adequate workability. For concrete mixture proportioning, Saak, Jennings, and Shah suggest the use of a self-flow zone (Figure 7.4) where both self-flow and segregation criteria are satisfied.

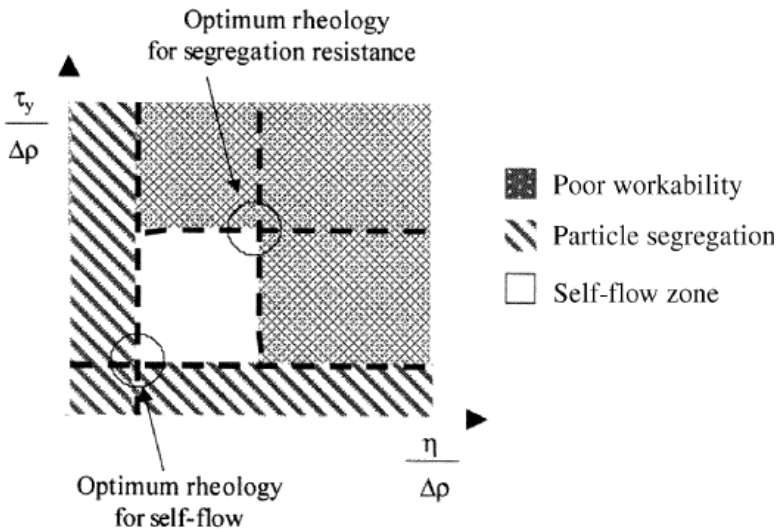


Figure 7.4 Proposed Self-Flow Zone (From Saak, Jennings, and Shah 2001)

Saak, Jennings, and Shah also point out that concrete is much more complex than the single aggregate model. In concrete, larger aggregates begin to segregate at a higher yield stresses than do smaller particles. Therefore, the fine aggregates create an additional upward force as the coarser aggregates begin to settle. The aggregate shape characteristics also contribute to segregation resistance.

The approaches of Beris et al. (1985); Jossic and Magnin (2001); and Saak, Jennings, and Shah (2001) are compared in Table 7.1. The equations are rewritten for direct comparison. The minimum necessary yield stress computed from the Saak, Jennings, and Shah equation is 14-times the yield stress determined from the Beris et al. (1985) solution. Bethmont et al. (2003) conducted experimental measurements to evaluate these three approaches. Different sized glass spheres were allowed to settle in cement pastes of varying yield stresses. As expected, it was found that increasing the yield stress or decreasing the sphere diameter reduced settlement of the sphere. The experimental results were found to match closely to the equation of Jossic and Magnin (2001) for the case of a rough sphere and the equation of Beris et al. (1985). The equation of Saak, Jennings, and Shah (2001), however, was found to be unsuitable.

Table 7.1 Comparison of Analyses of Yield Stress to Prevent Settlement

Source	Minimum Yield Stress to Prevent Settlement
Berris et al. (1985)	$\tau_0 \geq \frac{2}{3} Y_g g(\rho_{sphere} - \rho_{fluid})R = (0.09533)g(\rho_{sphere} - \rho_{fluid})R$
Jossic and Magnin (2001) (rough sphere)	$\tau_0 \geq Y_{max} g(\rho_{sphere} - \rho_{fluid})2R = (0.124)g(\rho_{sphere} - \rho_{fluid})R$
Saak, Jennings, and Shah (2001)	$\tau_0 \geq \frac{4}{3} g(\rho_{sphere} - \rho_{fluid})R$

It should be noted that segregation can occur in concrete mixtures where the paste yield stress is much higher than those shown in Table 7.1. For example, concrete used for paving applications can segregate. In traditionally vibrated concretes, vibration decreases the yield stress, such that segregation can occur. Further, the mechanisms responsible for segregation are different in high yield stress concretes that are more appropriately considered loose, granular materials rather than fluids. Therefore, the rheological parameters considered for SCC and other highly fluid concrete mixtures are not relevant to loose, granular materials.

In evaluating concrete, a distinction must be made between static and dynamic yield stress (Roussel 2006). Due to thixotropy, a flocculated structure builds up within at-rest paste. This built-up structure resists flow and prevents particles from settling. The static yield stress reflects the amount of stress needed to initiate flow in an at-rest material while the dynamic yield stress reflects the stress needed to maintain flow after the at-rest structure has been destroyed.

The difference between static and dynamic yield stress is illustrated conceptually in Figure 7.5. The static and dynamic yield stresses are equal immediately after mixing. The dynamic yield stress increases due to the loss of admixture efficacy and hydration. The static yield stress of un-agitated, at-rest SCC increases faster than the dynamic yield stress because of the build-up of an easily destroyed at-rest thixotropic structure, which acts in addition to the effects of reduced admixture efficacy and hydration. Concrete being placed in a precast plant is partially agitated during transport, which prevents the build-up of an at rest structure. When the concrete is placed, the resulting shearing fully breaks down the thixotropic at-rest structure, such

that the static yield stress again equals the dynamic yield stress. Once the concrete is again at rest in the formwork, the thixotropic at-rest structure rebuilds and the static yield stress increases.

In evaluating segregation susceptibility, the static yield stress should be considered instead of the dynamic yield stress because the concrete is at rest. The dynamic yield stress is important initially because the static and dynamic yield stresses are initially equal. The difference between static and dynamic yield stress at any given time reflects the extent of build-up of the thixotropic at-rest structure. It is the absolute magnitude of the static yield stress—not the amount of thixotropy per se—that ultimately determines whether an aggregate settles.

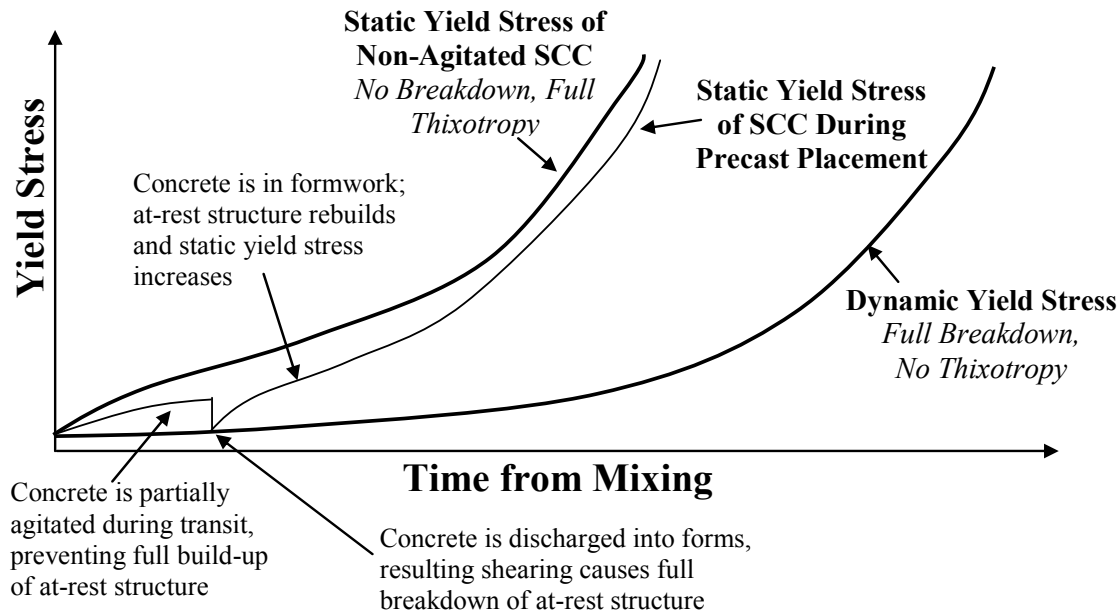


Figure 7.5 Conceptual Changes in Static and Dynamic Yield Stress with Time

7.1.2 Mixture Proportioning for Segregation

According to Khayat (1999) and El-Chabib and Nehdi (2006), achieving a moderate plastic viscosity is essential to achieving segregation resistance. Daczko (2003) states that segregation resistance is influenced by the cementitious materials, aggregates, water content, and admixtures. Specifically, increasing the cementitious materials content increases concrete viscosity, reducing segregation. The densities of the aggregates relative to the paste affect the yield stress needed to prevent segregation while the grading of the aggregates affects bleeding. The water content influences the paste rheology. Over-dosages of HRWRA can contribute to stability problems while the use of VMA and air entraining agents can reduce segregation.

Based on an experimental evaluation of 123 SCC mixtures, El-Chabib and Nehdi (2006) found that segregation increased with increasing w/cm and increasing HRWRA dosage. Increasing the cementitious materials content reduced segregation for low w/cm mixtures but increased segregation for high w/cm mixtures. The use of a certain threshold dosage of VMA resulted in the prevention of segregation. The ratio of coarse aggregate to fine aggregate had minimal effect on segregation.

According to Khayat (1999), stability can be improved by reducing the separation of solids and by minimizing bleeding. The separation of solids is reduced by limiting the coarse aggregate content, reducing the maximum aggregate size, or increasing the cohesion and viscosity. Bleeding is minimized by reducing the water content, reducing the water-to-powder ratio, using a powder with a high surface area, or using a viscosity modifying admixture.

Viscosity modifying admixtures can improve segregation resistance by imparting a shear-thinning character or by increasing thixotropy (Khayat 1998a). Shear thinning is beneficial because the high viscosity at low shear rates prevents segregation while the low viscosity at high shear rates ensures high flowability. Thixotropy enhances segregation resistance by increasing the static yield stress when the concrete is at rest. This yield stress is reduced to the dynamic yield stress when the concrete is subjected to shearing. In one example VMA application, Terpstra (2005) examined the use of a certain cellulose-based stabilizing agent that increased the yield stress but maintained a low plastic viscosity, resulting in enhanced segregations resistance with favorable flow conditions.

Figure 7.6 illustrates the effects of changing the aggregate specific gravity, fly ash rate, and w/cm on the paste yield stress to prevent segregation. In changing the proportions of SCC, the resulting differences in the paste yield stress to prevent segregation should be compared to the differences in paste yield stress. Due to the high sensitivity of SCC rheology to changes in mixture proportions, it is likely that any changes in materials or mixture proportions will have a larger effect on paste yield stress than paste specific gravity.

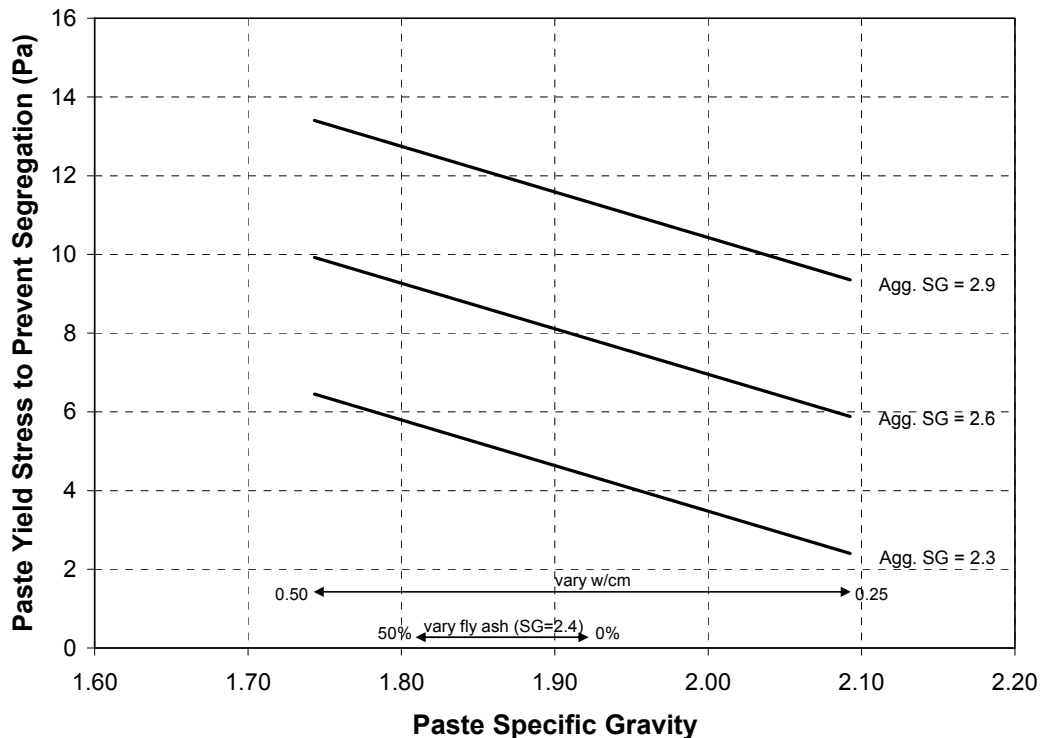


Figure 7.6 Paste Yield Stress to Prevent Segregate (Jossic and Magnin Equation, Rough Sphere)

7.2 Laboratory Testing Program

The laboratory testing was conducted in three phases. In the first phase, which consisted of 9 mixtures, the rheology over time was evaluated. In the second series, a central composite response surface was used to evaluate a range of mixture proportions. In the third phase, the effect of HRWRA and VMA dosage were evaluated.

7.2.1 Materials and Mixture Proportions

All mixtures incorporated Type III cement (PC-A), river gravel (RG), and natural sand (NS-A). Class F fly ash (FA-A) was used in all mixtures at a 25% mass replacement rate. A retarding admixture (RET-A) was used in all mixtures at a rate of 4 oz per 100 lb of cement. The HRWRA dosage (HRWRA-A) was adjusted in each mix to reach a slump flow of 29 +/- 1 inches unless noted otherwise. The first phase of testing consisted of 6-cubic foot batches while the other phases consisted to 3-cubic foot batches. Results from different batch sizes should not be compared directly due to the different degrees of dispersion achieved with different batch sizes. All mixture proportions and test results are in Appendix B.

7.2.1.1 Phase I: Evaluation of Rheology with Time

In the first phase, nine mixtures were compared by varying the paste volume between 30% and 38% and the w/cm between 0.24 and 0.39 (Table 7.2). The sand-aggregate ratio was held constant at 0.45. The mixtures were prepared in 6-cubic foot batches to enable four separate rheometer measurements over time. In addition, the hardened concrete column test was performed for these mixtures.

Table 7.2 Mixture Proportions for Phase I

Mixture ID	Indices		Proportions (lb/yd ³)				
	Paste Volume (%)	w/p w/cm	Cement	Fly Ash	Coarse Agg. (SSD)	Fine Agg. (SSD)	Water
S1	34.0	0.30	627.0	209.0	1584.0	1291.0	250.8
S2	30.0	0.30	548.6	182.9	1680.0	1369.2	219.4
S3	38.0	0.30	705.3	235.1	1488.0	1212.8	282.1
S4	34.0	0.24	691.3	230.4	1584.0	1291.0	221.2
S5	34.0	0.36	573.6	191.2	1584.0	1291.0	275.3
S6	36.4	0.264	713.0	237.7	1526.9	1244.5	251.3
S7	31.6	0.336	550.0	183.3	1641.1	1337.5	246.1
S8	34.0	0.33	599.1	199.7	1584.0	1291.0	263.6
S9	34.0	0.39	550.2	183.4	1584.0	1291.0	286.1
Note: All mixtures included 4 oz/cwt of RET-A. HR-A dosage was adjusted for a slump flow of 29 +/- 1 inches.							

7.2.1.2 Phase II: Central Composite Response Surface

In the second phase, a Box-Wilson central composite response surface experiment design was used to evaluate three factors: paste volume, water-cementitious materials ratio, and sand-aggregate ratio. The central composite design consisted of 18 mixtures including 8 factorial points (-1, 1), 6 start points (-1.68, 1.68), and 4 center points (0). The values of the factors are shown in Table 7.3.

Table 7.3 Central Composite Response Surface Factors for Phase II

	Coded Values				
	-1.68	-1	0	1	1.68
Paste Volume	0.30	0.316	0.34	0.364	0.38
w/p (w/cm)	0.24	0.264	0.30	0.336	0.36
S/A	0.40	0.420	0.45	0.480	0.50
Note: All mixtures included 4 oz/cwt of RET-A. HR-A dosage was adjusted for a slump flow of 29 +/- 1 inches.					

7.2.1.3 Phase III: Evaluation of VMA and HRWRA Dosage

In the third phase, the use of VMA was evaluated at two dosages in two separate mixtures and the HRWRA dosage was adjusted in one mixture to achieve higher and lower slump flows than used in all other mixtures (Table 7.4). The VMA was used at 2 oz/cwt and 14 oz/cwt of cementitious materials, which are the minimum and maximum recommended dosages from the manufacturer. The two mixtures selected for evaluation of VMA dosage exhibited segregation when tested without VMA. In mixtures A5 and A6, the slump flow was varied by changing the HRWRA dosage in a mixture that exhibited good segregation resistance when tested at a slump flow of 29 +/- 1 inches.

Table 7.4 Mixture Proportions for Phase III

Mixture ID	Indices		Proportions (lb/yd ³)					Change
	Paste Volume (%)	w/p w/cm	Cement	Fly Ash	Coarse Agg. (SSD)	Fine Agg. (SSD)	Water	
A1	34.0	0.24	691.3	230.4	1584.0	1291.0	221.2	VMA @ 2 oz/cwt
A2	34.0	0.24	691.3	230.4	1584.0	1291.0	221.2	VMA @ 14 oz/cwt
A3	34.0	0.36	573.6	191.2	1584.0	1291.0	275.3	VMA @ 2 oz/cwt
A4	34.0	0.36	573.6	191.2	1584.0	1291.0	275.3	VMA @ 14 oz/cwt
A5	34.0	0.30	627.0	209.0	1584.0	1291.0	250.8	22-inch slump flow
A6	34.0	0.30	627.0	209.0	1584.0	1291.0	250.8	32-inch slump flow
Note: All mixtures included 4 oz/cwt of RET-A. HR-A dosage was adjusted for a slump flow of 29 +/- 1 inches except as indicated in mixtures A5 and A6.								

7.2.2 Test Methods

Four empirical test methods (slump flow with T₅₀ and VSI, penetration apparatus, column segregation test, sieve stability test) and rheometer measurements were conducted for each mixture. In addition, the hardened concrete column test was performed on the mixtures in Phase

I. The hardened concrete column test was used as an absolute standard by which to compare other test methods because it best represents actual field conditions. The test procedures are described fully in Appendix A.

All tests were conducted in a consistent order after the end of mixing. First, concrete was sampled directly from the mixer and used in the slump flow test (cone in inverted position). If the slump flow did not reach the target value, the HRWRA dosage was adjusted, the concrete remixed, and the slump flow tested again. The penetration apparatus test was performed in conjunction with the slump flow test. As soon as the slump flow test was complete, concrete was discharged from the mixer to a wheelbarrow and then to the test methods in the following order: rheometer, column segregation test, sieve stability test, and hardened concrete column test. The first rheometer test was started immediately after filling the first rheometer container. Subsequent rheometer tests were conducted on separate samples that were left undisturbed in the rheometer container until testing. Rheometer measurements were performed at 0, 3, 7, and 15 minutes after filling the rheometer containers for Phase I and at 0 and 15 minutes after filling the rheometer containers for Phases II and III. Any remaining concrete was left undisturbed in a wheelbarrow. The slump flow and penetration apparatus tests were repeated after 15 minutes from concrete left in this wheelbarrow.

The ICAR rheometer was used to perform a stress-growth test and measure a flow curve. The stress growth test indicated the static yield stress while the flow curve indicated the dynamic yield stress, plastic viscosity, and breakdown area (a measure of thixotropy). The test regime, which is shown in Figure 7.7, consisted of an up and down curve. Each curve consisted of 8 speed points held for 5 seconds each. The speed ranged from 0.05 to 0.50 rps. After the up curve, the maximum speed was held constant for 20 seconds to ensure full breakdown of any effects due to thixotropy. No measurements were recorded during this extended breakdown time. The yield stress and plastic viscosity were computed for the down curve. The breakdown area was calculated as the area between the up and down flow curves. The first point of the up curve was also used for the stress growth test. In the stress growth test, the gradual build-up in torque over time was monitored and the maximum torque was determined and used to compute the yield stress.

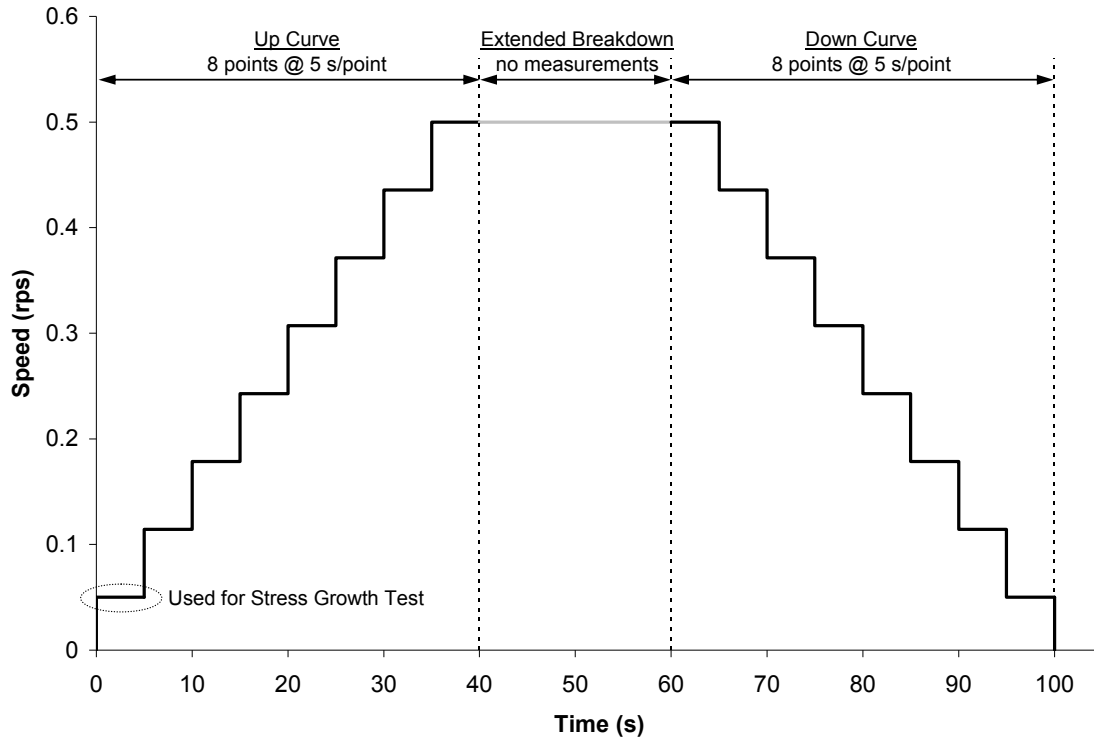


Figure 7.7 ICAR Rheometer Test Regime

7.2.3 Test Results: Factors Contributing to Stability

The factors contributing to stability were evaluated in terms of rheology and of in terms of materials and mixture proportions.

7.2.3.1 Rheology

Segregation resistance depends on the initial rheology and the changes in rheology over time. Typical rheological measurements are shown in Figure 7.8 and Figure 7.9 for Mixture S6, which exhibited good segregation resistance. In the flow curve measurements shown in Figure 7.8, the up curve for each measurement is above the down curve because of the presence of thixotropy. The intercept of the down curve—related to dynamic yield stress—the slope of the down curve—related to plastic viscosity—and the area between the up and down curves—a measure of thixotropy—increase with time. The up curve increases faster than the down curve because of thixotropy. In the stress growth measurements shown in Figure 7.9, the torque increases linearly before reaching a maximum. As the thixotropic, built-up structure is destroyed, the torque decreases. In the initial measurement, there is no decrease because no built-up structure has formed. At 15 minutes, the torque decreases after reaching a maximum because a built-up structure has formed. The maximum torque reached indicates the static yield stress. The dynamic yield stress from the flow curve and the static yield stress from the stress growth test are compared over time in Figure 7.10. As expected, the dynamic yield stress increases gradually due to hydration and the loss of admixture efficacy. The static yield stress

increases at a faster rate because of the build-up of a thixotropic structure. The static yield stress is greater than the dynamic yield stress at the initial measurement. This discrepancy is likely the result of not being able to start rheology measurements instantly after mixing. The dashed line shows the likely change in static yield stress if it could be measured starting immediately after mixing. The fact that most mixtures exhibited thixotropy (positive breakdown areas) in the initial measurement indicates that an at-rest structure did begin to form prior to the start of the initial rheometer measurement.

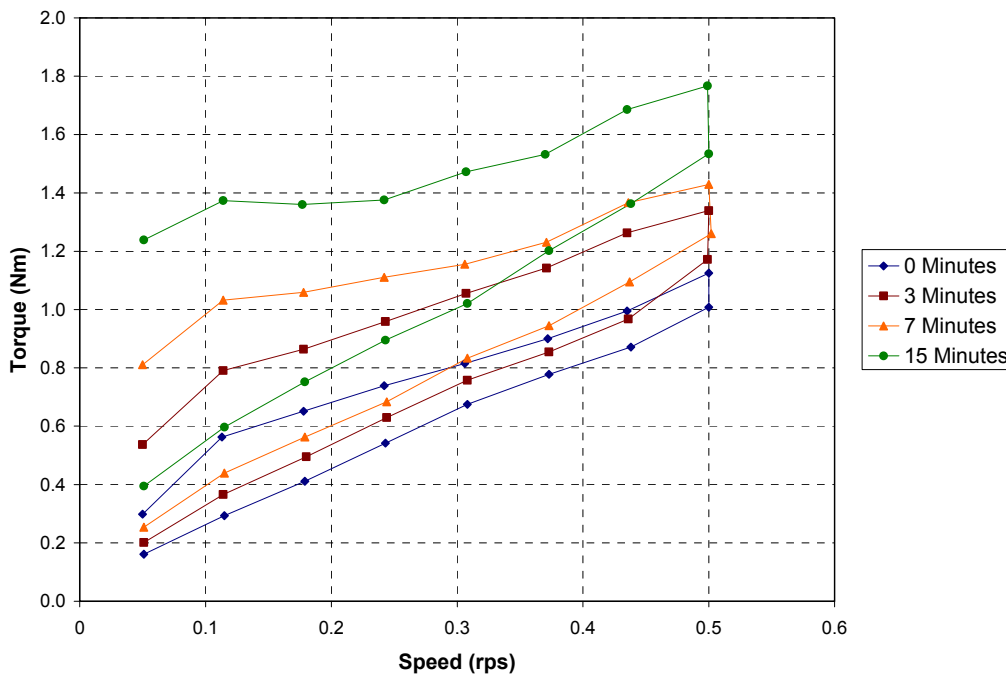


Figure 7.8 Typical Flow Curve Measurements (Mixture S6)

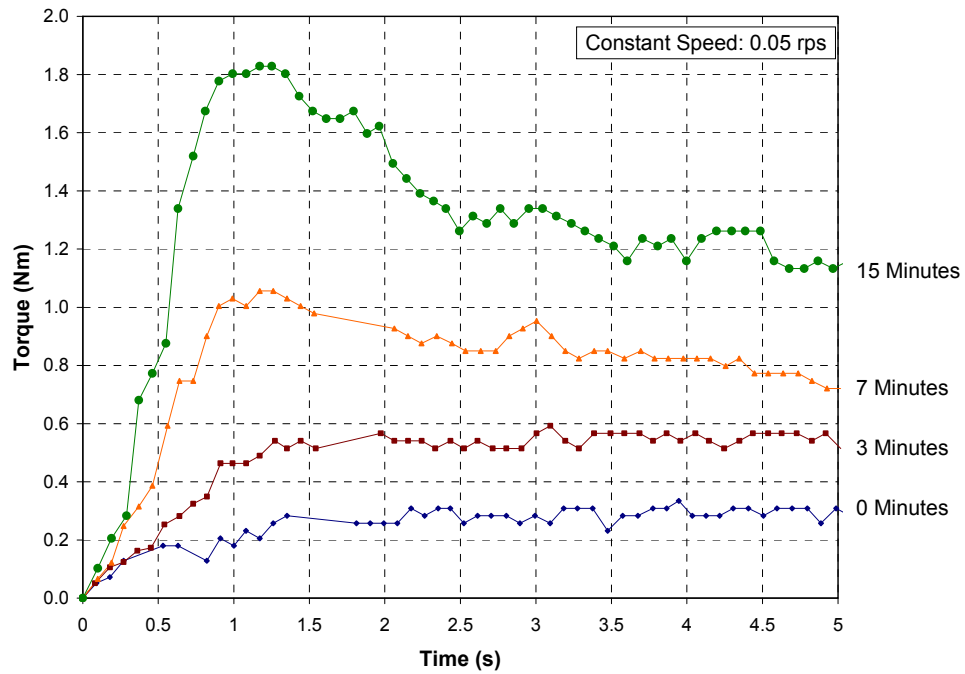


Figure 7.9 Typical Stress Growth Test Measurements (Mixture S6)

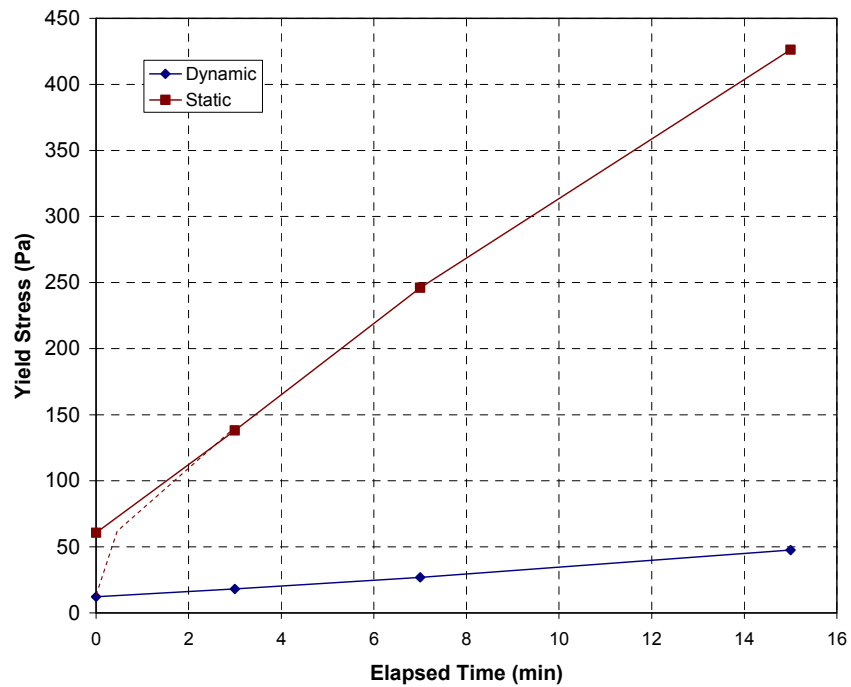


Figure 7.10 Change in Dynamic and Static Yield Stress with Time (Mixture S6)

To evaluate the effects of rheological parameters on segregation resistance, individual rheological parameters were first compared to the results of the column segregation test. Figure 7.11 indicates that increased initial dynamic yield stress and static yield stresses initially and at 15 minutes were associated with increased segregation resistance. The scatter, however, was high. Initially, the dynamic and static yield stresses are close because the thixotropic, built-up structure has not formed. While the initial yield stresses are important, they do not reflect the rate of increase in static yield stress. High initial yield stresses were associated with low segregation; however, some mixtures with low segregation had low initial yield stresses. If the static yield stresses in mixtures with low initial static yield stress increase quickly, the potential for segregation would be minimized. The static yield stress at 15 minutes cannot predict the segregation effectively because it does not distinguish between static yield stresses that are consistently high and static yield stresses that start too low but increase later. The thixotropic breakdown areas were not highly correlated with segregation resistance because it is the magnitude of the yield stress, not the amount of thixotropy per se, that determines segregation resistance. A mixture with high initial yield stress but low thixotropy could have similar segregation as a mixture with low initial yield stress but high thixotropy. Although it has been suggested that a moderate plastic viscosity is necessary for securing segregation resistance, the plastic viscosity was not correlated with segregation resistance. Although high viscosity can slow the rate of segregation, the yield stress, not plastic viscosity, stops segregation.

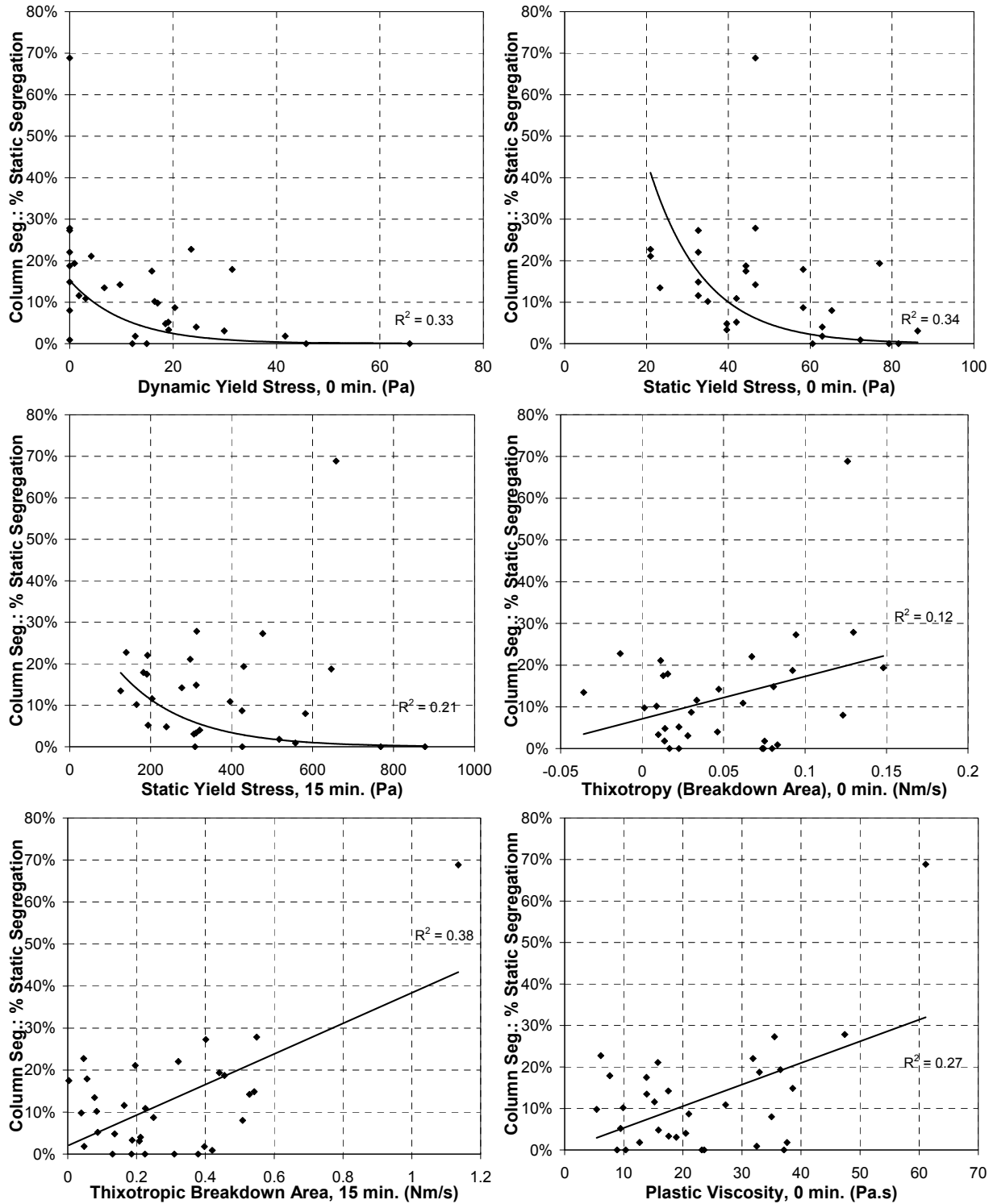


Figure 7.11 Relationships between Individual Rheological Parameters and Sieve Stability Test

Next, pairs of rheological parameters were compared to the column segregate test results, as indicated in Figure 7.12. As the initial dynamic yield stress is decreased, it is necessary to have higher plastic viscosity, increased degrees of thixotropy, and increased static yield stress.

Plastic viscosity reduces the extent of segregation until the static yield stress increases sufficiently to support aggregate particles. It is also correlated with the degree of thixotropy, as indicated in Figure 7.13, because small interparticle spacing between powder constituents increases both viscosity and thixotropy. It should be cautioned that, for a given slump flow, increases in plastic viscosity are typically associated with decreases in yield stress. The increased rate of thixotropic rebuilding increases the static yield stress quickly to prevent segregation. High static yield stresses at 15 minutes associated with low initial dynamic yield stresses reflect a high rate of thixotropic rebuilding over time.

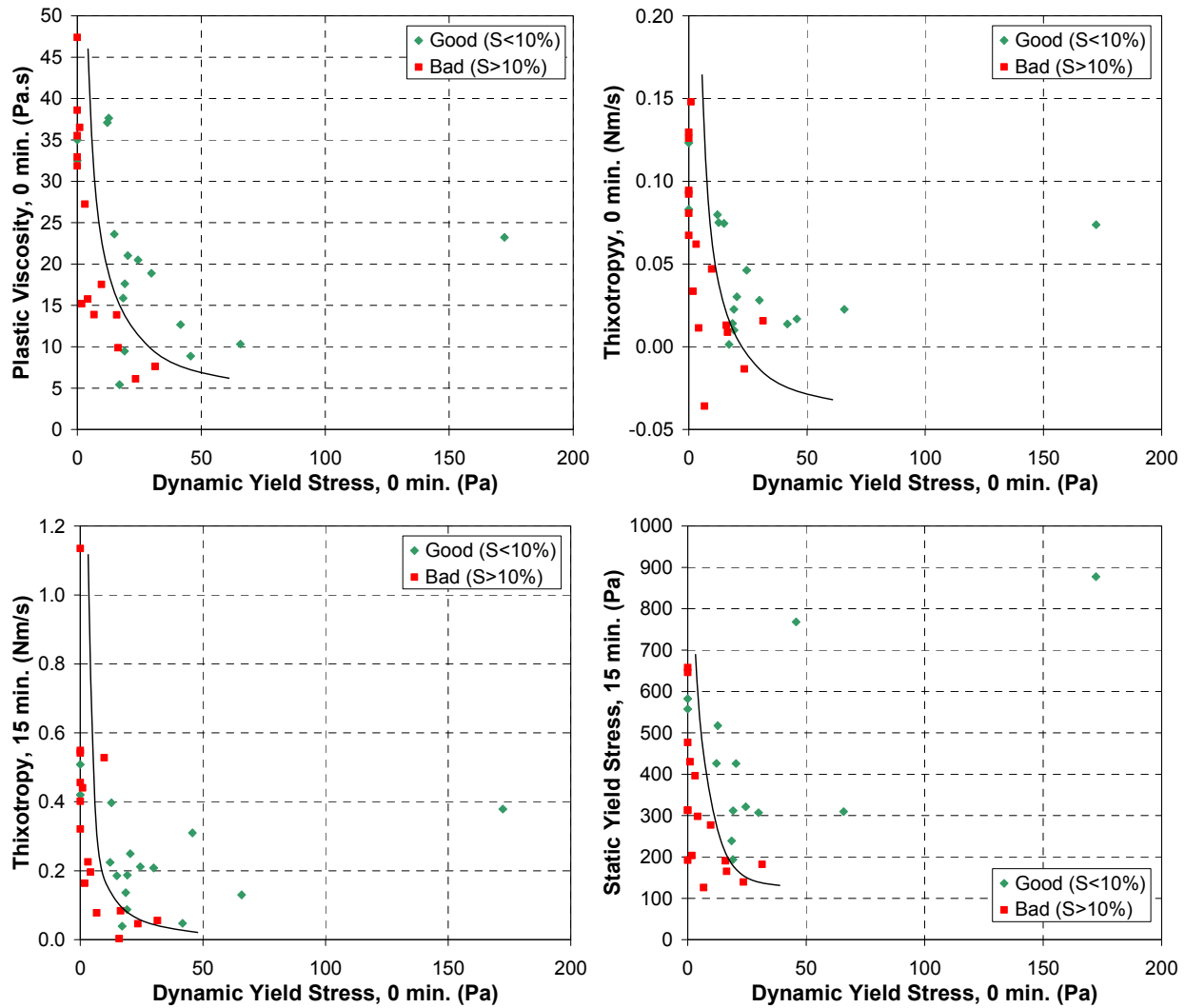


Figure 7.12 Effect of Initial Dynamic Yield Stress and Plastic Viscosity on Column Segregation Test (S)

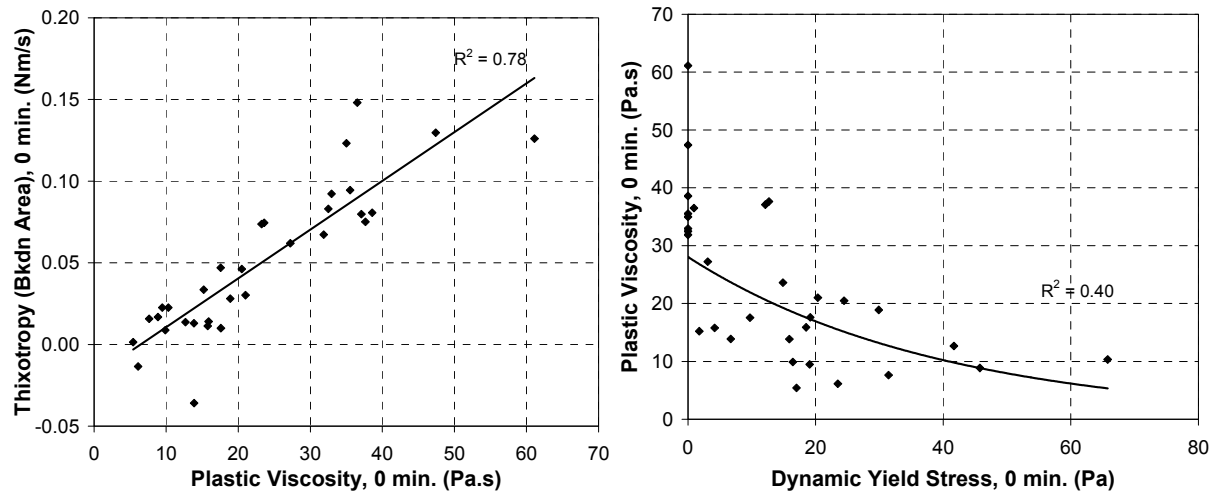


Figure 7.13 Relationships Between Rheological Parameters

Based on the literature review and laboratory test results, the following rheological parameters are key to ensuring segregation resistance:

- **Paste static yield stress with time.** The magnitude of the static yield stress ultimately determines whether aggregates sink. The static yield stress over time depends on the initial dynamic yield stress, the loss of workability, and the rate of thixotropic rebuilding.
 - **Initial paste dynamic yield stress.** Initially, the static yield stress is equal to the dynamic yield stress. The initial dynamic yield stress must be low enough for self-flow; however, this also increases the susceptibility to segregation. As soon as the shear rate drops, the static yield stress increases at a faster rate than the dynamic yield stress. However, the dynamic yield stress represents the starting point. If the dynamic yield stress is too low initially, some segregation will occur regardless of the rate of thixotropic rebuilding and loss of workability.
 - **Loss of workability.** Over time both the static and dynamic yield stresses increase due to the loss of workability resulting from hydration and the gradual reduction in admixture efficacy. Due to the short time scale relevant to segregation, the reduction in admixture efficacy is more consequential.
 - **Rate of thixotropic rebuilding.** The static yield stress increases at a faster rate than the dynamic yield stress because of the build-up of a thixotropic, at rest structure. As the dynamic yield stress is reduced, the rate of thixotropic rebuilding is more important. In conventionally placed concrete, the rate of thixotropic rebuilding is not as important as in SCC because the yield stress is higher.
- **Paste plastic viscosity.** If aggregates settle, the plastic viscosity affects the rate at which aggregates settle. Although the plastic viscosity does increase with time, it does not increase as quickly as the yield stress. High plastic viscosity may be associated with high thixotropy because both can be caused by close interparticle spacing of particles; however, high plastic viscosity does not assure segregation resistance in cases where the static yield stress remains too low.

It is not possible to compare the measured concrete rheological parameters to the equations in Table 7.1, which are for paste or mortar. The addition of aggregates to paste increases the yield stress and plastic viscosity of the concrete. The relationship between paste or mortar rheology and concrete rheology is complex. It is not feasible to measure paste and mortar separately because of the observer effect—that is, when paste or mortar is removed from concrete, they do not have the same rheological properties as when in the concrete. In concrete, the larger particles interact with the smaller particles in mortar or paste and affect the shear rate experienced by the paste. The use of concrete rheological parameters instead of paste or mortar parameters is an indirect approach that increases the experimental variation and makes the identification of trends more difficult.

(The rheology of a separate set of mortar mixtures corresponding to the Phase II concrete mixtures was measured to evaluate the effects of mortar rheology on coarse aggregate segregation. The relationships between the rheology of these mortar mixtures and the concrete segregation data were not as clear as the relationships between the concrete rheological properties and the concrete segregation data. In addition to the limitations described in the previous paragraph, batch-to-batch variation reduced any relationships because mortar and concrete were mixed separately. The different batch sizes and mixing energies in the mortar and concrete batches likely increased the variation in properties between mortar and concrete.)

The use of concrete rheology measurements is further complicated because the static and dynamic yield stress measurements may not fully reflect low shear rate behavior, which is most critical to segregation. Mixtures with shear-thinning behavior at low shear rates may have yield stresses lower than those estimated by flow curve measurements or stress growth measurements.

7.2.3.2 Materials and Mixture Proportions

With the rheological properties necessary to achieve segregation resistance established, it is next necessary to determine how concrete should be proportioned to achieve these rheological properties.

First, a multivariate regression analysis was conducted on the Phase II data to relate mixture proportions to the rheological parameters relevant to segregation resistance. The results, which are given in Table 7.5, are in terms of paste volume, water/powder ratio, and sand-aggregate ratio. Figure 7.14 indicates that reducing the paste volume and w/p and increasing the S/A increase HRWRA demand for a 29-inch slump flow. These results are consistent with the findings in Chapter 4. The HRWRA demand is important in the context of segregation because mixtures with high HRWRA dosage tend to have long workability retention (Assaad and Khayat 2006). Indeed, Figure 7.15 indicates that increased HRWRA dosage was associated with higher slump flows after 15 minutes. The higher HRWRA dosage ensures that more HRWRA is initially in suspension and able to adsorb on cement particles over time, resulting in increased workability retention (Chapter 2).

Figure 7.16 indicates that w/p has the biggest effect on plastic viscosity, with increased w/p resulting in reduced plastic viscosity. In addition, increases in paste volume and reductions in S/A are associated with reduced plastic viscosity. These results are consistent with the findings in Chapter 4 for T_{50} , which is correlated to plastic viscosity.

Figure 7.17 indicates that thixotropy, as measured by breakdown area, increases with reduced paste volume and w/p both initially and after 15 minutes. Reducing the w/p and paste

volume result in reduced spacing between powder particles, which increases the chance of powder particles contacting and flocculating.

Figure 7.18 indicates that increasing the paste volume and w/p increase the dynamic yield stress initially and after 15 minutes. This trend may seem counterintuitive; however, it must be noted that a constant slump flow was maintained in all mixtures. Mixtures with higher viscosity generally are associated with lower dynamic yield stresses for a given slump flow. In addition, mixtures with low paste volume and w/p required higher HRWRA dosages (Figure 7.14), which is associated with increased workability retention (Figure 7.15), which further explains the relationships between dynamic yield stress at 15 minutes and paste volume and w/p. In contrast, decreases in paste volume and w/p are associated with increased static yield stress after 15 minutes. In the case of static yield stress, the build-up of an at rest structure is more significant than the loss of workability. Despite the build-up of an at-rest structure over time, the near-zero dynamic yield stresses for highly viscous mixtures with high HRWRA dosages is potentially adverse.

Figure 7.19 further shows the effect of w/p on rheological parameters with time. Mixtures with low w/p required higher HRWRA dosages and exhibited near zero dynamic yield stress for the entire testing period. Although this trend reflected improved workability retention, the near zero dynamic yield stress increased the susceptibility to segregation. Mixtures with higher w/p had higher initial dynamic yield stresses and faster increases in dynamic yield stress with time due to the loss of workability retention. Increasing the w/p reduced the rate of static yield stress development. The initial measurement of static yield stresses for the mixture with w/p of 0.24 and 0.30 were near that of the mixture with w/p of 0.36. In theory, the static and dynamic yield stresses should be equal immediately after mixing—as indicated with the dashed lines in Figure 7.19. Because it was not possible to start the rheology test immediately after mixing, it was not possible to capture the static yield stress until approximately 20-30 seconds, at which time the static yield stress had already increased. The initial amount and rate of increase in thixotropy increased with reduced w/p. Therefore, the increase in static yield stress was dominated by increases in thixotropy for low w/p mixtures and loss of workability for high w/p mixtures. As expected, plastic viscosity increased with decreasing w/p. The plastic viscosity remained nearly constant for high w/p but increased over time for low w/p.

