NCHRP Web-Only Document 76 (Project 18-4B)

Final Report for Early-Opening-to-Traffic Portland Cement Concrete for Pavement Rehabilitation

Prepared for:

National Cooperative Highway Research Program

TRANSPORTATION RESEARCH BOARD
OF THE NATIONAL ACADEMIES

Submitted by:

T. J. Van Dam
K. R. Peterson
L. L. Sutter
A. Panguluri
J. Sytsma
Michigan Technological University
Houghton, MI

N. Buch
R. Kowli
P. Desaraju
Michigan State University
East Lansing, MI

June 2005

ACKNOWLEDGMENT

This work was sponsored by the American Association of State Highway and Transportation Officials (AASHTO), in cooperation with the Federal Highway Administration, and was conducted in the National Cooperative Highway Research Program (NCHRP), which is administered by the Transportation Research Board (TRB) of the National Academies.

CONTENTS

SUMMARY

This report contains the background information, experimental design, results, analysis, and conclusions of the study conducted under NCHRP Project 18-04B, "Durability of 'Early-Opening-to-Traffic' Portland Cement Concrete for Pavement Rehabilitation." Guidelines and recommendations for the design and construction of durable early-opening-to-traffic (EOT) portland cement concrete (PCC) mixtures for pavement rehabilitation were also developed in this study.

As motorists become increasingly less tolerant of delays caused by pavement rehabilitation, state highway agencies (SHAs) have responded by adopting techniques that minimize the time of lane closure. One such technique is to use EOT rehabilitation strategies that allow work to be completed at night or during periods of low traffic. Generally, PCC used in these applications is expected to become strong enough to carry traffic within 6 to 24 hours after placement. But concerns have been raised that such high early strength materials lack durability, resulting in the need for additional lane closures to fix failing repairs.

NCHRP Project 18-04B was conducted to evaluate the durability characteristics of EOT concrete to develop guidelines for materials, mixtures, and construction techniques that enhance long-term durability of EOT concrete for pavement rehabilitation. The research dealt with concrete mixtures that are suited for opening to traffic within (a) 6 to 8 hours and (b) 20 to 24 hours after placement and was limited to full-depth rehabilitation, such as a full-depth repair and slab replacement. In the course of the project, a review of literature was used to design an

experiment that evaluated both field- and laboratory-prepared EOT concrete mixtures. In the experiment, 6- to 8- hour and 20- to 24-hour EOT concrete mixtures obtained from four states (Ohio, Georgia, Texas, and New York) were evaluated to determine typical mixture properties and performance characteristics. Also, a laboratory study was undertaken to produce and test 28 different EOT concrete mixtures (two replicates or batches were made for each mixture for a total of 56 batches). The testing included assessment of the properties of the fresh concrete, volume change, freeze-thaw durability, microstructural characterization, and the absorption/porosity of the concrete. The results were analyzed to draw conclusions regarding the durability of the mixtures and to form the basis for the guidelines. It is expected that the application of these guidelines will enable SHAs to better understand mixture design, proportioning, and construction practices that affect EOT concrete durability and to achieve longer-lasting EOT concrete repairs.

The following general observations were made based on the results of this study:

- In general, the concrete obtained in the field and the concrete produced in the laboratory were of good quality. Difficulty was encountered in obtaining non-durable concrete for use in the analyses. This finding indicates that although some durability problems have been observed in EOT concrete repairs, durable, long-lasting EOT concrete can be produced both in the laboratory and in the field.
- One problem observed in both the field and the laboratory concrete was poorly formed air-void systems that were adequate to protect the concrete against freeze-thaw damage. In the laboratory, the creation of inadequate air-void systems appeared to be linked to interactions

between mixture constituents (e.g., the type of cement and high-range water reducer [HRWR] used in this study). Three of the four 6- to 8-hour EOT concrete and two of the 20- to 24-hour EOT concrete mixtures that were made with Type III cement and Type F HRWR had insufficient air contents as measured in the hardened concrete. It was difficult to control the air content in mixtures made with high cement contents, low water/cement (*w/c*) ratio, and multiple admixtures, especially if Type III cement was used. No specific conclusions can be drawn regarding the general use of Type III cement and/or Type F HRWR as only a single source of each was used in this study. Thus, testing must be conducted to determine if damaging interactions occur. It is noted that all mixtures made with the Type III cement in this study required the use of the Type F HRWR to obtain sufficient workability.

- In general, less homogeneous paste and more microcracking were observed in the field concrete as well as in the laboratory-prepared, 6- to 8-