Figure 7.20 indicates the changes in rheological parameters with time. The parameters initially and at 15 minutes were well correlated for dynamic yield stress and plastic viscosity. Mixtures with near-zero initial dynamic yield stresses tended to retain low dynamic yield stresses whereas mixtures with higher initial dynamic yield stresses tended to lose dynamic yield stress more quickly. Of the four parameters in Figure 7.20, plastic viscosity was the least affected by HRWRA efficacy at any time; therefore, the change in plastic viscosity with time was the smallest. The change in static yield stress with time exhibited the most scatter because its change with time is affected by thixotropy and workability loss. Lastly, thixotropy at 0 and 15 minutes were correlated. Part of the scatter in this relationship was likely due to the reduced precision in measuring low initial thixotropy values. The relationships in Figure 7.20 were established with the same HRWRA and cementitious materials and would likely be different if compared to mixtures with different admixtures or cementitious materials.

Table 7.5 Multivariate Regression Models for Phase II Laboratory Data

Equation	R ²
$HRWRA = 1/[-0.0784 + 3.112(V_p)(w/p) - 0.945(S/A)(w/p)]$	0.96
Dynamic yield stress, 0 min (Pa) = $[-9.496 + 122.85(V_p)(w/p)]^2$	0.60
Static yield stress, 0 min (Pa) = not statistically significant	--
Plastic viscosity, 0 min (Pa.s) = $\exp[6.293 - 45.39(V_p)(w/p) + 8.419(V_p)(S/A)]$	0.94
Thixotropy, 0 min (Nm/s) = $0.330 - 2.779(V_p)(w/p)$	0.72
Dynamic yield stress, 15 min (Pa) = $[-14.54 + 185.55(V_p)(w/p)]^2$	0.78
Static yield stress, 15 min (Pa) = $1/[-0.00691 + 0.103(V_p)(w/p)]$	0.72
Plastic viscosity, 15 min (Pa.s) = $[14.72 - 136.30(V_p)(w/p) + 33.26(V_p)(S/A)]^2$	0.93
Thixotropy, 15 min (Nm/s) = $[1.788 - 12.93(V_p)(w/p)]^2$	0.72
Regression details: full quadratic model; stepwise regression procedure with p-value = 0.05; transformations of dependent variables considered: y, 1/y, ln(y), sqrt(y); transformation with highest R ² selected; independent variables considered: paste volume (V _p), water-powder ratio (w/p), sand-aggregate ratio (S/A)	

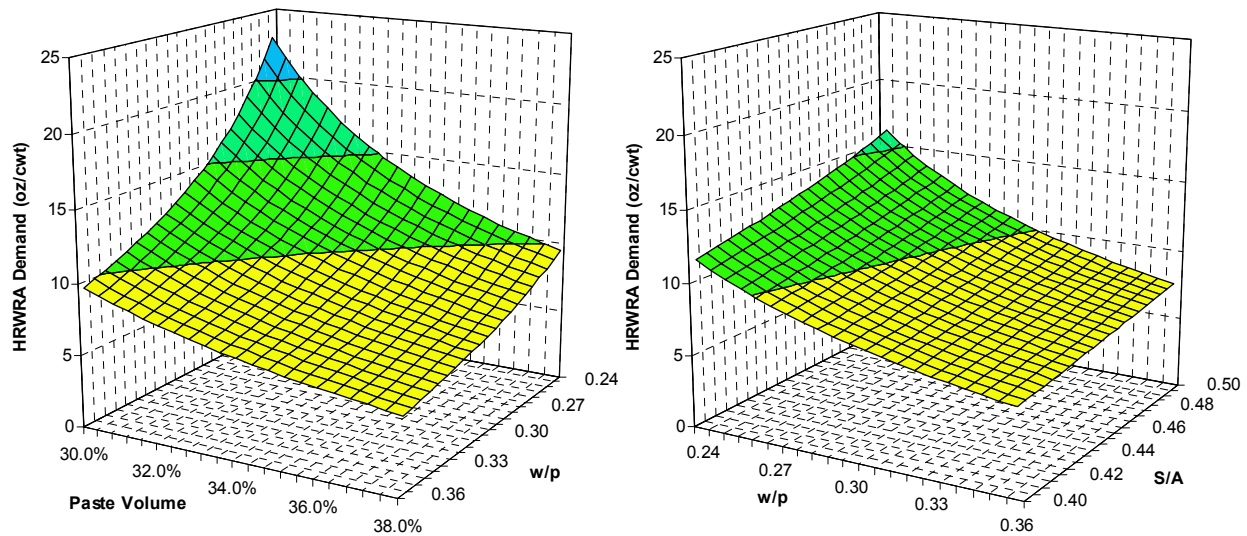


Figure 7.14 Effects of Mixture Proportions on HRWRA Demand for 29-inch Slump Flow (Multivariate Regression Analysis)

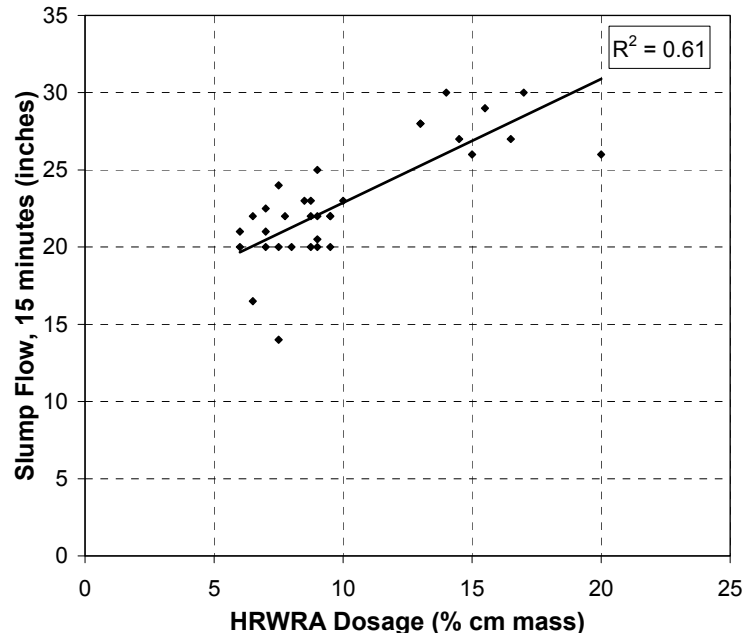


Figure 7.15 Effect of HRWRA Dosage on Slump Flow after 15 Minutes

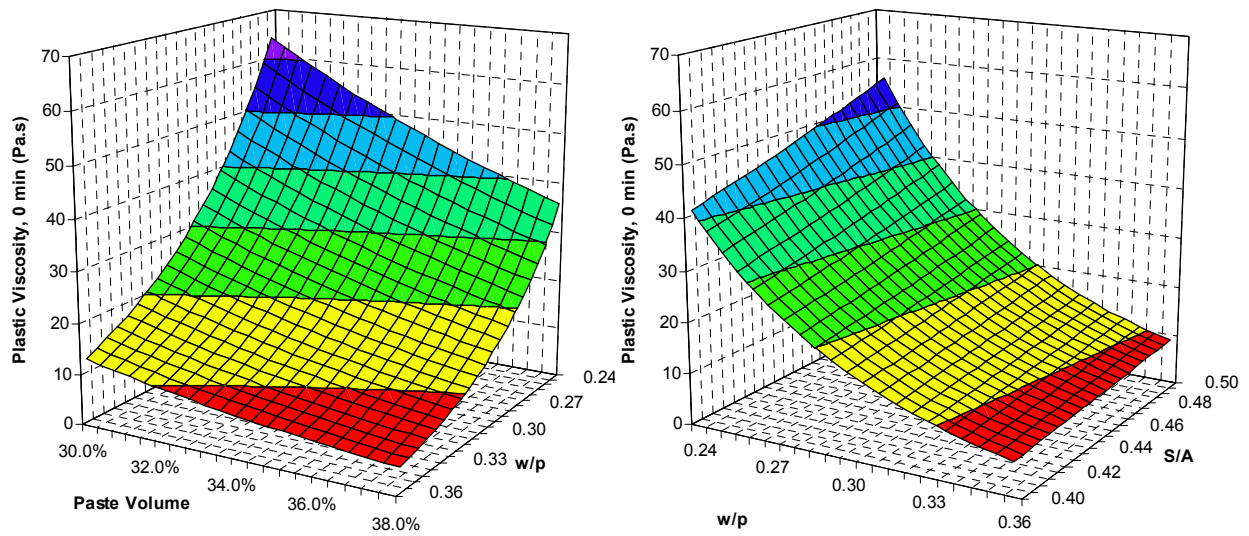


Figure 7.16 Effects of Mixture Proportions on Plastic Viscosity (Multivariate Regression Analysis)

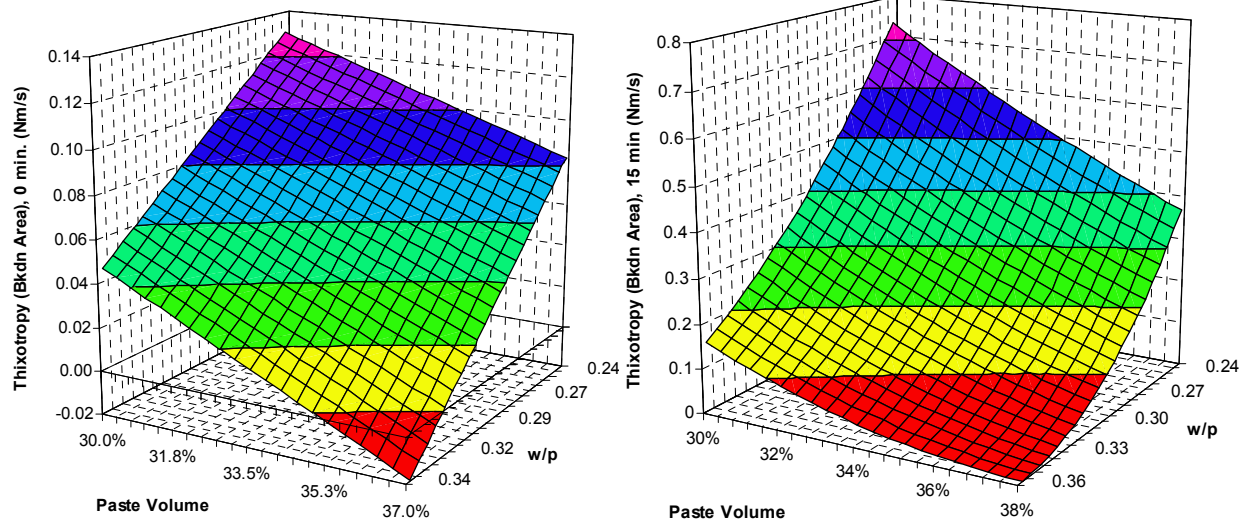


Figure 7.17 Effects of Mixture Proportions on Thixotropy (Multivariate Regression Analysis)

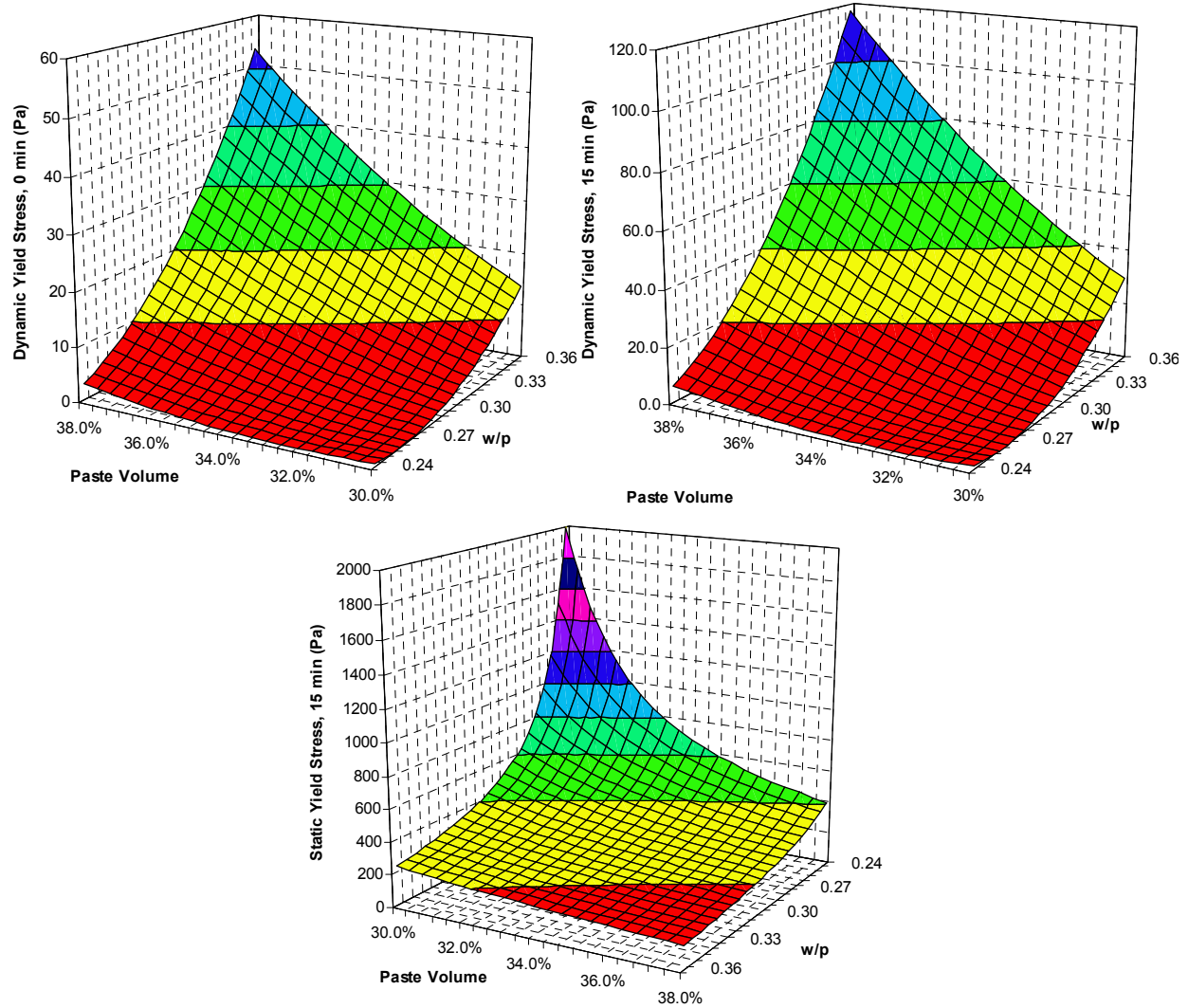
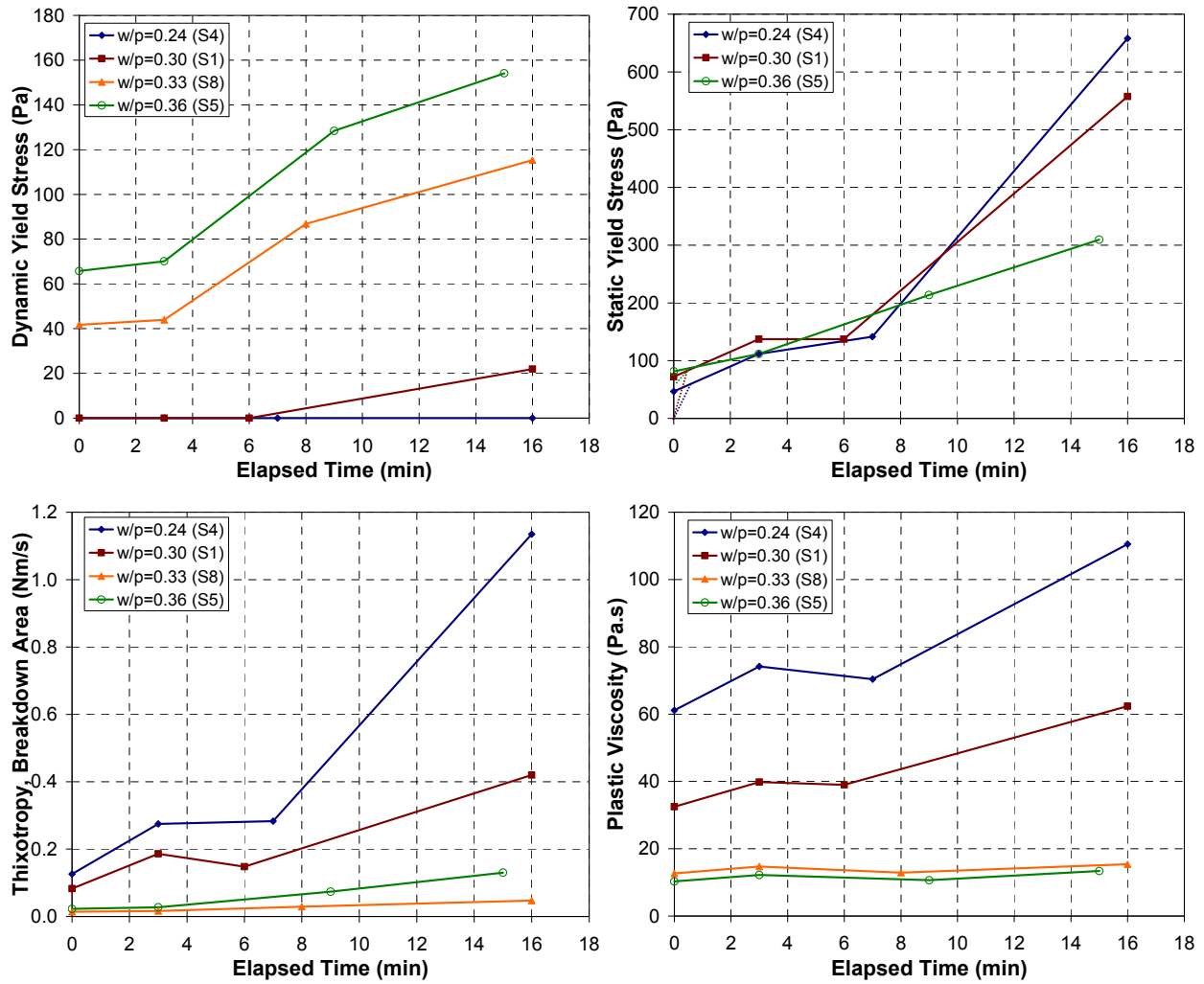


Figure 7.18 Effects of Mixture Proportions on Yield Stress (Multivariate Regression Analysis)



Column Segregation Test Results: S4: 68.9%, S1: 0.9%, S8: 1.8%, S5: 0.0%

Figure 7.19 Effect of w/cm on Rheological Parameters with Time

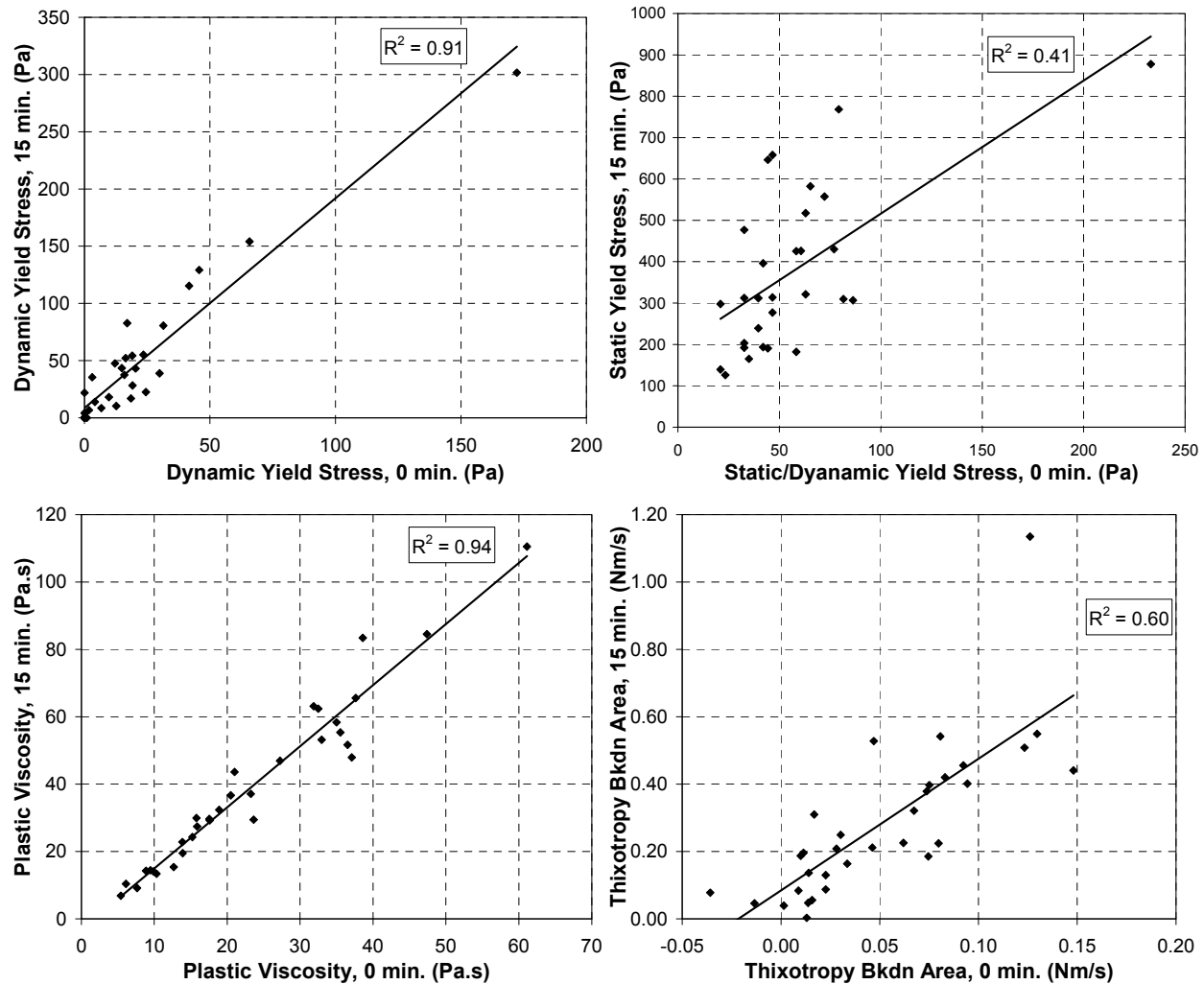


Figure 7.20 Change in Rheological Parameters with Time

The effect of VMA dosage was evaluated for a high viscosity and a low viscosity SCC mixture. The flow curves are shown in Figure 7.21 and the associated rheological parameters are shown in Figure 7.22 and Figure 7.23 for the high and low viscosity mixtures, respectively. For the high viscosity mixture, the dynamic yield stress was zero and remained zero regardless of the VMA dosage. The static yield stress; however, decreased with increased VMA dosage, especially after 15 minutes. This reduction was due to the decrease in thixotropy and not the loss of workability, as the dynamic yield stress remained zero. The use of VMA also decreased plastic viscosity.

The effect of VMA dosage was much different in the low viscosity mixture. The use of VMA increased the dynamic yield stress. This result was likely due to the shear thinning character associated with this particular VMA. Although a shear thinning character was not strongly evident in the flow curves, it would likely be present at lower shear rates. The use of VMA also increased thixotropy significantly, especially at 15 minutes. This increase in thixotropy, combined with the loss of workability, contributed to increases in static yield stress. In contrast, the use of VMA only increased plastic viscosity slightly.

Figure 7.24 indicates that the VMA was highly effective in reducing segregation in the low viscosity mixture but was ineffective in the high viscosity mixture.

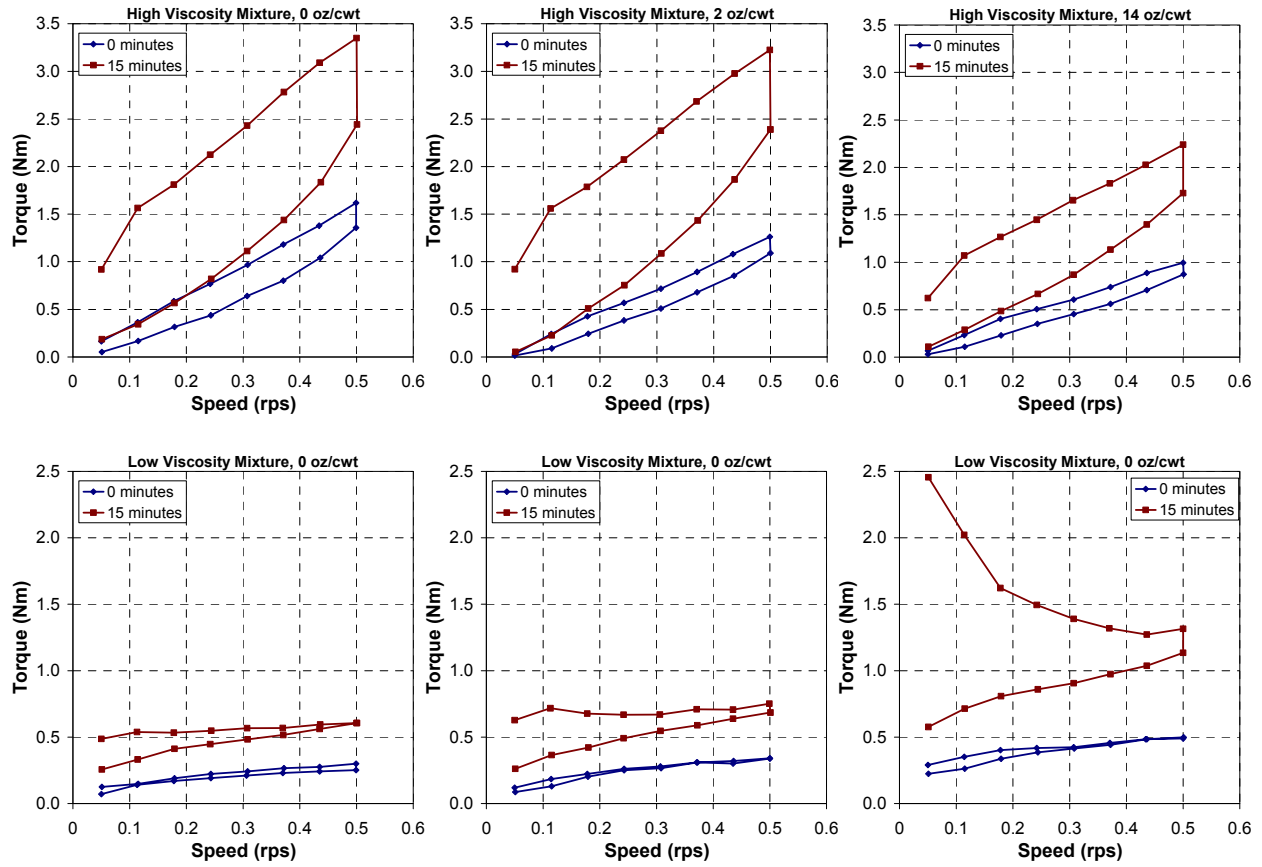


Figure 7.21 Effects of VMA Dosage on Rheology Flow Curves

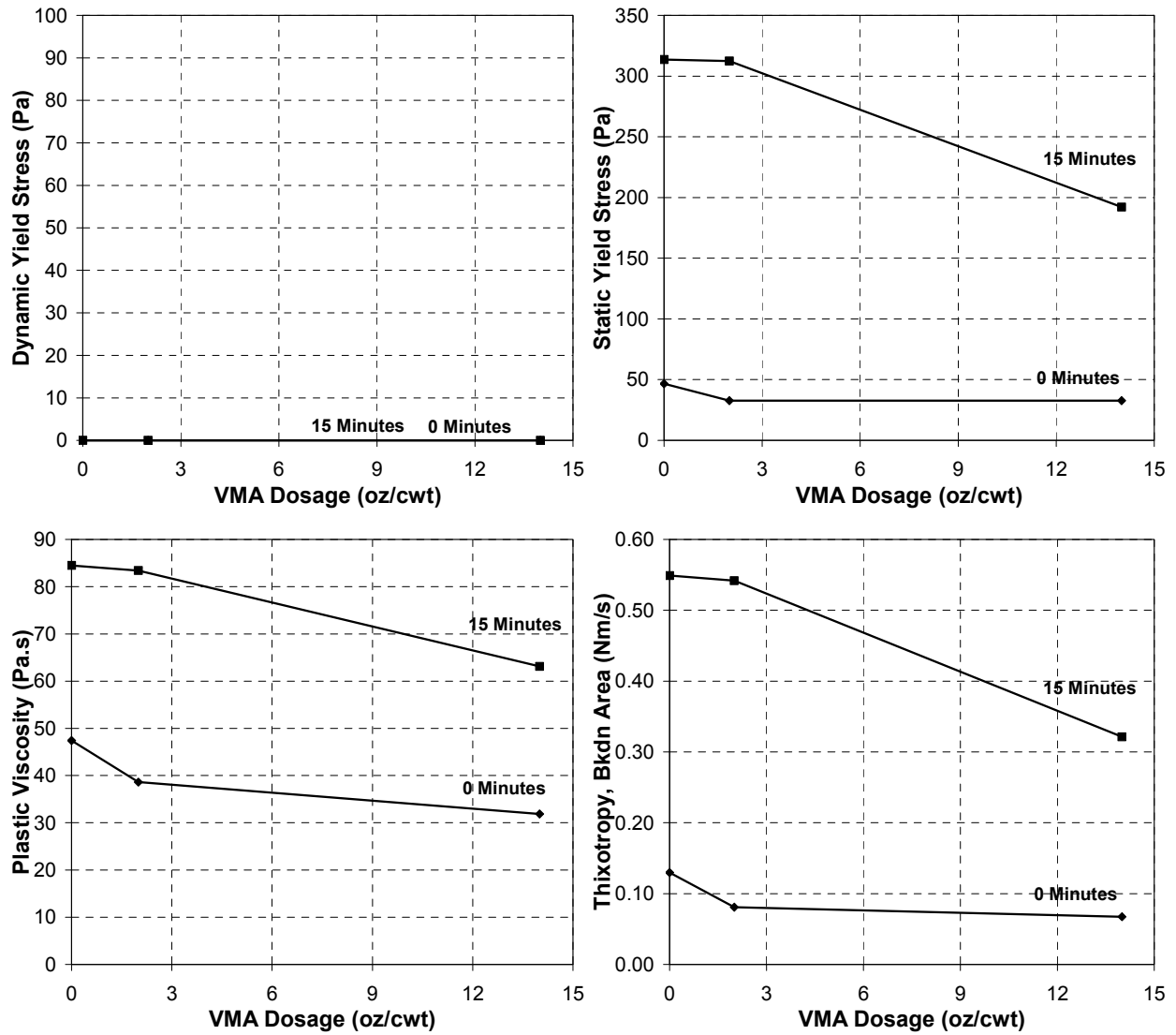


Figure 7.22 Effect of VMA Dosage on Rheological Parameters for High Viscosity Mixture

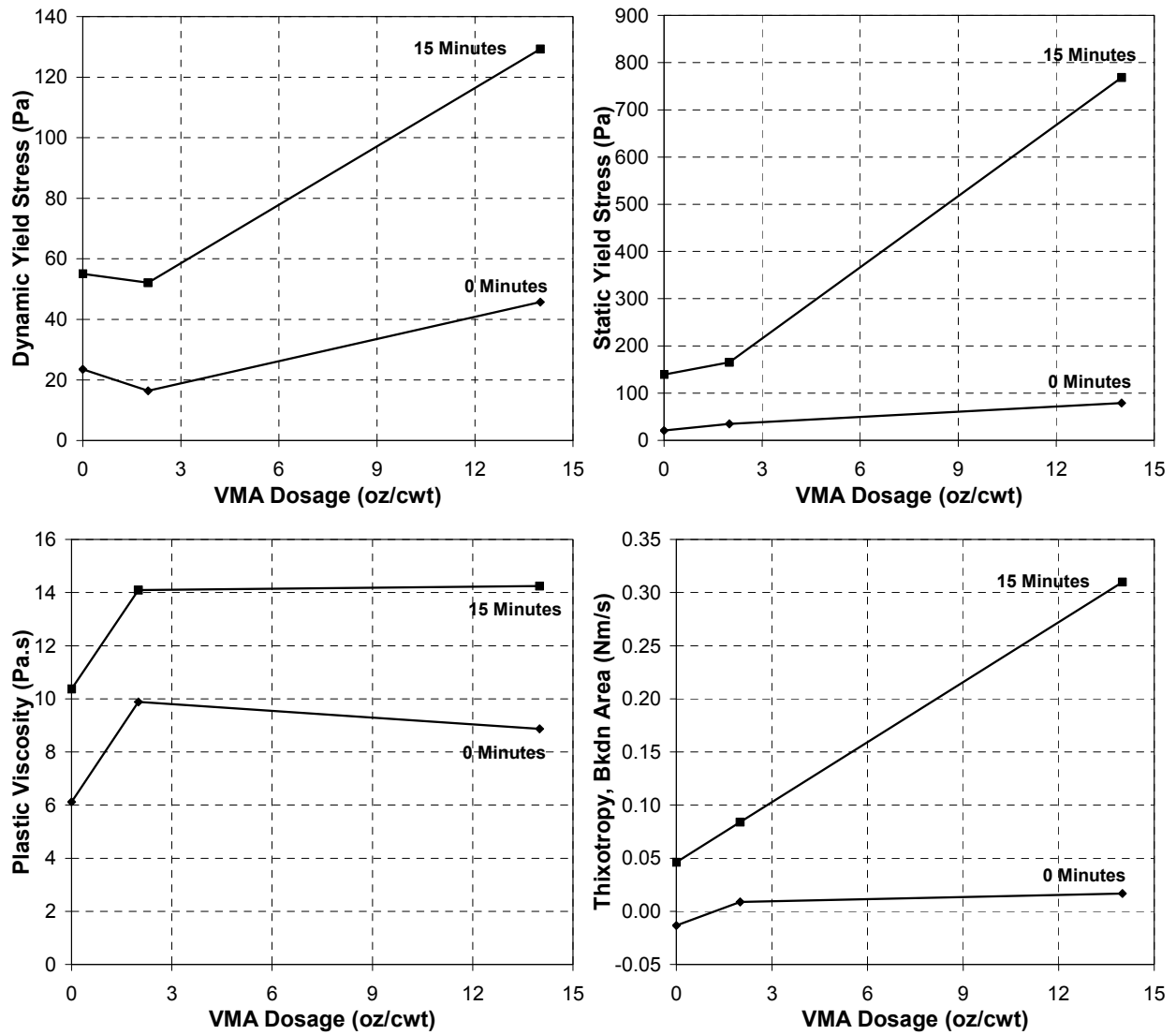


Figure 7.23 Effect of VMA Dosage on Rheological Parameters for Low Viscosity Mixture

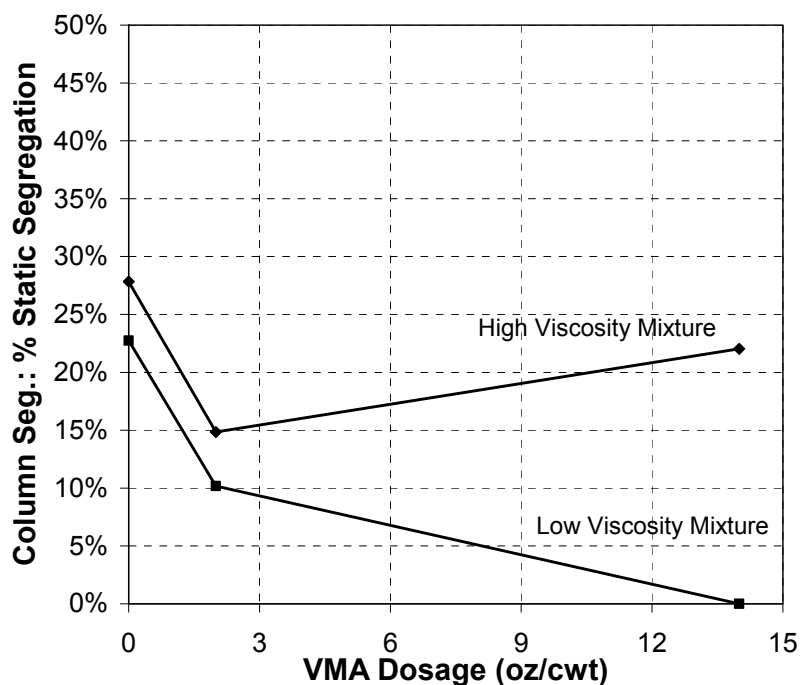


Figure 7.24 Effect of VMA Dosage on Column Segregation Test Results

Lastly, the effect of HRWRA dosage in a given concrete mixture was evaluated. HRWRA dosage is critical because HRWRA mainly affects yield stress, which is the most important parameter affecting segregation resistance. Figure 7.25 indicates that increasing the HRWRA dosage increased slump flow—which is inversely related to yield stress—and significantly increased segregation. (It was not possible to measure rheology in the mixture with the maximum HRWRA dosage due to the severe segregation.) At a low HRWRA dosage, the yield stress was high in the given mixture and no segregation occurred. As the HRWRA dosage was increased, the yield stress decreased and segregation increased. Therefore, in evaluating the effects of mixture proportions on segregation, HRWRA dosage—to the extent it affects yield stress—can outweigh all other considerations. Any mixture can be made to segregate by increasing the HRWRA dosage and decreasing the yield stress sufficiently. Segregation can be prevented by reducing the HRWRA dosage and increasing the yield stress sufficiently. However, it is important to have sufficiently low yield stress for self-flow.

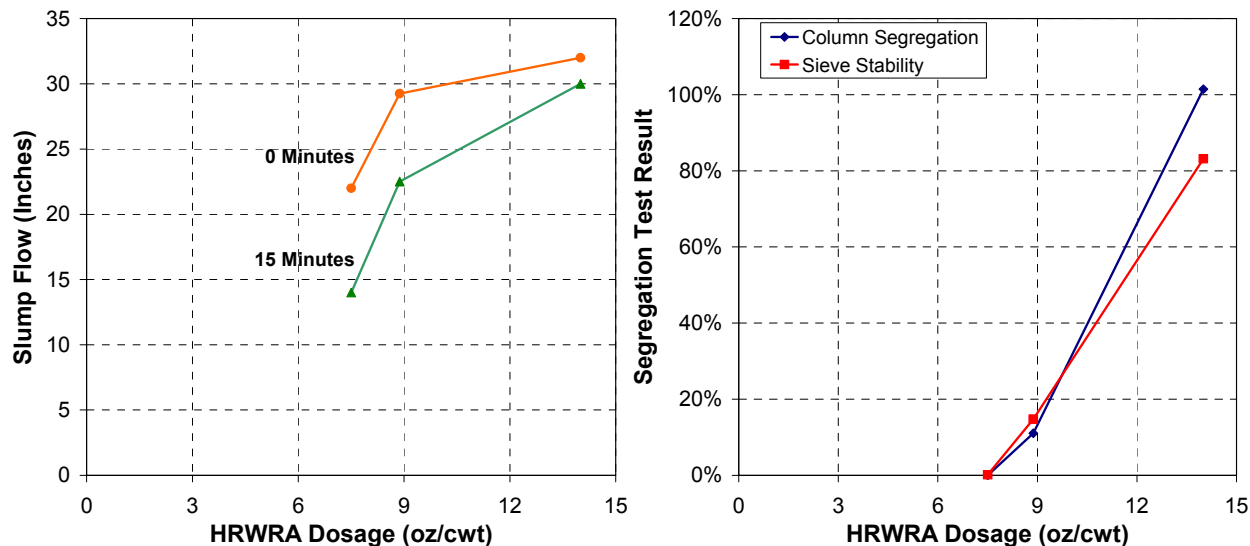


Figure 7.25 Effect of HRWRA Dosage on Slump Flow and Segregation Resistance

In proportioning SCC mixtures for segregation resistance, the maximum dynamic yield stress for self-flow should be balanced against the minimum static yield stress for segregation resistance. Based on the laboratory test results, the rheological parameters needed for segregation resistance can be established as follows:

- Paste volume and w/p.** The paste volume, w/p, or both should not be too high or too low. If the paste volume, w/p, or both are too high, the workability retention can be poor and the plastic viscosity too low. To maintain a constant slump flow, the yield stress is typically increased to compensate for the lower plastic viscosity. A low viscosity and yield stress can lead to severe segregation. Higher paste volume and w/p are also associated with reduced thixotropy; however, less thixotropy is needed when the yield stress is higher. In contrast, lower paste volume and w/p are associated with increased thixotropy and higher plastic viscosity; however, they also necessitate high HRWRA dosages, which results in extended workability retention. To maintain a constant slump flow, the yield stress is typically reduced to compensate for the higher plastic viscosity. The resulting low initial dynamic yield stresses may result in significant initial segregation, despite the higher thixotropy. In balancing the paste volume and w/p for self-flow, segregation resistance, and workability retention, it is likely that the plastic viscosity will be within a certain range (not too high or too low). Mixtures with low plastic viscosity likely reflect paste volumes and w/p that are too high. Mixtures with high plastic viscosity likely reflect paste volumes and w/p that are too low. That said, a certain range of plastic viscosity does not assure segregation resistance.
- Aggregate Grading.** The effect of S/A was minimal in this research. Aggregate grading was not evaluated directly in this research beyond the effects of S/A. Although aggregate grading is well known to affect segregation, the variations in paste rheology are so significant in SCC that paste rheology is likely much more consequential than aggregate grading (provided aggregate grading is within a reasonable range).
- VMA dosage.** The effects of VMA on concrete rheology can vary widely depending on the VMA properties and the mixture proportions. The VMA used in this research was

effective for the low viscosity mixture because it increased static yield stress—due to separate mechanisms of shear thinning and thixotropy—and did not affect plastic viscosity.

- **HRWRA dosage.** The HRWRA dosage is critical because it mainly affects the yield stress. In any given mixture, the HRWRA dosage can be reduced to reduce yield stress and segregation; however, the HRWRA dosage must be sufficient to ensure self-flow.

7.2.4 Test Results: Evaluation of Test Methods

The segregation test methods were evaluated in terms of their relationships to field conditions and to relevant rheological parameters and in terms of their suitability for use in the laboratory and field. The hardened concrete test was conducted to serve as an absolute standard for comparing the other test methods because it best represents field conditions. However, it is impractical for routine use in the laboratory or field.

7.2.4.1 Column Segregation Test

Discussion of Test

The column segregation test provides an independent measurement of segregation resistance by replicating static conditions in formwork and quantifying the segregation of coarse aggregate after a fixed time.

The results of the column segregation test are well correlated to those of the hardened column test (Figure 7.26) and the sieve stability test (Figure 7.27). For 7 of the 9 mixtures, the segregation measured in the hardened concrete column test was equal to or slightly greater than that measured in the column segregation test, which suggests that most but not all segregation occurs within the 15-minute duration of the column segregation test. The column segregation test is simpler to perform than the hardened concrete test and appears to be an appropriate simplification. A 15% static segregation in the column segregation test corresponds to a 15% reading from the sieve stability test; which was found to be appropriate by the European Testing SCC project (de Schutter 2005, Testing-SCC 2005). The ASTM standard for the column segregation test does not provide guidance on selecting a maximum test result for segregation resistance.

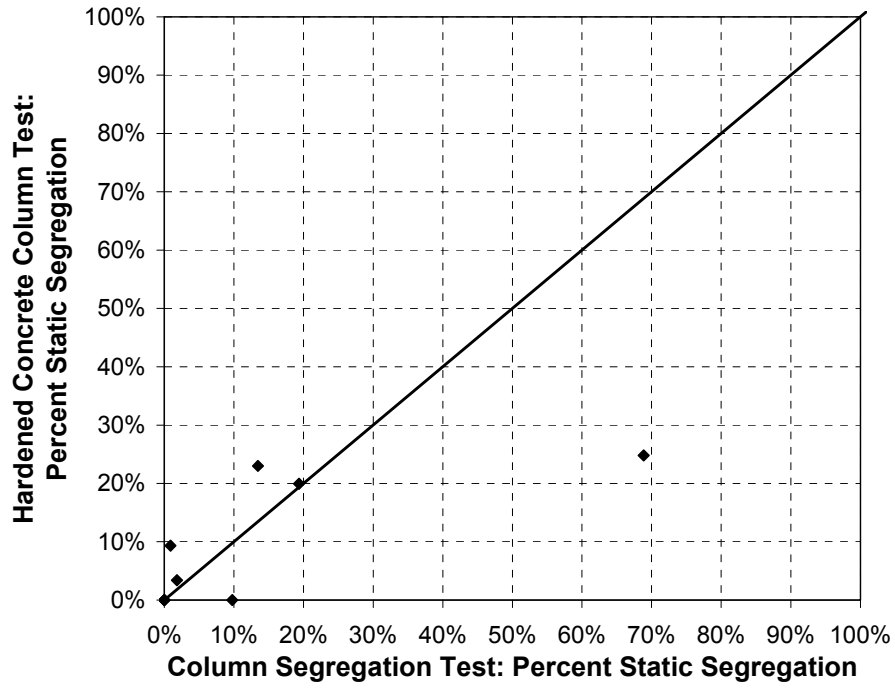


Figure 7.26 Relationship between Hardened Concrete Column Test and Column Segregation Test

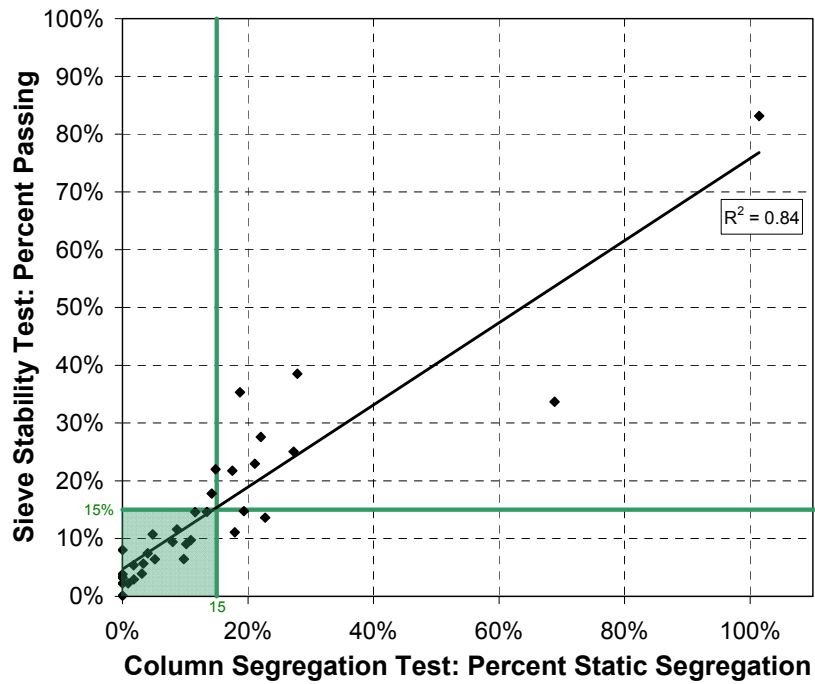


Figure 7.27 Relationship between Column Segregation Test and Sieve Stability Test

There are variations in the test apparatus, test procedure, and measurement of results that are important to interpreting results consistently. The test apparatus typically consists of four 6.5-inch long, 8-inch diameter pipe sections. The 8-inch diameter is representative of most field applications—reducing the diameter would likely reduce the amount of segregation recorded. The total height of 26 inches is adequate for measuring a significant difference in coarse aggregate mass between the top and bottom of the column. Other sizes of cylinders—varying from 4 by 8-inch cylinders to much larger columns—have been used to measure static segregation, though usually not by removing coarse aggregate in the same fashion as in the column segregation test. The version of the test evaluated by the European Testing SCC project featured a rectangular cross section and was attached to a drop table to accelerate segregation.

The test procedure mainly differs in when the coarse aggregate variation is measured. ASTM C 1621 requires the concrete to be left undisturbed for 15 minutes, which should be adequate for most cases. In other cases, concrete is allowed to harden and is then cut open to quantify the distribution of coarse aggregate. The ASTM C 1621 procedure allows aggregate mass to be determined when the aggregates are in saturated-surface dry condition, which enables the test to be completed sooner but may increase the variability of test results.

Several different ways of calculating test results have been reported. Results have been computed as a function of the relative amount of aggregate in just the top and bottom sections or in all four sections. The use of only the top and bottom sections is the preferred approach because it requires less work and the relative difference in aggregate mass in the middle two sections is likely to be low in most cases. A variety of ratios of aggregate mass in the top and bottom sections have been used; however, one is not clearly better than the others.

In performing the column segregation test, proper sampling is crucial. Concrete should not be segregated when it is first put into the column. Therefore, the source of the concrete—such as a wheelbarrow—should not be segregated and the act of filling the column should not cause segregation. Any dynamic segregation occurring during the loading of the concrete can influence test results. Because paste rheology strongly influences the degree of static segregation, the rheology of the concrete at the anticipated time of placement in the field should be considered. For instance, a laboratory-mixed concrete that is tested immediately after mixing may not be similar to the same mixture that is mixed in a truck, transported for 30 minutes, and then pumped to its final location. Mixtures with workability retention beyond the time of placement are more likely to segregate over time because the yield stress and plastic viscosity remain low for a longer time.

The column segregation test is difficult and time-consuming to perform. Therefore, the test is most appropriate for use in the laboratory for developing and pre-qualifying mixture proportions. The most difficult aspect of the test procedure is the removal of concrete from the pipe sections. Various collector plates have been developed; however, all require at least two people and do not adequately minimize the potential for spilling concrete. The test takes at least 30 minutes to perform—including filling the column, allowing the concrete to remain undisturbed for 15 minutes, collecting the concrete from the column, washing and sieving the aggregate, and drying the aggregate to its saturated surface-dry condition. If the aggregate is oven-dried, results are not available for at least several more hours. The need for a balance to determine aggregate mass makes the test further impractical for use in the field.

Advantages and Disadvantages

The advantages of the column segregation test include:

- The test provides an independent measurement of static stability.
- The test conditions generally reflect field conditions.

The disadvantages of the column segregation test include:

- The test does not measure dynamic stability.
- The test is difficult and time-consuming to perform and requires the use of a balance. Therefore, it is unsuitable for field use.
- Errors in sampling can influence test results significantly.

Recommendations

Either the column segregation test or the sieve stability test should be used to measure static segregation resistance. The results of the two tests are well-correlated; however, the sieve stability test is easier to perform. In performing the column segregation test, the procedure described in ASTM C 1621 is suitable. The test is not appropriate as a rapid field acceptance test. When using the test in the laboratory to qualify mixture proportions, mixtures should be prepared with the range of water contents and slump flows expected during production. If these mixtures exhibit adequate segregation resistance and the slump flow test is used in the field to control concrete rheology indirectly, it is not necessary to use the column segregation test in the field.

7.2.4.2 Penetration Apparatus Test

Discussion of Test

The penetration apparatus test is a rapid field test for segregation resistance. It was first proposed by Bui (Bui, Akkaya, and Shah 2002; Bui et al. 2002). Variations on the penetration concept—with different penetration heads, concrete specimen sizes, and time sequences—have been proposed. The European Testing SCC project found the sieve stability test to be preferable to the penetration apparatus test for measuring segregation resistance.

The test provides an independent measurement of static stability and does not provide an indication of dynamic stability. The test essentially measures the static yield stress. In fact, a similar penetration test for measuring yield stress was used successfully by Uhlherr et al. (2002) for Carbopol gels and TiO₂ suspensions. The yield stress to stop the descent of the penetration head can be calculated based on the difference between the buoyant force acting upward and the gravitational force acting downward divided by the surface area of the bottom and sides of the cylinder, as shown in Equation (7.9):

$$\tau = \frac{m_{head} - \rho \pi d (r_o^2 - r_i^2)}{\pi (r_o^2 - r_i^2) + 2 \pi d (r_i + r_o)} 9.81 \quad (7.9)$$

where τ is the stress to stop penetration head at given depth (Pa), m_{head} is the mass of the head (kg), ρ is the density of the concrete (kg/m³), d is the penetration depth (m), and r_o and r_i are

the outer and inner radii of the penetration head (m). The penetration apparatus test measures the static yield stress—as opposed to the dynamic yield stress—because the shear imposed by the descent of the head is minimal, resulting in negligible breakdown of any built-up in structure due to thixotropy (much like the descent of an aggregate).

The relationship between the penetration depth and the calculated and measured yield stresses are shown in Figure 7.28. The penetration apparatus tended to overestimate the dynamic yield stress and underestimate the static yield stress. The application of Equation (7.9) to the penetration apparatus test is complicated because of the heterogeneous nature of concrete, wall effects from the container, the extent of friction between the concrete and penetration head, and the rapid change in static yield stress in the first few minutes after mixing.

The heterogeneous nature of concrete complicates the penetration apparatus measurement because aggregates must be displaced in order for the penetration head to descend. If resistance to this displacement is provided from specimen boundaries, the stress required for penetration should increase. The smaller the container, the greater these wall effects will be. In addition, the penetration apparatus test may be further affected by a lack of aggregate particles near the top surface caused by any segregation prior to and during the descent of the penetration head. The friction between the concrete and the penetration head may vary for different concretes and different penetration head materials. The rapid change in static yield stress makes the timing of the test critical—any variation in timing can affect the penetration depth significantly.

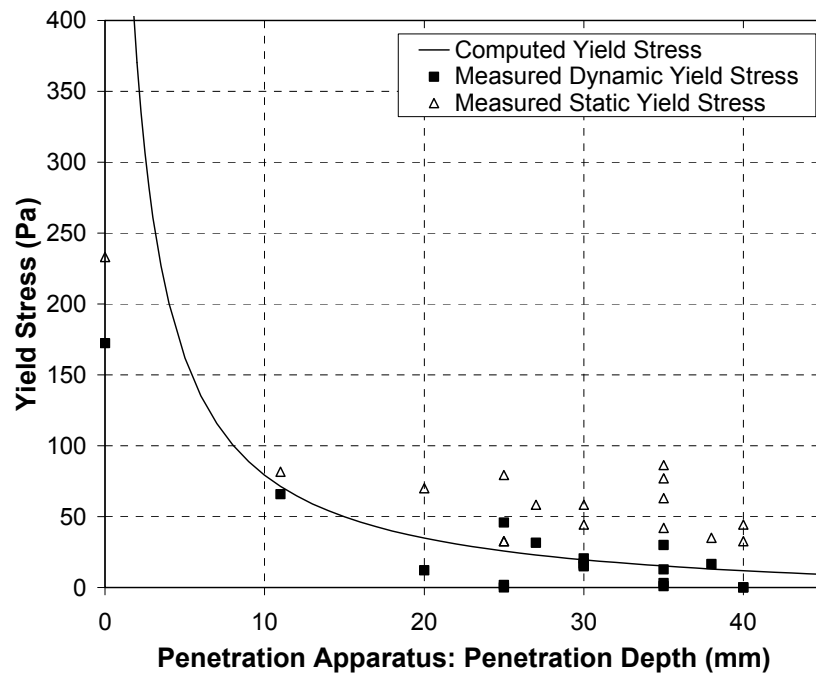


Figure 7.28 Relationship between Penetration Apparatus Depth and Yield Stress (Initial Measurements)

The results of the penetration apparatus test were not well correlated to other segregation test methods. As shown in Figure 7.29, there was poor correlation between the penetration apparatus test measured initially and after 15 minutes and both the column segregation and sieve stability tests. For tests conducted initially, mixtures with penetration apparatus measurements

below 20 mm exhibited no segregation; however, the penetration apparatus test gave false negatives as some mixtures exhibited the maximum penetration depth but no segregation. Although the test can assure segregation resistance for mixtures with low penetration apparatus measurements, it should be improved to eliminate false positives. The correlation between the penetration apparatus test and segregation resistance was better at 15 minutes; however, some mixtures exhibited unacceptably high segregation and zero penetration depth. In contrast, El-Chabib and Nehdi (2006) found good correlation between modified versions of the penetration apparatus test and column segregation test while Cussigh, Sonebi, and De Schutter (2003) found good correlation between the penetration apparatus test and the sieve stability test.

The inability of the penetration apparatus test to predict segregation resistance was likely due to the timing of the test procedure. It may be possible to improve the results and eliminate the false negatives by changing the test procedure. The timing of the test procedure is critical because the static yield stress can change rapidly in the first few minutes after mixing and is strongly dependent on the concrete shear history. The test is inherently limited, however, because it measures only one point in time whereas segregation resistance depends on the magnitude of the static yield stress over time. For instance, a low initial static yield stress may be acceptable if the rate of build-up in static yield stress is sufficient. In contrast, a mixture with a higher initial static yield stress but lower rate of increase in static yield stress may be unacceptable. A low initial yield stress does not correspond to segregation if thixotropy, loss of workability, or both increase the static yield stress quickly.

The shear history, which directly influences the magnitude of the static yield stress, can vary due to the shear history of the concrete prior to filling the cone, the extent of shearing during the filling of the cone, and the elapsed time from filling the cone until the penetration head is released. If the test is performed immediately after mixing or intense agitation, the test essentially measures the dynamic yield stress. If too much time transpires and the static yield stress increases too much, the penetration depth may be zero or near zero. In such a case, the penetration apparatus will not indicate the initial static yield stress and rate of build-up in static yield stress. The shear history prior to filling the cone should be similar to the shear history experienced in the field. Concrete that is tested in the lab immediately after mixing has a different shear history and likely different rheological properties than concrete in the field.

Therefore, in selecting a test protocol, the penetration apparatus should not be released too early or too late. The mixtures with false positives in Figure 7.29 did not stabilize within one minute for the penetration apparatus test. They did, however, stabilize sufficiently quickly to prevent segregation. Therefore, concrete should be left in the cone more than one minute prior to release of the penetration head. However, leaving concrete in the slump cone for an extended period of time can affect the slump flow measurements due to thixotropy and segregation. In addition, the shear history prior to testing must be well-defined.

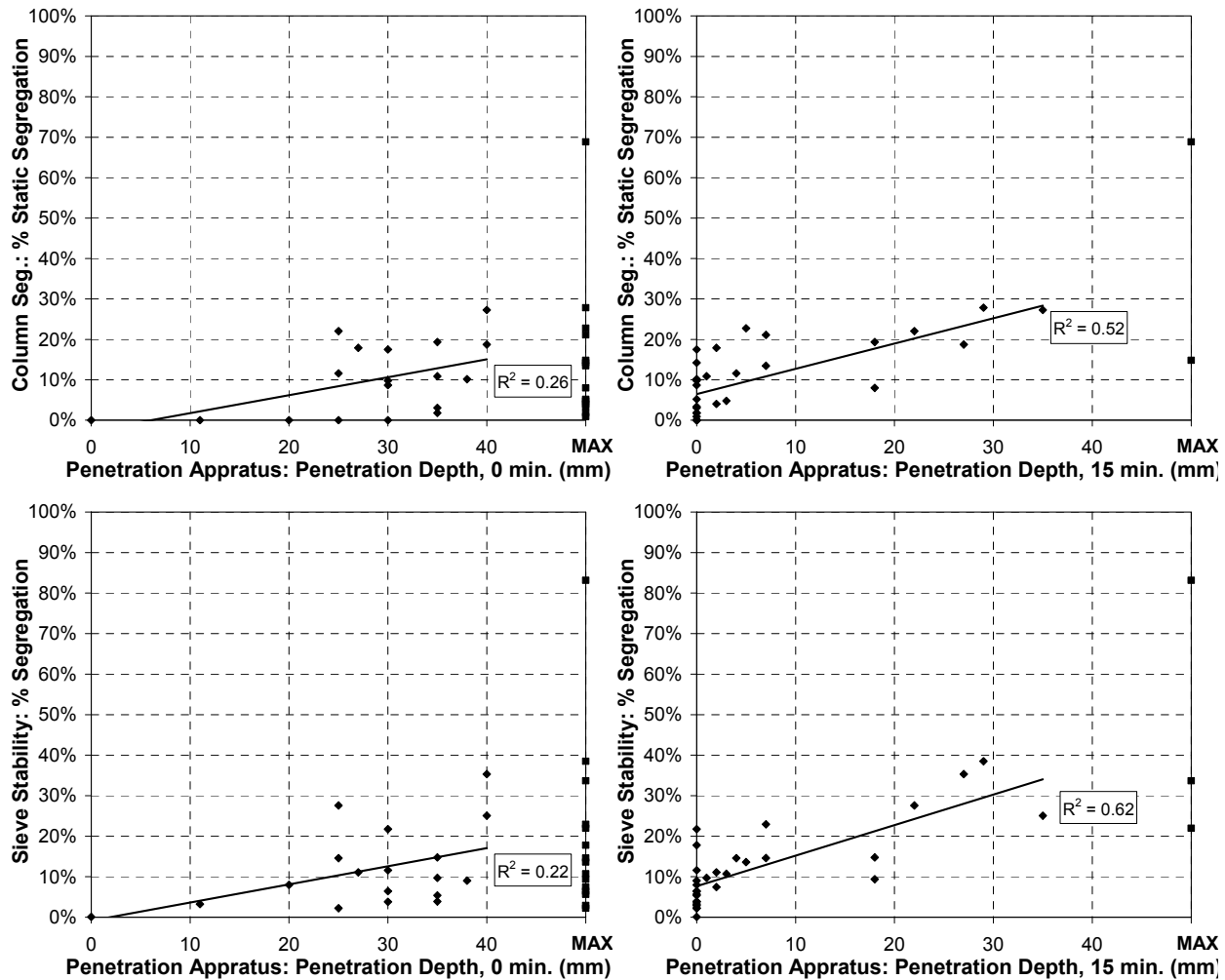


Figure 7.29 Relationships between Test and Penetration Apparatus Test at 0 and 15 Minutes and the Column Segregation Test and Sieve Stability Test

There are further variations in the test apparatus and test procedure that are important to interpreting results consistently. The test apparatus can vary significantly. The dimensions and mass of the penetration head can affect results significantly. The pressure exerted on the concrete, which is a function of the mass, diameter, and thickness of the penetration head—must be carefully matched to the range of yield stresses to be measured. If the pressure is too low, the penetration depth can be too low for reliable measurements. If the pressure is too high, the penetration depth can exceed the height of the penetration head. The pressure may need to be varied depending on the density of the aggregate relative to the paste. The concrete specimen size is also important. Short containers can limit the total amount of segregation that can occur. In narrow containers, the confinement and frictional resistance from the wall surface can reduce the amount of segregation. The use of a slump cone to contain concrete is a reasonable size and is a practical approach to conducting the test.

Advantages and Disadvantages

The advantages of the penetration apparatus test include:

- The test is fast, simple, and easy to perform, such that it could be used as a rapid field acceptance test.
- If used with the slump cone, the specimen size is small.

The disadvantages of the penetration apparatus test include:

- The test results are highly dependent on the amount of time the concrete remains at rest prior to releasing the cylinder.
- The test measures only one point in time, whereas segregation resistance depends on the magnitude of the static yield stress with time.
- The test does not measure dynamic segregation.

Recommendations

The penetration apparatus test may be suitable for measuring static segregation if it is better developed. The time sequence for performing the test must be carefully selected. In addition, the dimensions and mass of the penetration head and the concrete specimen size must be well defined. In the testing considered here, the time after mixing and the time for the concrete to remain in the slump cone were too short.

7.2.4.3 Sieve Stability Test

Discussion of Test

The sieve stability test for segregation resistance has been used mainly in Europe and was recommended by the European Testing SCC project for use as the reference test method for segregation resistance. The sieve stability test measures static segregation and—to some extent—dynamic segregation. When concrete is left undisturbed in the bucket for 15 minutes, any segregation or bleeding that occurs is due to static segregation. Segregation of coarse aggregate and bleeding lead to more mortar and paste at the top of the specimen, which is then poured onto and passes through a sieve. The amount of mortar passing the sieve depends to some extent on dynamic segregation resistance because viscous, cohesive mortar is less likely to pass through the sieve. Since this evaluation of dynamic segregation is determined after the concrete has remained undisturbed for 15 minutes, it may not reflect the dynamic segregation resistance of the concrete during placement conditions where the concrete is sheared continuously. It is likely that dropping the concrete onto the sieve does not fully breakdown the effects of thixotropy. The indication of dynamic segregation resistance would be more relevant to field conditions if done prior to the 15-minute rest period. If concrete were placed directly on the sieve, without the 15-minute period in the bucket, a much larger percentage of material would pass through the sieve because of the lack of thixotropic build-up.

As shown previously in Figure 7.27, the results of the sieve stability test and column segregation test were well correlated. In addition, Figure 7.30 shows that the relationship between the sieve stability test and hardened column test was acceptable.

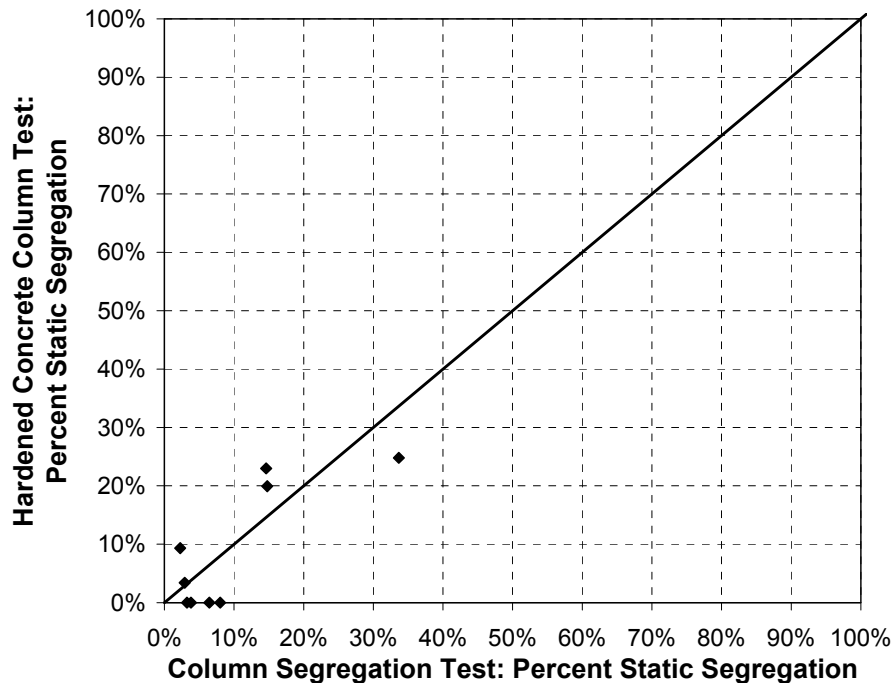


Figure 7.30 Relationship between Hardened Concrete Column Test and Sieve Stability Test

As with the column segregation test, the sampling of concrete is critical to the sieve stability test results. Further, concrete must be poured from the bucket in a consistent manner. The concrete in the bucket should not be agitated during pouring. If concrete is piled on one side of the sieve, less concrete may pass through the sieve than if the concrete is poured evenly over the sieve. The concrete must be allowed to remain on the sieve for a consistent time period. In some cases, it was found that material continued to pass the sieve after the standard two-minute period. Finally, the sieve must be removed carefully from the pan because any agitation could cause additional material to pass the sieve.

Although the sieve stability test is much simpler to perform than the column segregation test, it is not suitable for use as a rapid field acceptance test because of the amount of time required to perform the test and the need for a balance and a simple calculation. The test is simpler than the column segregation test because it does not require separating and cleaning the coarse aggregate. The test requires approximately 20 minutes to perform—including filling the bucket, waiting for the 15-minute rest period, pouring the concrete on the sieve and allowing it to remain there for 2 minutes, and measuring the final mass of material passing the sieve.

The European Testing SCC project preferred the sieve stability test over the column segregation test (rectangular cross section mounted on drop table) because the column segregation test is harder to perform and provides results that are no better than the sieve stability test.

Advantages and Disadvantages

The advantages of the sieve stability test include:

- The test provides an independent measurement of segregation resistance.
- The test is simpler to perform than the column segregation test.

The disadvantages of the sieve stability test include:

- The test conditions are not as directly representative of field conditions for static segregation as with the column segregation test.
- The test requires a balance.
- The test requires too much time for use as a rapid acceptance test in the field.
- The test does not fully measure dynamic stability.

Recommendations

Either the column segregation test or the sieve stability test should be used to measure static segregation resistance. The results of the two tests are well-correlated; however, the sieve stability test is easier to perform. The sieve stability test is not appropriate as a rapid field acceptance test. When using the test in the laboratory to qualify mixture proportions, mixtures should be prepared with the range of water contents and slump flows expected during production. If these mixtures exhibit adequate segregation resistance and the slump flow test is used in the field to control concrete rheology indirectly, it is not necessary to use the sieve stability test in the field. The possibility of measuring dynamic stability by dropping concrete onto the sieve without the 15-minute rest period should be evaluated further.

7.2.4.4 Visual Stability Index

Discussion of Test

The visual stability index provides an approximate visualization of concrete flow; however, it is not adequate to evaluate segregation resistance. The VSI does not reflect static segregation conditions in the field. Concrete mixtures may exhibit instability when observed for VSI determination but quickly improve when left undisturbed due to thixotropy. Conversely, mixtures that exhibit low VSI may exhibit gradual segregation that accumulates over time under static conditions but is not evident on the time scale of the slump flow test. The subjectivity of assigning the VSI also reduces the reliability of the index.

Indeed, Figure 7.31 shows the poor level of correlation between VSI and the column segregation and sieve stability tests. Some mixtures that exhibited a VSI of 0.0 exhibited segregation while other mixtures with a VSI of 2.0 exhibited minimal segregation. Elsewhere, Sedran and de Larrard (1999) found that the size of the mortar halo from the slump flow test was not correlated to the amount of segregation. Khayat (1999) and Khayat, Assaad, and Daczko (2004) also found the VSI inadequate for evaluating segregation resistance.

The mechanisms causing high VSI readings and poor static segregation resistance are similar, though different in several important aspects. The VSI reading mainly reflects the ability of the concrete to flow laterally. More specifically, it characterizes whether the paste exhibits adequate rheology to move aggregates to the periphery of the slump flow patty and to prevent a mortar halo and whether the concrete is susceptible to severe bleeding. The VSI is also problematic because it measures only one point in time. In some cases, a mixture may initially exhibit a poor VSI but improve very quickly; whereas, in other cases, the improvement may be slow. The VSI is better suited for characterizing dynamic stability because it is performed after partial or full breakdown of the thixotropic built-up structure and evaluates a short time duration. The subjective nature of the VSI determination further limits the precision of the test.

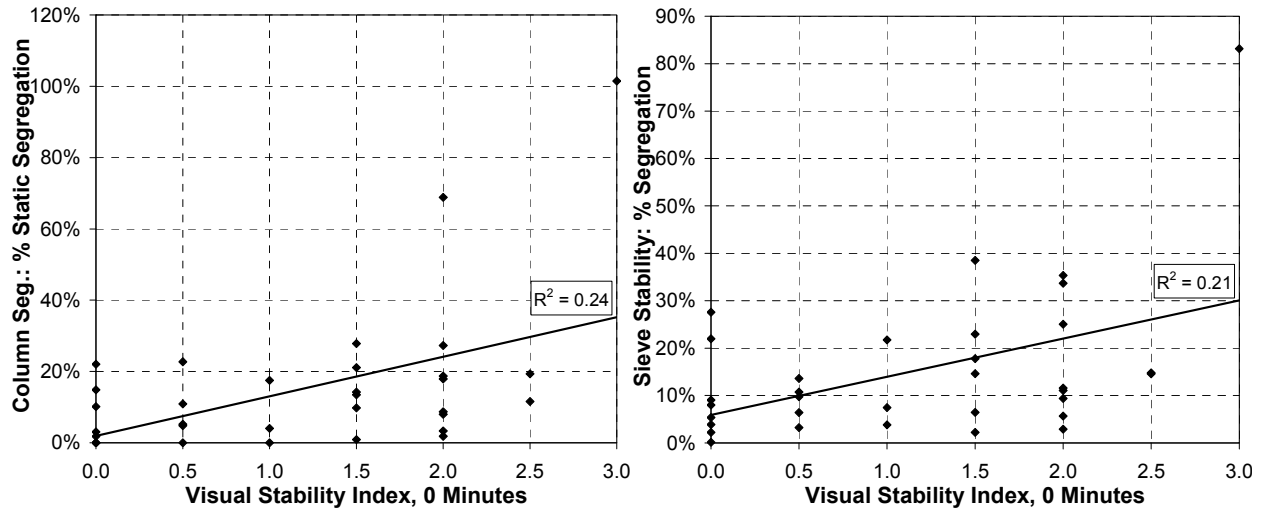


Figure 7.31 Relationship between Visual Stability Index and Column Segregation and Sieve Stability Tests

Advantages and Disadvantages

The advantages of the visual stability index include:

- The test is fast and simple and provides a visualization of concrete flow.
- The test may be able to identify significant segregation problems, especially inadequate paste volume and severe bleeding.

The disadvantages of the visual stability index include:

- The test is inadequate for characterizing stability. Mixtures with high VSI may exhibit good stability whereas mixtures with low VSI may exhibit poor stability.
- The short duration of the test may not capture segregation that occurs over a longer period of time.

Recommendations

The VSI should be recorded when performing the slump flow test to identify significant problems like inadequate paste volume and severe bleeding. If concrete exhibits a poor VSI, further investigation is warranted. Concrete with a good VSI should not be considered resistant to segregation.

7.3 Conclusions

The following conclusions can be drawn based on the testing presented in this chapter:

- The static yield stress with time is the most critical parameter for ensuring segregation resistance. In terms of rheology, yield stress is the main fundamental difference between the workability of SCC and conventionally placed concrete. The static yield stress must

be sufficiently high to prevent segregation while the dynamic yield stress must be sufficiently low for self-flow. The static yield stress at any time is a function of the starting yield stress, the loss of workability, and thixotropy. Higher plastic viscosity can slow the descent of particles; however, the magnitude of plastic viscosity provides no assurance of segregation resistance.

- Mixture should be proportioned for the proper dynamic and static yield stresses to ensure self-flow and segregation resistance. In particular, the paste volume, w/p, or both should not be too low or too high. The HRWRA type and dosage has a significant effect on the initial yield stress and rate of change in yield stress with time. VMAs that provide a shear thinning character and increase the degree of thixotropy can reduce segregation.
- The column segregation test and sieve stability test are acceptable for measuring static segregation resistance. Neither test is appropriate for use as a rapid field test. The sieve stability test is preferred to the column segregation test because it is easier to perform and the results of the two tests are well correlated. The column segregation test, however, is more likely to be used in the US because it has been standardized by ASTM International.

8. Evaluation of Workability Test Methods

When the research described in this report began, no workability test methods for SCC had been standardized in the United States. Although many test methods had been proposed, limited information was available on exactly what each test measured and on why certain tests should be used. Many of the details of each test, such as dimensions and procedures, varied throughout the world. As a result, seven test methods were selected for extensive evaluation to identify the best test methods for routine use based on sound, engineering justifications. The seven test methods evaluated were the column segregation test, j-ring test, l-box test, penetration apparatus test, sieve stability test, slump flow test, and v-funnel test. These test methods were evaluated as part of the research described in this report and a concurrent project at the University of Texas on the role of aggregates in SCC (Koehler and Fowler 2007). The data presented in this chapter are from both research projects and, therefore, cover a wide range of materials and mixture proportions. The specific test procedures are included in Appendix B. The evaluation of the segregation test methods (visual stability index, column segregation test, penetration apparatus test, and sieve stability test) was presented in Chapter 7.

8.1 Criteria for Evaluation of Test Methods

Each test method was evaluated based on its suitability for routine use in the laboratory for evaluating materials and developing mixture proportions and in the field for quality control. The following criteria were established to evaluate the test methods.

Well-Defined Results. The test results should clearly indicate filling ability, passing ability, segregation resistance (static or dynamic stability), a fundamental rheological parameter, or some other relevant property. The test results should be suitable for use in specifications.

Independent Measurements. Tests should measure filling ability, passing ability, and segregation resistance independently. By measuring only one of these properties at a time, the results can be used to identify specific problems with a mixture and implement solutions. In contrast, pass/fail-type tests that measure some combination of filling ability, passing ability, or segregation resistance indicate when a mixture is inadequate but provide little information for correcting problems. For instance, a test that measures both filling ability and passing ability simultaneously would be unsuitable because if the test results indicate inadequate workability, it would be impossible to determine whether the concrete lacks filling ability, passing ability, or both. To some extent, filling ability, passing ability, and segregation resistance are interrelated; therefore, some overlap is inevitable. Tests can, however, measure one aspect of workability predominately.

Simplicity. The equipment, test procedures, and interpretation of test results should be simple. The test should be standardized and the number of variations and options minimized. Minimal training should be required.

Use of Results. It should be possible to implement test results directly with minimal analysis. If concrete is unsuitable, the test should provide information on exactly why the concrete is unsuitable so action can be taken to rectify the problem or reject the mixture.

Use in Field. Test methods intended for use in the field must be lightweight, rugged, easy to perform in a variety of locations and circumstances, easy to clean, and low in cost. These same aspects are also desirable in tests intended for use primarily in the laboratory. When possible, the same tests should be performed in the laboratory and field.

Repeatability and Reproducibility. The test results must be robust and reliable, particularly given the potentially severe consequence of inadequate SCC workability.

8.2 Evaluation of Test Methods

8.2.1 J-Ring Test

8.2.1.1 Discussion of Test

The j-ring test provides an independent measurement of passing ability. Increasing the slump flow (filling ability) typically results in less j-ring blocking; however, it is likely that this trend is also present in field conditions. It is not affected by slump flow nearly to the extent as the l-box test. The European Testing-SCC project selected the j-ring, along with the l-box, as reference test methods for passing ability; however, they favored the l-box because of the availability of more field experience with the l-box.

There are variations in the test apparatus, test procedure, and measurement of results that are important to interpreting results consistently. The test apparatus can vary in the size and spacing of bars. Either smooth or deformed reinforcing bars can be used. The bar size is typically $\frac{1}{2}$ or $\frac{5}{8}$ inches. The spacing of bars, however, can vary widely. It is possible to vary either the reinforcing bars or the acceptance criteria—namely the change in height, change in slump flow, or test value—based on the application. While both approaches are acceptable, the use of constant bar spacing is more practical because it allows the same j-ring apparatus to be used in all cases without adjustment. The limitation of using the same bar spacing is that the standard bar spacing may not adequately represent field conditions for concrete with very large aggregate sizes or for applications with very narrow clear spacing. In the ASTM C 1621 standard, the clear spacing is approximately 1.75 inches, which appears to be an appropriate compromise. If the bar spacing is varied, it should be based on the actual bar spacing in the field and not the maximum aggregate size. The diameter of the ring is mostly consistent. The diameter of 12 inches is appropriate because it is small enough to evaluate mixtures with a wide range of slump flows and is large enough to contain a sufficient number of bars.

The main variation in the test procedure is the orientation of the slump cone. The use of the inverted slump cone orientation is recommended for the same reasons as for the slump test. In addition, if the cone is used in the inverted orientation, the foot pieces on the cone do not need to be removed so that the cone will fit within the j-ring.

The test results can be reported as the difference in height between the inside and outside of the j-ring, the change in slump flow spread with and without the j-ring, or the “test value”

which is a function of the height of concrete inside and outside and at the center of the j-ring. In some cases, T_{50} flow time is also measured. The change in height between the inside to outside of the ring is the best approach because of its simplicity, precision, and ability to best reflect the extent of passing ability. The j-ring test value (PCI 2003) is computed as shown in Equation (8.1) based on four measurements of the height of concrete inside (h_{inside}) and outside (h_{outside}) of the ring and one measurement at the center of the ring (h_{center}):

$$\text{J – Ring Test Value} = 2[\text{median}(h_{\text{inside}} - h_{\text{outside}})] - \text{median}(h_{\text{center}} - h_{\text{inside}}) \quad (8.1)$$

This calculation of the test value is unnecessarily complex. The difference in height between the inside and outside of the ring is much easier to determine. Figure 8.1 indicates a high correlation between the j-ring test value and change in height, suggesting the added calculation for the j-ring test value is of no benefit. The difference in height is typically measured at four locations equally spaced around the ring. The use of multiple measurements is important because some variation in blocking around the ring is possible. To simplify the determination of a single value, the median of three measurements can be used. The inside measurement can be made at the center of the ring or just inside the ring. Either approach is acceptable; however, the exact approach used must be indicated when reporting results.

The measurement of the difference in slump flow with and without the j-ring is unsuitable. First, the difference in slump flow with and without the j-ring is often within the precision of the slump flow test. According to ASTM C 1611, two slump flow tests conducted by the same operator on the same batch of concrete should not differ by more than 3 inches. ASTM C 1621 specifies that differences in slump flow measurements over 2 inches reflect “noticeable to extreme blocking”. This characterization of “extreme” blocking is not supported because it is within the expected precision of the slump flow test. Indeed, Figure 8.1 indicates a high degree of scatter between the change in slump flow and the change in height measurements. All plotted data were determined in the laboratory, where the potential for variation is likely to be less than in the field. Second, the difference in slump flow may not reflect the extent of blocking, notwithstanding the lack of precision. In some cases, the thickness of the concrete flowing out of the j-ring is thinner than for the concrete tested without the j-ring—due to differences in blocking—but the spread is approximately the same. This scenario is illustrated in Figure 8.2.

The measurement of T_{50} (or similar distance) with the j-ring is unnecessary because this same measurement made with the unobstructed slump flow test provides a better measurement of viscosity and the change in height between the inside and outside of the j-ring provides an adequate indication of passing ability.

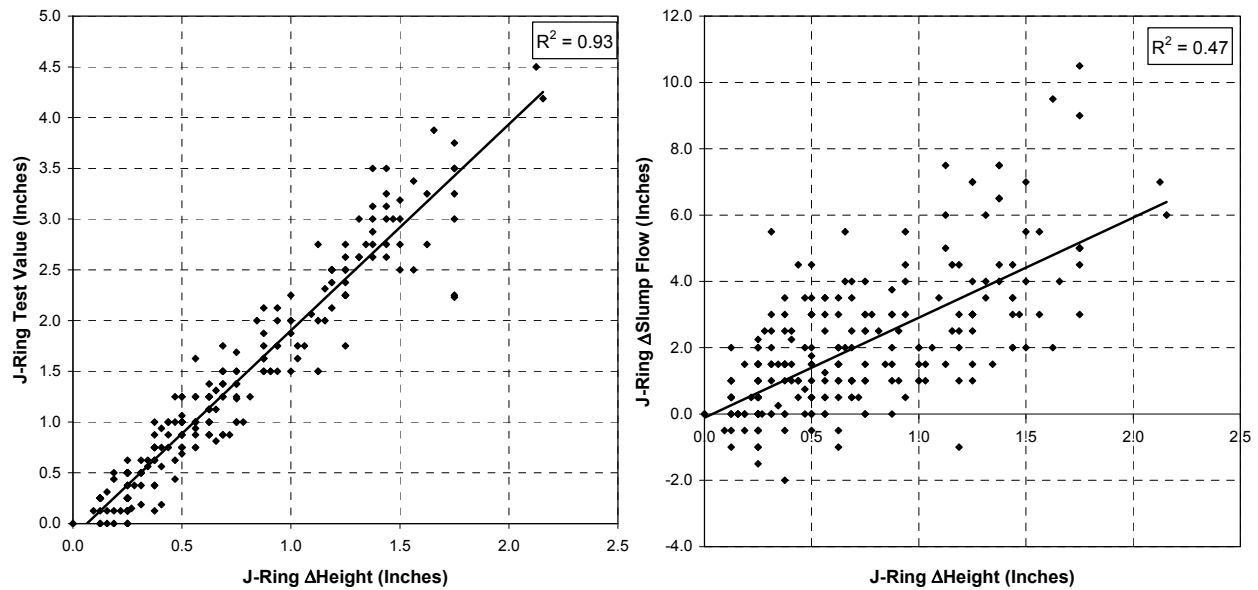


Figure 8.1 Relationship between J-Ring Test Value and Δ Height and Δ Slump Flow

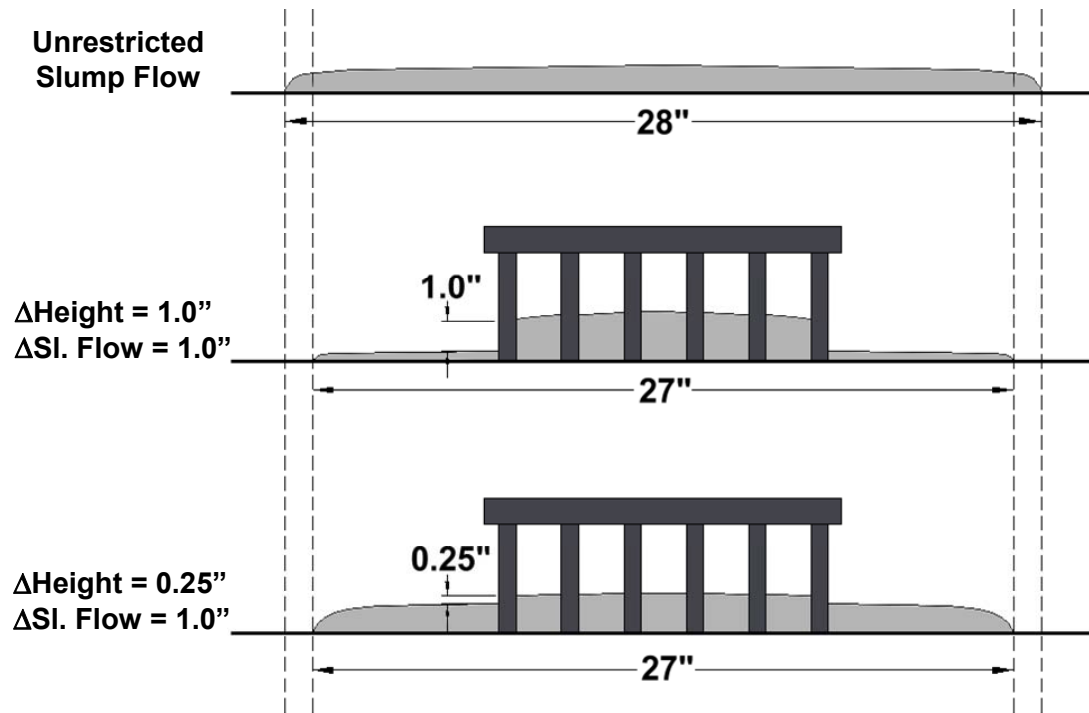


Figure 8.2 Representation of J-Ring Results with Same Restricted Slump Flows

The relationship between j-ring results and concrete field performance is not well established. In the field, the energy from a large mass of concrete moving through formwork can push concrete through the openings between reinforcing bars. The mass of concrete pushing concrete through the j-ring is much smaller by comparison. The effects of this lack of mass may

be exacerbated for highly viscous or highly thixotropic mixtures. In general, however, the test does reflect actual field conditions reasonably well and does effectively distinguish mixtures with varying degrees of passing ability due to changes in mixture proportions.

Advantages and Disadvantages

The advantages of the j-ring test include:

- The test independently measures passing ability.
- The test represents field conditions well and accurately distinguishes between mixtures with varying degrees of passing ability.
- The equipment is low in cost and portable. Although it is mainly needed in the laboratory, it can be easily used in the field (especially when compared to the l-box).

The disadvantages of the j-ring test include:

- Relationships between j-ring results and field performance are not well-established.
- The limited mass of concrete available to push concrete through the openings in the j-ring may not be representative of field conditions. (This limitation, however, is conservative.)
- The use of a single spacing of reinforcing bars for all tests may overestimate passing ability for highly congested sections.

Recommendations

The j-ring test is a simple and effective test for independently measuring passing ability and is appropriate for use in specifications. The test should be performed with the slump cone in the inverted position. Test results should be reported as the difference in height of concrete between the inside and outside of the j-ring (median of three equally spaced measurements). The measurement of the difference in slump flow with and without the j-ring is inappropriate and not advised. The reinforcing bar spacing should be constant—the spacing in ASTM C 1621 is reasonable—and the maximum acceptable change in height varied based on the application.

The test should be used in the laboratory when developing and qualifying mixture proportions. Because passing ability primarily depends on aggregate characteristics and paste volume and to a much lesser extent on paste rheology, the test does not need to be performed in the field if the slump flow test is used to control concrete rheology indirectly and the paste volume and aggregate characteristics remain reasonably constant.

8.2.2 L-Box Test

Discussion of Test

The l-box test provides a measurement of passing and filling ability. The l-box is similar to the u-box test. The l-box test was chosen for evaluation in the research described in this report because it is easier to visualize the flow of the concrete in the test—especially any blocking

behind the bars—and the apparatus is easier to clean. The l-box test has been used widely throughout the world and was selected by the European Testing SCC project as a reference test for passing ability.

The l-box test results are a function of both passing ability and filling ability because the extent to which concrete flows down the horizontal portion of the box depends on the yield stress (filling ability) of the concrete and the extent of blocking caused by the row of bars. Indeed, the degree of correlation between the l-box and j-ring is poor, as shown in Figure 8.3. Similarly, the European Testing SCC project found the correlation was “not very good” between l-box and j-ring. Therefore, the test is essentially a pass/fail test because it is not clear whether concrete with a low blocking ratio exhibits inadequate filling ability, passing ability, or both. Nguyen, Roussel, and Coussot (2006) found that for a homogenous yield stress fluid (no blocking), the blocking ratio is a function only of yield stress and density. While measuring the difference in blocking ratio with and without the bars would isolate the effects of filling ability and passing ability, such an approach would be much more time-consuming. It would not be feasible to measure passing ability independently by determining the difference in concrete height on either side of the bars because concrete may not completely flow out of the vertical portion of the box in all cases, including when the filling ability is inadequate.

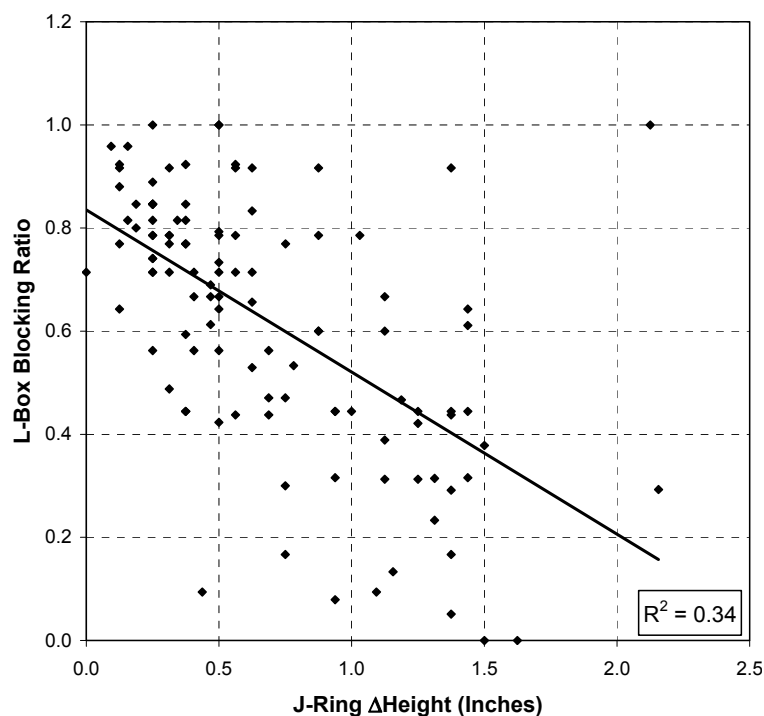


Figure 8.3 Relationship between L-Box and J-Ring Test Results

There are variations in the test apparatus, test procedure, and measurement of results that are important to interpreting results consistently. The test apparatus varies in dimensions, materials, and reinforcing bar spacing. The main differences in dimensions are in the cross section of the vertical portion and the total length of the horizontal portion of the box. These differences render it impossible to compare one box to another. The l-box is frequently constructed of plastic or plywood. Due to the large surface area in contact with concrete, the

surface finish is likely more important than in the slump flow test. Petersson, Gibbs, and Bartos (2003) found differences in wall surface finish to affect results significantly. The European Testing SCC project, however, found that differences in surface finish were negligible. The options for bar spacing are limited because no more than three bars can realistically be fit in the opening.

The main difference in the test procedure is the length of time the concrete is allowed to remain in the box before the gate is opened. Any delays in opening the box would likely reduce the blocking ratio because of any thixotropy or segregation.

In nearly all cases, the test results are computed in terms of the blocking ratio, defined as the ratio of concrete height in the horizontal portion to the vertical portion of the box. The term “blocking ratio” is a misnomer because higher blocking ratios correspond to less blocking, greater filling ability, or both. A term such as “passing ratio” or the use of the inverse of the blocking ratio as defined above would be more appropriate. In some cases, the time for concrete to flow a certain distance down the horizontal leg of the box is measured. This distance should be as long as possible to increase measurement precision. Figure 8.4 indicates that the correlation between slump flow T_{50} and l-box T_{40} is poor. The slump flow T_{50} is primarily related to plastic viscosity while the l-box T_{40} is related to both plastic viscosity and degree of blocking. The measurement of l-box T_{40} is unnecessary because plastic viscosity and degree of blocking are best measured independently with other tests.

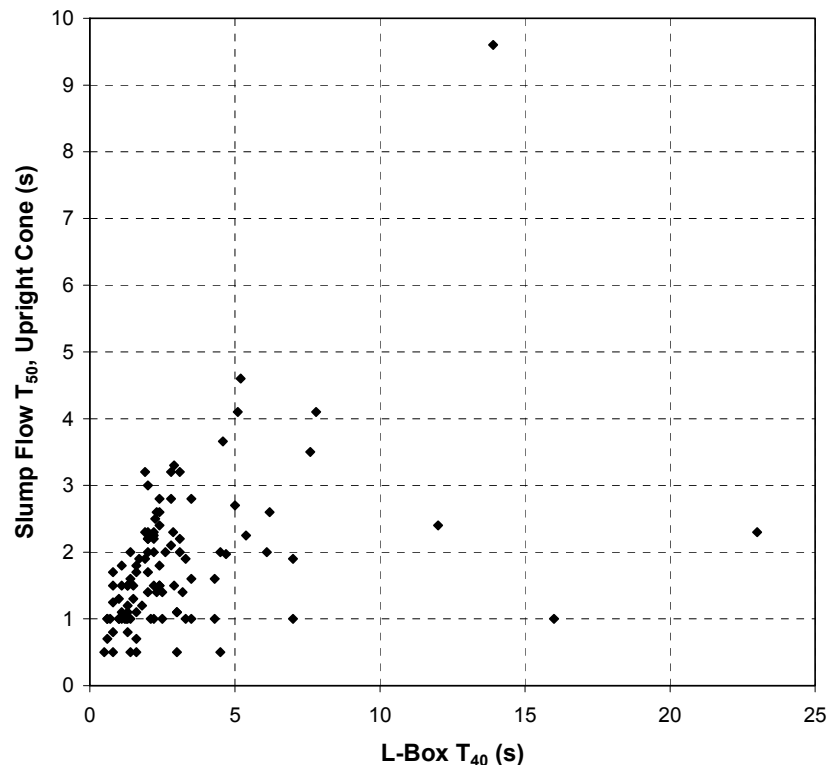


Figure 8.4 Comparison of L-Box T_{40} and Slump Flow T_{50} (Upright)

The calculation of blocking ratio requires three calculations: the heights of concrete in each end (from the distance from the top of the box to the concrete) and then the blocking ratio from the two heights. As such, results are not available immediately. It would be preferable to

measure one value and perform no calculations. Accordingly, the possibility of measuring just the distance from the top of the box to the concrete in the vertical or horizontal leg is evaluated in Figure 8.5. The precision of either measurement is insufficient. The difference in distance in the vertical leg is only 1 inch as the blocking ratio changes from 0.60 to 1.00.

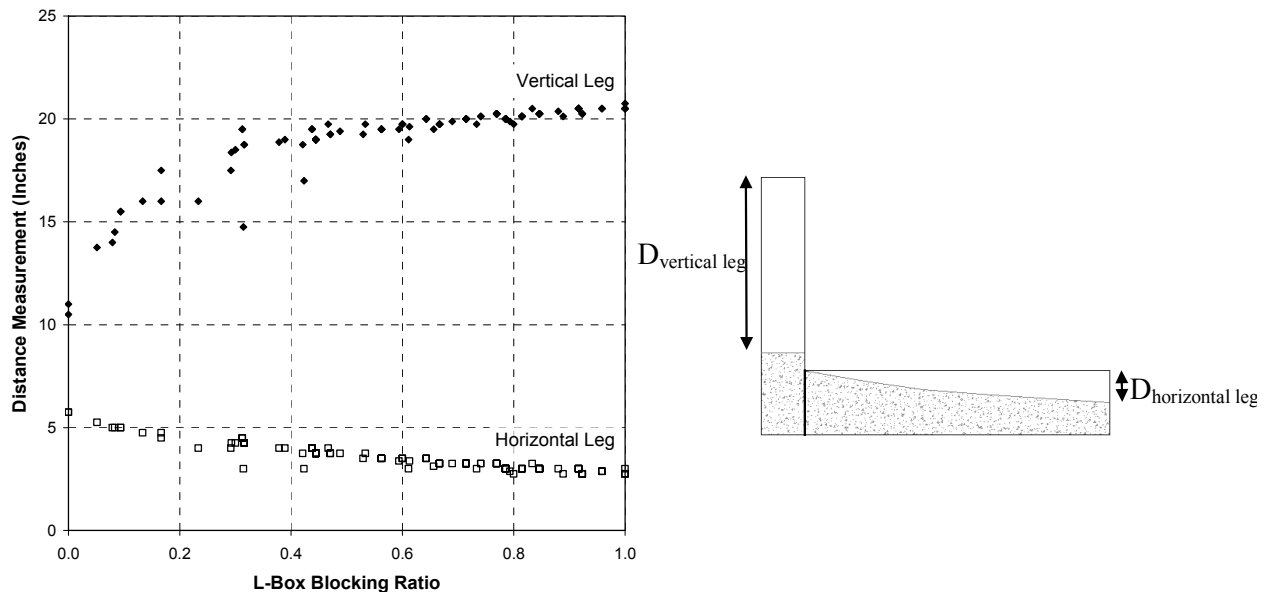


Figure 8.5 Simplified Measurements for L-Box Test

The l-box does reflect field conditions; however, the number of bars through which the concrete must pass is limited. The j-ring has more bars and would likely exhibit less variability from one test to another. The l-box, however, has an advantage over the j-ring in that a larger mass of concrete is available to push concrete through the bars.

Advantages and Disadvantages

The advantages of the l-box test include:

- The test provides a visualization of how concrete will flow in the field.
- The amount of mass available to push concrete through the bars is more representative of field conditions than in the j-ring test.
- The relationship between the test results and field performance is better established than for the j-ring test.

The disadvantages of the l-box test include:

- The test does not distinguish between passing ability and filling ability.
- The test apparatus is bulky, difficult to clean, and not well-suited for use in the field.
- The selection of rebar spacing is not well defined.
- The determination of blocking ratio requires two measurements and three separate calculations. A single measurement is not possible.

- The volume of concrete required is greater than for the j-ring test.

Recommendations

The use of the l-box test is not recommended because the measurement of passing ability is not sufficiently independent of filling ability and because the test is bulky and difficult to clean. The j-ring test is preferred for measuring passing ability. The l-box is preferred to the u-box.

8.2.3 Slump Flow Test (with T_{50} and VSI)

Discussion of Test

The slump flow test is the most well-known and widely used test for characterizing SCC and is extremely easy and straightforward to perform. The slump flow (yield stress) is the main fundamental difference between SCC and conventionally placed concrete. The slump flow test provides a measure of filling ability. The horizontal spread reflects the ability of the concrete to flow under its own mass (yield stress) while the T_{50} time and VSI provide indications of the plastic viscosity and segregation resistance, respectively. The test does not provide a complete description of filling ability because it does not fully reflect harshness and the ability to fill all corners of the formwork. The test does, however, provide a valuable visualization of concrete flow.

There are variations in the test apparatus, test procedure, and measurement of test results that are important to interpreting results consistently. The test apparatus may vary in the material used for the base plate and the moisture condition of the base plate. A smooth, plastic base plate is typically best. It is particularly important that the plate be level, flat, and free of any standing water, all of which can affect results. Any appreciable amount of water not only increases the slump flow but may also reduce the observed stability. A squeegee should be used to remove any standing water.

The main variation in the test procedure is the orientation of the cone—namely inverted or upright. The final spread is the same regardless of the orientation; however, the T_{50} time is greater with the inverted orientation. The inverted orientation is preferred because (1) the larger end of the cone can be more easily filled with less spillage, (2) the mass of concrete in the cone is sufficient to hold the cone down—eliminating the need for a person to stand on the foot pedals of the cone—and (3) the T_{50} is greater and can be measured with increased precision. The test results may also be influenced by the speed with which the concrete is lifted. The 4-inch diameter of the bottom of the cone is sufficiently large such that test results are not typically influenced by passing ability.

The main difference in the measurement of test results involves the determination of T_{50} . In some cases a longer or shorter distance is used for high or low slump flows, respectively. Given that most SCC exhibits slump flow greater than 22 inches (560 mm) and T_{50} greater than 2-3 seconds (inverted orientation), the use of 50 mm is appropriate for the vast majority of SCC mixtures. Using various distances, while technically sound, reduces the simplicity and practicality of the test. Another variation occurs in determining the precise time to stop the T_{50} measurement. If concrete does not flow at the same rate in all directions, which is common, all concrete will not reach the T_{50} line at the same time. Therefore, it is important to specify

whether T_{50} should be determined when concrete first touches the T_{50} line or completely reaches the entire T_{50} line.

The meaning of the slump flow test results is well-defined. The slump flow spread reflects the ability of concrete to flow under its own mass and is related to the yield stress (Roussel, Stefani, and Leroy 2005). For a given concrete mixture over a wide range of slump flow measurements, the correlation between yield stress and slump flow is high, as shown in Figure 8.6. For the narrower range of slump flow values mainly associated with SCC—namely 22 to 30 inches—and for a wide variety of materials and mixture proportions, the variation in yield stress for a given change in slump flow is very small, such that a strong correlation between the two values cannot be established. Indeed, Figure 8.7 indicates the scatter in the relationship between slump flow and yield stress is high over this narrower range of slump flows. Plastic viscosity also affects the final slump flow. Figure 8.8 indicates that for a constant slump flow, increasing the yield stress requires a lower plastic viscosity.

The T_{50} measurement is well correlated to plastic viscosity (Figure 8.9), particularly when considering the precision of the T_{50} test. This relationship is valid for nearly the full range of slump flows associated with SCC. Determining the plastic viscosity—either directly or indirectly—is particularly important because, with the yield stress relatively unchanged over the range associated with SCC, the plastic viscosity is often the main factor distinguishing the workability of one mixture from another. Changes in plastic viscosity can directly reflect changes in materials or mixture proportions, making the T_{50} measurement particularly valuable for quality control.

As discussed in Chapter 8, the visual stability index fails to measure stability adequately.

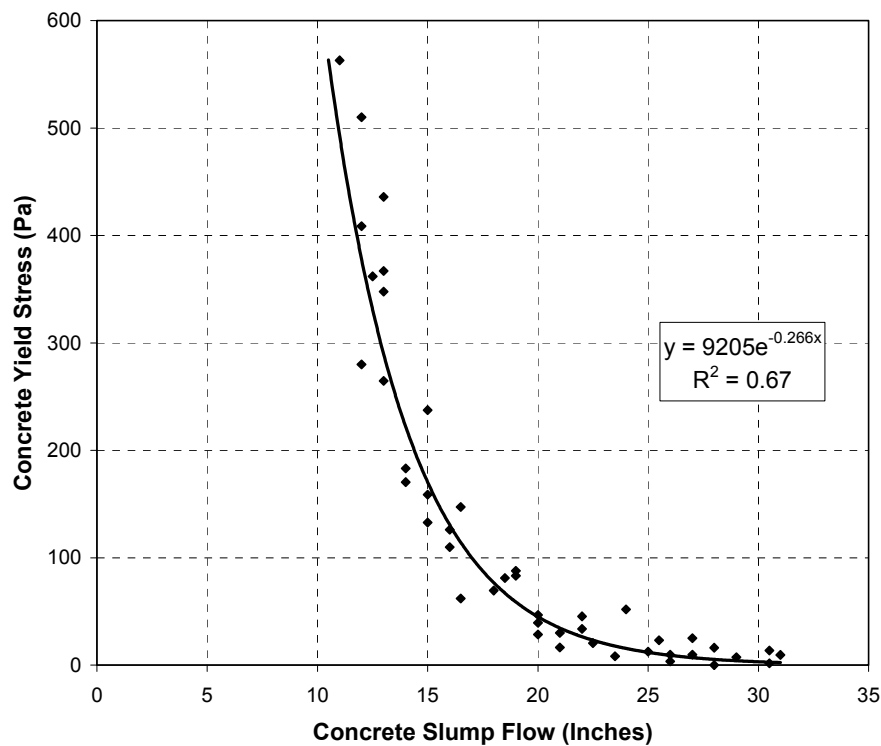


Figure 8.6 Relationship between Slump Flow and Yield Stress for Constant Mixture Proportions (Variable HRWRA Type and Dosage)

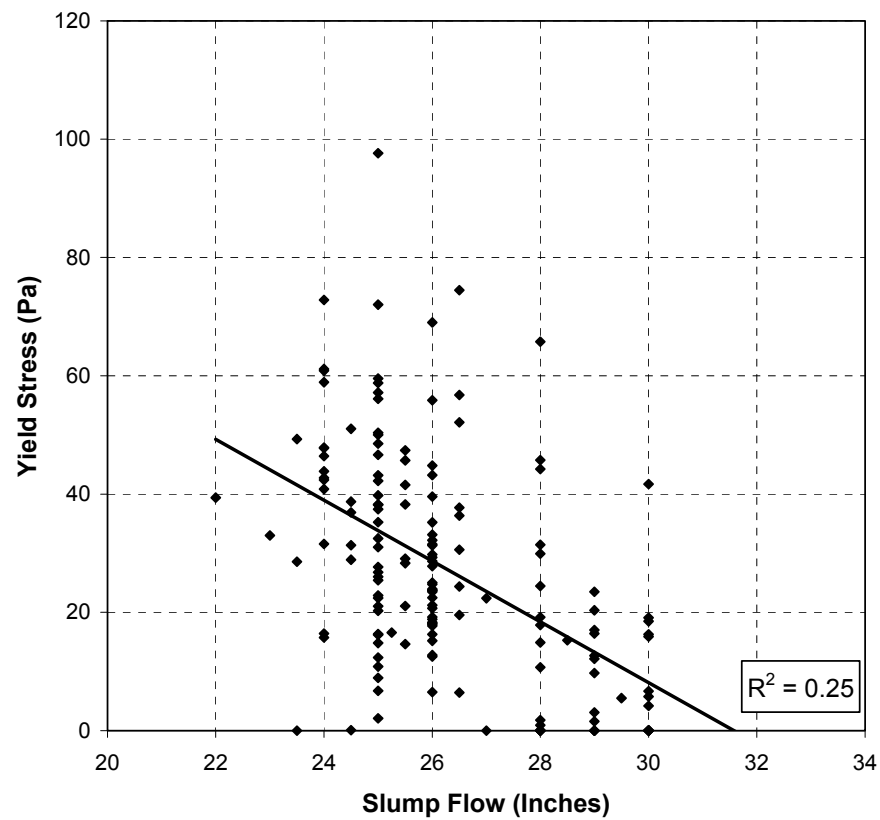


Figure 8.7 Relationship between Slump Flow and Yield Stress for SCC Mixtures (Various Materials and Mixture Materials)

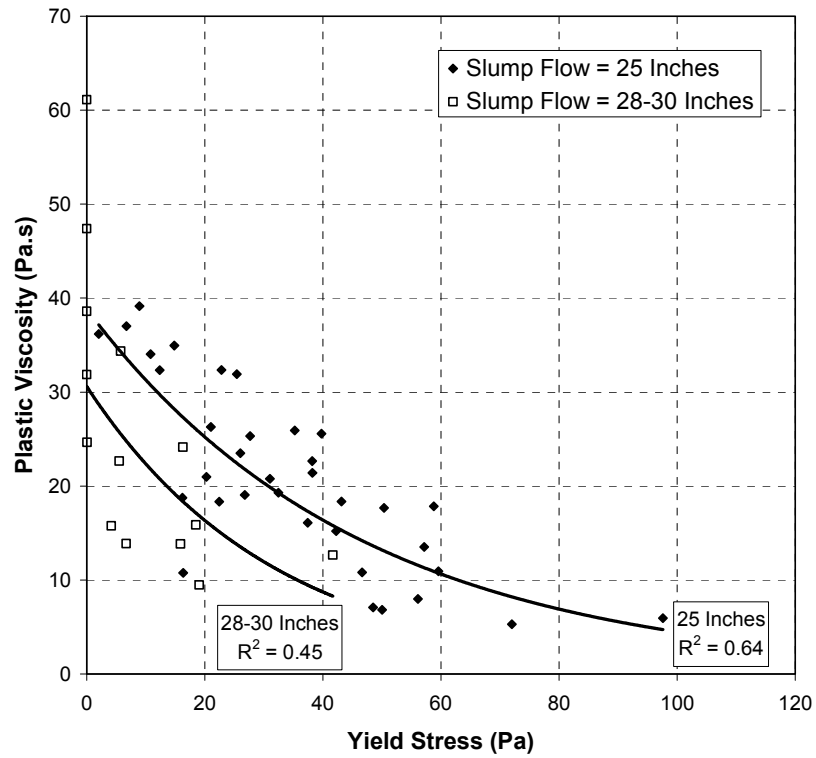


Figure 8.8 Relationship between Yield Stress and Plastic Viscosity at Various Slump Flows

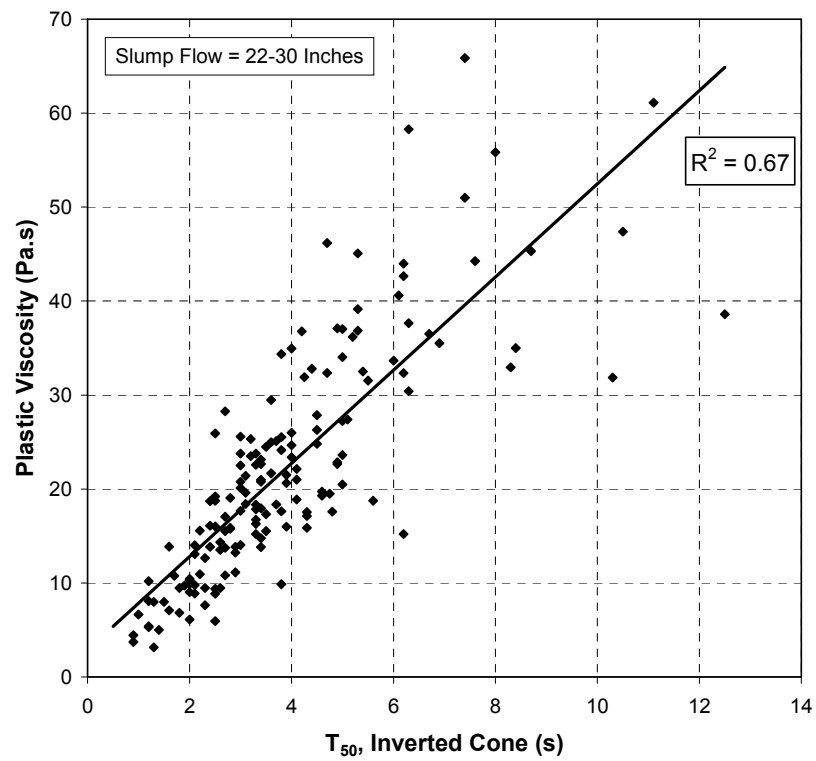


Figure 8.9 Relationship between T_{50} Time (Inverted Cone) and Plastic Viscosity

Advantages and Disadvantages

The advantages of the slump flow test include:

- The test provides an independent measurement of filling ability.
- The test is well-known, widely used, and simple to perform.
- The test is inexpensive and easily portable.
- The specimen size is small.
- The test is robust and repeatable.
- The spread is related to yield stress and T_{50} is related to plastic viscosity.
- The test provides a visualization of concrete flow.

The disadvantages of the slump flow test include:

- The VSI is inadequate for ensuring segregation resistance.
- The test results do not reflect all aspects of filling ability and do not indicate the harshness of mixtures.
- The test must be conducted on a flat base plate with no standing water.

Recommendations

The slump flow test is a simple, inexpensive, robust, and effective test for measuring filling ability. The ability of the test to measure indirectly the fundamental rheological properties of yield stress and plastic viscosity is especially valuable. In addition to slump flow, which is related to yield stress, T_{50} should always be measured because it is related to plastic viscosity. The test should always be performed with the cone in the inverted orientation because this orientation makes the test easier to perform and the use of consistent orientation ensures accurate comparisons between tests.

The slump flow test can be used in both the laboratory and field. For many cases, the slump flow test is the only test needed in the field for quality control. The slump flow spread should be used to adjust the HRWRA dosage to ensure the ability of the concrete to flow under its own mass. T_{50} should be used in the laboratory for developing and qualifying mixtures to assess plastic viscosity and should be used in the field to detect unexpected changes in materials and mixture proportions. The VSI can be used to catch cases of severe segregation; however, it is not reliable as an assurance of adequate segregation resistance. Mixtures with high VSI should be investigated further but not necessarily rejected.

8.2.4 Funnel Test

Discussion of Test

The v-funnel test measures a single value that is related to filling ability, passing ability, and segregation resistance. Therefore, the test may be suitable as a pass/fail test but cannot provide an independent indication of filling ability, passing ability, or segregation resistance. Low v-funnel times can be associated with good flow properties, but the test provides no information for troubleshooting mixtures with high v-funnel times.

For a homogenous fluid with no segregation, the v-funnel test results have been shown to be a function of yield stress and plastic viscosity (Roussel and Le Roy 2005). By determining yield stress and plastic viscosity, the test provides a measure of filling ability. Since yield stress does not vary over a wide range for SCC, the v-funnel time of self-flowing concretes that can be idealized as homogenous, non-segregating fluids is mainly a function of plastic viscosity. As the size and volume of aggregate increase, the potential for blocking of aggregate across the opening increases. Therefore, the v-funnel is affected by passing ability in some cases. Any segregation that occurs from when the concrete is loaded into the v-funnel until the concrete flows out of the v-funnel increases the v-funnel time. Even if the gate of the v-funnel is opened as soon as practical, it is possible for some segregation to occur.

Error! Reference source not found. indicates that the relationship between plastic viscosity and v-funnel time is poor for concrete. For v-funnel times less than 10 seconds, a better correlation between v-funnel time and plastic viscosity appears to exist. The scatter is much greater at higher v-funnel times due to any harshness, blocking, or segregation—which increase v-funnel time but do not increase plastic viscosity by a proportionate amount. For mortar, the relationship between v-funnel time and plastic viscosity is better due to the reduced blocking and segregation.

There are variations in the test apparatus, test procedure, and measurement of test results that are important to interpreting results consistently. The test apparatus mainly varies in the dimensions. Alternative shapes are available, such as an o-shaped cross section and the orimet, which consists of a cylinder with a narrowed opening at the bottom. Smaller versions of funnels are available for mortar and paste. Even for the v-shape version for concrete, the dimensions vary. The test procedure mainly varies in the amount of time from filling the funnel to opening the gate. This period can be lengthened to measure segregation. Whatever period is chosen, it should be consistent for all tests. Care should be taken to load the concrete in a consistent time frame—such as filling quickly with a single bucket of concrete or more gradually with a scoop. The measurement of test results can be reported as the v-funnel time or the average rate of flow. The calculation of average rate of flow is an unnecessary extra step.

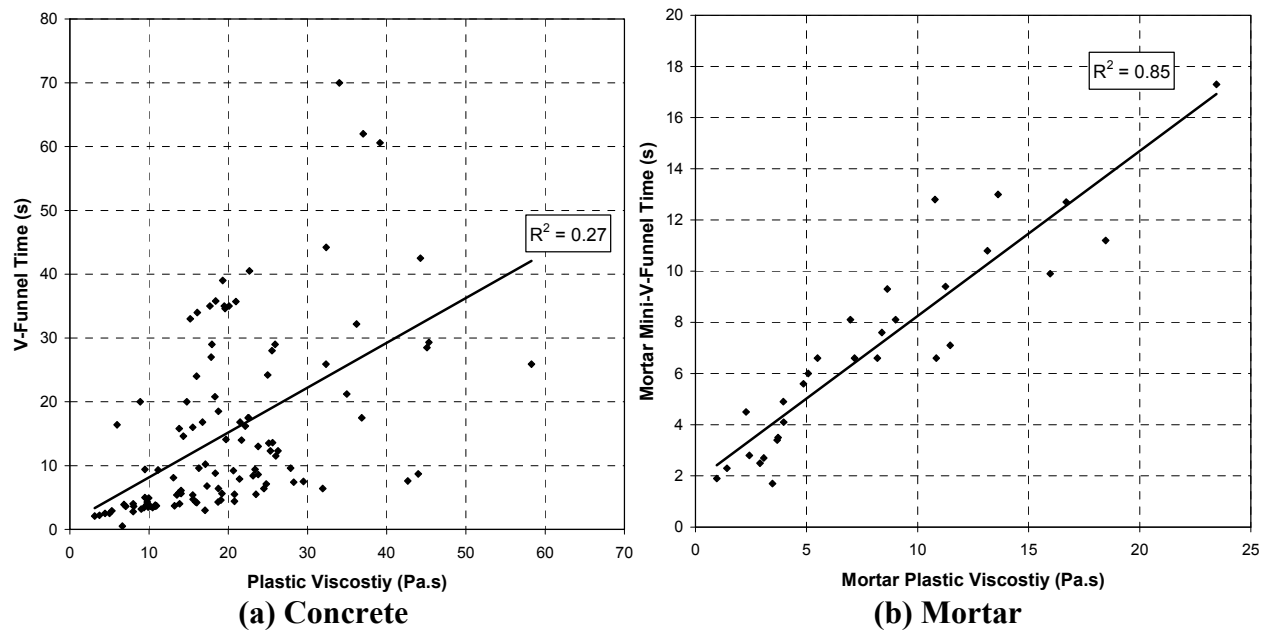


Figure 8.10 Relationship between Plastic Viscosity and V-Funnel Time

Advantages and Disadvantages

The advantages of the v-funnel test include:

- The test is relatively simple to perform and results are expressed in a single value related to filling ability, passing ability, and segregation resistance.
- For paste, mortar, and concrete mixtures that can be idealized as homogenous, non-segregating materials, the results are a function of yield stress and plastic viscosity. For such materials that are also self-flowing (near-zero yield stresses), the results are primarily a function of plastic viscosity.

The disadvantages of the v-funnel test include:

- The test does not provide an independent indication of filling ability, passing ability, or segregation resistance.
- The test frame is large, bulky, and must be placed on a level surface.

Recommendations

The use of the v-funnel test is not recommended because the results are affected by filling ability, passing ability, and segregation resistance. Although the test does provide indications of each of these three characteristics, it should not be relied upon as conclusive confirmation of any one of these characteristics. The test can be used as a pass/fail test; however, no information is provided to troubleshoot problematic mixtures.

8.3 Conclusions

The following conclusions can be drawn based on the information presented in this chapter:

- In evaluating the workability of SCC, tests should measure filling ability, passing ability, and segregation resistance independently. Such an approach is preferred to pass/fail-type tests that measure multiple aspects of workability. Measuring each property individually provides a more direct insight into the performance of the concrete and allows more effective troubleshooting. These advantages outweigh the need to conduct multiple tests.
- To evaluate the workability of SCC, the slump flow test (with T_{50} and VSI) should be used for filling ability, the j-ring test for passing ability, and the column segregation test or sieve stability test for segregation resistance.
- For quality control measurements in the field, only the slump flow test is needed in most cases. (This recommendation matches that of the European Testing SCC project.) The slump flow spread should be used to adjust the HRWRA dosage to achieve proper slump flow for self-flow, T_{50} should be used to measure indirectly plastic viscosity and to detect changes in materials and mixture proportions, and VSI should be used to identify significant segregation.

9. Field Testing

The TTI research team cast two full-scale AASHTO Type A beams—one with conventionally placed concrete and the other with SCC—for structural property testing. The CTR research team planned the placement procedures to ensure the concrete would exhibit adequate workability and measured the workability and the early-age temperature and strength development. This chapter summarizes the field data and observations.

9.1 Field Testing Procedures

The field testing was conducted in two phases at Texas Concrete Company in Victoria, TX. In Phase I, the selected conventionally placed concrete and SCC mixtures were cast into 14-foot long AASHTO Type IV beams. The purpose of the Phase I testing was to verify the field performance of the concrete mixtures—which had previously only been tested in the laboratory—in terms of workability and early-age temperature and compressive strength development. In Phase II, the selected concrete mixtures were cast into the 40-foot long Type A beams for structural testing. In both phases, the conventionally placed concrete mixture was RG-5-C and the SCC mixture was RG-5-40. The mixture proportions—including admixture dosages—used in all field batches were as shown in Table 4.12. The material properties of the cement and aggregates are shown in Table 9.1 and Table 9.2.

The beam forms for Phase I and II are shown in Figure 9.1. The Type IV beams for Phase I were essentially unreinforced—they contained single untensioned strands in the top and bottom flanges. The Type A beams used in Phase II contained the reinforcement designed by the TTI research team. This reinforcement, which is shown in Figure 9.2, consisted of ten ½-in. strands in the bottom flange (6 in the bottom layer and 4 in the top layer) and two ½-in. strands in the top flange (one layer). These strands were spaced at 2 in. on center. The amount of transverse steel varied over the length of the beam, with the most steel in the end regions. The end regions consisted of #5 vertical bars spaced at 4 in. on center and #4 bars spaced at 4 in. on center, resulting in average clear spacing of approximately 1.5 in. Untensioned longitudinal steel was also present in the end regions. Figure 9.3 shows the extent of steel congestion in the bottom flanges in the end regions.

The concrete was mixed in a central mix plant and transported by forklift in the bucket-auger equipment shown in Figure 9.4. Upon reaching the form, the gate at the end of the horizontal tube was opened and the auger was operated to convey concrete into the beam, as shown in Figure 9.5. The conventionally placed concrete mixtures were placed with normal procedures. Namely, each conventionally placed concrete mixture was deposited along the length of the beam near its final location and was vibrated with internal vibrators. The SCC mixtures were placed in two lifts. For the first lift, the concrete was deposited at one end of the beam and allowed to flow to the other end. For the second lift, the concrete was deposited along the length of the beam because less concrete mass was available to push the concrete down the length of the beam.

After placement, the beams were covered with tarps, as shown in Figure 9.6. The forms for both types of beams were elevated to leave an air space below the forms, which allowed the temperature beneath the beam to be greater than if the beams rested directly on the ground (Figure 9.7).

The weather conditions, which are shown in Table 9.3, were similar for Phase I and Phase II testing.

Table 9.1 Cement Properties for Field Testing, PC-A (Reported by Manufacturer)

Chemical Tests		Physical Tests	
Silicon Dioxide (SiO ₂), %	20.6	Wagner Fineness, m ² /kg	273
Aluminum Oxide (Al ₂ O ₃), %	4.5	Blaine Fineness, m ² /kg	541
Iron Oxide (Fe ₂ O ₃), %	3.6	Initial Set (Gillmore), min	140
Calcium Oxide (CaO), %	64.6	Final Set(Gillmore), min	250
Magnesium Oxide (MgO), %	0.7	Compressive Strength	
Sulfur Trioxide (SO ₃), %	3.7	1 day, MPa	26.5
Total Alkalies (as Na ₂ O _{eq}), %	0.50	3 day, MPa	36.6
Insoluble Residue, %	0.13	7 day, MPa	43.3
C ₃ S, %	56.6	28 day, MPa	47.0
C ₂ S, %	16.3	Air Content, %	6
C ₃ A, %	5.8	Loss on Ignition, %	1.9
C ₄ AF, %	11.0	Amount Retained on #325 Sieve, %	0.5

Table 9.2 Aggregate Grading for Field Testing (Reported by Precaster)

		River Gravel (RG)	Natural Sand (NS-A)
Fineness Modulus			3.08
Grading (% Passing)	1" (25 mm)		
	3/4" (19 mm)	90	
	1/2" (13 mm)		
	3/8" (9.5 mm)	20	100
	#4	1	98
	#8	0	80
	#16		67
	#30		34
	#50		11
	#100		2
	#200		0



Figure 9.1 Type IV Beam (Left) and Type A Beam



Figure 9.2 Reinforcement in Type A Beams



Figure 9.3 Reinforcement Congestion in End Region Bottom Flange of Type A Beams



Figure 9.4 Concrete Transport Equipment: Side and Top View



Figure 9.5 Placement of Conventional Placed Concrete (Left) and SCC



Figure 9.6 Tarp Covering Forms After Placement (Type A Beams)



Figure 9.7 Typical Air Space beneath Forms (Type A Beams)

Table 9.3 Weather Conditions for Field Testing

	Phase I (March 12, 2007)	Phase II (March 26, 2007)
Placement Time	3:30-5:30 pm	3:15-5:00 pm
Weather Prior to Placement	Daytime temperatures in mid- to upper-60s with overcast skies, heavy rainfall (over 1 in.) previous night	Daytime temperatures near 70°F with overcast skies, scattered light showers during day
Placement Weather	71°F, 70% relative humidity, mostly cloudy, calm wind	72°F, 80% relative humidity, overcast, moderate wind
Weather after Placement (Overnight)	64°F low temperature, 90% relative humidity, calm wind	66°F low temperature, 90% relative humidity, light wind, thunderstorms producing approximately ¼ in. of rain
Sources: weather station next to beam (March 12, 2007, placement and overnight weather) and National Weather Service (all other weather)		

9.2 Phase I: Full-Scale Production Trials

9.2.1 Workability

The concrete was mixed in two 2 yd³ batches for each beam. The first batch filled to just below the bottom of the top flange. The workability of each batch is shown in Table 9.4.

The target slump of the conventionally placed concrete was 6 to 8 in. The first batch of conventionally placed concrete had a slump of approximately 4-6 in. (visual estimate) and was consolidated adequately with vibration. The HRWRA dosage was increased for the second batch, resulting in a slump of 10 in. and a slump flow of 20 in. Despite this slump flow, the concrete did not have adequate filling ability and stability to be considered SCC and was placed with vibration.

The first SCC batch had excellent workability. It was discharged in one end of the beam and flowed to the other end, resulting in a nearly level surface along the length of the beam (Figure 9.8). For the first SCC batch, the difference in concrete height from the end of the beam where the concrete was discharged was 0.75 in. at midspan and 1.25 in. at the opposite end. The concrete appeared to remain uniform as it flowed and to resist segregation, as evidenced by the coarse aggregates visible on the top surface of the concrete (Figure 9.9). The second batch was too stiff to be SCC when discharged—as evidenced by a low slump flow—and lost workability quickly. At the time of placement, the slump flow appeared to be approximately 18-20 inches. Therefore, the concrete was able to consolidate reasonably well without vibration but not adequately. Due to the low slump flow of the second mixture, a third batch was mixed for workability testing but was not cast into the beam. This mixture had higher initial slump flow than the second batch, but also lost workability quickly.

The batch-to-batch variation in SCC workability was likely due to two factors. First, the heavy rain from the previous evening increased the moisture content of the aggregate stockpiles and resulted in greater variability in moisture content. Even a small change in water content—with constant HRWRA dosage—can cause a large change in slump flow. Second, the poor workability retention associated with the HRWRA (HR-A) and cement (PC-A), which was documented during the laboratory testing, made rendering a direct comparison between mixtures impossible because of the rapidly changing workability. Even if a mixture exhibited excellent workability in the mixer, it could potentially lose too much workability with even a slight delay in placement. Because of the poor workability retention, it was not possible to make a direct comparison of the workability of one mixture to another and to isolate the effects of moisture content variations or any other variations.

The changes in workability of the SCC mixture were mainly associated with the yield stress and were reflected in the slump flow. The static yield stress of the first batch was sufficiently high to prevent segregation and the dynamic yield stress sufficiently low to enable self-flow. In the subsequent batches, the dynamic yield stress was too high for self-flow. The plastic viscosity—though only measured on the third batch—was likely similar for all batches even as workability decreased with time. The plastic viscosity of approximately 30 Pa.s was similar to that measured in the laboratory and was adequate to ensure stability and to allow the concrete to flow quickly from one end of the form to the other. Therefore, the SCC mixture was well proportioned for the application; however, a better HRWRA, retarder, or both were needed to ensure adequate workability retention. HRWRA mainly affects yield stress, which must be reduced sufficiently for self-flow, and affects plastic viscosity by a comparatively small amount. Therefore, a different HRWRA, retarder, or both would ensure adequate slump flow for sufficient time for placement without significantly changing the plastic viscosity or stability.

The formed surface finish of the conventionally placed concrete and SCC beams were similar, as shown in Figure 9.10. (Only the formed surface finish of the SCC beam corresponding to the location of the first SCC batch, which had excellent workability, was considered.) Both beams exhibited surface voids (bugholes) of similar size and quantity. For the SCC mixture, the fewest number of bugholes were present in the bottom flange, likely due to the greater mass of concrete above it. The conventionally placed concrete mixture exhibited the greatest number of bugholes in the bottom flange, likely due to the reduced intensity of internal vibration near the form.

Table 9.4 Workability Measurements for Phase I

	Conventionally Placed Concrete	SCC
Target	Slump: 6-8 in.	Slump Flow: 25-28 in.
Typical Laboratory Testing Results	Slump: 7.5 in. Yield Stress: 294 Pa Plastic Viscosity: 42 Pa.s	Slump Flow: 30 in T50: 3.8 s VSI: 0.5 J-Ring: 0.19 in. Column Segregation: 3% Dynamic Yield Stress: 16 Pa Plastic Viscosity: 24 Pa.s
First Batch	Batch Time: 3:30 pm Slump: 4-6 in. (visual estimate)	Batch Time: 4:20 pm Slump Flow: 25-27 in. (visual estimate)
Second Batch	Batch Time: 3:45 pm HRWRA dosage increased to 80.5 oz from 68 oz/yd ³ Slump: 10 in. Slump Flow: 20 in. Air: 1.6% Concrete Temperature: 79°F Dynamic Yield Stress: 94 Pa Plastic Viscosity: 23 Pa.s	Batch Time: 4:35 pm Visual Assessment: The concrete was not SCC when discharged and lost workability very quickly. The slump flow at placement was likely 18-20 in. (visual estimate)
Third Batch	n/a	Batch Time: 5:05 pm Note: Concrete lost workability quickly. Slump Flow: 20 in. T ₅₀ : 4.6 s VSI: 0 Air: 2.3% Column Segregation: 0% Concrete Temperature: 81°F Dynamic Yield Stress: 84 Pa Plastic Viscosity: 28 Pa.s



Figure 9.8 First Batch of SCC in Form



Figure 9.9 Top of First Batch of SCC: End Opposite of Placement (Left) and Near Midspan



Figure 9.10 Formed Surface Finish of SCC (Left) and Conventional Placed Concrete Beams

9.2.2 Temperature and Compressive Strength Development

The temperatures in the two beams during the first 24 hours are compared in Figure 9.11 and Figure 9.12. The conventionally placed concrete mixture had a pre-set time of just over 4 hours and a maximum temperature of 120°F. The preset time of the SCC mixture was approximately one hour longer; however, the maximum temperature was 125-130°F. The compressive strengths at 16 hours were close to those predicted by the maturity relationships developed in Chapter 5, as indicated in Table 9.5.

Semi-adiabatic calorimetry measurements were conducted in the field on the SCC mixture, as shown in Table 9.6 and Figure 9.13. The SCC mixture tested in the field generated less initial adiabatic temperature rise compared to the same mixture tested in the laboratory; however, it generated greater adiabatic temperature rise later. If the field concrete had generated adiabatic temperature rise at the same rate as the laboratory concrete, the maximum temperature in the beam would likely have been higher.

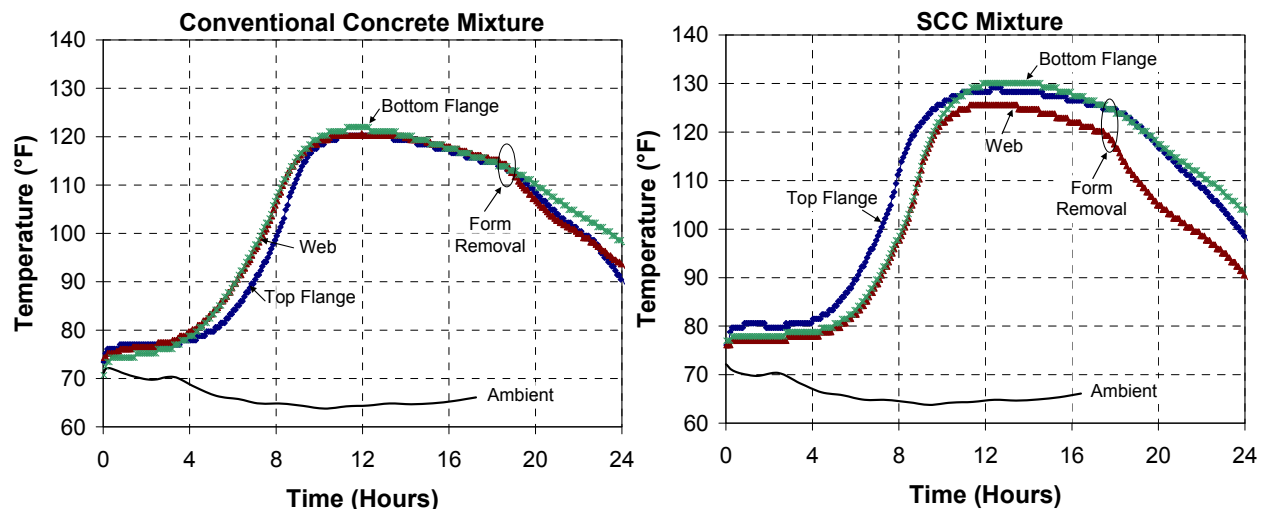


Figure 9.11 Heat Generation in Conventional and SCC Mixtures

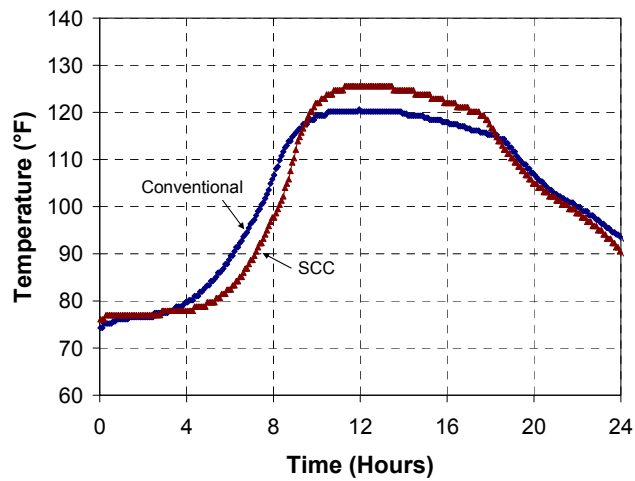


Figure 9.12 Comparison of Web Temperatures for Conventional and SCC Mixtures (Data Used for Match Curing; Separate Thermocouples from Data in Figure 9.11)

Table 9.5 Compressive Strength Development for Phase I Mixtures

	Conventional Placed Concrete (2 nd Batch)	SCC (3 rd Batch)
Equivalent Age at 16 Hours, Hours at 23°C	35.1	33.5
Predicted 16-Hour Compressive Strength, psi	5957	6101
Actual 16-Hour Compressive Strength psi	6210	5410
28-Day Compressive Strength, psi	9963	10251
Notes: Maturity and strength at 16 hours based on match cured cylinders with web temperature; predicted strength based on maturity equations developed in Chapter 5; 28-day compressive strength measured on cylinders cured next to beam and under tarp overnight and then moist-cured until testing.		

Table 9.6 Semi-Adiabatic Calorimetry Results

Mixture	E_a	H_u	Hydration Parameters (Eq. (5.4))			Adiabatic Temp. Rise	
			α_u	τ	β	16 hr	100 hr
	<i>kJ/mol</i>	<i>J/kg</i>				<i>°F</i>	<i>°F</i>
RG-5-C (lab)	35	466	0.769	11.740	1.135	83.6	97.7
RG-5-40 (lab)	35	398	0.670	12.857	1.060	96.3	113.2
RG-5-40 (field)	35	398	0.670	16.107	0.893	76.6	107.5
E_a determined from isothermal measurements; H_u computed with Equation (5.1)							

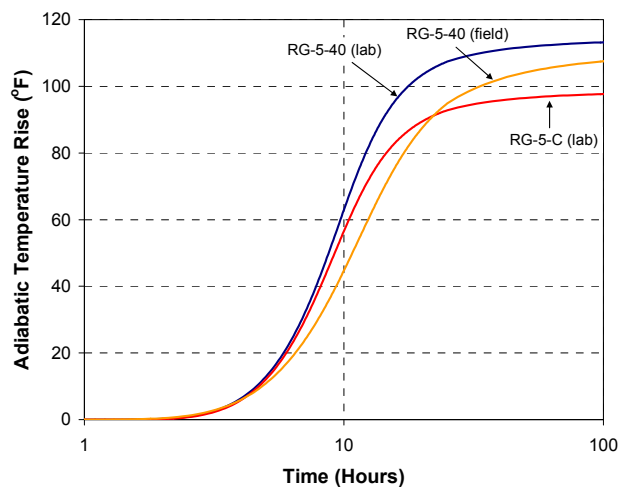


Figure 9.13 Calculated Adiabatic Temperature Rise from Semi-Adiabatic Calorimetry Results

9.3 Phase II: Casting of AASHTO Type A Beams

9.3.1 Workability

The concrete was mixed in two batches for each beam. The workability of each batch is shown in Table 9.7.

For the conventionally placed concrete beam, the first batch was 3.25 yd³ and filled nearly the entire beam. The 8-inch slump was within the target range and the concrete exhibited adequate workability.

Both SCC batches had high slump flows but poor stability—despite having the same proportions and admixture dosages as the SCC batches in the Phase I testing. Both batches exhibited segregation in the mixer. Upon reaching the form, they exhibited less segregation. The second batch had slightly better stability than the first, despite the higher measured slump flow. The first batch of SCC was deposited in one end of the beam and flowed to the opposite end; however, a paste-rich layer formed in the top 2-3 inches along much of the length of the beam due to the poor stability. It was not clear how much aggregate flowed from one end of the beam to the other. The second batch of concrete was deposited along the length of the beam. Figure 9.14 shows that although concrete did flow down the length of the beam, the coarse aggregate distribution did not appear to be uniform at all locations due to the poor stability. The extent of this lack of uniformity was not quantified. The top surface of the concrete after placement was smooth (Figure 9.15); however, aggregate was present within 1 in. or less along the full length of the beam. The segregation of both SCC batches was exacerbated in the transport buckets due to the vibration generated during transport. Although remixing would have re-homogenized the concrete, the transport buckets used in the plant did not have this capability. As a result, the initial concrete placed into the form had an excess of coarse aggregate and resulted in a large volume of voids visible on the formed surface finish in the location where it was first deposited (Figure 9.16). The passing ability appeared to be adequate for both SCC batches.

The formed surface finish of the conventionally placed concrete beam in Phase II, shown in Figure 9.17, was similar to that of the Phase I beam. The formed surface finish of the SCC beam in Phase II, shown in Figure 9.18, was better than that of the SCC beam in Phase I. With the exception of the location shown in Figure 9.16, the Phase II SCC beam exhibited fewer and smaller bugholes than the Phase I SCC beam. This improvement in formed surface finish was likely attributable to the higher slump flow.

The differences in the workability of the SCC mixtures between Phase I and II was likely due to variations in moisture contents—the nominal proportions and admixture dosages were otherwise identical. Both mixtures were cast on days with overcast skies and rain showers in the previous 24 hours, which likely increased the potential for variations in aggregate moisture contents. The stability problems could be prevented by reducing variations in moisture contents, using lower slump flows, and improving the robustness of the mixture. Furthermore, the direct comparison of mixtures was complicated by the short workability retention of the mixtures.

Table 9.7 Workability Measurements for Phase II

	Conventionally Placed Concrete	SCC
Target	Slump: 6-8 in.	Slump flow: 25-28 in.
First Batch	Batch Time: 3:15 pm Slump: 8 in. Air: 1.8% Concrete Temperature: 81°F	Batch Time: 4:25 pm Slump flow: 27 in. T ₅₀ : 3.3 s VSI: 1.0 Air: 0.9% Concrete Temperature: 80°F
Second Batch	Batch Time: 3:35 pm Visual Assessment: Workability similar to first batch	Batch Time: 4:45 pm Slump flow: 28.5 in. T ₅₀ : 3.6 s VSI: 1.5 Concrete temperature: 81°F



Figure 9.14 Placement of SCC Mixture



Figure 9.15 Top Surface of SCC Mixture in Form Immediately After Placement



Figure 9.16 Voids at Location of SCC Discharge for First Batch



Figure 9.17 Formed Surface Finish of Conventional Placed Concrete Beam



Figure 9.18 Formed Surface Finish of SCC Beam

9.3.2 Temperature and Compressive Strength Development

For the Phase II testing, only the web temperature in each beam was measured. Figure 9.19 indicates that the SCC and conventionally placed concrete mixture had similar pre-set times; however, the maximum temperature in the SCC mixture was approximately 20°F greater than in the conventional mixture. Due to its greater maturity at 16 hours, the SCC mixture exhibited higher 16-hour compressive strength, as indicated in Table 9.8. The 16-hour strengths of both mixtures, however, were less than predicted by the compressive strength-maturity relationships developed in Chapter 5.

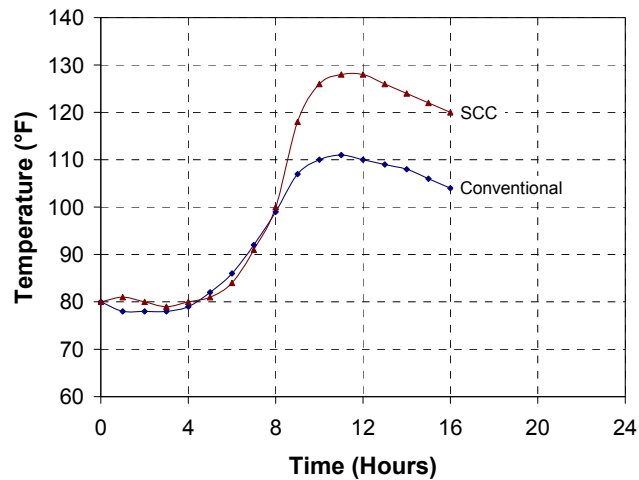


Figure 9.19 Comparison of Web Temperatures for Conventional and SCC Mixtures

Table 9.8 Compressive Strength Development

	Conventional Placed Concrete	SCC
Equivalent Age at 16 Hours, Hours at 23°C	29.3	37.3
Predicted 16-Hour Compressive Strength, psi	5723	6314
Actual 16-Hour Compressive Strength psi	5080	5714
Notes: Maturity and strength at 16 hours based on match cured cylinders with web temperature. Predicted strength based on maturity equations developed in Chapter 5.		

9.4 Conclusions

Based on the field testing, the following conclusions can be reached:

- The field testing results were generally consistent with the laboratory testing results in terms of workability, heat generation, and compressive strength development when the variability of the field testing results is considered.
- The variability of the SCC workability was too high. Therefore, the design of robust mixtures and the implementation of field quality control are critical to the success of SCC production. All SCC mixtures should be visually evaluated prior to being placed into beams to ensure adequate workability.
- The yield stress (and associated slump flow) must be carefully controlled during production. The SCC mixture exhibited excellent workability when the yield stress was within an acceptable range. When the yield stress was too high—potentially due to moisture content variations and poor workability retention—the concrete did not adequately consolidate under its own mass (Phase I, second batch). When the yield stress was too low—potentially due to moisture content variations—the concrete exhibited poor stability (Phase II batches).

- The passing ability of the SCC mixtures appeared to be adequate. Therefore, the j-ring criteria used in the laboratory testing was acceptable for this application and may be relaxed if justified with further testing.
- The maximum horizontal free-flow distance specified by TxDOT can be increased for SCC depending on the concrete rheology.
- The formed surface finishes of the SCC beams were as good as or better than the conventionally placed concrete beams.
- The SCC mixtures exhibited slightly higher maximum temperature than the conventionally placed concrete mixtures; however, the temperatures were well below the TxDOT-specified maximum temperatures.
- The field testing was limited in scope and did not explore all aspects of the field implementation of SCC. The placement issues that must be addressed may vary depending on the particular application, mixture proportions, plant equipment, construction practices, and weather conditions.

10. Recommendations for Specifying and Inspecting SCC

To use SCC, modifications are necessary to the way concrete is specified, produced, and inspected. This chapter discusses how to establish, test, and achieve target workability and hardened properties and the importance of material characteristics and mixture proportions. With this background, suggested changes to the 2004 TxDOT Standard Specifications are presented. The discussions in this chapter are intended to relate primarily to precast concrete in Texas and are based on the laboratory and field testing in this report and guidelines published by PCI (2003), EFNARC (2005), and Koehler and Fowler (2007).

10.1 Background

10.1.1 Workability Requirements

SCC is defined in terms of its workability. Compared to conventionally placed concrete, the workability of SCC is more sensitive to changes in materials and mixture proportions and the consequences of inadequate workability are much greater. If conventionally placed concrete does not exhibit sufficient workability, the amount of vibration can be increased accordingly to ensure consolidation. With SCC, however, there is no opportunity to compensate for inadequate workability once the concrete is discharged from the mixer. The potential for segregation can be much more severe for SCC than for conventionally placed concrete. Therefore, the control of SCC workability is much more critical.

Many terms have been used to describe SCC workability and numerous test methods have been developed to measure various aspects of SCC workability. However, it is only necessary to define and evaluate SCC workability in terms of three properties: filling ability, passing ability, and segregation resistance. Each of these three workability characteristics should be evaluated independently. The extent to which SCC must exhibit filling ability, passing ability, and segregation resistance can vary widely depending on the application and should be established separately for each application. In selecting target workability properties, it is important to only design for what is needed. Increasing the levels of filling ability, passing ability, and segregation resistance will often result in increased costs. Further, the susceptibility to segregation typically increases as the filling ability is increased. Filling ability, passing ability, and segregation resistance are described in detail in the following subsections.

10.1.1.1 Filling Ability

Description. Filling ability describes the ability of concrete to flow under its own mass and completely fill formwork. It is essential because without it, concrete would not consolidate adequately. It is primarily related to slump flow—increasing slump flow is associated with

greater filling ability—however, concrete must have adequate stability at a given slump flow to achieve filling ability.

Application Dependency: Medium. Members with tight spaces—such as with narrow widths or congested reinforcement—and applications where concrete must flow long horizontal distances may require greater filling ability. High placement energy—such as that generated by pumping or by gravity acting on a large mass of concrete—may reduce filling ability requirements.

Achieving. To achieve filling ability, concrete must have adequate paste volume and paste rheology for the given combined aggregate. Sufficient paste volume ensures that voids between aggregates are filled and that sufficient spacing is provided between aggregates. If the concrete contains insufficient paste volume, the paste will not convey the aggregates regardless of the rheology of the paste. In this case, increasing the HRWRA dosage may result in very low paste viscosity and severe bleeding. Paste with very low viscosity will quickly flow out of the aggregates without mobilizing the aggregates. In the slump flow test, the concrete will not achieve the desired slump flow with adequate stability, if at all. Even with the proper paste volume, concrete must also have proper rheology, which is directly affected by the paste rheology. Proper paste rheology ensures that the paste can convey aggregates uniformly as the concrete flows and that the concrete can fill all corners of the formwork. Concrete that is too viscous may be difficult to pump and place. Low concrete viscosities may result in poor dynamic stability. Harsh concrete mixtures can occur when the paste volume or paste viscosity is too low. In such a case, the concrete does not flow smoothly and may not completely fill all corners of the formwork and produce a smooth top-surface finish.

Testing: Slump Flow (ASTM C 1611). Filling ability should be tested with the slump flow test, including measurements of the time to spread 50 mm (T_{50}) and visual stability index (VSI). The test should be conducted with the inverted cone orientation. The slump flow spread ensures that the yield stress is sufficiently low for the concrete to flow under its own mass. The final adjustment of slump flow should be made by varying the HRWRA dosage. The T_{50} value should not be too low, which would result in segregation, or too high, which would result in concrete that is difficult to place. The VSI is a quick but approximate indication of the stability of the mixture; however, an acceptable VSI does not ensure adequate stability. In addition, a visual assessment of harshness can be made. When testing concrete in the laboratory or producing it in the field, a constant slump flow should be maintained for all mixtures because slump flow (yield stress) is the main characteristic distinguishing the workability of SCC from that of conventionally placed concrete. With the slump flow constant, the effects of changing proportions on filling ability, passing ability, and segregation resistance can be evaluated. Typically, the range of HRWRA dosages corresponding to the range of slump flows associated with SCC is small.

Target. Maximum and minimum slump flows should be established. The value of the required slump flow depends on the application and can vary from approximately 21 to 30 inches, with slump flows of 24-27 inches appropriate for most applications. The susceptibility to segregation increases significantly as the slump flow is increased above 27 inches. The ability to achieve higher slump flows than needed without segregation is one indication of robustness. Given the sensitivity of slump flow to small changes in materials and mixture proportions in general and HRWRA dosage in particular, a realistic range of slump flows—no less than 3 inches—should be used. It is advisable to measure T_{50} to detect variations in materials or mixture proportions in a given mixture; however, target values of T_{50} should be set broadly

because SCC mixtures with a wide range of T_{50} values can be placed successfully. When proportioning, lower values of T_{50} are preferably; however, sufficiently low T_{50} values can be associated with poor stability. T_{50} values between 2 and 7 seconds are appropriate for most applications (inverted cone orientation). It is advisable to monitor VSI; however, concrete should not be accepted or rejected on the basis of VSI.

10.1.1.2 Passing Ability

Description. Passing ability describes the ability of concrete to flow through confined conditions, such as the narrow openings between reinforcing bars. Although increasing the filling ability typically increases passing ability, a high level of filling ability does not assure passing ability.

Application Dependency: High. Applications may range from unreinforced or lightly reinforced sections with no passing ability requirements to narrow sections containing highly congested reinforcement with strict passing ability requirements.

Achieving. Passing ability is primarily affected by the aggregate characteristics and the paste volume. Reducing the maximum aggregate size and coarseness of an aggregate grading and improving the aggregate shape and angularity result in increased passing ability. Increasing the paste volume reduces the volume of aggregates and reduces the interparticle friction between aggregates. In addition, reducing the paste yield stress or viscosity improves passing ability.

Testing: J-Ring (ASTM C 1621). Passing ability should be measured with the j-ring because it provides an independent measurement of passing ability. The j-ring test can be evaluated by measuring either the difference in height between the inside and outside of the ring (Δ height) or the difference in slump flow measured with and without the ring. It is strongly recommended that the difference in height be measured because (1) the difference in slump flow with and without the j-ring is often within the precision of the slump flow test and (2) the difference in slump flow may not reflect the extent of blocking (such as when the thickness of the concrete flowing out of the j-ring is thinner than for the concrete tested without the j-ring—due to differences in blocking—but the spread is approximately the same). The size and spacing of reinforcement bars should remain constant while the maximum value for the change in height should be established for the application.

Target. The maximum j-ring Δ height should be specified based on the amount of reinforcement and clear spacing between reinforcement. Additional field testing is needed to establish specific values of j-ring Δ height for different applications. There is no need to measure passing ability for unreinforced or lightly reinforced elements.

10.1.1.3 Segregation Resistance

Description. Segregation resistance describes the ability of concrete to remain uniform in terms of composition during placement and until setting. Segregation includes both static and dynamic stability. Static stability describes segregation resistance when concrete is at rest. Dynamic stability describes segregation resistance with concrete is not at rest—such as during mixing and placing.

Application Dependency: Low. All mixtures must exhibit segregation resistance. Requirements for dynamic stability may be higher for sections with highly congested reinforcement, conditions where concrete is subjected to vibration, or applications where concrete is dropped from vertical heights or required to flow long horizontal distances.

Achieving. Segregation resistance encompasses both static and dynamic stability. Static stability is affected by the relative densities of the aggregate and paste, the rheology of the paste with time, the aggregate shape and grading, and the characteristics of the element (such as width and spacing of reinforcement). Changing the paste rheology is generally the most productive means of improving static stability. Improving the aggregate grading is also effective for reducing segregation resistance, though to a much lesser extent than changing the paste rheology. An SCC mixture with an aggregate that is well-graded for segregation resistance can exhibit severe segregation if the paste rheology is improper. The paste should have sufficiently high yield stress, plastic viscosity, and thixotropy. Dynamic stability is mainly affected by the cohesiveness and passing ability of the concrete.

Testing: Column Segregation (ASTM C 1610). Static stability should be measured with the column segregation or sieve stability test. In the United States, it is likely that the column segregation test will be used because it has been standardized by ASTM International. No test method is available for dynamic stability; therefore, dynamic stability is usually measured indirectly with measurements of filling and passing ability.

Target. For the column segregation test, the maximum segregation should be less than 15% for most cases but may need to be reduced in some applications. The sampling conditions should be well defined. To qualify mixtures prior to production, tests should be performed over the range of water contents and slump flows possible during production. Concrete with adequate filling ability, passing ability, and static stability are likely to exhibit dynamic stability. For cases with demanding dynamic stability requirements—such as with long lateral flow or free-fall distances—full-scale field mock-ups may be necessary.

10.1.2 Workability Testing

Testing requirements vary between the laboratory and field. To qualify mixtures in the laboratory prior to production, the slump flow, j-ring, and column segregation tests should be used to evaluate filling ability, passing ability, and segregation resistance, respectively. The robustness of each of these characteristics should be evaluated by varying the water content and slump flows over the ranges expected to be encountered in production. Full-scale mock-ups of proposed mixtures should be tested in the field prior to production. During production, it is often only necessary to perform the slump flow test. It is not generally necessary to measure passing ability during production because passing ability mainly depends on the aggregates, paste volume, and slump flow. If the aggregates and paste volume remain reasonably constant and the slump flow is controlled during production, further testing of passing ability is not required. Further, it is not generally necessary to measure segregation resistance during production because segregation resistance mainly depends on the paste rheology. If the paste rheology is controlled by ensuring that slump flow and T_{50} are within the ranges associated with acceptable segregation resistance established during mixture pre-qualification, further testing of segregation resistance is not required.

The slump flow spread should be used in the field to verify that the HRWRA dosage is correct while T_{50} should be used to evaluate unexpected variations in mixture proportions (most likely water content). The j-ring test does not normally need to be used in the field because passing ability primarily depends on the aggregates and paste volume and to a much lesser extent on paste rheology. As long as the aggregates and paste volume remain reasonably consistent in the field and the slump flow test is used to ensure proper concrete rheology, it is not necessary to measure passing ability in the field. The column segregation test is too time-consuming for use in the field. In performing the column segregation test in the laboratory, representative sampling is crucial. When using the column segregation test to qualify mixtures, it is especially important to test a range of water contents and slump flows because (1) segregation resistance is highly dependent on paste rheology and (2) it is possible for the paste rheology to vary substantially due to small variations in slump flow and water content (such as from variations in aggregate moisture conditions). If tests are conducted in the laboratory with the range of paste rheology expected to be encountered during production—by varying the water content and slump flow (by adjusting HRWRA dosage)—no further segregation testing is required in the field provided the slump flow test is used to monitor concrete rheology indirectly (with slump flow and T_{50}).

Given the greater sensitivity of SCC workability to changes in mixture proportions and the greater consequences of inadequate workability, workability tests during production should be performed with no less frequency for SCC than for conventionally placed concrete. When starting production—with a new mixture, new materials, or different placement conditions—all batches should be tested for slump flow until satisfactory results can be obtained consistently. Subsequent tests can be conducted at similar frequency as the slump test for conventionally placed concrete. At a minimum, a visual assessment should be made of every batch. It is also advisable to monitor mixer amperage as a way of detecting variations in workability.

10.1.3 Other Considerations

10.1.3.1 Rheology

Rheology can be used to characterize concrete flow characteristics and to optimize mixtures for filling ability, passing ability, and segregation resistance. Rheology involves measuring yield stress, plastic viscosity, and thixotropy. Yield stress describes the stress to initiate (static yield stress) or maintain (dynamic yield stress) flow. The yield stress should be near zero to ensure concrete flows under its own mass. Plastic viscosity describes the resistance to flow once the yield stress is exceeded. Mixtures with high plastic viscosity appear sticky and cohesive. Plastic viscosity should not be too low, which would result in instability, or too high, which would result in mixtures that are difficult to pump and place. Thixotropy describes the reversible, time-dependent reduction in viscosity of a concrete subjected to deformation (shearing). Thixotropy is caused by the build-up of a structure in fresh concrete at rest. This structure, which provides an initial resistance to deformation, is destroyed upon application of sufficient deformation to the concrete. Thixotropy, which is manifested in the difference between static and dynamic yield stress or the breakdown area between upward and downward rheometer flow curves, contributes to increased segregation resistance and reduced formwork pressures. Too much thixotropy; however, reduces placeability.

Concrete rheology is a function of the aggregates, paste volume, and paste rheology. Angular and poorly shaped aggregates increase yield stress and plastic viscosity. Increasing the

paste volume reduces yield stress and plastic viscosity. If the aggregates and paste volume are held constant, changes in paste rheology are generally matched in concrete rheology (e.g. increasing paste yield stress and viscosity increases concrete yield stress and viscosity). To increase filling ability and passing ability, the yield stress and plastic viscosity should be reduced. If the yield stress and plastic viscosity are too low; however, the concrete may become unstable, resulting in reduced filling and passing abilities. To increase segregation resistance, the yield stress and plastic viscosity should generally be increased.

Rheology is normally measured with a rheometer; however, certain empirical tests are correlated with rheological parameters. Specifically, reductions in yield stress generally result in higher slump flows while increases in plastic viscosity generally result in higher T_{50} and v-funnel flow times. Even if rheology parameters are not measured with a rheometer, considering workability in terms of rheology is often useful.

10.1.3.2 Form Surface Finish

The form surface finish can be better with SCC than conventionally placed concrete; however the quality of the form surface finish depends on the workability of the SCC, the formwork, and the form oil (Djelal et al. 2002; EFNARC 2005).

10.1.3.3 Vibration

Not only is vibration not needed for SCC, it may be detrimental (EFNARC 2005; Chan; Chen, and Liu 2003; Safawi, Iwaki, and Miura 2005). Vibration decreases the already low yield stress and destroys any built-up thixotropic structure, thereby greatly increasing the susceptibility to segregation, especially for low-viscosity concrete. Vibration should only be used in cases where prior testing has demonstrated that vibration will not cause adverse results. Potential instances where vibration may be appropriate include the location of cold joints and in placements of concrete with low slump flow. Vibration should not be used as extra “insurance” to ensure full consolidation without prior testing.

Similarly, vibration applied to SCC during transit, such as from jolts as the transport equipment passes over bumps, can be detrimental. Concrete that has segregated due to such vibration can be remixed to restore uniformity. It may be necessary to limit slump flows if the available transport equipment is not capable of remixing concrete prior to discharge.

10.1.3.4 Cold Joints

Cold joints are possible if extended delays occur in placement (Roussel 2006; EFNARC 2005; PCI 2003). The build-up of a thixotropic, at-rest structure in concrete may reduce the mixing and bonding of lifts. In conventionally placed concrete placements, vibration is used to destroy the thixotropic, at rest structure and, therefore, promote mixing and bonding between layers.

10.1.3.5 Air Content

The air content of non-air entrained concrete is typically similar to that in fully consolidated conventionally placed concrete. SCC can be successfully air entrained. Due to the potential for variations in air content because of variations in certain materials such as fly ash and HRWRA, it may be advisable to test the air content of non-air-entrained SCC.

10.1.3.6 Horizontal Flow Distance and Free-Fall Height

Due to the cohesive nature and high segregation resistance of SCC, it may be possible to increase the permissible lateral flow distance and free-fall height (PCI 2003). It is still advisable to minimize the lateral flow distance and free-fall height and to conduct tests to establish maximum permissible distances (EFNARC 2005).

10.1.3.7 Workability Retention

Workability retention and setting time are different properties and should be evaluated separately. Workability retention depends on the HRWRA type and dosage, retarder type and dosage, mixture proportions, concrete temperature, weather conditions, and degree of agitation. It is critical that SCC exhibit proper workability at the time of placement. The extent of workability retention should be tailored to the application because excessive workability retention is unnecessary and may increase formwork pressure and susceptibility to segregation.

10.1.3.8 Setting Time

For a given application, the setting times of SCC are typically equal to or slightly longer than those for conventionally placed concrete, although they may be shorter. As with conventionally placed concrete, the setting times should be evaluated under the anticipated placement temperatures. The application of accelerated curing may need to be adjusted accordingly.

10.1.3.9 Hot and Cold Weather Placement

The same practices for hot and cold weather concreting for conventionally placed concrete are generally appropriate for SCC (PCI 2003). Likewise, the minimum and maximum concrete curing temperature limits do not need to be changed for SCC.

10.1.4 Evaluation of SCC Mixture Proportions

10.1.4.1 Materials

SCC is generally made with the same materials as conventionally placed concrete; however, the effects of material characteristics are often greater in SCC mixtures. Therefore, specifications for materials do not need to be changed; however, it is advisable to monitor material characteristics more frequently. When any changes in materials occur, trial mixtures should be reevaluated for workability and hardened properties.

SCC mixtures always include HRWRA. HRWRAs are typically polycarboxylate-based and may be tailored by the manufacturer for a given application, such as for SCC in precast concrete in certain regions of the country. The characteristics of HRWRAs can vary widely between individual products. Therefore, testing with local materials is critical to understanding the characteristics of a HRWRA particularly with respect to rheology, workability retention, and setting time. All HRWRAs can reduce the yield stress to near zero; however, the associated plastic viscosity can vary widely. In some cases, a water-reducing admixture or mid-range water-reducing admixture can be used to reduce HRWRA demand. A retarder can be added to increase workability retention. Some commercial HRWRA products may include a retarder or VMA.

Viscosity modifying admixtures are used in certain cases, such as when high coarse aggregate contents, poorly shaped aggregates, or high water-powder ratios are used. They can enhance stability and reduce the effects of changes in materials and mixture proportions. VMA tend to be most effective at higher w/p. The chemical formulation and effects on workability and hardened properties vary between commercial products.

10.1.4.2 Mixture Proportions

Mixture proportions can vary depending on the available materials and placement conditions, the workability and hardened property requirements, and the mixture proportioning procedure used. For a given set of materials and application requirements, there is commonly more than one acceptable mixture proportioning solution. Mixture proportions that perform successfully in one plant are likely of little relevance for a different plant with different materials.

The selection of mixture proportions should be the responsibility of the precaster and should be developed by someone experienced with SCC. The suitability of mixture proportions should be evaluated in both laboratory and production trial batches. The following indices should be considered when evaluating mixture proportions:

- **Aggregates.** Aggregates are defined in terms of maximum size, grading, and shape and angularity. All three of these factors should be considered together. For instance, adding a poorly shaped aggregate to improve grading may be adverse.
 - **Maximum Size.** The maximum aggregate size guidelines for conventionally placed concrete (such as ACI 211 requirements of one-fifth the narrowest dimension between side forms, one-third the depth of slabs, or three-fourth the

minimum clear spacing of bars) are generally applicable to SCC. Reducing the maximum aggregate size may improve passing ability and segregation resistance; however, these benefits must be measured against any necessary increases in paste volume and associated direct or indirect effects on hardened properties.

- **Grading (Sand-Aggregate Ratio).** The combined grading of all aggregates should be considered. The S/A ratio is normally between 0.40 and 0.50 for SCC to ensure passing ability and segregation resistance. Although the use of higher S/A usually results in lower coarse aggregate volume, it may allow a lower paste volume (and higher total aggregate volume). Severely gap-graded aggregate blends should be avoided; however, slightly gap-graded mixtures can be acceptable and may be preferable to more uniform gradings. Although grading does affect segregation, the rheology of the paste is likely to be significantly more influential.
- **Shape and Angularity.** Aggregate shape and angularity significantly affect workability; however, aggregates of any shape and angularity can be accommodated in SCC. Well-rounded aggregates with few or no flat or elongated particles allow the use of lower paste volume and result in lower viscosity and HRWRA demand. Texture does not affect workability significantly; however, it can affect hardened properties substantially.
- **Paste Volume.** The paste volume, which can range from 28% to 40% in SCC, must be sufficient for ensuring filling ability and passing ability. The minimum required paste volume primarily depends on the aggregate characteristics. Well-shaped and well-graded aggregates with high packing density require significantly less paste volume. The paste volume should be minimized, which results in improved hardened properties and economy. Increasing the paste volume can enhance robustness with respect to any variations in aggregate properties.
- **Paste Composition.** The paste composition is defined in terms of the relative amounts of water, powder, and air and the blend of powder. The same volume of paste is needed regardless of whether powder-type SCC ($w/p < \sim 0.40$ and high powder content) or VMA type ($w/p > \sim 0.45$ and lower powder content) is used. The required paste composition for a given set of rheological characteristics depends on the aggregates and paste volume. Once the paste composition is set, admixture dosages are established.
 - **Water Content.** The water content per unit volume of concrete in SCC is often similar to or slightly higher than in conventionally placed concrete.
 - **Water-Cement Ratio.** The w/c mainly relates to early-age hardened properties when SCMs and fillers with minimal early-age activity are used. In such cases, the w/c in SCC should normally be similar to that in conventionally placed concrete.
 - **Water-Cementitious Materials Ratio.** The w/cm , in conjunction with the powder blend, mainly relates to long-term hardened properties.
 - **Water-Powder Ratio.** The w/p , in conjunction with the powder blend, mainly relates to workability. Depending on the mixture proportioning approach, it may range from 0.25 to 0.45 or higher. The need for VMA increases as the w/p increases above 0.40. If no non-cementitious powders are used, the w/cm is equal to the w/p .

- **Powder Content and Blend.** The powder content must be set in conjunction with the w/p to ensure proper workability. The blend of powders affects workability and hardened properties.
 - **Cement Content.** The cement content should generally be minimized and the remainder of the powder content should be composed of SCMs, mineral fillers, or both. For a given release-of-tension strength level, the cement content and w/c in SCC should normally be similar to that in conventionally placed concrete depending on the reactivity of any SCMs or mineral fillers.
 - **SCMs and Mineral Fillers.** As with conventionally placed concrete, the size distribution and shape of the SCMs and mineral fillers mainly affect the workability. Due to their low reactivity at early ages, fly ash, slag, and mineral fillers typically reduce early heat of hydration as compared to cement. SCMs can increase long-term strength and durability.
- **Air Content.** SCC can be air entrained. For non-air entrained concrete, air contents of 1-2% are typical.

As the aggregates, paste volume, or paste composition are changed, the amount of HRWRA required to reach the target slump flow varies. HRWRA mainly affects slump flow. Therefore, the HRWRA dosage is typically adjusted to reach the target slump flow once the aggregates, paste volume, and paste composition are set. A certain target slump flow can usually be achieved with a wide range of aggregates, paste volumes, and paste compositions merely by adjusting the HRWRA dosage. In addition, VMA and other chemical admixtures may be added to assure adequate concrete performance.

The roles of aggregates, paste volume, and paste composition in achieving adequate filling ability, passing ability, and segregation resistance are summarized in Table 10.1.

Hardened properties should be evaluated in the same manner as for conventionally placed concrete. The relationships between hardened properties and materials and mixture proportions for conventionally placed concrete generally apply to SCC. Certain modifications to mixture proportions needed to ensure workability may affect hardened properties. These modifications may include higher paste volume, increased sand-aggregate ratio, and reduced maximum aggregate size. Conversely, requirements for hardened properties may result in limits on certain parameters important to achieving workability, such as cement content, paste volume, and water-cementitious materials ratio. In some applications, the low water-cementitious materials ratios and use of SCMs used to achieve workability result in hardened properties that significantly exceed design requirements.

Modifications to mixture proportions during production should be kept to a minimum. Once mixing commences, modifications should be only made with HRWRA or VMA or by pausing to let the concrete lose workability with time. It is advisable to pre-test mixtures to understand the effects of re-tempering with HRWRA or VMA. For example, the change in slump flow per change in dosage of HRWRA can be established. The criteria for accepting and modifying mixtures should be clearly established and personnel responsible for evaluating mixtures and making changes should be well trained.

Table 10.1 Proportioning for SCC Workability

Property	Aggregate	Paste Volume	Paste Composition
Filling Ability	Improve shape and angularity to reduce interparticle friction, use finer grading to reduce harshness or coarser grading to reduce viscosity	Ensure sufficient minimum paste volume to fill voids between aggregates and reduce interparticle friction between aggregates	Ensure viscosity is not too high (sticky) or too low (instability); increase HRWRA dosage to increase slump flow
Passing Ability	Reduce amount of larger particles by reducing coarseness of grading or maximum aggregate size, improve shape and angularity to reduce interparticle friction	Increase paste volume to reduce aggregate volume and interparticle friction between aggregates	Reduce paste viscosity or increase HRWRA dosage to increase slump flow
Segregation Resistance	Use more uniform grading (avoid gap gradings), reduce coarseness of aggregate grading or maximum aggregate size	Increase paste volume	Ensure paste viscosity not too high or too low, reduce slump flow (lower HRWRA dosage), optimize workability retention (accelerate loss of slump flow in formwork), use VMA

10.2 Suggested Changes to TxDOT Specifications and Test Methods

The following TxDOT documents were reviewed with respect to precast concrete to identify changes necessary for the use of SCC:

- 2004 TxDOT “Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges”
 - Item 421. “Hydraulic Cement Concrete,” as amended by Special Provision 421-024
 - Item 424. “Precast Concrete Structures (Fabrication)”
- Department Material Specifications: DMS-7300 “Precast Concrete Fabrication Plants” effective January 2006
- TxDOT Test Procedures
 - 400-A Series (Concrete)
 - 700-I Series (Structural)

To specify SCC, it is first necessary to define SCC. SCC differs from conventionally placed concrete in terms of mixture proportions and workability. The benefits of SCC are mostly realized during the placement process. Therefore, the following two definitions are needed for specifications:

Self-Consolidating Concrete Workability: The ability of concrete to flow under its own mass without vibration, pass through intricate geometrical configurations, and resist segregation, all to the extent necessary for the given application. At a minimum, concrete must exhibit a slump flow of 21 inches and resist segregation to be considered to have SCC workability.

Self-Consolidating Concrete: Concrete that, during placement, exhibits self-consolidating concrete workability.

SCC workability must be controlled to ensure that the hardened concrete is adequately consolidated and uniform in composition. The mixture proportions of SCC must be controlled to ensure that adequate workability and long-term performance are achieved. A concrete with SCC workability has SCC mixture proportions and the hardened properties associated with those mixture proportions; however, a concrete with SCC mixture proportions and the hardened properties associated with those mixture proportions does not necessarily have SCC workability.

10.2.1 General Approach for SCC Specifications

As with conventionally placed concrete, the specification of SCC involves two tasks—mixture qualification and quality control/quality assurance.

- **Mixture Qualification.** The performance of each proposed mixture—in terms of workability and hardened properties—must be demonstrated prior to placement in preliminary trial batches (conducted in laboratory) and pilot test batches (conducted under production conditions).
- **Quality Control (Producer)/Quality Assurance (TxDOT).** Due to sensitivity of SCC workability to small changes in materials or mixture proportions and to the potentially severe consequences of improper workability, proper quality control of all SCC batches is imperative. The constituent material properties should be monitored. The slump flow must be maintained within the range established during the qualification of mixtures. In addition, a minimum T_{50} should be maintained. In doing so, the yield stress, viscosity, and thixotropy are likely to be suitable to ensure adequate filling ability, passing ability, and segregation resistance. The producer should submit a detailed quality control plan for review by TxDOT.

The yield stress—which is related to slump flow—is the main fundamental difference between with workability of SCC and conventionally placed concrete. The yield stress is likely to vary most significantly during production. Changes in yield stress are likely to have the largest effect on workability performance (filling ability, passing ability, and segregation resistance). In addition, variations in water content due to changes in aggregate moisture content can have significant effects on workability. Therefore, to qualify mixtures, the workability should be evaluated in preliminarily laboratory trial batches over a range of water contents and slump flows anticipated during production and the associated filling ability, passing ability, and segregation resistance should be evaluated. Then, pilot trial batches should be conducted to ensure that the filling ability, passing ability, and segregation resistance measured with the laboratory tests can be achieved in field production conditions. During regular production, the slump flow should be maintained within a range of slump flows associated with suitable filling ability, passing ability, and segregation resistance, as established earlier in laboratory testing. A minimum T_{50} should be maintained to ensure adequate viscosity.

10.2.2 Changes to TxDOT Standard Specifications

The 2004 TxDOT Standard Specifications include provisions that preclude or limit the use of SCC due to provisions restricting mixture proportions and workability. All of these

provisions, however, include language for TxDOT to allow exceptions (e.g. “unless otherwise approved”). To permit the use of SCC in precast members, the following provisions must be addressed:

- **Required gradings for fine and coarse aggregates (§421.2.E.).** The fine and coarse aggregates gradings in Table 3 and 4, respectively, can be used successfully in SCC; however, they may be unnecessarily limited for SCC. Although this provision does not restrict the use of SCC, it may limit the ability of precasters to optimize SCC proportions. In particular, the use of an intermediate sized aggregate may be beneficial in some cases. The provision of a maximum aggregate size of at least 1/2 inch is acceptable. Gradings optimized for conventionally placed concrete may not be suitable for SCC.
- **Required use of ACI 211 or other approved procedure to develop mixture designs. (§421.4.A.).** The ACI 211 mixture design procedure is inappropriate for SCC. The paste volumes are likely to be too low, the coarse aggregate contents too high, and the water-powder ratios and aggregates gradings inappropriate. Numerous mixture design methods are available for SCC. These methods vary widely in scope, approach, and results. It is not recommended that TxDOT endorse or require the use of a specific procedure.
- **Maximum cementitious materials content of 700 lb/yd³, unless otherwise specified approved (§421.4.A.1.).** SCC can be produced with powder contents (cementitious materials and mineral fillers) of less than 700 lb/yd³; however, in order to achieve simultaneously the high levels of passing ability and high early strengths, it is highly likely that total powder contents above 700 lb/yd³ will be needed. In most cases in Texas, the powder content is likely to be composed of cementitious materials only. Therefore, the limit on cementitious materials content should be eliminated. The maximum curing temperature limits in §424.3.B.7. can be used to control temperature directly. Alternatively, the cement content, not cementitious materials content, could be limited to 700 lb/yd³, unless otherwise specified or approved. It may be necessary to specify or approve cement contents greater than 700 lb/yd³ to ensure release-of-tension strength, but not to ensure workability because high power contents—not high cement contents—may be needed for workability.
- **Maximum slump of 6.5 inches without HRWRA and 9 inches with HRWRA, unless otherwise specified (§421.4.A.5. and §424.3.B.5.).** For SCC, the requirements for slump should be replaced by requirements for filling ability (slump flow test), passing ability (j-ring test), and segregation resistance (column segregation test).

The 2004 Standard Specifications include provisions that do not preclude the use of SCC, that are useful for ensuring concrete quality, and that can be retained. These provisions include but are not limited to the following:

- Requirements for material properties (§421.2.).
- Maximum water-cementitious materials ratio of 0.45 for precast concrete (Class H) (§421.4.A.).
- Mixture design options to increase durability (§421.4.A.6.).
- Requirement of trial batches to verify that concrete conforms to specification requirements (§421.4.B.).
- Limits on fresh concrete placement temperature of 50 to 95°F and hot and cold weather concrete placement requirements (§424.3.B.5.).

- Curing requirements, including limits on maximum concrete temperature during curing (§424.3.B.7.).

The 2004 Standard Specifications include provisions that can be changed to allow precasters to take advantage of the properties of SCC. These provisions include the following:

- **Limits on lateral flow and free-fall height (§424.3.B.5).** Testing is required to determine the extent to which these limits can be increased.
- **Requirements on vibration (§424.3.B.5).** These restrictions can be voided for SCC. Vibration is only necessary if a concrete mixture does not exhibit SCC workability at the time of placement. Vibration should not be applied to concrete with SCC workability unless it can be shown to not be adverse.

In addition to these changes, it is necessary to include provisions specific to SCC to ensure the quality of the finished concrete product. The suggested specifications for SCC workability are shown in Table 10.2. The testing requirements in §421.4.B. and §421.4.G.2. should be updated so that SCC workability is demonstrated in terms of slump flow and T_{50} during production and in terms of slump flow, T_{50} , j-ring, and column segregation for the qualification of mixtures (preliminary laboratory trial batches).

The specifications in Table 10.2 require that the producer establish a target slump flow range to be used during production. The advantage of a wide slump flow range is greater tolerance for variations during production. The disadvantage of a wide slump flow range is that mixtures with higher slump flows generally require greater robustness, especially to ensure segregation resistance, and are, therefore, more difficult to proportion. Mixtures with high slump flows are more likely to exhibit segregation because of their low yield stresses. Therefore, in selecting a target slump flow, the producer must balance the benefit of a wide slump flow range with the challenge of demonstrating adequate robustness in high slump flow mixtures. Due to the minimum slump flow range of 3 inches, all producers should, in effect, be able to produce mixtures with slump flows of 24 to 27 inches. Producers with better mixture proportions and quality control measures will be able to produce a wider range of slump flows. No producer will be able to use a mixture that has not been proven to be robust in terms of passing ability and segregation resistance for the given slump flow range.

To demonstrate robustness, the producer must modify simultaneously the water content and slump flow over the range expected to be encountered in production. The slump flow should be varied by adjusting the HRWRA dosage. (If the HRWRA dosage is held constant as the water content is varied, the variation in slump flow is likely to be large and it is likely to be extremely difficult, if not impossible, to demonstrate adequate robustness. During production, the HRWRA dosage should also be adjusted to maintain the target slump flow range.) The water content should be varied based on the expected variations in moisture content. The required 3% variation in water content is based on an approximate 0.5% variation in fine aggregate moisture content in typical SCC mixtures and can be changed to be more representative of specific mixtures and producers. For each variation in water content and slump flow, the concrete should exhibit the required workability properties shown in the mixture qualification column in Table 10.2 and be subject to the additional specification requirements shown in Table 10.3.

The limits on slump flow were selected because slump flows lower than 24 inches may not consolidate fully in prestressed members and slump flows greater than 30 inches are typically much more susceptible to segregation. In lightly reinforced or unreinforced precast members,

lower slump flows may be acceptable. The minimum T_{50} of 2 seconds is provided for stability. A maximum T_{50} is not needed because higher T_{50} values are generally not adverse. Producers will likely prefer lower T_{50} values because of the increased placeability. Limited data is available to establish a maximum j-ring Δ height value. The value of 0.5 inches is likely conservative and may be increased with further field testing. The maximum column segregation test result is based on the laboratory segregation test results in this research project and the correlation to the sieve stability test, for which more field experience is available in the literature. The values for the slump flow, j-ring, and column segregation tests are conservative and may be modified as experience with SCC in prestressed concrete beams in Texas increases.

Although SCC may have higher paste volume, higher powder content, lower coarse aggregate content, and higher sand-aggregate ratio than conventionally placed concrete, it is not recommended that limits be placed on these parameters. Producers should be encouraged to develop mixture proportions that minimize paste volume and powder content and to optimize aggregate grading in general and to maximize coarse aggregate content in particular. These parameters must be optimized within the context of the available materials and application requirements and may vary widely. Therefore, it is not appropriate to impose broad limits on mixture proportioning parameters.

Table 10.2 Suggested Target Properties and Specification for SCC Workability for Precast Concrete

General Guidelines (Not for Specifications)			Specification Requirements	
			Mixture Qualification	Production QC
Slump Flow (in.)	21-24	Appropriate for members with light or no reinforcement, short lateral flow distances, or high placement energy	24-30 in. at the expected time of placement (lower slump flows permissible in lightly reinforced or unreinforced members)	24-30 in. at the time of placement (lower slump flows permissible in lightly reinforced or unreinforced members)
	24-27	Ideal for most applications		
	27-30	Appropriate for members with highly congested reinforcement, long lateral flow distances, or low placement energy		
	30-33	Possible, but a high risk of segregation		
T ₅₀ (s)	<2	Poor stability	>2 s (inverted cone orientation)	>2 s (inverted cone orientation)
	2-7	Acceptable, should not vary over range of 3 s between batches		
	>7	Possible if needed due to limits on hardened properties; may reduce placeability		
J-Ring Δheight (in.)	<0.5	Appropriate for members with highly congested reinforcement	<0.5 for highly congested reinforcement <1.0 for moderately congested reinforcement None for unreinforced or light reinforcement	None
	0.5-1.0	Appropriate for members with moderately congested reinforcement		
	1.0	Appropriate for unreinforced or lightly reinforced members		
	>1.0	Appropriate for unreinforced or lightly reinforced members		
Column Segregation (%)	<5	Highly segregation resistant	<15%	None
	5-10	Segregation resistant		
	10-15	Borderline segregation resistant		
	>15	Not segregation resistant		

Table 10.3 Additional Specification Requirements

Qualification of Mixtures.

Laboratory Trial Batches. Each proposed SCC mixture shall be accepted on the basis of slump flow, T_{50} , j-ring Δ height, and column segregation measured in laboratory trial batches. The producer shall propose and the engineer shall approve a range of target slump flows that will be used in production. The range of target slump flows shall be at least 3 inches. The proposed SCC mixture shall exhibit adequate T_{50} , VSI, j-ring Δ height, and column segregation results in laboratory trial batches when measured over the range of target slump flows and the expected variation in water content (+/- 3% of water content unless specified otherwise). The HRWRA dosage should be varied as the water content is changed to reach the target slump flows. The range of slump flows and water contents shall be tested simultaneously (maximum and minimum slump flows with minimum water content, maximum and minimum slump flows with maximum water content). Mixtures must exhibit adequate workability retention to achieve the target slump flow and associated workability at the expected time of placement.

Pilot Test Batches. The producer shall, at the Engineer's option, construct a full-scale mock-up section to demonstrate that the proposed mixture exhibits filling ability, passing ability, and segregation resistance, as evidenced by visual observations of the concrete flow around reinforcing bars and the lack of defects such as excessive honeycombing, aggregate or mortar pockets, and surface air voids (bugholes). Only one batch within the range of target slump flows is necessary for the pilot test batches.

Testing During Production. The slump flow and T_{50} shall be tested during production. The frequency of testing shall be established by the Engineer and shall not be less than the slump test for conventionally placed concrete. Mixtures with slump flows outside the target range at the time of placement shall be retempered, allowed to lose slump flow over time, or not used. Mixtures allowed to lose slump flow over time must be remixed prior to placement.

Production Quality Control Plan. The producer shall submit to the Engineer a quality control plan. The quality control plan must address how the producer will ensure consistent quality for all batches. At a minimum, all batches should be visually evaluated by qualified quality control personnel. A certain minimum number of batches should be measured for slump flow and T_{50} (e.g. first X batches and every subsequent Y^{th} batch). Other potential quality control measures include increased monitoring of moisture content and material properties, monitoring of mixer performance (amperage meter), and increased slump flow testing. Procedures for accepting and modifying mixtures should be established.

Adjusting Mixtures During Production. If the slump flow is less than the target, the mixture can be retempered with HRWRA. If the slump flow is greater than the target, the mixture can be held until the slump flow is within the target range or VMA can be added (subject to maximum dosage limits). If the T_{50} is too low, VMA can be added (subject to maximum dosage limits). VMA maximum dosage limits shall be established for each mixture based on the effects on hardened properties.

10.2.3 Changes to Department Material Specifications

The following provisions of DMS-7300 should be modified:

- **Section I.B.1. and I.B.2 (Equipment, Furnishings and Laboratory Equipment, Multi-project Fabrication Plant and Project-specific Fabrication Plant).** In addition to the equipment listed in this section, it is necessary to furnish equipment for the slump flow test, j-ring test, and column segregation test. The equipment for performing the slump test need not be furnished when only SCC is being produced.
- **Section III. (Personnel Qualifications).** For the plant types listed, the quality control supervisor and technicians should be certified to perform the slump flow, j-ring, and column segregation tests. At the time of this writing, the certification programs listed in the specification do not address the testing of SCC.

- **Section IV. (Quality Responsibilities).** The following shall apply to Contractors at all plant types listed in this section.
 - **Quality Control Procedures.** The written quality control procedures should address SCC placement—including the target slump flow range, procedures to ensure consistent workability, and procedures for modifying mixtures during production.
 - **Material Sampling and Testing.** Table 10.2 should be amended to require the slump flow test to be performed at a frequency not less than the slump test (1 test for the first batch and 1 for each set of compressive strength cylinders; at least 3 total tests per casting line for prestressed members). The slump test does not need to be performed for SCC. It may be advisable to require that measurements of the moisture contents of fine and coarse aggregates be conducted at greater frequency.
 - **Inspection.** The sixth bullet item should be clarified to ensure that concrete is at minimum visually inspected for each batch and that concrete exhibits adequate filling ability, passing ability, and segregation resistance.

10.2.4 Changes to TxDOT Test Methods

Several of the TxDOT test methods should be modified for use with SCC. These suggested modifications are listed in Table 10.4. In addition, new test methods are needed for the slump flow, j-ring, and column segregation tests. These three test methods can be used directly as ASTM C 1611 (slump flow), ASTM C 1621 (j-ring), and ASTM C 1610 (column segregation), except as noted in Table 10.5.

Table 10.4 Suggested Changes to Existing TxDOT Test Methods

Test Method	Proposed Change
Tex-407-A, Sampling Freshly Mixed Concrete (08/99)	Section 2, Second Bullet Point: Modify as follows: All water <u>and chemical admixtures</u> used in mixing the concrete should be introduced into the batch and properly mixed prior to taking the sample. Section 3, Step 4: Modify as follows: Start tests for slump <u>or slump flow</u> and/or air content within five minutes after sampling is complete (see NOTE 2). <u>Do not perform test for air content if concrete is visibly segregated.</u> Section 3, Step 5: Modify as follows: Start molding of specimens for strength tests within 15 minutes after sampling is complete. <u>Do not mold specimens if concrete is visibly segregated.</u>
Tex-414-A, Air Content of Freshly Mixed Concrete by the Volumetric Method (12/04)	Section 4, Step 1: Add the following note: <u>NOTE: For concrete with slump flow greater than 610 mm (24 inches), place concrete in bowl in one layer. No external consolidation (by rodding or internal vibration) is required.</u>
Tex-415-A, Slump of Hydraulic Cement Concrete (12/04)	This test is not applicable to SCC.
Tex-416-A, Air Content of Freshly-Mixed Concrete by the Pressure Method (08/99)	Section 3, Procedures, Determining Air Content of Concrete, Step 2: Add the following note: <u>NOTE: For concrete with slump flow greater than 610 mm (24 inches), place concrete in measuring bowl in one layer. No external consolidation (by rodding or internal vibration) is required.</u>
Tex-417-A, Unit Weight, Yield, and Air Content (Gravimetric) of Concrete (08/99)	Section 4, Procedure: Obtaining Unit Weight of Concrete: Add the following note: <u>NOTE: For concrete with slump flow greater than 610 mm (24 inches), place concrete in the measure in one layer. No external consolidation (by rodding or internal vibration) is required.</u>
Tex-430-A, Slump Loss of Hydraulic Cement Concrete (02/05)	This test is not applicable to SCC.
Tex-440-A, Initial Time of Set of Fresh Concrete (08/99)	Section 4, Procedures, Preparing Mortar Specimen: Add the following note <u>NOTE: For concrete with slump flow greater than 610 mm (24 in.), consolidation of mortar by rodding is not required.</u>
Tex-447-A, Making and Curing Concrete Test Specimens (12/04)	Section 2, Part I, Compressive Strength Specimens (Cylinders), Consolidation: Add the following note: <u>NOTE: For concrete with slump flow greater than 610 mm (24 in.), place concrete in mold in one layer. No external consolidation (by rodding or vibration) is required.</u> Section 3, Part II, Flexural Strength Specimens (Beams): Add the following note: <u>NOTE: For concrete with slump flow greater than 610 mm (24 in.), place concrete in mold in one layer. No external consolidation (by rodding or vibration) is required.</u>
Tex-498-A, Minimum Standards for Acceptance of a Laboratory for Concrete and Aggregate Testing (08/99)	Add requirements for slump flow, j-ring, and column segregation tests.
Tex-704-I, Making, Curing, and Testing Compression Test Specimens for Precast Concrete (08/00)	No changes are needed if changes indicated above to Tex-447-A, Section 2, Part I are made.
Tex-715-I, Curing Release of Tension Strength Cylinders for Precast/Prestressed Concrete Products Using Match-Cure Technology (08/00)	No changes are needed if changes indicated above to Tex-447-A, Section 2, Part I are made.

Table 10.5 Suggested Changes to ASTM Test Methods

Test Method	Suggested Changes
ASTM C 1611/C 1611M-05, Slump Flow of Self-Consolidating Concrete	1.) Use only Filling Procedure B (Inverted Mold). 2.) The procedure to pre-moisten the base plate should be clarified to ensure that no standing water is on the base plate. It should be permissible to use a squeegee to remove standing water. 3.) The measurement of T_{50} should be mandatory and described in the main body of the standard. Measurements of T_{50} should only be recorded for slump flows greater than 21 inches. The measurement of VSI should remain non-mandatory, if it is included at all.
ASTM C 1621/C1621M-06, Passing Ability of Self-Consolidating Concrete by J-Ring	1.) The definition of passing ability should be given as “the ability of concrete to flow through confined conditions, such as the narrow openings between reinforcing bars.” 2.) The base plate should have inscribed circles to indicate the locations of the slump cone and j-ring. 3.) Use only Filling Procedure B (Inverted Mold). 4.) The procedure to pre-moisten the base plate should be clarified to ensure that no standing water is on the base plate. It should be permissible to use a squeegee to remove standing water. 5.) The measuring device should be capable of measuring to the nearest 1/8 inch. 6.) The difference in slump flow measured with and without the j-ring should not be measured. All language related to this measurement should be removed from the standard. 7.) The degree of passing ability should be determined as the average difference in concrete height between the immediate inside and outside of the j-ring, measured at 4 locations equally spaced around the j-ring.
ASTM C 1610/C 1610M-06, Static Segregation of Self-Consolidating Concrete Using Column Technique	1.) The aggregates may be brought to an oven-dry condition rather than a saturated surface dry condition. All aggregates should be brought to a similar moisture condition. 2.) If the concrete is visibly segregated prior to placing concrete into the column, every effort should be made to prevent segregation in the concrete used to fill the column. This condition should be noted and reported with the test results.

11. Summary and Conclusions

Research was conducted to evaluate the suitability of self-consolidating concrete for prestressed concrete bridge beams in Texas. Materials, test conditions, and application requirements were selected to be representative of those in prestressed concrete bridge beam plants in Texas. The workability of SCC was related to materials and mixtures proportions. A series of 16 SCC mixture proportions was developed and subsequently evaluated in terms of workability, early-age engineering properties (up to the time of release of tension), and shrinkage. The early-age engineering properties evaluated included setting time, calorimetry, and compressive strength and modulus of elasticity development. Separately, researchers at the Texas Transportation Institute evaluated the longer-term engineering properties of these mixtures. Test methods for workability were evaluated. An analysis of the factors contributing to segregation resistance was conducted. Field testing was conducted and recommendations were developed for specifying and inspecting SCC.

Based on the results of this research project, the following main conclusions can be reached:

- SCC is suitable for use in prestressed concrete bridge beams based on the properties addressed in this research report. The ability of SCC to pass through congested reinforcement and fully consolidate under its own mass and without segregation can result in improved concrete quality and increased construction productivity. However, greater quality control is needed to ensure SCC workability is obtained consistently. Any changes in hardened properties associated with SCC should be attributed to specific differences in mixture proportions and to the improved dispersion and extent of consolidation in SCC.
- The main challenge in developing SCC mixture proportions for use in prestressed concrete beams was in achieving simultaneously adequate passing ability, adequate release-of-tension compressive strength, and a cementitious materials content of less than 700 lb/yd³. It was necessary to exceed the maximum limit of 700 lb/yd³ of cementitious materials required by the 2004 TxDOT Standard Specifications. (Higher cementitious materials contents can be allowed under these specifications.) For a given nominal 16-hour compressive strength, the cement contents were similar in SCC and conventionally placed concrete mixtures; however, it was necessary to increase the total powder content by adding fly ash to ensure adequate paste volume, powder volume, and w/p for workability.
- A wide range of SCC mixture proportions can be developed for prestressed concrete bridge beam applications. Compared to conventionally placed concrete mixtures, SCC mixtures may exhibit some combination of higher paste volume, higher powder volume, lower water-powder ratio, increased sand-aggregate ratio, and reduced maximum aggregate size. The extent of the changes in these mixture proportioning parameters depends on the local materials, application requirements, and mixture proportioning procedure. For the 16 final SCC mixture proportions, which were developed with nominal 16-hour compressive strengths levels at 5,000 and 7,000 psi based on a defined curing temperature history and with either a river gravel aggregate set or crushed limestone aggregate set, it was necessary to increase the paste volume to ensure passing

ability. Within the SCC mixtures, the required paste volumes were higher for mixtures with lower sand-aggregate ratios and with the angular crushed limestone aggregate set. For a given nominal 16-hour compressive strength level, most of the increase in paste volume for the SCC mixtures was achieved with the addition of fly ash—the water content was only slightly increased and the cement content was approximately unchanged.

- The workability of SCC mixtures can be highly sensitive to changes in materials and mixture proportions. The ranges of HRWRA dosages and water contents associated with SCC workability are often narrow. The consequences of inadequate workability can be much greater in SCC than in conventionally placed concrete.
- The setting times of the SCC mixtures were similar to the conventionally placed concrete mixtures. The setting times of the SCC mixtures were slightly longer for the nominal 5,000 psi 16-hour strength level and equal or slightly shorter for the nominal 7,000 psi 16-hour strength level, when compared to the conventionally placed concrete mixtures.
- The heat generated by the SCC mixtures, which was evaluated with isothermal and semi-adiabatic calorimetry, was slightly delayed at early ages compared to conventionally placed concrete. Over time, however, the SCC mixtures generated greater total heat.
- As with conventionally placed concrete, the early-age compressive strengths of the SCC mixtures varied widely depending on the test time and curing temperature. For instance, the nominal 5,000 psi 16-hour compressive strength mixtures varied in compressive strength from 4,500 to 8,000 psi at 16 hours depending on the curing temperature history.
- For a given compressive strength level, the static modulus of elasticity was slightly lower for SCC mixtures than for the comparable conventionally placed concrete mixture. The static modulus of elasticity was measured from 8 hours to 28 days. The S/A ratio had no effect on static modulus of elasticity.
- For a given 16-hour nominal compressive strength level, the 112-day shrinkage of the SCC and conventionally placed concrete mixtures were similar. Only the SCC mixtures containing the PT-1482 admixture exhibited higher shrinkage than the comparable conventionally placed concrete mixtures with the same 16-hour nominal compressive strength level.
- SCC is susceptible to segregation due to the low yield stresses needed for self-flow. The static yield stress must be sufficiently high for segregation resistance while the dynamic yield stress must be sufficiently low for self-flow. The static yield stress at any time is a function of the initial yield stress, the loss of workability, and thixotropy. Higher plastic viscosity can slow the descent of particles; however, the magnitude of plastic viscosity provides no assurance of segregation resistance.
- SCC workability should be defined and measured in terms of filling ability, passing ability, and segregation resistance. Rheology can be used to provide additional insights into workability. The main difference between SCC and conventionally placed concrete workability is the low yield stress (high slump flow) required to achieve self-flow in SCC. The plastic viscosity should not be too low, which would result in poor stability, or too high, which would result in reduced placeability. HRWRA is mainly responsible for the reduction in yield stress. Aggregate characteristics, paste volume, and paste composition can be varied to reduce the HRWRA demand for a given slump flow and to ensure proper filling ability, passing ability, and segregation resistance.

- Filling ability should be measured with the slump flow test, including both measurements of slump flow and T_{50} . Passing ability should be measured with the j-ring test. The change in height of concrete between the inside and outside of the j-ring should be measured instead of the difference in slump flow with and without the j-ring. Segregation resistance should be measured with the column segregation or sieve stability test methods. These two methods give similar results; however, the column segregation test is more likely to be used in the US because it is standardized by ASTM International.
- If SCC is to be used in precast concrete for TxDOT, the 2004 TxDOT specifications must be modified. Existing provisions that limit the use of SCC must be modified or removed. Provisions must be added for the qualification of SCC mixture proportions and for quality control requirements.

The research described in this report demonstrated that SCC is currently suitable for use in prestressed concrete beams in Texas based on the properties addressed in this research report; however, areas for additional research and innovation in SCC remain. SCC is an evolving technology. It is likely that new research, new materials—especially new chemical admixtures—and increased industry experience with SCC will further improve the properties and production of SCC for both precast and ready mixed concrete.

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Appendix A: Test Procedures

A.1. Concrete Mixing Procedures

Two concrete mixing procedures were used throughout the research project. Procedure B was found to be preferable to Procedure A. Procedure A was used for all mixtures presented in Chapter 4 except for the mixtures used to develop the final mixture proportions for the 7,000 psi nominal compressive strength level. Procedure B was used for all other testing.

Concrete Mixing Procedure A

1. Add aggregates and approximately 2/3 of mixing water to mixer. Run mixer to blend ingredients.
2. Add cementitious materials.
3. Start mixer and add remaining water. The remaining water should contain any admixtures other than HRWRA.
4. Mix for 3 minutes.
5. Stop mixing for 3 minutes. Scrape sides of mixer. Add HRWRA near end of this period.
6. Mix for 6 minutes. Adjust HRWRA dosage to reach desired slump flow.
7. Measure slump flow. If slump flow is too low, add more HRWRA and mix for at least 1 minute.
8. Discharge concrete from mixer upon reaching the desired initial slump flow.

Note: For extremely dry mixtures where severe clumping of the cement would occur, a portion of the HRWRA was added to the final addition of the mixing water. In such a case, the procedure was modified to prevent combining admixtures per manufacturers' recommendations.

Concrete Mixing Procedure B

1. Add aggregates and cementitious materials to mixer. Run mixer for 1 minute to blend ingredients.
2. With mixer still running, gradually add water with approximately 50-70% of expected HRWRA dosage and full dosage of all other admixtures.
3. Mix for 10 minutes. Adjust HRWRA dosage beginning after 3 minutes.
4. Measure slump flow. If slump flow is too low, add more HRWRA and mix for at least 1 minute.
5. Discharge concrete from mixer upon reaching the desired initial slump flow.

A.2 Concrete Curing Temperature Profiles

Table 11.1: Concrete Curing Temperature Profiles

		Mild Temperature Scenario						Hot Temperature Scenario		Cold Temperature Scenario	
Start Temp (°F)		75	75	75	75	75	75	95	95	50	50
Preset Time (Hours)		4	6	8	8	4	8	4	4	8	8
Max Temp (°F) ¹		120	120	120	145	170	170	145	170	80	105
Time (Hours)	0	75.0	75.0	75.0	75.0	75.0	75.0	95.0	95.0	50.0	50.0
	1	76.3	75.5	75.1	76.5	80.0	77.5	97.6	98.9	50.0	50.0
	2	77.5	76.0	75.3	78.0	85.0	80.0	100.3	102.9	50.0	50.0
	3	78.8	76.5	75.4	79.5	90.0	82.5	102.9	106.8	50.0	50.0
	4	80.0	77.0	75.5	81.0	95.0	85.0	105.5	110.8	50.0	50.0
	5	85.0	77.5	75.6	82.5	113.0	87.5	115.0	125.0	50.0	50.0
	6	90.0	78.0	75.8	84.0	132.0	90.0	125.0	140.0	50.0	50.0
	7	99.0	83.0	75.9	85.5	143.0	92.5	130.8	148.7	50.0	50.0
	8	107.0	89.0	76.0	87.0	153.0	95.0	136.1	156.6	50.0	50.0
	9	113.0	96.0	80.0	94.0	160.0	106.0	139.7	162.1	52.7	55.0
	10	117.0	103.0	85.0	104.0	165.0	118.0	142.4	166.1	56.1	61.3
	11	119.0	110.0	93.0	116.0	168.0	132.0	143.9	168.4	61.6	71.3
	12	120.0	115.0	100.0	125.0	170.0	144.0	145.0	170.0	66.4	80.0
	13	120.0	118.0	106.0	133.0	170.0	154.0	145.0	170.0	70.5	87.5
	14	119.0	120.0	112.0	139.0	168.5	162.0	144.0	169.0	74.5	95.0
	15	118.0	120.0	117.0	143.0	167.0	167.0	143.0	168.0	78.0	101.3
	16	117.0	119.0	120.0	145.0	165.5	170.0	142.0	167.0	80.0	105.0
	17	116.0	118.0	120.0	145.0	164.0	170.0	141.0	166.0	82.0	107.0
	18	115.0	117.0	119.0	144.0	162.5	168.5	140.0	165.0	84.0	109.0
¹ Between 0 and 16 hours											

A.3 Workability Test Methods

The workability test methods are described in this appendix as they were performed in the research described in this report and may differ slightly from standardized test methods.

A.3.1 Column Segregation Test

Apparatus

1. PVC pipe sections, 8 inches in diameter and 6.5 inches in height, with seals and clips to accept clamps (4). (Alternative: replace 2 middle sections with single 13-inch long section)
2. Spring clamps (12)
3. Base plate (the bottom PVC pipe section is permanently attached to the base plate)
4. Collector plate
5. No. 4 sieve (at least 1, preferably 2)
6. Scoop or bucket to load concrete into column
7. Stopwatch
8. Drying containers or dishes, minimum 5 liters (2)
9. Oven or microwave
10. Balance

Concrete Volume

0.76 ft³ (21.4 l)

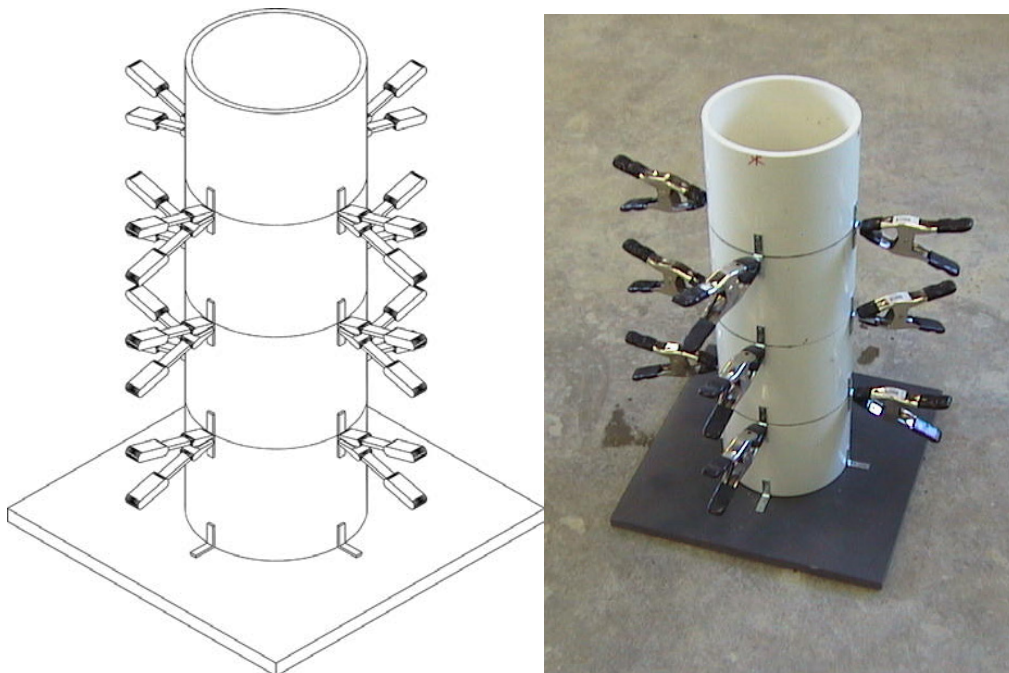


Figure 11.1: Column Segregation Test (Collector Plate not Shown)

Procedure

1. Assemble the PVC pipe sections. Use the clamps to secure each PVC pipe section firmly and to ensure a water-tight seal.
2. Place the assembled apparatus on a firm, level surface.
3. Fill the column with concrete with no external compaction effort.
4. Allow the concrete to remain undisturbed for 15 minutes.
5. Use the collector plate to remove individually each PVC pipe section with the concrete material inside.
6. Individually transfer the contents of the top and bottom pipe section to separate No. 4 sieves. Discard the contents of the middle section(s). Wash each concrete sample over the No. 4 sieve to remove all paste and fine aggregate, leaving behind only clean coarse aggregates on each sieve.
7. Collect the coarse aggregates retained on each sieve in a separate container for each pipe section. Dry each sample in an oven or microwave until it reaches a constant mass.
8. **Measure** the mass of each sample of coarse aggregates.

Results

1. Percent Static Segregation:

$$\text{Percent Static Segregation} = \left\{ \begin{array}{ll} \frac{M_{bottom} - M_{top}}{M_{bottom} + M_{top}} \times 100\% & \text{if } M_{bottom} > M_{top} \\ 0\% & \text{if } M_{bottom} < M_{top} \end{array} \right\}$$

Where: M_{bottom} = mass of aggregate retained on No. 4 sieve from bottom pipe section
 M_{top} = mass of aggregate retained on No. 4 sieve from top pipe section.

Notes

1. This test method is standardized as ASTM C 1610.
2. ASTM C 1610 allows aggregates to be dried to saturated-surface dry condition instead of oven-dried. In such a case, towels are needed to dry surface moisture from aggregates.
3. In the testing performed for this report, only the coarse aggregate content in the top and bottom sections was measured, which is in accordance with the ASTM C 1610 procedure, even though the apparatus pictured in Figure 11. has four pipe sections.

A.3.2 Concrete Rheometer (ICAR Rheometer)

Apparatus

The prototype of the ICAR rheometer used in the research described in this report is shown in Figure 11.. The rheometer features a 4-bladed vane (5-inches in diameter and height) that is rotated axially in the center of the container. The rheometer electronics control the speed of the vane and record the torque acting on the vane as it rotates in concrete. The resulted torque versus speed measurements are used to calculate the Bingham model parameters of yield stress and plastic viscosity. The container size is selected based on the maximum aggregate size. The gap between the vane and concrete specimen boundaries should be at least 4 times the maximum aggregate size. For a $\frac{3}{4}$ -inch maximum aggregate size, the resulting 3-inch gap would require an 11-inch diameter, 11-inch tall concrete specimen. The container includes vertical strips to prevent slippage of the concrete against the container walls.

Concrete Volume

For 11 by 11-inch container: 0.68ft^3 (19.2 l)

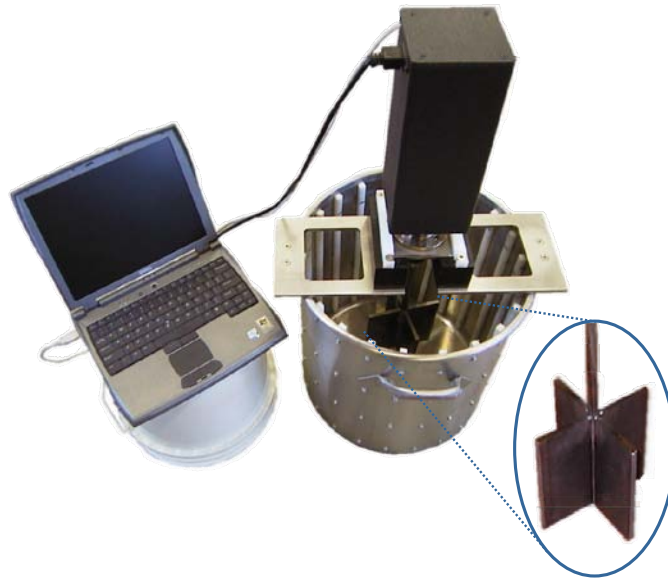


Figure 11.2: ICAR Rheometer

Procedure and Results

The container is filled with concrete to the appropriate height. The vane is then inserted into the concrete. The computer operates the test, records data, and computes test results. The rheometer can perform a flow curve test or stress growth test. In a typical flow curve test, the vane is rotated at a constant speed to breakdown the effects of thixotropy. The speed is then decreased in steps from maximum to minimum. The resulting torque and speed data are used to compute the Bingham parameters of yield stress and plastic viscosity. In a stress growth test, the vane is rotated at a constant, low speed and the gradual increase in torque is monitored. The torque increases to a maximum value, then decreases. The maximum torque is recorded and used to calculate the static yield stress.

Notes

For the testing presented in this report, an 11-inch by 11-inch container was used. Except as noted otherwise, the following test procedures were used:

- Flow Curve Tests: Ascending and descending flow curves were measured. Eight ascending flow curve points from 0.05 to 0.50 rps were measured, the speed was held constant at 0.50 rps for 20 seconds, and then 8 descending flow curve points from 0.50 to 0.05 rps were measured. The time for each flow curve point was 5 seconds. The Bingham parameters of yield stress and plastic viscosity were calculated from the descending flow curve points. Thixotropy was calculated as the area between the up and down curves on the torque versus rotation speed plot. In cases where thixotropy was not determined, the ascending flow curve was not measured.
- Stress Growth Tests: The first point on the ascending flow curve (0.05 rps) was used as a stress growth test. The maximum torque was recorded and used to calculate the static yield stress.

A.3.3 J-Ring Test

Apparatus

1. J-ring (Figure 11.), 300 mm diameter with 17 equally spaced, 16 mm-diameter reinforcement bars (deformed)
2. Rigid, non-absorbent plate, at least 32 inches square, with concentric circles marked at diameters of 300 mm (12 in.) and either 100 mm (4 in.) for the inverted cone orientation or 200 mm (8 in.) for the upright cone orientation..
3. Slump cone (ASTM C 143)
4. Scoop or bucket to load concrete into slump cone
5. Measuring tape or ruler

Concrete Volume

0.20 ft³ (5.6 l)

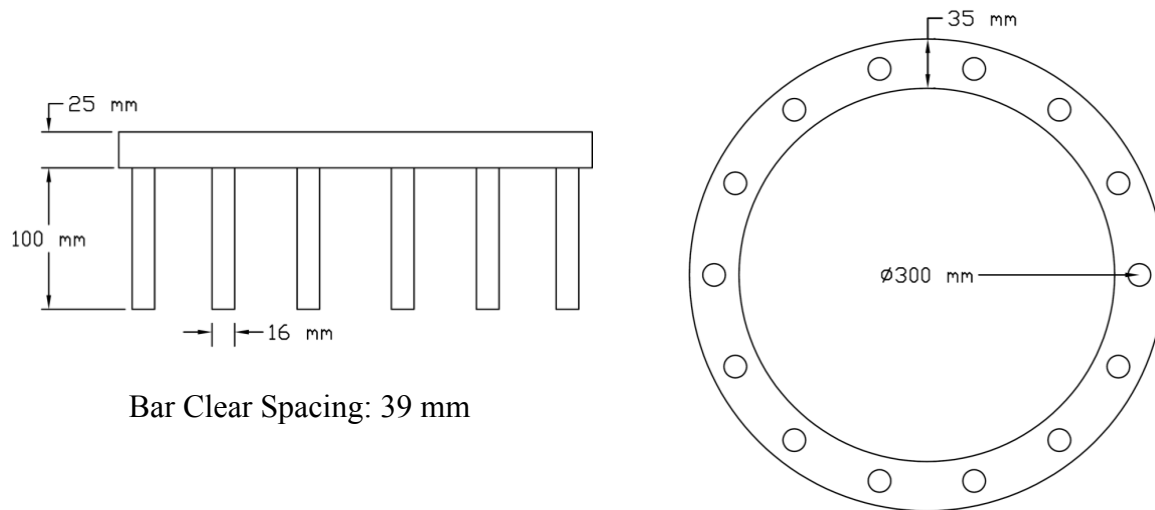


Figure 11.3: J-Ring (Bar Spacing Can Vary)

Procedure

1. Attach reinforcement bars on one side of the j-ring to achieve the desired clear spacing between bars.
2. Dampen the slump cone and plate (ensure there is no standing water). Place the plate on firm, level ground. Center the j-ring on the plate (use the 12-inch concentric circle as a guide). Center the slump cone on the plate (use the 8-inch or 4-inch concentric circle as a guide) and hold down firmly. The slump cone can be oriented upright or inverted.
3. Fill the slump cone with concrete. Do not apply any external compaction effort. Strike off any excess concrete above the top of the slump cone. Remove any concrete on the plate.
4. Remove the slump cone by lifting it vertically upward, being careful not to apply any lateral or torsional motion. Allow the concrete to spread horizontally and cease flowing.
5. **Measure** the height of concrete inside the ring (H_{in}) and outside the ring (H_{out}) at four locations around the ring (See Figure 11.).
6. **Measure** the height of concrete in the center of the ring (H_{center}).

7. **Measure** the final horizontal slump flow in two orthogonal directions.

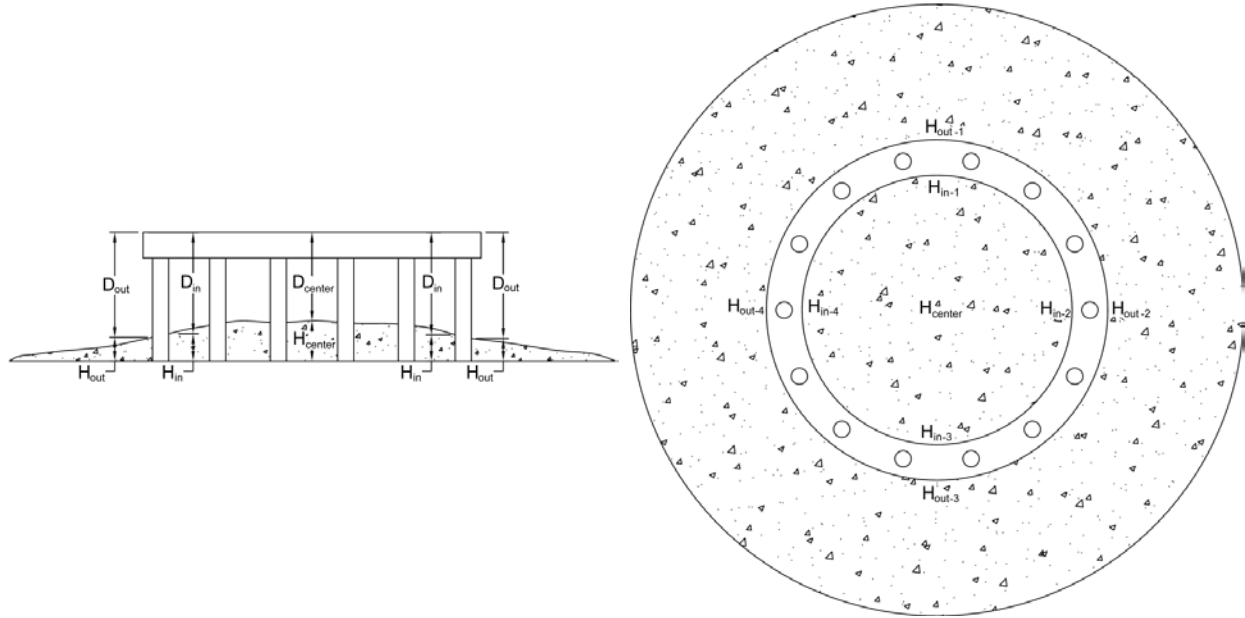


Figure 11.4: J-Ring Measurement Locations

Results

1. J-ring Δ height = $\text{mean}(H_{in}-H_{out})$, in inches or mm
2. J-ring test value = $2 * \text{median}(H_{in}-H_{out}) - \text{median}(H_{center}-H_{in})$, in inches or mm
3. J-ring Δ slump flow = $(\text{Slump Flow})_{\text{without J-ring}} - (\text{Slump Flow})_{\text{with J-ring}}$, in inches or mm

Notes

1. This test method is now standardized as ASTM C 1621.
2. Bar spacing can vary. Changes in bar spacing can be facilitated by threading top of reinforcing bars to screw into tapped holes on top of ring. Different bar spacing can be used on either side of j-ring.
3. For all tests performed in this research, 17 equally spaced, 16-mm diameter deformed reinforcing bars were used. The j-ring in ASTM C 1621 uses 16 equally-spaced 5/8 inch-diameter smooth bars.

A.3.4 L-Box Test

Apparatus

1. L-box (Figure 11.), with 3 equally spaced, 16 mm-diameter, deformed reinforcement bars (clear spacing = 38 mm)
2. Scoop or bucket to load concrete into l-box
3. Stopwatch
4. Measuring tape or ruler

Concrete Volume

0.44 ft³ (12.6 l)

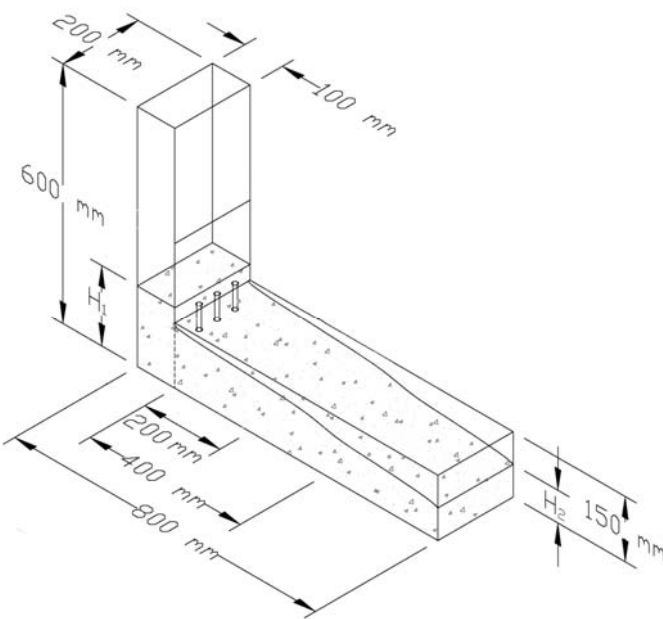


Figure 11.5: L-Box Apparatus

Procedure

1. Place the l-box on a firm, level surface. Close the gate.
2. Fill vertical portion of the l-box with concrete. Do not apply any external compaction effort.
3. Allow the concrete to remain undisturbed in the l-box for one minute.
4. Open the gate fully.
5. **Measure** the time for the concrete to reach point marked at 400 mm (T_{40}) down the length of the box.
6. **Measure** the heights H_1 and H_2 at each end of the box (see Figure 11.) after concrete flow has ceased.

Results

1. Time for the concrete to flow to a point 400 mm down the box (T_{40}), in seconds
2. Blocking Ratio = H_2/H_1

A.3.5 Hardened Concrete Column Test (Segregation)

Apparatus

1. Cylindrical column form, 8 inches in diameter and 28 inches in height
2. Scoop or bucket to load concrete into form
3. Concrete saw
4. Transparent plastic sheet with grid

Concrete Volume

0.81 ft³ (23.0 l)

Procedure

1. Fill concrete into cylindrical column form. Do not apply any external compaction effort.
2. Allow concrete to harden.
3. Make 3 horizontal cuts spaced 1-inch apart from both the top and bottom of the column.
4. Superimpose grid on each cut section. (Superimpose grid only on one side of each cut section; superimpose consistently on the same side of section for each series of cuts.)
5. **Measure** the volume of coarse aggregate in each cut section by counting the number of points on the grid that overlap a coarse aggregate (count 1 when grid point overlaps coarse aggregate, ½ when grid point overlaps edge of coarse aggregate). The volume of coarse aggregate is equal to the number of counts divided by the number of grid points.

Results

1. Percent Static Segregation:

$$\text{Percent Static Segregation} = \begin{cases} \frac{V_{bottom} - V_{top}}{V_{bottom} + V_{top}} \times 100\% & \text{if } V_{bottom} > V_{top} \\ 0\% & \text{if } V_{bottom} < V_{top} \end{cases}$$

Where: V_{bottom} = average volume of aggregate in the bottom cut sections
 V_{top} = average volume of aggregate in the top cut sections

Notes

1. The calculation of coarse aggregate volume is described in ASTM E 562.
2. For the research described in this report, the grid consisted of 78 points, spaced ¾-inch apart, resulting in a total of 234 points each for the top and bottom of the column.

A.3.6 Penetration Apparatus Test

Apparatus

1. Concrete specimen container (e.g. slump cone)
2. Penetration cylinder (aluminum, 45 g)
3. Cylinder positioning apparatus with length scale
4. Stopwatch

Concrete Volume

0.20 ft³ (5.6 l) for slump cone

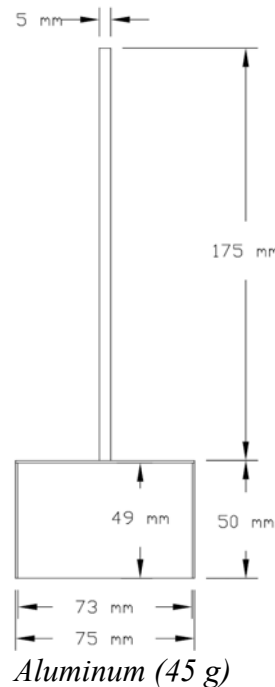


Figure 11.6: Penetration Apparatus

Procedure

1. Fill the specimen container with no external compaction effort and ensure top surface of concrete is level.
2. Immediately put the cylinder positioning apparatus into place such that the penetration cylinder is centered above the specimen. Position the bottom of the penetration cylinder just above the top surface of the concrete and tighten the set screw to secure the cylinder in this position.
3. Allow the concrete to remain undisturbed for one minute from the completion of the filling the specimen container.
4. Release the set screw and allow the penetration cylinder to sink into the concrete under its own mass.
5. **Measure** the penetration depth after 30 seconds.
6. OPTIONAL: Repeat the procedure an additional two times.

Results

1. Average Penetration Depth

Notes

1. For the research described in this report, test was conducted once on each mixture. The specimen container for concrete was an inverted slump cone; however, the test method is not limited to this concrete specimen container.
2. Other sizes of penetration cylinders can be used.

A.3.7 Sieve Stability Test

Apparatus

1. Container, minimum 10-12 liter capacity, with lid, provide line at 10 liters (3-gallon bucket with 10.5-inch diameter used in testing in this report)
2. Balance (accuracy +/- 20g)
3. No. 4 Sieve
4. Pan (the sieve should be easily removed from the pan so as not to cause extra mortar to pass through the pan)
5. Frame to position container 500 mm over sieve
6. Stopwatch



Figure 11.7: Sieve Stability Test

Concrete Volume

0.35 ft³ (10 l)

Procedure

1. Fill the container with 10 +/-0.5 liters of concrete with no external compaction effort. Cover the container.
2. Allow the concrete to remain undisturbed in the container for 15 +/- 0.5 minutes.
3. Place the pan and sieve on the scale. **Measure** the mass of the pan.
4. Pour 4.8 +/- 0.2 kg of concrete from a height of 500 +/- 50 mm onto the sieve. **Measure** the mass of concrete poured onto the sieve.
5. After 2 minutes, remove the sieve. **Measure** the mass of the pan and any mortar that has passed into the pan.

Results

1. Sieved Portion = $(\text{Mass}_{\text{pan+passed mortar}} - \text{Mass}_{\text{pan}}) / (\text{Mass}_{\text{concrete poured on sieve}}) * 100\%$

A.3.8 Slump Flow Test

Apparatus

1. Rigid, non-absorbent plate, at least 32 inches square, with concentric circles marked at diameters of 500 mm (20 in.) and either 100 mm (4 in.) for the inverted cone orientation or 200 mm (8 in.) for the upright cone orientation..
2. Slump cone (ASTM C 143)
3. Scoop or bucket to load concrete into slump cone
4. Stopwatch
5. Measuring tape or ruler

Concrete Volume

0.20 ft³ (5.6 l)

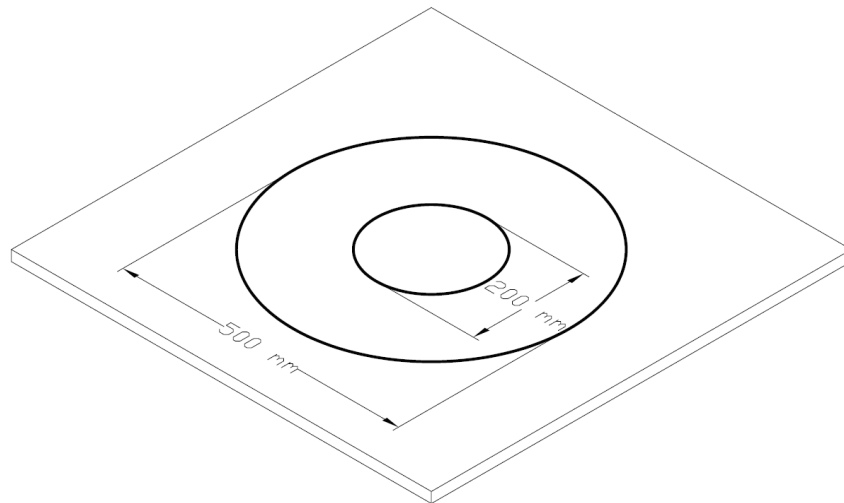


Figure 11.8: Slump Flow Plate

Procedure

1. Dampen the slump cone and plate (ensure there is no standing water). Place the plate on firm, level ground. Center the slump cone on the plate (use the 8-inch or 4-inch concentric circle as a guide) and hold down firmly. The slump cone can be oriented upright or inverted.
2. Fill the slump cone in one lift. Do not apply any external compaction effort. Strike off any excess concrete above the top of the slump cone. Remove any concrete on the plate.
3. Remove the slump cone by lifting it vertically upward, being careful not to apply any lateral or torsional motion.
4. **Measure** the time for the concrete to spread to a diameter of 500 mm (T_{50})
5. **Measure** the final slump flow in two orthogonal directions after the concrete has ceased flowing.
6. **Assign** the visual stability index (VSI) to the nearest 0.5 based on the criteria in Table 11..

Table 11.2: Visual Stability Index Ratings (Daczko 2002)

VSI	Criteria
0	No evidence of segregation in slump flow patty or in mixer drum or wheelbarrow.
1	No mortar halo or aggregate pile in the slump flow patty but some slight bleed or air popping on the surface of the concrete in the mixer drum or wheelbarrow.
2	A slight mortar halo (< 10 mm) and/or aggregate pile in the slump flow patty and highly noticeable bleeding in the mixer drum and wheelbarrow.
3	Clearly segregating by evidence of a large mortar halo (>10 mm) and/or a large aggregate pile in the center of the concrete patty and a thick layer of paste on the surface of the resting concrete in the mixer drum or wheelbarrow.

Results

1. Average slump flow, in inches or mm
2. T₅₀, in seconds
3. Visual stability index

Notes

1. This test method is standardized as ASTM C 1611.
2. The slump cone can be used in the inverted or upright orientation. The inverted orientation is preferred.
3. The visual stability index ratings vary slightly in ASTM C 1611 from Table 11., which were used for the research described in this report.

A.3.9 V-Funnel Test

Apparatus

1. V-funnel (Figure 11.)
2. Bucket (minimum capacity = 0.35 ft^3 or 10.0 l)
3. Scoop or bucket to load concrete into v-funnel
4. Stopwatch

Concrete Volume

0.35 ft^3 (10.0 l)

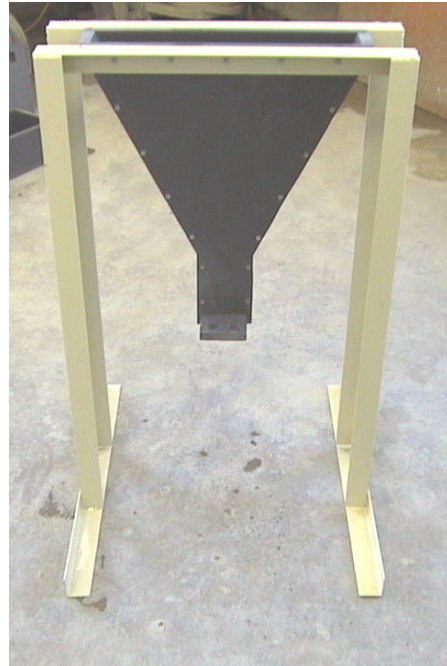
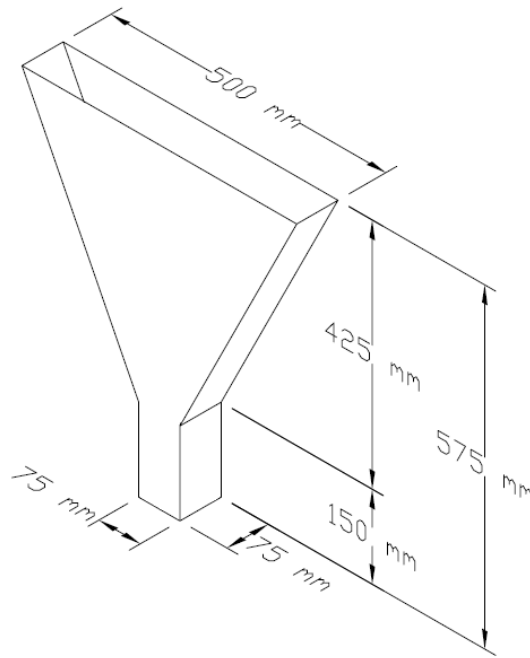


Figure 11.9: V-Funnel

Procedure

1. Place the v-funnel frame on firm, level ground. Position the bucket below the opening in the v-funnel.
2. Dampen the inside of the v-funnel. Leave the bottom gate open for sufficient time so that once the gate is closed, water does not drain and collect on the gate.
3. Close the bottom gate.
4. Fill the v-funnel with concrete. Do not apply any external compaction effort. Strike off any excess concrete above the top of the v-funnel.
5. Allow the concrete to remain undisturbed in the v-funnel for one minute.
6. Open the gate of the v-funnel and allow the concrete to flow into the bucket.
7. **Measure** the time from the opening of the gate to the point when light is first visible through the bottom hole.

Optional steps (for v-funnel time after 5 minutes of rest):

8. Close the gate and refill the v-funnel. Do not apply any external compaction effort. Strike off any excess concrete above the top of the v-funnel. It is not necessary to clean the v-funnel for this subsequent test.
9. Allow the concrete to remain undisturbed for 5 minutes.
10. Open the gate and allow the concrete to flow into the bucket.
11. **Measure** the time from the opening of the gate to the point when light is first visible through the bottom hole.

Results

1. Standard v-funnel time (T_{std}), in seconds
2. Five-minute v-funnel time ($T_{5 \text{ min}}$), in seconds

Notes

1. Only the standard v-funnel time was recorded for the research described in this report.
2. Separate mini-v-funnel used for mortar.

Appendix B: Test Data

Table 11.3: Development of Mixture Proportions, River Gravel Set (1 of 2)

ID	Date	Cem-ent	Fly Ash	Proportions (SSD)										Mixture Indices										Slump Flow				J-Ring				L-Box				Comp. Strength	
				Coarse Agg.	Fine Agg.	Water	HR-A	HR-B	VMA-A	VMA-B	MR-A	MR-B	RET-A	RET-B	ACC-A	ACC-B	Paste Vol.	S/A	w/cm	w/c	Fly Ash	ASTM C231	Flow	T ₅₀	VSI	Δh	F _u	T ₄₀	Bkng Ratio	H ₂ /H ₁	72°F	16-hr	28-d				
		lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	oz/cwt	oz/cwt	oz/cwt	oz/cwt	oz/cwt	oz/cwt	oz/cwt	oz/cwt	oz/cwt	oz/cwt	oz/cwt	oz/cwt	oz/cwt	oz/cwt	oz/cwt	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	psi	psi	psi				
1	4/14/05	490.0	210.0	13	13	162.9	141.1	210.0	13					29.1	0.458	0.30	0.429	30	2.6	27	3.3	1.0	2.13	-7.0	2.9	1.00	4525	12078									
2	4/14/05	490.0	210.0	10.5	10.5	162.9	136.8	245.0	13					31.1	0.458	0.35	0.50	30	3.4	28	1.0	1.5	0.56	-3.5	2.2	0.92	3287	10085									
3	4/14/05	490.0	210.0	7.5	7.5	158.7	132.5	280.0	7.5					33.2	0.458	0.40	0.571	30	1.2	23.5	1.0	0.0	1.38	-6.5	0.7	0.44	2682	8833									
4	4/14/05	490.0	210.0	6.5	6.5	155.1	130.3	301.0	6.5					34.5	0.458	0.43	0.614	30	0.6	26	1.0	0.5	1.13	-7.5	0.6	0.60	2195	7541									
5	4/14/05	560.0	240.0	6	6	147.5	123.9	320.0	6					37.7	0.458	0.40	0.571	30	0.6	26	1.0	0.5	1.13	-2.0	0.6	0.64	2609	7745									
6	4/14/05	560.0	240.0	13	13	158.7	133.4	240.0	13					32.9	0.458	0.30	0.429	30	1.7	28.5	2.0	2.0	0.88	-2.0	3.1	0.92	4633	11778									
7	4/26/05	560.0	140.0	9	9	159.7	133.7	280.0	9					31.7	0.458	0.40	0.50	20	1.9	26.5	1.5	0.0	0.56	-3.5	1.3	0.44	3310	9080									
8	4/26/05	700.0		9.5	9.5	161.3	135.2	280.0	9.5					31.8	0.458	0.40	0.40	0	1.4	29.5	1.5	1.5	0.69	-3.5	1.3	0.56	4758	9747									
9	4/26/05	700.0		12.5	12.5	162.8	139.7	245.0	12.5					29.7	0.458	0.35	0.35	0	2.0	29.0	2.0	2.0	0.75	-3.0	4.5	0.17	5909	11491									
10	4/26/05	560.0	140.0	12	12	164.9	137.9	245.0	12					30.7	0.458	0.35	0.438	20	1.7	29	1.0	1.0	0.88	-3.0			3882	10730									
11	5/3/05	700.0		10	10	161.3	135.6	280.0	10					31.8	0.458	0.40	0.40	0	1.1	28.5	1.0	1.5	0.50	-4.5	2.1	0.42	4838	10112									
12	5/3/05	700.0		10.5	10.5	148.7	148.1	280.0	10.5					31.8	0.500	0.40	0.40	0	1.6	27	1.1	0.5	0.75	-4.0	3.0	0.30	4915	10229									
13	5/3/05	700.0		9	9	178.5	118.5	280.0	9					31.8	0.400	0.40	0.40	0	1.1	30	1.0	1.5	0.75	-7.0	7.0	0.08	4774	9953									
14	5/3/05	700.0		8.5	8.5	193.8	103.7	280.0	8.5					31.8	0.350	0.40	0.40	0	0.9	29	1.0	2.0	1.63	-9.5	16.0	0.00	4685	9457									
15	5/19/05	700.0		10	10	178.5	118.5	280.0	10					31.8	0.400	0.40	0.40	0	0.8	29.5	0.5	2.0	1.38	-7.5	4.5	0.29	5237	10756									
16	5/19/05	700.0		10	10	178.5	118.5	280.0	10					31.8	0.400	0.40	0.40	0	1.5	26	1.5	1.0	1.13	-5.0	2.4	0.31	4934	9549									
17	5/19/05	700.0		11	11	178.5	118.5	280.0	11					31.8	0.400	0.40	0.40	0	2.8	29	1.0	2.0	1.25	-3.0	3.3	0.42	4947	9618									
18	5/19/05	700.0		10	10	178.5	118.5	280.0	10					31.8	0.400	0.40	0.40	0	1.9	25	1.0	0.5	0.44	-4.5	3.5	0.09	4766	9065									
19	5/19/05	700.0		11	11	178.5	118.5	280.0	11					31.8	0.400	0.40	0.40	0	4.5	31	0.5	2.0	1.31	-6.0	3.0	0.23	4116	8419									
20	5/26/05	775.0		8	8	170.4	112.9	310.0	8					35.0	0.400	0.40	0.40	0	0.6	30	0.5	1.5	1.25	-7.0	1.6	0.31	4850										
21	5/26/05	850.0		6	6	161.8	107.4	340.0	6					38.2	0.400	0.40	0.40	0	0.7	27.5	0.5	0.5	1.13	-6.0	0.8	0.39	4571										
22	5/26/05	560.0	140.0	8	8	176.7	116.9	330.0	8					32.7	0.400	0.40	0.50	20	1.9	28.5	0.7	1.5	1.38	-7.5	1.6	0.17	3650										
23	5/26/05	680.0	170.0	5	5	158.8	105.4	340.0	5					39.3	0.400	0.40	0.50	20	1.7	28	0.5	1.5	1.25	-7.0	0.5	0.44	3612										
24	6/7/05	640.0	160.0	10	10	142.3	141.8	280.0	10					34.8	0.500	0.35	0.438	20	1.2	30.5	1.5	1.5	0.00	0.0	1.1	0.71											
25	6/7/05	640.0	160.0	9	9	156.5	127.6	280.0	9					34.8	0.450	0.35	0.438	20	1.0	31	1.0	1.5	0.13	-0.5	1.0	0.77											
26	6/7/05	640.0	160.0	8.5	8.5	170.8	113.4	280.0	8.5					34.8	0.400	0.35	0.438	20	1.0	30	1.0	1.0	0.88	-3.0	1.4	0.79											
27	6/7/05	640.0	160.0	15	15	147.5	146.6	240.0	15					32.4	0.500	0.30	0.375	20	2.4	29	2.2	0.5	0.25	1.5	2.0	1.00											
28	6/9/05	840.0	160.0	9	9	142.3	141.8	280.0	9					34.8	0.500	0.35	0.438	20	2.4	29	1.0	0.5	0.31	-2.0	1.1	0.71	4093	4897	10629								
29	6/9/05	640.0	160.0	8	8	156.5	127.6	280.0	8					34.8	0.450	0.35	0.438	20	3	27.5	1.1	0.0	0.31	-3.0	1.3	0.49	4359	4916	10058								
30	6/9/05	640.0	160.0	7.5	7.5	170.8	113.4	280.0	7.5					34.8	0.400	0.35	0.438	20	1.7	27.5	1.3	0.0	0.38	-3.5	1.5	0.44	4356	5035	10233								
31	6/7/05	640.0	160.0	10.5	10.5	142.3	141.8	280.0	10.5					34.8	0.500	0.35	0.438	20	1.5	30	1.4	1.0	0.63	-3.0	2.5	0.83											
32	6/7/05	640.0	160.0	10	10	142.3	141.8	280.0	10					34.8	0.500	0.35	0.438	20	1.1	30	1.1	1.0	0.88	-1.5	3.0	0.60											
33	6/7/05	640.0	160.0	10	10	142.3	141.8	280.0	10					34.8	0.500	0.35	0.438	20	0.8	31	1.2	1.5	0.50	-0.5	1.8	1.00											
34	6/7/05	640.0	160.0	9.5	9.5	142.3	141.8	280.0	9.5					34.8	0.500	0.35	0.438	20	1.0	31	1.1	1.5	0.38	-1.5	1.6	0.77											
35	6/7/05	640.0	160.0	10	10	142.3	141.8	280.0	10					34.8	0.500	0.35	0.438	20	0.8	31	1.1	1.0	0.31	-1.0	1.1	0.92											
36	6/14/05	800.0		9.5	9.5	144.6	144.1	280.0	9.5					33.7	0.500	0.35	0.35	0	1.7	26	2.0	0.0	1.19	-4.5	2.0	0.47	6018	6373	11329								
37	6/14/05	800.0		9	9	144.6	144.1	280.0	9					33.7	0.500	0.35	0.35	0	1.0	30	1.3	1.0	1.38	-6.5	0.8	0.32	5832	5924	12185								
38	6/14/05	640.0	160.0			147.5	146.6	240.0						32.4	0.500	0.30	0.375	20	3	30	1.8	1.0	0.50	-3.0	1.1	1.00	3743	3654	11061								
39	6/14/05	640.0	160.0			142.3	141.8	280.0						34.8	0.500	0.35	0.438	20	2.4	26.5	1.5	0.0	0.56	-3.0		0.71	3814	4960	9929								
40	6/16/05	640.0	160.0	8.25	8.25	142.3	141.8	280.0	8.25					34.8	0.500	0.35	0.438	20	2.3	29.5	1.0	1.0	0.94	-5.5	2.5	0.44											
41	6/16/05	640.0	160.0																																		

Table 11.4: Development of Mixture Proportions, River Gravel Set (2 of 2)

ID	Date	Cem-ent	Fly Ash	Proportions (SSD)										Mixture Indices										Air (ASTM)	Slump Flow		J-Ring		L-Box		Comp. Strength	
				UFA	Coarse Agg.	Fine Agg.	Water	HRA	HR-B	VMA-A	VMA-B	MR-A	MR-B	RET-A	RET-B	ACC-A	ACC-B	Paste Vol.	S/A	w/cm	w/c	Fly Ash	Flow		T ₅₀	VSI	Δh	F-u	T ₆₀	Bkng Ratio	H ₂ /H ₁	psi
		lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³
53	7/28/05	800.0		1446.7	1441.1	280.0	12.5																									
54	8/11/05	555.0	253.0	1501.0	1514.0	242.4	14																									
55	8/11/05	490.0	210.0	1580.0	1328.5	280.0	8.75																									
56	9/15/05	700.0		1551.3	1545.3	231.0	16																									
57	9/15/05	700.0		1524.1	1518.2	232.0	12																									
58	9/15/05	700.0		1496.9	1491.1	273.0	11.5																									
59	9/15/05	800.0		1467.4	1461.7	264.0	12																									
60	9/15/05	900.0		1383.6	1378.2	297.0	10																									
61	9/20/05	800.0		1467.4	1461.7	264.0	12																									
63	9/20/05	704.0		96.0	1458.5	1452.9	284.0	8.5																								
64	9/20/05	656.0		144.0	1454.1	1448.4	264.0	9.5																								
65	9/20/05	704.0		96.0	1510.3	1504.5	224.0	15																								
66	9/27/05	700.0		1861.5	1236.2	231.0	14																									
67	9/27/05	700.0		1828.9	1214.3	232.0	10.8																									
68	9/27/05	700.0		1796.2	1192.9	273.0	9																									
69	9/27/05	800.0		1760.9	1169.4	264.0	9																									
70	9/27/05	900.0		1660.3	1102.6	297.0	8.5																									
71	9/27/05	800.0		1467.4	1461.7	264.0	11																									
72	9/29/05	935.9		1667.8	1107.6	280.8	9																									
73	9/29/05	983.7		1667.8	1107.6	265.6	13																									
74	9/29/05	759.7		1461.8	1456.2	281.1	9																									
75	9/29/05	685.6	166.4	1461.8	1456.2	246.3	11																									
76	9/29/05	1000.0		1559.7	1035.8	330.0	7																									
77	10/6/05	622.0	155.5	1461.8	1456.1	264.4	9																									
79	10/6/05	672.1	168.0	1429.1	1423.6	268.8	8.5																									
80	10/6/05	819.3		1461.8	1456.2	282.2	11																									
81	10/6/05	845.9		1461.8	1456.2	253.8	12																									
82	10/6/05	886.8		1429.1	1423.6	266.0	11																									
83	10/12/05	643.1	160.3	1461.8	1456.2	256.4	9.5																									
85	10/12/05	724.1	310.3	1570.9	1043.2	289.6	7																									
86	10/12/05	666.9	285.8	1649.5	1095.4	266.8	8																									
87	10/12/05	730.6	243.5	1649.5	1095.4	263.0	8																									
88	10/17/05	845.9		1461.8	1456.2	253.8	11.5																									
89	10/17/05	631.5	221.9	1560.0	1271.4	260.3	9.25																									
90	10/17/05	739.4	208.6	1649.5	1095.4	265.4	8																									
91	10/17/05	739.4	208.6	1649.5	1095.4	265.4	7.5																									
92	10/17/05	739.4	208.6	1649.5	1095.4	265.4	7.75																									
93	10/17/05	739.4	208.6	1649.5	1095.4	265.4	7.75																									
94	10/20/05	617.0	245.9	1560.0	1271.4	254.6	9																									
95	10/20/05	645.7	198.4	1560.0	1271.4	265.9	8																									
96	10/20/05	623.8	156.0	1461.8	1456.2	257.3	9.5																									
97	10/20/05	633.3	298.0	1649.5	1095.4	260.8	7.5																									
98	10/20/05	739.4	208.6	1649.5	1095.4	265.4	7.5																									
99	10/20/05	739.4	208.6	1649.5	1095.4	265.4	7.5																									
100	10/21/05	645.7	238.8	1536.0	1251.9	265.4	9																									
101	10/21/05	665.1	198.7	1536.0	1251.9	276.4	8																									

Notes: Accelerator and retarder dosages expressed as oz/cwt of cement; all other admixtures expressed as oz/cwt of cementitious materials; Fly ash is FA-C in mixture 54, UHFA in mixture 63-65, FA-A in all other mixtures.
Cement=PC-A, Coarse=RG, Fine=NS-A

Notes: Accelerator and retarder dosages expressed as oz/cwt of cement; all other admixtures expressed as oz/cwt of cementitious materials; Fly ash is FA-C in mixture 54, UFFA in mixture 63-65, FA-A in all other mixtures; Cement=P-C-A, Coarse=RG, Fine=NS-A

Table 11.5: Development of Mixture Proportions, Crushed Limestone Aggregate Set

ID	Date	Proportions (SSD)					Mixture Indices					Air (ASTM C231)		Slump Flow		J-Ring		L-Box		Rheology		Comp. Strength				
		Cem-ent	Fly Ash	Coarse Agg.	Fine Agg.	Water	HR-A	HR-B	RET-A	Paste Vol.	S/A	w/cm	w/c	Fly Ash	Flow	T ₅₀	VSI	Δh	r-u	T ₄₀	Bkling Ratio	t ₀	μ	72°F	100°F	28-d
		lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	oz/cwt	oz/cwt	oz/cwt	%	%	%	%	%	in.	s		in.	in.	s	H ₂ /H ₁	Pa	Pa.s	psi	psi	psi
1	7/21/05	800.0	0.0	1446.7	1452.3	280.0		9.5		33.9	0.502	0.35	0.35	0	29	1.4	2.0	0.38	-2.0	2.0	0.92			5385		12048
2	7/21/05	700.0	0.0	1533.1	1539.0	245.0		12.5		30.0	0.502	0.35	0.35	0	28	2.7	1.5	1.38	-4.5	5.0	0.44			4362		9083
3	7/21/05	800.0	0.0	1736.0	1161.8	280.0		8		34.0	0.402	0.35	0.35	0	29.5	1.4	1.5	1.44	-2.0	3.2	0.61			6069		12062
4	7/21/05	700.0	0.0	1839.8	1231.2	245.0		9		30.1	0.402	0.35	0.35	0	27	2.3	1.0	1.50	-7.0	23.0	0.00			6034		12057
5	11/18/05	700.0	0.0	1563.2	1557.2	231.0	25		4	28.9	0.500	0.33	0.33	0	27	9.6	2.0	2.16	-6.0	13.9	0.29	0.0	69.3			
6	11/18/05	767.0	0.0	1506.6	1500.9	253.1	19.8		4	31.5	0.500	0.33	0.33	0	28	4.1	2.0	1.03	-1.5	5.1	0.79	0.0	39.7			
7	11/18/05	833.0	0.0	1450.9	1445.3	274.9	10.8		4	34.0	0.500	0.33	0.33	0	27.5	3.2	1.0	0.50	-1.0	1.9	0.79	0.0	21.0	6010	7361	
8	11/18/05	900.0	0.0	1394.2	1388.9	297.0	8.5		4	36.6	0.500	0.33	0.33	0	26.5	2.0	0.5	0.50	-1.5	1.4	0.73	21.4	21.7			
9	11/18/05	708.1	125.0	1432.6	1427.2	274.9	9		4	34.8	0.500	0.33	0.388	15	30											6152
10	11/21/05	800.0	0.0	1774.5	1178.5	264.0	9		4	32.7	0.400	0.33	0.33	0	26.5	2.4	0.5	1.38	-4.0	12.0	0.05	20.9	27.6			
11	11/21/05	900.0	0.0	1673.1	1111.1	297.0	7.25		4	36.6	0.400	0.33	0.33	0	28	1.9	1.5	0.78	-3.0	1.9	0.53	6.3	19.0			
12	11/21/05	1000.0	0.0	1571.7	1043.8	330.0	5.35		4	40.4	0.400	0.33	0.33	0	26	1.7	1.5	0.63	-2.0	0.8	0.66	25.5	10.6			
13	11/21/05	1100.0	0.0	1470.3	976.5	363.0	4.23		4	44.3	0.400	0.33	0.33	0	26	0.7	1.5	0.19	0.5	0.6	0.80	20.2	6.9			
14	11/21/05	666.4	168.6	1426.6	1421.1	274.9	4.59		4	35.1	0.500	0.33	0.41	20	28.5	2.5	1.5							4052	5698	
15	11/21/05	624.8	208.3	1420.5	1415.1	274.9	4.05		4	35.4	0.500	0.33	0.44	25	25	3.4	1.0									
16	12/5/05	1000.0	0.0	1309.8	1304.7	330.0	7.5		4	40.4	0.500	0.33	0.33	0	27.5	1.0	0.5	0.13	-1.0	1.3	0.88	48.0	11.1			
17	12/5/05	634.1	271.8	1385.2	1379.9	271.8	6.5		4	37.0	0.500	0.30	0.43	30	28	2.3	0.0	0.25	-2.3	2.2	0.81	14.7	37.2	3732	6280	5619
18	12/5/05	681.6	367.0	1556.7	1033.8	283.1	5.5		4	41.0	0.400	0.27	0.42	35	26.5	3.2	0.0	0.50	-3.5	2.8	0.67	137.4	17.9	4312	6626	6165
19	12/5/05	636.5	342.7	1475.3	1202.4	274.2	6		4	39.0	0.450	0.28	0.43	35	28	2.6	1.0	0.25	-0.5	2.3	0.89	48.2	30.7	3866	6379	5635
20	12/7/05	636.6	390.2	1556.7	1033.8	287.5	5.5		4	41.0	0.400	0.28	0.45	38	27	3.2	1.0	0.47	-2.0	3.1	0.61	0.0	24.5	2791	5492	5305
21	12/7/05	639.6	426.4	1530.3	1016.3	287.8	6		4	42.0	0.400	0.27	0.45	40	27	3.0	0.5	0.25	-1.0	2.0	0.74	2.3	33.9	2990	5732	5252
22	12/7/05	603.8	297.4	1385.2	1379.9	270.4	7		4	37.0	0.500	0.30	0.45	33	29	2.3	0.5	0.19	-1.5	2.0	0.85	3.8	35.3	2767	5826	5093
23	12/7/05	604.0	370.2	1475.3	1202.4	272.8	6		4	39.0	0.450	0.28	0.45	38	29	2.8	1.0	0.16	0.0	2.4	0.81	0.0	39.6	3269	5861	5455
Notes: Retarder dosage expressed as oz/cwt of cement; HR-A and HR-B expressed as oz/cwt of cementitious materials; Cement=PC-A, Fly Ash=F-A-A, Coarse=LS, Fine=NS-B																										

Table 11.6: Development of Nominal 7,000 psi Mixtures, River Gravel Mixtures: Proportions

ID	Proportions (SSD)																	Indicies						
	Cement	Fly Ash	Coarse Agg.	Fine Agg.	Water	HR-A	Experimental Admixture								LN	ACC-B	RET-A	PT-1482	VMA-A	Paste Vol	S/A	w/cm	w/c	Fly Ash
							A	B	C	D	E	F	G	H										
	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	oz/yd ³	oz/yd ³	oz/yd ³	oz/yd ³	oz/yd ³	oz/yd ³	oz/yd ³	oz/yd ³	oz/yd ³	oz/yd ³	oz/yd ³	oz/yd ³	%					%	
T1	808	143	1351	1351	266.7	100	331											38.0	0.50	0.28	0.33	15		
T2A	808	143	1346	1346	266.7	111		414										38.2	0.50	0.28	0.33	15		
T2B	808	143	1346	1346	266.7	124		476										38.2	0.50	0.28	0.33	15		
T3	808	143	1340	1340	266.7	116			546									38.5	0.50	0.28	0.33	15		
T4	808	143	1357	1357	266.7	124				274								37.7	0.50	0.28	0.33	15		
T2C	808	143	1346	1346	266.7	102		414										38.2	0.50	0.28	0.33	15		
T2D	808	143	1321	1321	285.7	101		414										39.3	0.50	0.30	0.354	15		
T3B	808	143	1340	1340	266.7	103			496									38.5	0.50	0.28	0.33	15		
T6	808	143	1345	1345	266.7	118					331						83	38.2	0.50	0.28	0.33	15		
T7	808	143	1357	1357	266.7	138				248								37.7	0.50	0.28	0.33	15		
T5	808	143	1356	1356	266.7	120						83			158			37.7	0.50	0.28	0.33	15		
T8	810	160	1339	1339	262.3	147		422										38.5	0.50	0.27	0.324	16.5		
T9	810	160	1336	1336	262.3	131							464					38.6	0.50	0.27	0.324	16.5		
T10	810	160	1339	1339	262.3	119		422										38.5	0.50	0.27	0.324	16.5		
T11	810	160	1336	1336	262.3	114							464					38.6	0.50	0.27	0.324	16.5		
T12	760	190	1412	1303	256.9	110		318										37.7	0.48	0.27	0.338	20		
T13	760	190	1405	1297	256.9	105		413										38.0	0.48	0.27	0.338	20		
T14	760	190	1399	1291	256.9	107		508										38.2	0.48	0.27	0.338	20		
T15	760	190	1393	1286	256.9	117							580					38.5	0.48	0.27	0.338	20		
T16	760	190	1391	1284	256.9	120		617										38.6	0.48	0.27	0.338	20		
T17	760	190	1384	1278	256.9	116		712										38.9	0.48	0.27	0.338	20		
T18	760	190	1368	1262	256.9	125		950										39.6	0.48	0.27	0.338	20		
T19	710	235	1389	1282	255.5	122		614										38.7	0.48	0.27	0.36	25		
T20	720	180	1442	1331	243.4	133		481										36.3	0.48	0.27	0.338	20		
T21	720	180	1432	1322	243.4	136		585										36.8	0.48	0.27	0.338	20		
T22	730	170	1440	1330	243.4	134		482										36.4	0.48	0.27	0.333	18.9		
T23	730	170	1429	1319	243.4	136		585										36.9	0.48	0.27	0.333	18.9		
T24	720	180	1431	1321	243.4	136											578	36.8	0.48	0.27	0.338	20		
T25	720	180	1423	1313	243.4	134											702	37.2	0.48	0.27	0.338	20		
T26	720	180	1409	1300	261.4	105											578	37.8	0.48	0.29	0.363	20		
T27	720	180	1400	1292	261.4	103											702	38.2	0.48	0.29	0.363	20		
T27	720	180	1400	1292	261.4	112											702	38.2	0.48	0.29	0.363	20		
T28	720	180	1391	1284	261.4	104		585								234		38.6	0.48	0.29	0.363	20		
T29	720	180	1400	1292	261.4	112											702	38.2	0.48	0.29	0.363	20		
T30	720	180	1488	1267	243.3	115											578	4	36.8	0.46	0.27	0.338	20	
T31	720	180	1543	1213	243.3	121											578	4	36.7	0.44	0.27	0.338	20	
T32	720	180	1599	1158	243.3	114											578	4	36.7	0.42	0.27	0.338	20	
T33	695	174	1429	1319	252.3	110											678	5	36.9	0.48	0.29	0.363	20	
T34	695	174	1484	1264	252.3	120											678	5	36.9	0.46	0.29	0.363	20	
T35	695	174	1539	1209	252.3	110											678	5	36.9	0.44	0.29	0.363	20	
T36	720	180	1431	1321	243.4	121											578		36.8	0.48	0.27	0.338	20	
T37	720	180	1488	1267	243.3	121											578		36.8	0.46	0.27	0.338	20	
T38	720	180	1543	1213	243.3	121											578	4	36.7	0.44	0.27	0.338	20	
T39	720	180	1599	1158	243.3	121											578	8	36.7	0.42	0.27	0.338	20	
T41	720	180	1513	1289	243.3	122							216						35.7	0.46	0.27	0.338	20	
T42	720	180	1569	1233	243.3	114							216						35.7	0.44	0.27	0.338	20	
T43	720	180	1625	1176	243.3	114							216					8	35.7	0.42	0.27	0.338	20	

Mixtures developed with BASF Construction Chemicals, LLC; only mixtures tested at University of Texas shown

Note: HR-A dosage indicates total dosage (including additions after first test)

LN is ASRx30 LN admixture from BASF Construction Chemicals, LLC

FINAL MIXTURES: T20, T30-T32 (paste volume and composition, admixtures)

Table 11.7: Development of Nominal 7,000 psi Mixtures, River Gravel Mixtures: Test Results

ID	Workability												Strength				Comments
	Initial Reading				Ini. +10 min				Ini. +20 min				16-Hr f _{ci}				
	Sl. F.	T ₅₀	VSI	PA	Sl. F.	T ₅₀	VSI	PA	Sl. F.	T ₅₀	VSI	PA	72°F	Bckt.	100°F	Nom.	
	in.	s		mm	in.	s		mm	in.	s		mm	psi	psi	psi	psi	
T1	26.5								24.5				4598			6643	
T2A	28								22				4967			6968	
T2B	24								23.5				4740			7039	
T3	30								24.8				4885			6461	
T4	28.5								21.5				4485			6201	
T2C	24.5								22.8				4903	5877		6654	
T2D	26.8								22.5				4361	5616		6205	
T3B	26.5								20.5				5359	6213		6941	
T6	15								20.5				4846	5720		6533	
T7	32								24.5				3443	4146		5202	
T5	19								21.3				3839	5163		6266	
T8	33	3.1	2.0		33	3.5	2.0		29.5	4.1	1.5		5320	6174		6894	
T9	28.5	4.2	0.5		25.5	4.4	0.0		23	5.7	0.0		5679	6489		7071	
T10	30.5	4.3	1.0		27.5	4.8	0.5		23	4.8	0.0		5836	7029		7318	
T11	28	5.9	1.0		28.5	5.3	0.5		23.5	5.8	0.0		5050	6425		7162	
T12	31	3.4	2.0		29	3.4	1.5		24.8	3.8	0.0		4724	5625		6368	
T13	28.5	5.4	0.5		28	3.8	0.5		24.5	3.7	0.0		3341	4801		6735	
T14	28	5.5	1.0		24.5	5.5	0.0		25	3.3	0.0		4891	5895		6938	
T15	30	4.6	2.0		27	4.6	1.0		27	3.7	0.5		4598	5659		6880	
T16	26	5.3	0.0		28	4.0	0.0		26	3.6	0.0		5685	6657		7346	added 8 oz/yd ³ of HR-A after ini. test
T17	28.5	4.6	0.5		24	5.2	0.0		24.5	4.6	0.0		5035	6239		7094	
T18	29	4.7	1.0		24	6.4	0.0		20.5	7.2	0.0		4340	5638		6948	
T19	30	4.4	1.5		27	4.2	0.0		26.5	3.8	0.0		3932	5288		6278	
T20	29.5	5.5	1.0		28.5	5.1	0.5		28.5	3.5	0.0		4411	5576		6571	
T21	28	7.4	0.5		24	8.1	0.0		25	5.8	0.0		4421	5575		6747	
T22	27	6.8	0.0		24	7.6	0.0		22.5	7.3	0.0		3977	5030		6300	
T23	30.5	6.6	2.0		27.5	6.5	0.5		28	4.8	0.5		4296	5363		6747	
T24	26	7.6	0.0		27	5.7	0.5		24.5	5.6	0.0		5806	6779		7332	added 4 oz/yd ³ HR-A after ini. test
T25	26	6.9	0.5		24.5	6.8	0.5		21	8.0	0.0		5198	6283		7304	
T26	24	8.3	0.5	8	29.5	5.0	2.0	max	27.5	4.0	2.0	28	4256	5050		5624	added 9 oz/yd ³ HR-A after ini. test
T27	23.5	8.2	0.5	4	27.5	4.1	0.5	4	26	3.5	0.5	12	4660	5904		6847	added 6 oz/yd ³ HR-A after ini. Test
T27	26.5	7.2	0.5	8	28	4.3	2.0	20	26	3.8	1.5	13	4717	5987		6701	
T28	23.5	7.8	0.5	5	27.5	5.8	2.0	20	26	5.7	1.0	8	3989	5192		6010	added 10 oz/yd ³ HR-A after ini. Test
T29	25.5	6.0	0.5	6	28	4.4	1.0	11	25.5	5.6	0.5	5	5191	6517		7327	added 8 oz/yd ³ HR-A after ini. Test
T30	26.5	11.3	0.0	9	23	8.2	0.0	4	23	8.5	0.0	2	5294	6172		6889	
T31	28.5	6.4	0.5	19	25.5	7.0	0.0	2	26	6.4	0.0	4	5228	6152		6886	
T32	26	7.5	0.0	8	25	7.2	0.0	7	25	5.3	0.0	5	5246	6164		6873	
T33	23	7.5	0.0	1	25	5.4	0.0	3	26	3.7	0.0	4	4851	5751		6716	added 9 oz/yd ³ HR-A after ini. Test
T34	28	3.8	1.0	24	25.5	5.0	0.0	4	27	3.8	0.0	14	4993	5947		6933	
T35	27	6.8	0.5	12	24	5.0	0.0	1	22.5	6.1	0.0	3	4776	5620		6487	
T36	29	6.2	0.0	22									4894	5833	7960	7463	
T37	28	6.9	0.0	14									5304	6232	8035	7322	
T38	27.5	7.4	0.0	22									4888	5884	7923	7273	
T39	26	4.5	0.0	4									5260	6074	7671	7051	
T41	30	6.6	2.0	max	27	6.6	1.5	26	21	8.3	1.0	0	4214	4841	6494	5689	
T42	27	6.8	0.5	15	20	n/a	0.0	2	17	n/a	n/a	0	4916	5529	6629	6011	
T43	29	7.6	0.5	38	22.5	8.2	0.0	5	21	8.7	0.0	0	4512	5403	6809	5974	
Mixtures developed with BASF Construction Chemicals, LLC; only mixtures tested at University of Texas shown																	
Note: concrete mixed continuously between workability tests; cylinders obtained when mixtures were stable; PA = penetration apparatus depth																	
Curing: 72°F and 100°F: cylinders at ambient temperature; Bckt.: 3 4x8 in. cylinders in 5-gal plastic bucket, bucket stored in 72°F ambient;																	
nominal: 8-hr preset, 120°F maximum temperature																	
FINAL MIXTURES: T20, T30-T32 (paste volume and composition, admixtures)																	

Table 11.8: Development of Nominal 7,000 psi Mixtures, Crushed Limestone Aggregate Set

ID	Proportions (SSD)					Indices					Workability				Strength			Comments												
	Cem-ent lb/yd ³	Fly Ash lb/yd ³	Coarse Agg. lb/yd ³	Fine Agg. lb/yd ³	Water lb/yd ³	HR-A oz/yd ³	PT- VMA- 1482 oz/yd ³	Paste Vol %	S/A w/cm	w/c w/c	Ash %	Initial Reading		In. +10 min		In. +20 min			16-Hr f _{cd}											
												SI. F. in.	T ₅₀ s	VS in.	PA mm	SI. F. in.	T ₅₀ s		VS in.	PA mm	SI. F. in.	T ₅₀ s	VS in.	PA mm	72°F Bckt. psi	100°F Nom. psi				
T48	720	180	1381	1381	243.3	114	578	8	36.8	0.50	0.27	0.338	20	29	11.8	0.0	45	29	5.8	0.5	5	5780	7003	7655	added 4 oz/yd ³ VMA after ini. test, 4 oz/yd ³ VMA after 2nd test					
T49	720	180	1492	1271	243.3	111	578	8	36.8	0.46	0.27	0.338	20	25.5	11.5	0.0	8	29	9.3	0.5	44	5618	6819	7503	added 13 oz/yd ³ HR-A after ini. test, 8 oz/yd ³ VMA after 2nd test					
T50	720	180	1603	1161	243.3	118	578	8	36.8	0.42	0.27	0.338	20	26	13.4	0.0	11	29	13.6	0.5	47	5869	7142	7621	added 19 oz/yd ³ HR-A and 8 oz/yd ³ VMA after ini. test					
T51	715	185	1369	1369	252.4	117	578	0	37.4	0.50	0.28	0.353	20.6	27	9.1	0.0	15	27	9.2	0.0	13	3446	4843	6767						
T52	715	185	1478	1259	252.4	112	578	0	37.4	0.46	0.28	0.353	20.6	28	10.9	0.5	35	27.5	7.8	0.5	22	27.5	7.3	0.0	16	4705	6077	7478		
T53	715	185	1587	1149	252.4	105	578	4	37.4	0.42	0.28	0.353	20.6	29	7.3	1.0	50	29	7.1	0.5	30	27.5	7.4	0.5	33	5607	6744		added 21 oz/yd ³ HR-A after ini. test	
T54	720	180	1352	1352	265.9	107	578	4	38.2	0.50	0.295	0.369	20	24	7.2	0.0	0	29	4.0	0.5	18	26	4.0	0.0	4					
T55	720	180	1460	1244	265.9	95	578	4	38.1	0.46	0.295	0.369	20	27	6.0	0.0	10	26	4.8	0.0	4	23.5	5.6	0.0	2					
T56	720	180	1568	1136	265.9	91	578	6	38.1	0.42	0.295	0.369	20	26.5	6.1	0.0	18	25	5.9	0.0	2	23.5	6.1	0.0	3					
T54R	720	180	1352	1352	265.9	105	578	4	38.2	0.50	0.295	0.369	20	29	4.0	0.5										4249	5350	6529	strength only	
T55R	720	180	1460	1244	265.9	99	578	4	38.1	0.46	0.295	0.369	20	26	5.4	0.0										3998	5417	6162	strength only	
T56R	720	180	1568	1136	265.9	103	578	6	38.1	0.42	0.295	0.369	20	28	5.3	1.0										3658	5095	5945	strength only	
T57	720	180	1362	1362	256.9	110	578	0	37.7	0.50	0.285	0.357	20	25	7.7	0.0	2	27	6.0	0.0	19	25	5.7	0.0	8	5467	6411	8025	7453	added 9 oz/yd ³ HR-A after ini. test
T58	720	180	1471	1253	256.9	108	578	0	37.7	0.46	0.285	0.357	20	28.5	6.9	0.5	16	27	6.0	0.0	16	25	7.9	0.0	2	4897	5959	7610	6878	
T59	720	180	1580	1144	256.9	105	578	3	37.7	0.42	0.285	0.357	20	28	7.5	1.0	15	27	5.9	0.0	20	26	6.4	0.0	10	4715	5677	7501	6960	

Mixtures developed with BASF Construction Chemicals, LLC; only mixtures tested at University of Texas are shown
Note: concrete mixed continuously between workability tests; cylinders obtained when mixtures were stable; PA = penetration apparatus depth
HR-A dosage indicates total dosage (including additions after first test)
Curing: 72°F and 100°F; cylinders at ambient temperature; Bckt.: 3 4x8 in. cylinders in 5-gal plastic bucket, bucket stored in 72°F ambient; nominal: 8-hr preset, 120°F maximum temperature
FINAL MIXTURES: T57, T58, T59

Table 11.9: Effect of HRWRA Type on Dosage Response (Sensitivity Analysis)

	HRWRA Dosage % cm mass	Elap. Time min.	Workability				
			S. Flow	T ₅₀	VSI	τ ₀	μ
			in.	s		Pa	Pa.s
HR-A	0.219%	0				1192.6	16.6
	0.269%	5	21.0	8.3	0	67.3	41.0
	0.309%	10	26.5	3.8	0	18.1	25.5
	0.349%	16	28.5	3.2	0.5	17.8	22.3
	0.389%	20	29.5	2.9	0.5	10.1	22.0
HR-B	0.196%	0	16.0			184.3	35.2
	0.231%	5	23.5	5.7	0	19.7	28.2
	0.258%	10	26.0	3.9	0	0.0	23.4
	0.294%	15	27	2.5	0	10.9	19.3
	0.329%	19	28	2.4	0	1.9	18.8
HR-D	0.164%	0	14.0			393.9	29.7
	0.192%	5	19.0			124.2	26.7
	0.219%	9	25.0	4.8	0	15.4	26.3
	0.247%	13	27	3.7	0	0.9	21.7
	0.274%	18	28	3.6	0	0.0	18.3
HRWRAs tested in RG-5-50a without RET-A							

Table 11.10: Effect of HRWRA Type on Workability Retention (Sensitivity Analysis)

Elap. Time min.	HR-A (0.35% cm mass)					HR-B (0.29% cm mass)					HR-D (0.25% cm mass)				
	S. Flow	T ₅₀	VSI	τ ₀	μ	S. Flow	T ₅₀	VSI	τ ₀	μ	S. Flow	T ₅₀	VSI	τ ₀	μ
	in.	s		Pa	Pa.s	in.	s		Pa	Pa.s	in.	s		Pa	Pa.s
0	27.0	3.9	0.0	15.7	17.7	27.0	3.7	0.0	7.3	18.9	27.0	2.7	0.0	11.8	15.5
10	17.5			136.4	11.7	25.0	3	0	19.2	15.3	17.5			186.7	7.4
20	14.0			372.1	11.0	19.0			71.9	13.9	15.0			304.4	8.3
30				693.3	14.1	16			186.7	11.4				504.7	7.3
40						13			296.6	12.755					
HRWRAs tested in RG-5-50a without RET-A															

Table 11.11: Compressive Strength-Maturity Data, River Gravel Aggregate Set (Mild Temperature)

Preset Time Hours	Max Temp °F	Equivalent Age		Compressive Strength							
		E _a =35 kJ/mol Hours	E _a =25 kJ/mol Hours	RG-5-C psi	RG-5-50 psi	RG-5-40 psi	RG-5-50a psi	RG-7-C psi	RG-7-50 psi	RG-7-42 psi	
4	170	87.1	51.6	7099	8705	8254	8094	8456	8767	9020	
8	170	53.2	35.7	6441	6857	6835	6745	8260	8119	8324	
8	145	37.5	28.4	5929	6224	6035	5963	7475	7829	7815	
4	120	34.6	27.3	6247	6643	6297	6444	8152	8250	8522	
6	120	29.8	24.5	5741	5806	5937	5942	7397	7826	7727	
8	120	24.5	21.3	5784	5622	5501	5521	6871	7108	7296	
0	75	15.8	15.8	4618	4792	4815	4499	5284	4926	5520	
Equivalent age calculated at reference temperature of 23 °C											

Table 11.12: Compressive Strength-Maturity Data, Crushed Limestone Aggregate Set (Mild Temperature)

Preset Time Hours	Max Temp °F	Equivalent Age		Compressive Strength							
		E _a =35 kJ/mol Hours	E _a =25 kJ/mol Hours	LS-5-C psi	LS-5-50 psi	LS-5-40 psi	LS-5-50a psi	LS-7-C psi	LS-7-50 psi	LS-7-42 psi	
4	170	87.1	51.6	6969	10435	9579	8706	9073	9213	9090	
8	170	53.2	35.7	6018	7345	7075	6850	8473	8621	8640	
8	145	37.5	28.4	5563	6466	6141	6020	7394	7794	8134	
4	120	34.6	27.3	5899	7147	6612	6692	7941	8375	7984	
6	120	29.8	24.5	5555	6448	6137	5816	7437	7944	8276	
8	120	24.5	21.3	5260	6112	5641	5603	7165	7422	7657	
0	75	15.8	15.8	3974	4738	4620	4613	5648	5353	5401	
Equivalent age calculated at reference temperature of 23 °C											

Table 11.13: Compressive Strength-Maturity Data (Hot Temperature)

Pre-Set	Start	Max	RG-5-50a			RG-7-50			RG-5-C		
Time	Temp	Temp	Test Age	Eq. Age	f _c	Test Age	Eq. Age	f _c	Test Age	Eq. Age	f _c
Hours	°F	°F	Hours	Hours	psi	Hours	Hours	psi	Hours	Hours	psi
4	95	145	16.0	62.7	6843				16.1	62.7	6193
4	95	70				16.1	55.3	8651	16.1	94.2	6570
Constant Ambient Temp (95 °F)			16.0	30.7	5650	16.1	25.7	7673	16.1	30.8	5694
			21.1	39.8	6205	21.1	33.6	8452	21.1	40.1	6212
			27.3	51.1	6698	26.9	42.5	9161	26.5	49.6	6486
			40.0	73.6	7480	39.6	61.4	9748	39.2	72.1	6595
			48.3	88.0	7861	47.9	73.9	10361	47.5	86.6	6970

Equivalent age calculated at reference temperature of 23 °C based on measured concrete temperatures

Table 11.14: Compressive Strength-Maturity Data (Cold Temperature)

Pre-Set	Start	Max	RG-5-50a			RG-7-50			RG-5-C		
Time	Temp	Temp	Test Age	Eq. Age	f _c	Test Age	Eq. Age	f _c	Test Age	Eq. Age	f _c
Hours	°F	°F	Hours	Hours	psi	Hours	Hours	psi	Hours	Hours	psi
8	50	80	16.1	11.8	2305	16.1	13.0	1188	16.1	11.7	1574
8	50	105	16.0	15.1	3297	16.1	15.2	2345	16.0	15.0	4068
Constant Ambient Temp (50 °F)			16.1	9.6	620	16.1	11.6	614	16.1	9.9	580
			40.1	23.4	4477	40.0	27.3	4567	40.0	23.3	4931
			64.4	36.1	5841	64.1	42.4	6503	63.7	35.6	5996
			99.6	54.4	6934	99.2	64.4	7960	98.9	53.9	6935
			144.3	77.7	7815	144.0	92.5	8794	143.6	77.2	7623

Equivalent age calculated at reference temperature of 23 °C based on measured concrete temperatures

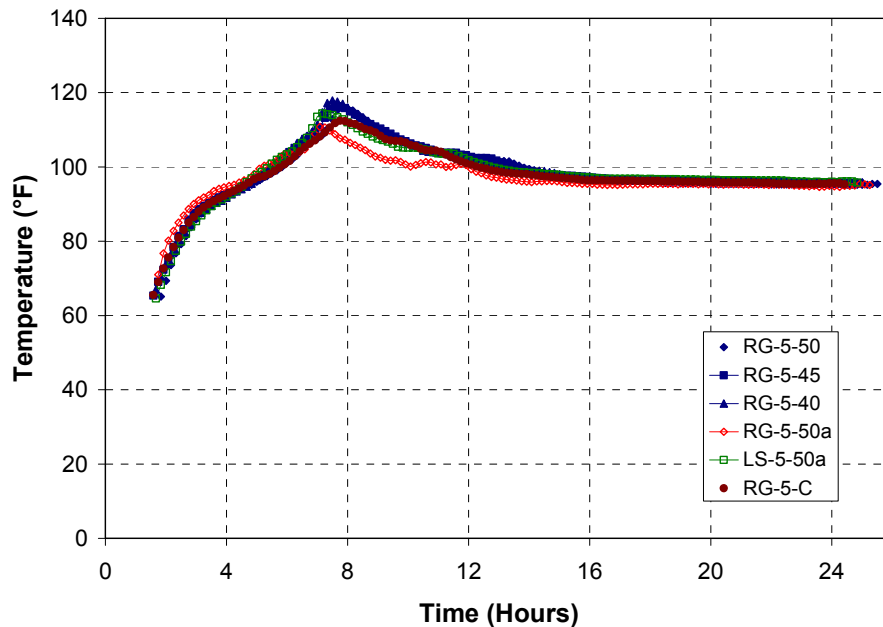


Figure 11.10: Measured Temperature Histories of Specimens for Static and Dynamic Moduli Measurements

Table 11.15: Static Elastic Modulus Measurements

Test Time	RG-5-50				RG-5-45				RG-5-40			
	Eq. Age (23°C)	Comp. Strength	Elastic Modulus	Poisson's Ratio	Eq. Age (23°C)	Comp. Strength	Elastic Modulus	Poisson's Ratio	Eq. Age (23°C)	Comp. Strength	Elastic Modulus	Poisson's Ratio
	hours	psi	ksi		hours	psi	ksi		hours	psi	ksi	
8	12.8	2341	3358	0.15	13.2	2371	3658	0.08	13.4	3060	3757	0.10
12	22.1	5104	4723	0.14	22.7	5054	5000	0.10	22.8	5104	5087	0.10
16	29.9	6122	5568	0.12	30.5	5837	5575	0.12	30.5	6008	5308	0.14
24	44.1	7311	5726	0.12	44.8	7028	5711	0.11	44.8	7181	5495	0.14
72	94.1	8943	6350	0.16	94.4	8838	6385	0.15	94.5	8859	6314	0.15
168	191.0	9887	6469	0.15	191.3	9769	6593	0.15	191.4	10108	6452	0.16
672	696.4	12320	7112	0.15	696.6	12334	7043	0.15	696.7	12023	7154	0.16
Test Time	RG-5-50a				LS-5-50a				RG-5-C			
	Eq. Age (23°C)	Comp. Strength	Elastic Modulus	Poisson's Ratio	Eq. Age (23°C)	Comp. Strength	Elastic Modulus	Poisson's Ratio	Eq. Age (23°C)	Comp. Strength	Elastic Modulus	Poisson's Ratio
	hours	psi	ksi		hours	psi	ksi		hours	psi	ksi	
8	13.2	2864	4022	0.11	13.1	2525	3155	0.14	12.4	2904	4820	0.10
12	21.5	4363	4889	0.13	22.1	4818	4415	0.17	21.7	5192	5743	0.08
16	28.7	5391	5032	0.11	29.8	5724	4598	0.19	29.2	6219	6163	0.10
24	42.7	6221	5443	0.11	44.2	6882	5131	0.18	43.4	6845	6304	0.12
72	92.4	8025	6269	0.15	93.6	8645	5663	0.19	92.8	7797	6876	0.14
168	189.3	8790	6270	0.15	190.5	9801	5603	0.21	189.7	8568	7038	0.14
672	694.6	10597	7159	0.16	695.9	12011	6347	0.22	695.1	9602	7405	0.14

Table 11.16: Dynamic Elastic and Shear Moduli Measurements

Test Time	RG-5-50					RG-5-45					RG-5-40				
	Eq. Age (23°C)	P-Wave Modulus	Elastic Modulus	Shear Modulus	Poisson's Ratio	Eq. Age (23°C)	P-Wave Modulus	Elastic Modulus	Shear Modulus	Poisson's Ratio	Eq. Age (23°C)	P-Wave Modulus	Elastic Modulus	Shear Modulus	Poisson's Ratio
	hours	ksi	ksi	ksi		hours	ksi	ksi	ksi		hours	ksi	ksi	ksi	
1	1.0	6				1.0	6				1.0	7			
3.5	3.5	17				3.7	22				3.7	22			
4.5	5.2	75				5.3	67				5.3	396			
8	12.8	5061	4019	1581	0.27	13.2	5778	5162	2140	0.21	13.4	5493	4799	1964	0.22
11.3	20.8	6014	5234	2140	0.22	21.2	6281	5627	2338	0.20	21.4	5930	5329	2220	0.20
24	44.1	7325	6132	2458	0.25	44.8	7424	6808	2876	0.18	44.8	7195	6571	2766	0.19
72	94.1	8144	7285	3024	0.20	94.4	7772	7037	2944	0.19	94.5	8748	7754	3200	0.21
168	191.0	8448	7519	3111	0.21	191.3	8283	7452	3105	0.20	191.4	9077	7921	3240	0.22
672	696.4	9063	7886	3220	0.22	696.6	8595	7731	3220	0.20	696.7	9348	8073	3285	0.23
Test Time	RG-5-50a					LS-5-50a					RG-5-C				
	Eq. Age (23°C)	P-Wave Modulus	Elastic Modulus	Shear Modulus	Poisson's Ratio	Eq. Age (23°C)	P-Wave Modulus	Elastic Modulus	Shear Modulus	Poisson's Ratio	Eq. Age (23°C)	P-Wave Modulus	Elastic Modulus	Shear Modulus	Poisson's Ratio
	hours	ksi	ksi	ksi		hours	ksi	ksi	ksi		hours	ksi	ksi	ksi	
1	1.0	4				1.0	4				1.0	7			
3.5	3.9	14				3.5	30				3.7	45			
4.5	5.6	412				5.1	350				5.3	426			
8	13.2	5892	5239	2166	0.21	13.1	5778	4520	1771	0.28	12.8	6587	4551	1722	0.32
11.3	20.1	6521	5848	2431	0.20	20.7	6281	5411	2199	0.23	20.6	7577	6033	2371	0.27
24	42.7	7236	6506	2711	0.20	44.2	7325	6543	2714	0.21	43.7	8575	7088	2826	0.25
72	92.4	8094	7030	2869	0.23	93.6	8512	7397	3019	0.23	92.8	8857	7373	2948	0.25
168	189.3	8283	7291	2996	0.22	190.5	9124	7818	3168	0.23	189.7	9197	7840	3169	0.24
672	694.6	8595	7501	3068	0.22	695.9	9496	8040	3240	0.24	695.1	9610	8109	3261	0.24

Table 11.17: Dynamic Elastic Modulus Measurements: Raw Data for Compression Wave Measurements

Mixture RG-5-50 (Specimen A)										Mixture RG-5-50a (Specimen A)										Mixture LS-5-50a (Specimen A)									
Test Time (hours)	Measured Travel Time (μs)	Corrected Travel Time (μs)	Vp (fps)	P-Wave Modulus (ksi)	Input Freq. (Hz)	λ _(Note-2) (in)	d/λ _(Note-3)	Test Time (hours)	Measured Travel Time (μs)	Corrected Travel Time (μs)	Vp (fps)	P-Wave Modulus (ksi)	Input Freq. (Hz)	λ _(Note-2) (in)	d/λ _(Note-3)	Test Time (hours)	Measured Travel Time (μs)	Corrected Travel Time (μs)	Vp (fps)	P-Wave Modulus (ksi)	Input Freq. (Hz)	λ _(Note-2) (in)	d/λ _(Note-3)						
1	1640	1606	415	6	400	12.5	0.64	1	1560	1526	437	6	400	13.1	0.92	1	1890	1856	359	4	400	10.8	1.11						
3.5	900	866	770	19	2000	4.6	2.60	3.5	1040	1006	663	14	2000	4.0	3.02	3.5	727	693	962	30	2000	5.8	2.08						
4.5	460	416	1603	83	2000	9.6	1.25	4.5	211	177	3766	458	2000	22.6	0.53	4.5	217	183	3643	428	2000	21.9	0.55						
8.0	84	50	13333	5733	20000	8.0	1.50	8.0	85.2	51.2	13021	5468	20000	7.8	1.54	8.0	82	48	13889	6221	20000	8.3	1.44						
11.3	82	48	13889	6221	20000	8.3	1.44	11.3	82.3	49	13605	5970	20000	8.2	1.47	11.3	80	46	14493	6774	20000	8.7	1.38						
24.0	77	43	15504	7752	20000	9.3	1.29	24.0	78.6	44.6	14948	7206	20000	9.0	1.34	24.0	77	43	15504	7752	20000	9.3	1.29						
72.0	75	41	16260	8527	30000	6.5	1.85	72.0	74	40	16667	8958	30000	6.7	1.80	72.0	74.7	40.7	16380	8653	30000	6.6	1.83						
168.0	74.1	40.1	16625	8914	40000	5.0	2.41	168.0	73.1	39.1	17050	9375	40000	5.1	2.35	168.0	73.04	39.04	17077	9404	40000	5.1	2.34						
672.0	73	39	17094	9424	40000	5.1	2.34	672.0	72.4	38.4	17361	9720	40000	5.2	2.30	672.0	72.4	38.4	17361	9720	40000	5.2	2.30						
Mixture RG-5-50 (Specimen B)										Mixture RG-5-50a (Specimen B)										Mixture LS-5-50a (Specimen B)									
Test Time (hours)	Measured Travel Time (μs)	Corrected Travel Time (μs)	Vp (fps)	P-Wave Modulus (ksi)	Input Freq. (Hz)	λ _(Note-2) (in)	d/λ _(Note-3)	Test Time (hours)	Measured Travel Time (μs)	Corrected Travel Time (μs)	Vp (fps)	P-Wave Modulus (ksi)	Input Freq. (Hz)	λ _(Note-2) (in)	d/λ _(Note-3)	Test Time (hours)	Measured Travel Time (μs)	Corrected Travel Time (μs)	Vp (fps)	P-Wave Modulus (ksi)	Input Freq. (Hz)	λ _(Note-2) (in)	d/λ _(Note-3)						
1	1440	1406	474	7	400	14.2	0.84	1	1390	1356	492	8	400	14.7	0.81	1	1830	1796	371	4	400	11.1	1.08						
3.5	990	956	697	16	2000	4.2	2.87	3.5	700	666	1001	32	2000	6.0	2.00	3.5	708	674	989	32	2000	5.9	2.02						
4.5	480	446	1495	72	2000	9.0	1.34	4.5	233	199	3350	362	2000	20.1	0.60	4.5	254	220	3030	296	2000	18.2	0.66						
8.0	89	55	12121	4738	20000	7.3	1.65	8.0	83.3	49.3	13523	5897	20000	8.1	1.48	8.0	84	50	13333	5733	20000	8.0	1.50						
11.3	82	48	13889	6221	20000	8.3	1.44	11.3	81.7	47.7	13976	6300	20000	8.4	1.43	11.3	82	48	13889	6221	20000	8.3	1.44						
24.0	78	44	15152	7404	20000	9.1	1.32	24.0	77.2	43.2	15432	7680	20000	9.3	1.30	24.0	78	44	15152	7404	20000	9.1	1.32						
72.0	75.5	41.5	16064	8322	30000	6.4	1.87	72.0	73.6	39.6	16835	9140	30000	6.7	1.78	72.0	74	40	16667	8958	30000	6.7	1.80						
168.0	74.91	40.91	16296	8564	40000	4.9	2.45	168.0	73.04	39.04	17077	9404	40000	5.1	2.34	168.0	72.9	38.9	17138	9472	40000	5.1	2.33						
672.0	73.2	39.2	17007	9328	40000	5.1	2.35	672.0	72.6	38.6	17271	9620	40000	5.2	2.32	672.0	72	38	17544	9926	40000	5.3	2.28						
Mixture RG-5-45 (Specimen A)										Mixture RG-5-50a (Specimen A)										Mixture RG-5-5C (Specimen A)									
Test Time (hours)	Measured Travel Time (μs)	Corrected Travel Time (μs)	Vp (fps)	P-Wave Modulus (ksi)	Input Freq. (Hz)	λ _(Note-2) (in)	d/λ _(Note-3)	Test Time (hours)	Measured Travel Time (μs)	Corrected Travel Time (μs)	Vp (fps)	P-Wave Modulus (ksi)	Input Freq. (Hz)	λ _(Note-2) (in)	d/λ _(Note-3)	Test Time (hours)	Measured Travel Time (μs)	Corrected Travel Time (μs)	Vp (fps)	P-Wave Modulus (ksi)	Input Freq. (Hz)	λ _(Note-2) (in)	d/λ _(Note-3)						
1	1610	1576	423	6	400	12.7	0.95	1	1890	1856	359	4	400	10.8	0.74	1	1500	1466	455	7	400	13.6	0.88						
3.5	867	833	800	21	2000	4.8	2.50	3.5	1000	966	690	15	2000	4.1	2.90	3.5	691	657	1015	33	2000	6.1	1.97						
4.5	533	499	1336	58	2000	8.0	1.50	4.5	213	179	3724	447	2000	22.3	0.54	4.5	221	187	3565	410	2000	21.4	0.56						
8.0	84	50	13333	5733	20000	8.0	1.50	8.0	82	48	13889	6221	20000	8.3	1.44	8.0	80	46	14493	6774	20000	8.7	1.38						
11.3	82	48	13889	6221	20000	8.3	1.44	11.3	79.7	45.7	14588	6863	20000	8.8	1.37	11.3	77.5	43.5	15326	7575	20000	9.2	1.31						
24.0	77.84	43.84	15207	7458	20000	9.1	1.32	24.0	77.52	43.52	15319	7568	20000	9.2	1.31	24.0	75	41	16260	8527	20000	9.8	1.23						
72.0	76.45	42.45	15705	7954	30000	6.3	1.91	72.0	75.45	41.45	16084	8342	30000	6.4	1.87	72.0	74.5	40.5	16461	8738	30000	6.6	1.82						
168.0	74.85	40.85	16320	8589	40000	4.9	2.45	168.0	75	41	16260	8527	40000	4.9	2.46	168.0	73.7	39.7	16793	9094	40000	5.0	2.38						
672.0	74	40	16667	8958	40000	5.0	2.40	672.0	74.3	40.3	16543	8825	40000	5.0	2.42	672.0	73	39	17094	9424	40000	5.1	2.34						
Mixture RG-5-45 (Specimen B)										Mixture RG-5-50a (Specimen B)										Mixture RG-5-5C (Specimen B)									
Test Time (hours)	Measured Travel Time (μs)	Corrected Travel Time (μs)	Vp (fps)	P-Wave Modulus (ksi)	Input Freq. (Hz)	λ _(Note-2) (in)	d/λ _(Note-3)	Test Time (hours)	Measured Travel Time (μs)	Corrected Travel Time (μs)	Vp (fps)	P-Wave Modulus (ksi)	Input Freq. (Hz)	λ _(Note-2) (in)	d/λ _(Note-3)	Test Time (hours)	Measured Travel Time (μs)	Corrected Travel Time (μs)	Vp (fps)	P-Wave Modulus (ksi)	Input Freq. (Hz)	λ _(Note-2) (in)	d/λ _(Note-3)						
1	1480	1446	461	7	400	13.8	0.87	1	1890	1856	359	4	400	10.8	1.11	1	1440	1406	474	7	400	14.2	0.84						
3.5	805	771	865	24	2000	5.2	2.31	3.5	1070	1036	644	13	2000	3.9	3.11	3.5	538	504	1323	56	2000	7.9	1.51						
4.5	455	421	1584	81	2000	9.5	1.26	4.5	222	188	3546	406	2000	21.3	0.56	4.5	213	179	3724	447	2000	22.3	0.54						
8.0	82	48	13889	6221	20000	8.3	1.44	8.0	83	49	13605	5970	20000	8.2	1.47	8.0	81	47	14184	6489	20000	8.5	1.41						
11.3	80	46	14493	6774	20000	8.7	1.38	11.3	80.5	46.5	14307	6629	20000	8.6	1.40	11.3	77.2	43.2	15432	7680	20000	9.3	1.30						
24.0	76.59	42.59	15653	7902	20000	9.4	1.28	24.0	78	44	15152	7404	20000	9.1	1.32	24.0	74.5	40.5	16461	8738	20000	9.9	1.22						
72.0	76	42	15873	8125	30000	6.3	1.89	72.0	75.3	41.3	16142	8403	30000	6.5	1.86	72.0	73.7	39.7	16793	9094	30000	6.7	1.79						
168.0	74.95	40.95	16280	8547	40000	4.9	2.46	168.0	74.8	40.8	16340	8610	40000	4.9	2.45	168.0	74	39	17094	9424	40000	5.1	2.34						
672.0	74.3	40.3	16543	8825	40000	5.0	2.42	672.0	74	40	16667	8958	40000	5.0	2.40	672.0	72	38	17544	9926	40000	5.3	2.28						

Notes:

1. Corrected Travel Time = Measured Travel Time - Calibration Time (34 μs)

2. Wavelength (λ) calculated using the relationship : [Vp = φ × λ]

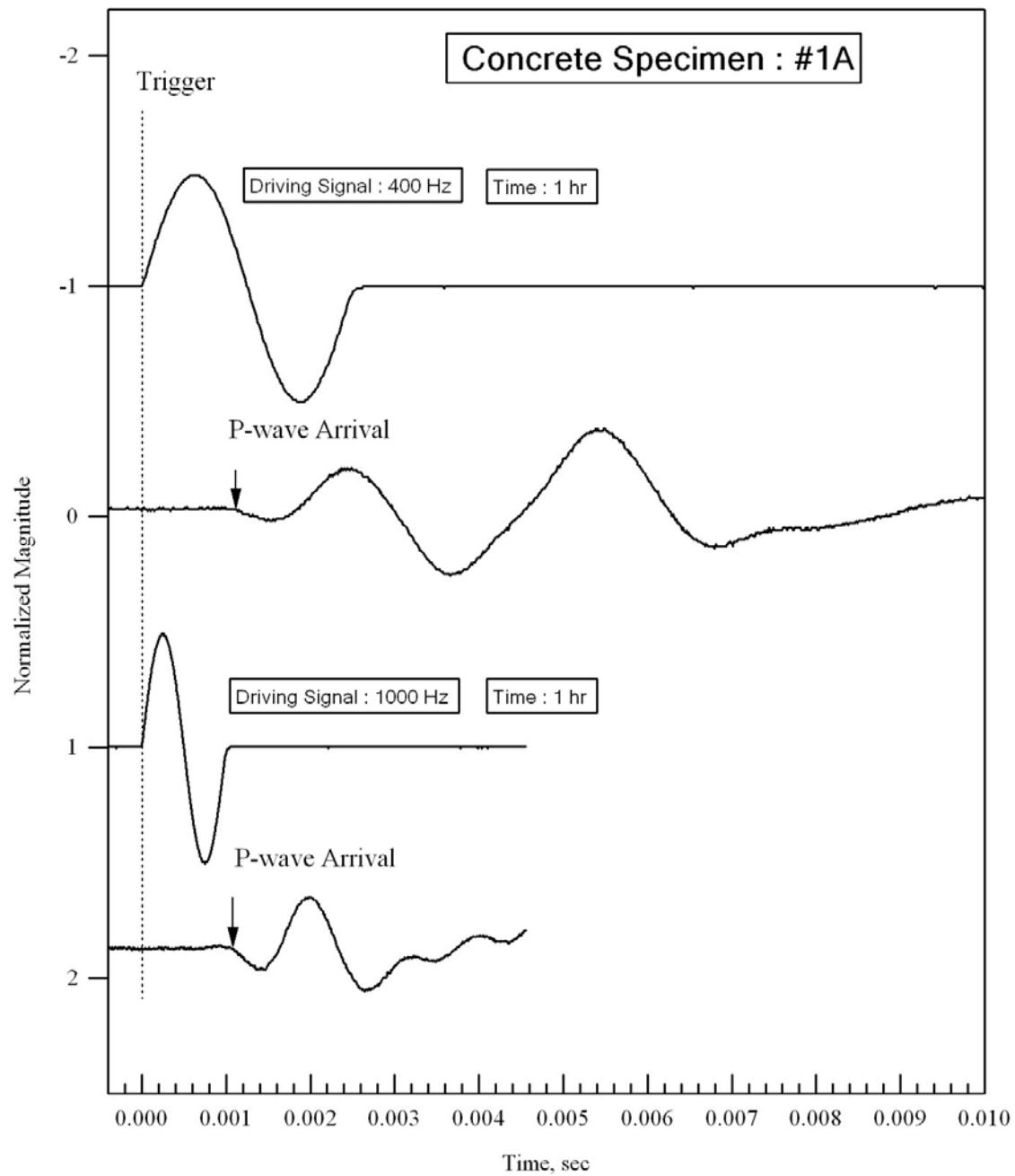
3. Number of wavelengths in travel distance (d = 8 in.)

Notes:
 1. Corrected Travel Time = Measured Travel Time - Calibration Time (34 μs)
 2. Wavelength (λ) calculated using the relationship: $[Vp = \phi \times \lambda]$
 3. Number of wavelengths in travel distance (d = 8 in.)

Table 11.18: Dynamic Elastic Modulus Measurements: Raw Data for Shear Wave Measurements

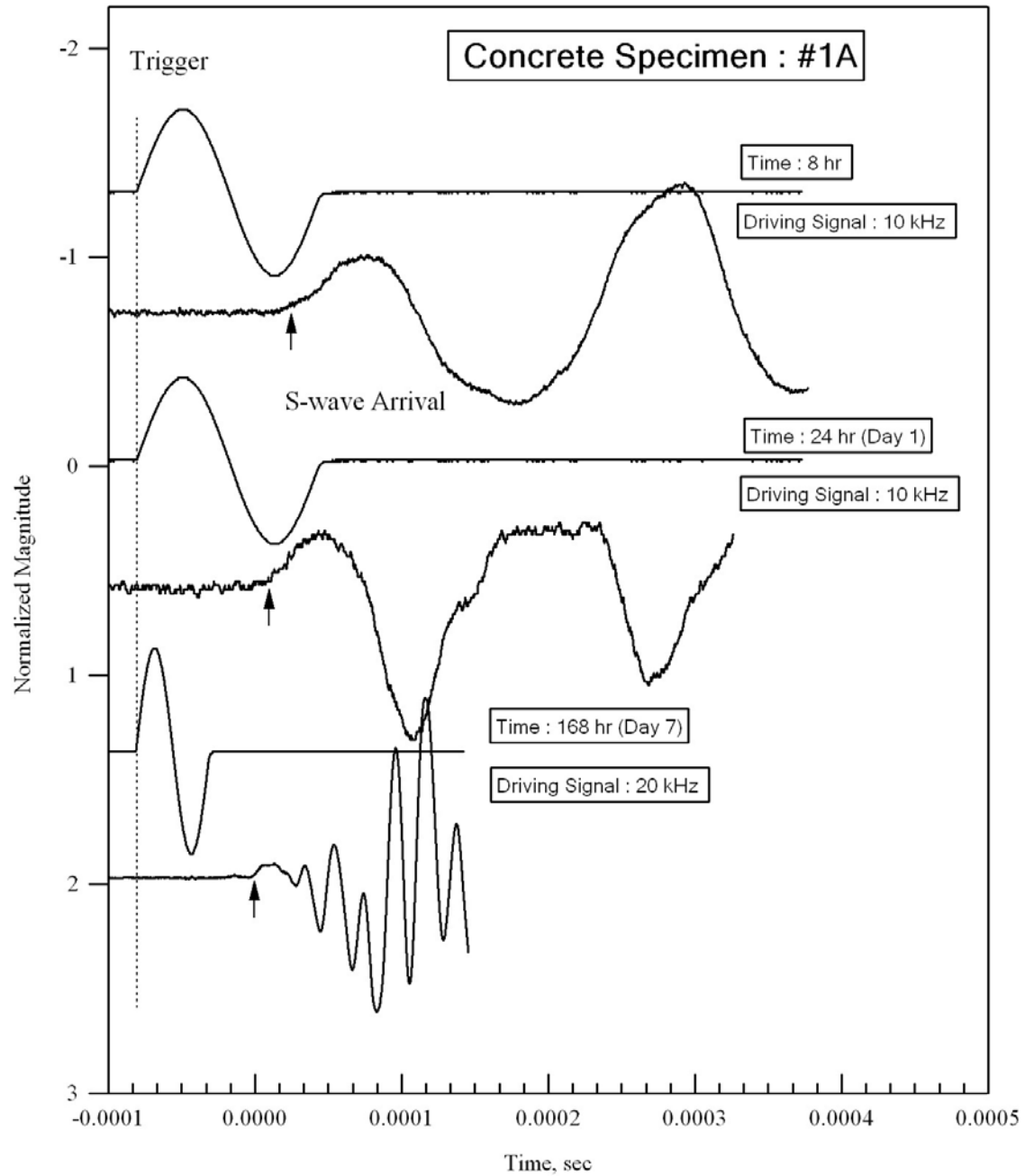
Mixture RG-5-50 (Specimen A)										Mixture RG-5-50a (Specimen A)										Mixture LS-5-50a (Specimen A)									
Test Time (hours)	Measured Travel Time (μs)	Corrected Travel Time ¹ (μs)	Vs (fps)	Shear Modulus (ksi)	Input Freq. (Hz)	λ ^(Note 2) (in)	d/λ ^(Note 3)	Test Time (hours)	Measured Travel Time (μs)	Corrected Travel Time ¹ (μs)	Vs (fps)	Shear Modulus (ksi)	Input Freq. (Hz)	λ ^(Note 2) (in)	d/λ ^(Note 3)	Test Time (hours)	Measured Travel Time (μs)	Corrected Travel Time ¹ (μs)	Vs (fps)	Shear Modulus (ksi)	Input Freq. (Hz)	λ ^(Note 2) (in)	d/λ ^(Note 3)						
8	100	87	7663	1894	10000	9.2	1.31	8	98	85	7843	1984	10000	9.4	1.28	8	103	90	7407	1770	10000	8.9	1.35						
11.3	92	79	8439	2297	20000	5.1	2.37	11.3	92	79	8439	2297	20000	5.1	2.37	11.3	90	77	8658	2417	20000	5.2	2.31						
24	87.2	74.2	8985	2603	20000	5.4	2.23	24	85	72	9259	2765	20000	5.6	2.16	24	83	70	9524	2925	20000	5.7	2.10						
72	80.2	67.2	9921	3174	20000	6.0	2.02	72	79	66	10101	3290	20000	6.1	1.98	72	81	68	9804	3100	20000	5.9	2.04						
168	79	66	10101	3290	20000	6.1	1.98	168	78.8	65.8	10132	3310	20000	6.1	1.97	168	78.7	65.7	10147	3321	20000	6.1	1.97						
672	78	65	10256	3392	20000	6.2	1.95	672	78.5	65.5	10178	3341	20000	6.1	1.97	672	79.7	66.7	9995	3222	20000	6.0	2.00						
Mixture RG-5-50 (Specimen B)										Mixture RG-5-50a (Specimen B)										Mixture LS-5-50a (Specimen B)									
Test Time (hours)	Measured Travel Time (μs)	Corrected Travel Time ¹ (μs)	Vs (fps)	Shear Modulus (ksi)	Input Freq. (Hz)	λ ^(Note 2) (in)	d/λ ^(Note 3)	Test Time (hours)	Measured Travel Time (μs)	Corrected Travel Time ¹ (μs)	Vs (fps)	Shear Modulus (ksi)	Input Freq. (Hz)	λ ^(Note 2) (in)	d/λ ^(Note 3)	Test Time (hours)	Measured Travel Time (μs)	Corrected Travel Time ¹ (μs)	Vs (fps)	Shear Modulus (ksi)	Input Freq. (Hz)	λ ^(Note 2) (in)	d/λ ^(Note 3)						
8	115	102	6536	1378	10000	7.8	1.53	8	96	83	8032	2081	10000	9.6	1.25	8	100	87	7663	1894	10000	9.2	1.31						
11.3	95	82	8130	2132	20000	4.9	2.46	11.3	92	79	8439	2297	20000	5.1	2.37	11.3	95	82	8130	2132	20000	4.9	2.46						
24	89	76	8772	2482	20000	5.3	2.28	24	82.6	69.6	9579	2959	20000	5.7	2.09	24	86	73	9132	2690	20000	5.5	2.19						
72	81.2	68.2	9775	3082	20000	5.9	2.05	72	78.6	65.6	10163	3331	20000	6.1	1.97	72	80.57	67.57	10015	3234	20000	5.9	2.03						
168	80.5	67.5	9877	3146	20000	5.9	2.03	168	78	65	10256	3392	20000	6.2	1.95	168	79.57	66.57	10015	3234	20000	6.0	2.00						
672	79.2	66.2	10070	3271	20000	6.0	1.99	672	77.4	64.4	10352	3456	20000	6.2	1.93	672	79	66	10101	3290	20000	6.1	1.98						
Mixture RG-5-45 (Specimen A)										Mixture RG-5-50a (Specimen A)										Mixture RG-5-C (Specimen A)									
Test Time (hours)	Measured Travel Time (μs)	Corrected Travel Time ¹ (μs)	Vs (fps)	Shear Modulus (ksi)	Input Freq. (Hz)	λ ^(Note 2) (in)	d/λ ^(Note 3)	Test Time (hours)	Measured Travel Time (μs)	Corrected Travel Time ¹ (μs)	Vs (fps)	Shear Modulus (ksi)	Input Freq. (Hz)	λ ^(Note 2) (in)	d/λ ^(Note 3)	Test Time (hours)	Measured Travel Time (μs)	Corrected Travel Time ¹ (μs)	Vs (fps)	Shear Modulus (ksi)	Input Freq. (Hz)	λ ^(Note 2) (in)	d/λ ^(Note 3)						
8	84	71	8130	2132	10000	9.8	1.23	8	92	79	8439	2297	10000	10.1	1.19	8	106	93	7168	1657	10000	8.6	1.40						
11.3	82	69	8547	2356	20000	5.1	2.34	11.3	88	75	8889	2548	20000	5.3	2.25	11.3	91	78	8547	2356	20000	5.1	2.34						
24	77	64	9524	2925	20000	5.7	2.10	24	85	72	9259	2765	20000	5.6	2.16	24	85	72	9259	2765	20000	5.6	2.16						
72	75	62	9704	3037	20000	5.8	2.06	72	82	69	9662	3011	20000	5.8	2.07	72	83	70	9524	2925	20000	5.7	2.10						
168	74.1	61.1	10086	3281	20000	6.1	1.98	168	81	68	9804	3100	20000	5.9	2.04	168	81	68	9804	3100	20000	5.9	2.04						
672	73	60	10256	3392	20000	6.2	1.95	672	80.2	67.2	9921	3174	20000	6.0	2.02	672	80.2	67.2	9921	3174	20000	6.0	2.02						
Mixture RG-5-45 (Specimen B)										Mixture RG-5-50a (Specimen B)										Mixture RG-5-C (Specimen B)									
Test Time (hours)	Measured Travel Time (μs)	Corrected Travel Time ¹ (μs)	Vs (fps)	Shear Modulus (ksi)	Input Freq. (Hz)	λ ^(Note 2) (in)	d/λ ^(Note 3)	Test Time (hours)	Measured Travel Time (μs)	Corrected Travel Time ¹ (μs)	Vs (fps)	Shear Modulus (ksi)	Input Freq. (Hz)	λ ^(Note 2) (in)	d/λ ^(Note 3)	Test Time (hours)	Measured Travel Time (μs)	Corrected Travel Time ¹ (μs)	Vs (fps)	Shear Modulus (ksi)	Input Freq. (Hz)	λ ^(Note 2) (in)	d/λ ^(Note 3)						
8	92	79	8439	2297	10000	10.1	1.19	8	94	81	8230	2185	10000	9.9	1.22	8	102	89	7491	1810	10000	9.0	1.34						
11.3	89	76	8772	2482	20000	5.3	2.28	11.3	89	76	8772	2482	20000	5.3	2.28	11.3	90	77	8658	2417	20000	5.2	2.31						
24	81.84	68.84	9684	3025	20000	5.8	2.07	24	84	71	9390	2843	20000	5.6	2.13	24	83	70	9524	2925	20000	5.7	2.10						
72	81.5	68.5	9732	3055	20000	5.8	2.06	72	83	70	9524	2925	20000	5.7	2.10	72	82	69	9662	3011	20000	5.8	2.07						
168	80.52	67.52	9874	3144	20000	5.9	2.03	168	81	68	9804	3100	20000	5.9	2.04	168	79.1	66.1	10086	3281	20000	6.1	1.98						
672	79.2	66.2	10070	3271	20000	6.0	1.99	672	80.2	67.2	9921	3174	20000	6.0	2.02	672	78	65	10256	3392	20000	6.2	1.95						

Notes:
 1. Corrected Travel Time = Measured Travel Time - Calibration Time (13 μs)
 2. Wavelength (λ) calculated using the relationship: $[V_p = \phi \times \lambda]$
 3. Number of wavelengths in travel distance (d = 8 in.)



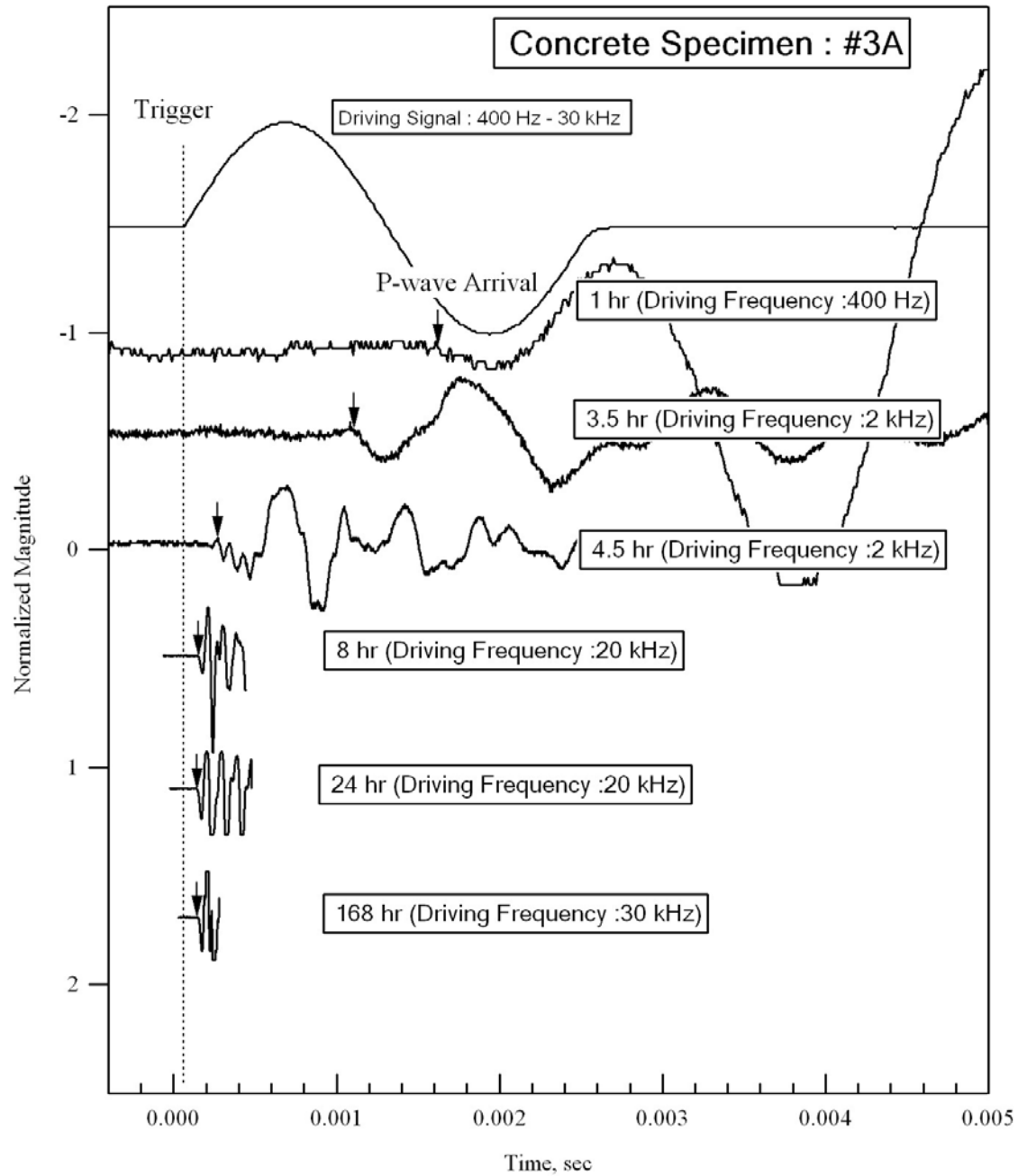
(Note: The initial arrival of P-wave is indicated with the arrow.)

Figure 11.11: Typical P-Wave Signals for Concrete Specimen #1A (Time: 1 hour), Mixture RG-5-50



(Note: The initial arrival of S-wave is indicated with the arrow.)

Figure 11.12: Typical S-Wave Signals for Concrete Specimen #1A (Waterfall Plot), Mixture RG-5-50



(Note: Trigger shown in plot is generic; actual trigger varied for each measurement time. The initial arrival of P-wave is indicated with the arrow.)

Figure 11.13: Typical P-Wave Signals for Concrete Specimen #3A (Waterfall Plot), Mixture RG-5-40

Table 11.19: Shrinkage Measurements

	Shrinkage (μ-strain)											112-d CoV
	initial	4 hr	8 hr	24 hr	48 hr	4 d	7 d	14 d	28 d	56 d	112 d	
RG-5-C	0	-70	-83	-120	-153	-183	-223	-257	-300	-360	-397	1.5%
RG-5-50	0	-93	-93	-130	-133	-173	-207	-280	-327	-353	-387	6.5%
RG-5-45	0	-47	-50	-120	-147	-180	-223	-277	-320	-353	-397	8.9%
RG-5-40	0	-70	-107	-137	-160	-190	-223	-273	-310	-343	-397	2.9%
RG-5-50a	0	-83	-83	-113	-140	-173	-213	-257	-283	-347	-393	3.9%
RG-7-C	0	-97	-107	-160	-193	-260	-273	-310	-373	-393	-423	9.5%
RG-7-50	0	-70	-130	-155	-195	-245	-295	-385	-445	-465	-505	1.4%
RG-7-46	0	-63	-80	-167	-217	-250	-300	-353	-407	-440	-480	3.6%
RG-7-42	0	-93	-137	-187	-217	-263	-317	-380	-423	-450	-507	1.1%
LS-5-C	0	-100	-103	-137	-167	-237	-283	-353	-450	-497	-537	5.4%
LS-5-50	0	-93	-103	-130	-157	-220	-280	-353	-403	-487	-510	3.9%
LS-5-45	0	-87	-103	-150	-167	-230	-290	-353	-437	-480	-503	12.9%
LS-5-40	0	-83	-93	-123	-167	-213	-253	-310	-393	-453	-477	4.4%
LS-5-50a	0	-107	-117	-140	-187	-207	-260	-353	-423	-470	-503	4.6%
LS-7-C	0	-87	-107	-150	-183	-247	-290	-343	-383	-463	-467	6.5%
LS-7-50	0	-90	-120	-183	-217	-293	-373	-447	-530	-583	-587	4.3%
LS-7-46	0	-103	-127	-177	-213	-290	-353	-410	-507	-567	-583	2.6%
LS-7-42	0	-93	-93	-127	-197	-247	-320	-417	-477	-530	-553	2.8%
RG-5-50a (no RET-A)	0	-70	-77	-113	-133	-157	-217	-277	-333	-397	-403	5.2%
RG-7-50 (no PT-1482)	0	-60	-70	-110	-127	-160	-217	-267	-310	-353	-363	3.2%

Table 11.20: Segregation Resistance Test Results, Phase I (1 of 2)

ID	Proportions (SSD)							Indices			Slump Flow Test							
	Cement	Fly Ash	Coarse Agg.	Fine Agg.	Water	HR-A	RET-A	Paste Vol.	S/A	w/cm	0 Minutes				15 Minutes			
											Flow	T ₅₀	VSI	PA	Flow	T ₅₀	VSI	PA
	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	oz/cwt	oz/cwt	%			in.	sec		mm	in.	sec		mm
S1	627.0	209.0	1584.0	1291.0	250.8	9.0	4	34.0	0.45	0.30	28	5.4	1.5	max	20.5	10.4	0.0	0
S2	548.6	182.9	1680.0	1369.2	219.4	14.5	4	30.0	0.45	0.30	28	6.7	2.5	35	27	8.2	0.5	18
S3	705.3	235.1	1488.0	1212.8	282.1	7.5	4	38.0	0.45	0.30	30	1.6	1.5	max	24	2.7	0.0	7
S4	691.3	230.4	1584.0	1291.0	221.2	17.0	4	34.0	0.45	0.24	30	11.1	2.0	max	30	10.1	1.0	max
S5	573.6	191.2	1584.0	1291.0	275.3	6.5	4	34.0	0.45	0.36	28	2.0	0.5	11	16.5		0.0	0
S6	713.0	237.7	1526.9	1244.5	251.3	10.0	4	36.4	0.45	0.264	29	4.9	0.0	20	23	7.7	0.0	0
S7	550.0	183.3	1641.1	1337.5	246.1	9.5	4	31.6	0.45	0.336	28	5.0	1.0	30	20		0.0	0
S8	599.1	199.7	1584.0	1291.0	263.6	7.5	4	34.0	0.45	0.33	30	2.3	2.0	max	20		0.0	0
S9	550.2	183.4	1584.0	1291.0	286.1	6.0	4	34.0	0.45	0.39	29	1.2	1.5	30	20		0.0	0

Note: PA = penetration apparatus depth; HRWRA and VMA per cementitious materials, retarder per cement

Table 11.21: Segregation Resistance Test Results, Phase I (2 of 2)

ID	Rheology																Segregation		
	0 min				3 min				7 min				15 min				Col. Seg.	Sieve Stab.	Hard. Col.
	τ_0	μ	τ_0	Thixo-tropy	τ_0	μ	τ_0	Thixo-tropy	τ_0	μ	τ_0	Thixo-tropy	τ_0	μ	τ_0	Thixo-tropy			
	(dyn.)		(static)		(dyn.)		(static)		(dyn.)		(static)		(dyn.)		(static)				
	Pa	Pa.s	Pa	Nm/s	Pa	Pa.s	Pa	Nm/s	Pa	Pa.s	Pa	Nm/s	Pa	Pa.s	Pa	Nm/s	%	%	%
S1	0.01	32.5	72.3	0.083	0.01	39.8	137.2	0.186	0.01	39.0	137.2	0.148	22.0	62.4	557.5	0.42	0.9	2.3	9.3
S2	0.9	36.5	76.9	0.148	15.0	39.4	153.8	0.209	0.0	49.1	261.9	0.366	0.0	51.7	430.0	0.44	19.4	14.8	20.0
S3	6.7	13.9	23.3	-0.04	6.6	16.8	60.0	0.016	7.7	16.0	72.1	0.012	8.6	19.5	126.1	0.078	13.5	14.6	23.0
S4	0.0	61.1	46.6	0.126	0.0	74.2	111.5	0.275	0.0	70.4	141.6	0.283	0.0	110.5	658.0	1.135	68.9	33.7	24.8
S5	65.8	10.3	81.6	0.023	70.1	12.2	111.5	0.027	128.4	10.7	213.6	0.074	154.1	13.4	309.7	0.13	0.0	3.3	0.0
S6	12.1	37.1	60.6	0.08	18.2	40.6	138.0	0.146	26.9	42.2	246.1	0.179	47.5	47.9	426.2	0.224	0.0	8.0	0.0
S7	14.9	23.6		0.075	15.7	26.4		0.088	47.5	23.6		0.117	43.5	29.5		0.185	0.0	3.8	0.0
S8	41.7	12.7		0.014	43.9	14.7		0.017	86.8	12.9		0.029	115.3	15.4		0.047	1.8	2.9	3.4
S9	17.0	5.4		0.001	15.5	7.7		0.007	27.6	7.6		0.024	82.7	6.8		0.039	9.8	6.4	0.0

Note: thixotropy expressed in terms of breakdown area

Table 11.22: Segregation Resistance Test Results, Phase II

ID	Proportions (SSD)							Indices				Slump Flow Test						Rheology								Seg-regregation		
	Cement lb/yd ³	Fly Ash lb/yd ³	Coarse Agg. lb/yd ³	Fine Agg. lb/yd ³	Water lb/yd ³	HR-A oz/cwt	RET-A oz/cwt	Paste Vol. %	S/A	w/cm	0 Minutes				15 Minutes				0 min				15 min					
											Flow in.	T ₅₀ sec	VSI mm	PA mm	Flow in.	T ₅₀ sec	VSI mm	PA mm	τ ₀ (dyn.)	μ _s Pa.s	τ ₀ (static)	Thixotropy Nm/s	τ ₀ (dyn.)	μ _s Pa.s	τ ₀ (static)	Thixotropy Nm/s		
																											Col. Seg.	Sieve Stab.
R1	614.4	204.8	1729.8	1249.2	216.5	13.0	4	31.6	0.42	0.26	28	8.3	2.0	40	28	9.7	1.0	27	0.01	33	44.3	0.092	0.0	53.2	646.3	0.456	18.7	35.3
R2	713.0	237.7	1609.5	1162.3	251.3	8.8	4	36.4	0.42	0.26	29	5.0	0.5	35	22	5.3	0.0	1	3.1	27.2	42.0	0.062	35.4	46.9	396.3	0.226	10.9	9.7
R3	550.0	183.3	1729.8	1249.2	246.1	8.5	4	31.6	0.42	0.34	28	6.2	2.5	25	23	5.4	0.0	4	1.8	15.2	32.6	0.034	6.8	24.3	203.4	0.164	11.6	14.6
R4	638.3	212.8	1609.5	1162.3	285.7	6.0	4	36.4	0.42	0.336	28	2.3	2.0	27	21	4.6	0.0	2	31.4	7.6	58.3	0.016	80.6	9.2	182.2	0.056	17.9	11.1
R5	614.4	204.8	1552.4	1425.9	216.5	20.0	4	31.6	0.48	0.264	29	6.9	2.0	40	26	10.8	0.0	35	0.0	35.5	32.6	0.094	0.0	55.4	476.7	0.401	27.3	25.0
R6	713.0	237.7	1444.4	1326.7	251.3	9.5	4	36.4	0.48	0.264	29	6.3	0.0	35	22	9.4	0.0	0	12.7	37.6	62.9	0.075	10.4	65.5	517.4	0.397	1.8	5.4
R7	550.0	183.3	1552.4	1425.9	246.1	9.5	4	31.6	0.48	0.336	28	4.8	2.0	max	22		0.0	0	19.2	17.6	39.6	0.01	28.3	29.3	311.8	0.187	3.3	5.7
R8	638.3	212.8	1444.4	1326.7	285.7	7.0	4	36.4	0.48	0.336	30	1.8	0.5	max	20		0.0	0	19.1	9.5	42.0	0.023	54.4	14.4	193.6	0.087	5.2	6.4
R9	548.6	182.9	1680.0	1369.2	219.4	15.0	4	30.0	0.45	0.3	28	8.4	2.0	max	26	12.0	1.5	18	0.0	35.0	65.3	0.123	4.2	58.4	582.4	0.508	8.0	9.4
R10	705.3	235.1	1488.0	1212.8	282.1	6.5	4	38.0	0.45	0.3	30	2.4	1.0	30	22	4.3	0.0	0	15.9	13.9	44.3	0.013	37.5	22.8	190.7	0.003	17.5	21.8
R11	691.3	230.4	1584.0	1291.0	221.2	13.0	4	34.0	0.45	0.24	30	10.5	1.5	max	28	15.5	0.5	29	0.0	47.4	46.6	0.13	0.0	84.5	313.7	0.549	27.8	38.5
R12	573.6	191.2	1584.0	1291.0	275.3	7.0	4	34.0	0.45	0.36	29	2.0	0.5	max	23	3.2	0.0	5	23.5	6.1	21.0	-0.01	55.1	10.4	139.5	0.046	22.7	13.6
R13	627.0	209.0	1728.0	1147.6	250.8	8.0	4	34.0	0.40	0.3	28	4.1	0.0	35	20		0.0	0	29.9	18.9	86.2	0.028	38.8	32.4	306.8	0.208	3.1	3.9
R14	627.0	209.0	1440.0	1434.4	250.8	9.0	4	34.0	0.50	0.3	29	4.1	2.0	30	20		0.0	0	20.4	21.0	58.3	0.03	43.2	43.6	425.7	0.249	8.7	11.6
R15	627.0	209.0	1584.0	1291.0	250.8	9.0	4	34.0	0.45	0.3	30	2.8	1.5	max	25	4.6	0.0	7	4.2	15.8	21.0	0.011	13.9	29.9	297.8	0.196	21.1	22.9
R16	627.0	209.0	1584.0	1291.0	250.8	8.8	4	34.0	0.45	0.3	28	5.0	1.0	max	20		0.0	2	24.5	20.5	62.9	0.046	22.6	36.7	321.3	0.211	4.0	7.5
R17	627.0	209.0	1584.0	1291.0	250.8	8.8	4	34.0	0.45	0.3	30	4.3	0.5	max	23	5.2	0.0	3	18.5	15.9	39.6	0.014	17.0	27.4	239.0	0.136	4.8	10.7
R18	627.0	209.0	1584.0	1291.0	250.8	9.0	4	34.0	0.45	0.3	29	4.3	1.5	max	22	6.5	0.0	0	9.7	17.5	46.6	0.047	18.1	29.7	276.8	0.528	14.2	17.8

Note: PA = penetration apparatus depth; HRWRA and VMA per cementitious materials, retarder per cement; thixotropy expressed in terms of breakdown area

Note: PA = penetration apparatus depth; HRWRA and VMA per cementitious materials, retarder per cement; thixotropy expressed in terms of breakdown area

Table 11.23: Segregation Resistance Test Results, Phase III

ID	Proportions (SSD)								Indices			Slump Flow Test								Rheology								Segregation	
	Cement	Fly Ash	Coarse Agg.	Fine Agg.	Water	HR-A	VMA	RET-A	Paste Vol.	S/A	w/cm	0 Minutes				15 Minutes				0 min				15 min				Col. %	Sieve %
												Flow	T ₅₀	VSI	PA	Flow	T ₅₀	VSI	PA	τ ₀ (dyn.)	μ	τ ₀ (static)	Thixotropy	τ ₀ (dyn.)	μ	τ ₀ (static)	Thixotropy		
lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	oz/cwt	oz/cwt	oz/cwt	%				in.	sec.	mm.		in.	sec.	mm.		Pa.	Pa.s	Pa	Nm/s	Pa.	Pa.s	Pa	Nm/s	%	%
A1	691.3	230.4	1584.0	1291.0	221.2	15.5	2	4	34.0	0.45	0.24	30	12.5	0.0	max	29	13.6	0.0	max	0.01	38.6	32.6	0.081	0.0	83.4	312.4	0.542	14.8	22.0
A2	691.3	230.4	1584.0	1291.0	221.2	16.5	14	4	34.0	0.45	0.24	30	10.3	0.0	25	27	11.2	0.0	22	0.0	31.9	32.6	0.067	0.0	63.1	192.2	0.321	22.0	27.6
A3	573.6	191.2	1584.0	1291.0	275.3	7.0	2	4	34.0	0.45	0.36	29	3.8	0.0	38	21	4.3	0.0	0	16.4	9.9	35.0	0.009	52.1	14.1	165.1	0.084	10.2	9.0
A4	573.6	191.2	1584.0	1291.0	275.3	7.8	14	4	34.0	0.45	0.36	28	2.5	0.0	25	22	5.4	0.0	0	45.7	8.9	79.3	0.017	129.3	14.2	768.3	0.31	0.0	2.2
A5	627.0	209.0	1584.0	1291.0	250.8	7.5		4	34.0	0.45	0.3	22	5.5	0.0	0	14		0	0	172.3	23.2	233.1	0.074	301.7	37.1	877.7	0.379	0.0	0.1
A6	627.0	209.0	1584.0	1291.0	250.8	14.0		4	34.0	0.45	0.3	32	2.2	3.0	max	30	3.2	2.0	max									101.4	83.2

Note: PA = penetration apparatus depth; HRWRA and VMA per cementitious materials, retarder per cement; thixotropy expressed in terms of breakdown area

Note: PA = penetration apparatus depth; HRWRA and VMA per cementitious materials, retarder per cement; thixotropy expressed in terms of breakdown area

Table 11.24: Segregation Test Results (Mortar Testing)

Conc. ID	Mortar Proportions (SSD)						Indices		Slump Flow Test				Rheology							
									0 Minutes		15 Minutes		0 min			15 min				
	Cem- ent	Fly Ash	Fine Agg.	Water	HR-A	RET-A	Paste Vol.	w/cm	Mini- Flow	Mini- VF	Mini- Flow	Mini- VF	τ ₀	μ	Thixo- trophy	τ ₀	μ	τ ₀ (static)	Thixo- trophy	
									(dyn.)	μ	Thixo- trophy	τ ₀	μ	τ ₀ (static)	Thixo- trophy					
	lb/yd ³	lb/yd ³	lb/yd ³	lb/yd ³	oz/cwt	oz/cwt	%		in.	s	in.	s	Pa	Pa.s	Nm/s	Pa	Pa.s	Pa	Nm/s	
R1	1017.8	339.3	2069.5	358.7	13.0	4	52.4	0.26	12.5	8.1	12.3	13	0.01	7	0.002	0.0	13.6	82.2	0.051	
R2	1129.7	376.6	1841.5	398.1	8.8	4	57.6	0.26	12.8	6.6	10.5	9.4	2.6	5.5	-0.01	4.5	11.3	236.1	0.057	
R3	911.1	303.7	2069.5	407.8	8.5	4	52.4	0.34	13.3	2.7	11.5	3.4	0.0	3.1	-0	0.0	3.7	53.2	0.015	
R4	1011.2	337.1	1841.5	452.6	6.0	4	57.6	0.336	14.5	1.9	12.0	2.3	0.0	1.0	-0	4.1	1.4	42.0	0.005	
R5	953.6	317.9	2213.2	336.1	20.0	4	49.1	0.264	12	9.3	11.0	11.2	0.0	8.6	-0	1.9	18.5	58.5	0.019	
R6	1065.8	355.3	1983.1	375.6	9.5	4	54.4	0.264	12	8.1	9.5	12.7	1.4	9.0	0.018	6.1	16.7		0.092	
R7	853.6	284.5	2213.2	382.1	9.5	4	49.1	0.336	11	6.6	8.5	9.9	0.0	10.8	0.011	6.2	16.0		0.073	
R8	954.1	318.0	1983.1	427.0	7.0	4	54.4	0.336	12.8	2.8	10.5	3.5	7.3	2.4	-0.01	17.8	3.7	59.8	0.006	
R9	892.0	297.3	2226.4	356.8	15.0	4	48.8	0.3	12	6.0	12.0	7.1	0.8	5.1	-0	3.2	11.5		0.058	
R10	1070.3	356.8	1840.3	428.1	6.5	4	57.7	0.3	11.3	4.1	9.5	5.6	3.4	4.0	0.009	18.1	4.9	73.3	0.02	
R11	1085.2	361.7	2026.7	347.3	13.0	4	53.4	0.24	12.8	12.8	12.0	17.3	0.0	10.8	-0.01	0.0	23.5		0.152	
R12	900.5	300.2	2026.7	432.2	7.0	4	53.4	0.36	13	1.7	10.8	2.5	0.0	3.5	0.002	7.0	2.9		0.006	
R13	1038.0	346.0	1899.9	415.2	8.0	4	56.3	0.3	12	4.5	9.8	6.6	28.6	2.3	-0.02	10.3	7.2		-0	
R14	935.8	311.9	2141.0	374.3	9.0	4	50.7	0.3	9.75	7.6	8.0	10.8	27.1	8.4	-0	25.3	13.1	219.4	0.059	
R15-18	984.3	328.1	2026.7	393.7	9.0	4	53.4	0.3	13	4.9	10.8	6.6	20.9	4.0	-0.01	18.0	8.2	162.7	0.041	

Appendix C: Concrete Works Verification

The validity of *Concrete Works* in predicting precast concrete temperatures was evaluated by comparing *Concrete Works* analysis results with the field testing data collected on March 12 and 13, 2007 (Phase I Testing, Chapter 9). The input parameters for the *Concrete Works* analysis, which are listed in Table 11., were selected to reflect field conditions as closely as possible.

The *Concrete Works* analysis results are compared to the field data in Figure 11. and Figure 11.. For the conventionally placed concrete mixture (Figure 11.), the *Concrete Works* analysis predicted the maximum temperature to within +/- 5°F and accurately predicted the preset time and time of maximum temperature. For the SCC mixture (Figure 11.), the *Concrete Works* analysis under-predicted the preset time by about 1 hour, under-predicted the maximum temperature by 5°F or less, and accurately predicted the time of maximum temperature. For both mixtures, the *Concrete Works* analysis predicted a slightly greater discrepancy between the temperatures in the web and the two flanges than was recorded in the field.

These *Concrete Works* analysis results are considered to be an excellent prediction of the field test data, especially when considering the difficulty of determining field parameters for use in the analysis (such as the R-value of the tarps and the concrete hydration parameters) and the difficulty of modeling temperature in a relatively small element (a precast beam as compared to a mass concrete element such as a footing or large column). Further testing is needed for a wider range of concrete mixtures and weather conditions to evaluate the validity of the *Concrete Works* analysis results more fully.

Table 11.25: Input Parameters for *Concrete Works* Analysis

Parameter	Conventionally Placed Concrete	Self-Consolidating Concrete
Location	Victoria, TX	Victoria, TX
Date	March 12, 2007	March 12, 2007
Placement Time	3:00 pm	4:00 pm
Formwork Removal	18 hours	17 hours
Mixture	RG-5-C	RG-5-40
Hydration Parameters ¹		
Activation Energy	35,000 kJ/mol	35,000 kJ/mol
α_u	10.47	11.14
τ	1.546	1.410
β	0.682	0.587
H_u	457,000	391,000
Form Type	Steel	Steel
Form Color	Red	Red
Age for Curing Start (Application of Blanket)	1 hour	1 hour
Fresh Concrete Temperature	79°F	81°F
Blanket R-Value	1.0 hr-ft ² -°F/BTU	1.0 hr-ft ² -°F/BTU
Ambient Temperature (Both Days)		
Minimum	64°F	64°F
Maximum	72°F	72°F
Ambient Humidity (Both Days)		
Minimum	70 %	70 %
Maximum	90 %	90 %
Wind Speed (Maximum, Both Days)	5 mph	5 mph
Percent Cloud Cover (Maximum, Both Days)	75 %	75 %
¹ Hydration parameters (α_u , τ , and β) calculated based on first 24 hours of laboratory semi-adiabatic calorimeter data to most accurately reflect early-age heat evolution.		

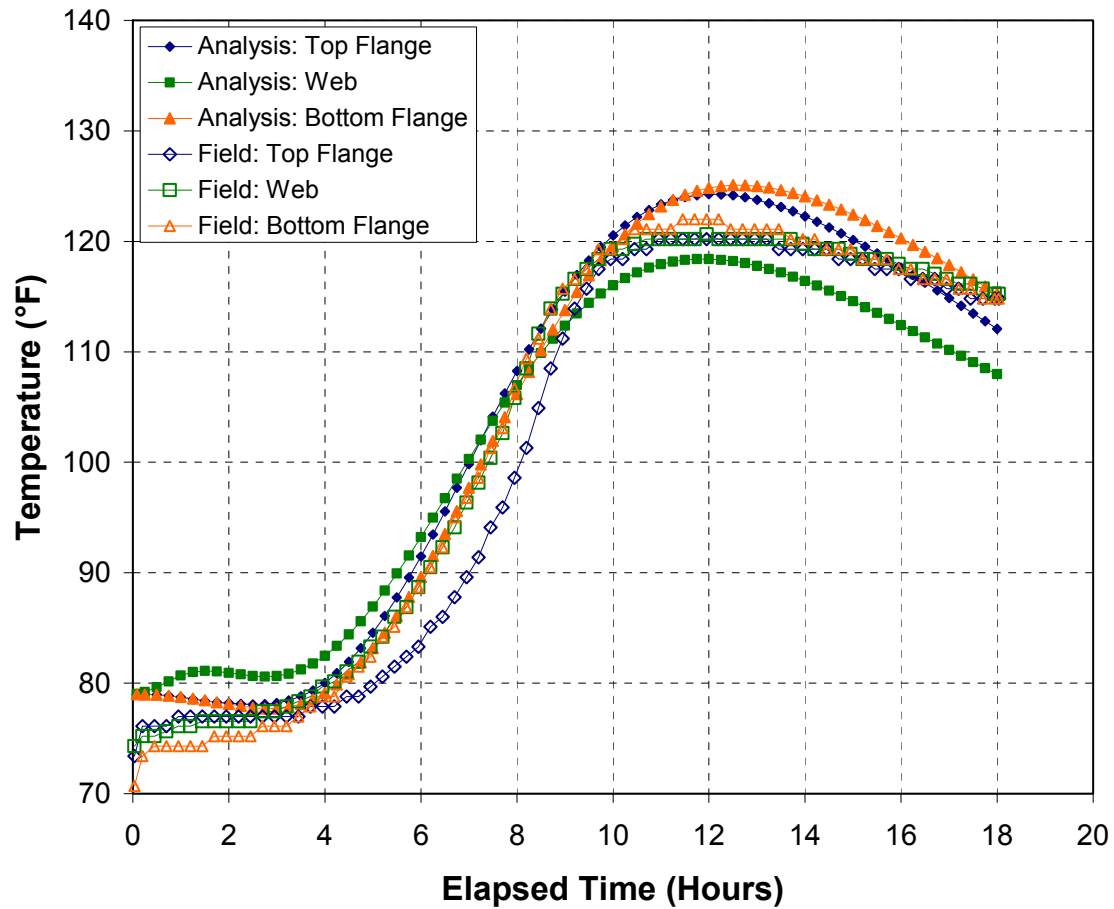


Figure 11.14: Comparison of *Concrete Works* Analysis with Field Data (Conventionally Placed Concrete Mixture)

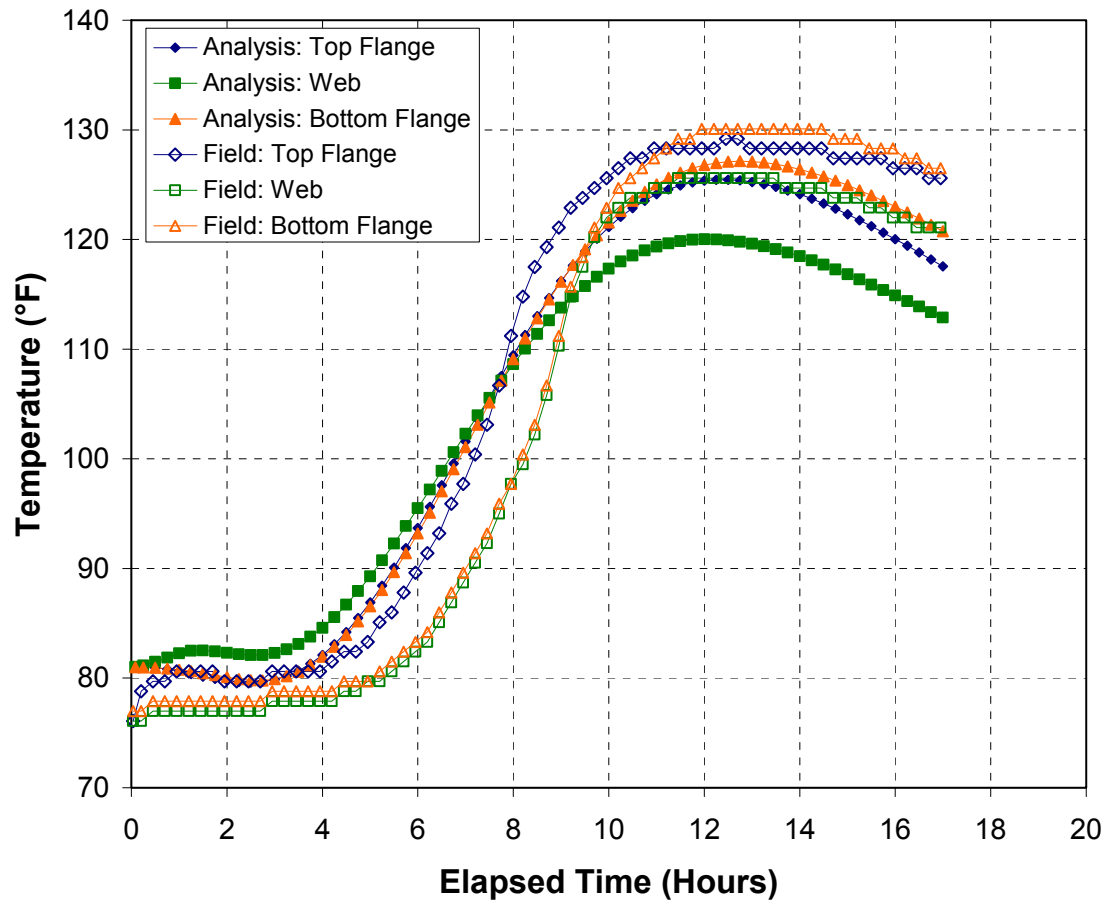


Figure 11.15: Comparison of *Concrete Works* Analysis with Field Data (SCC Mixture)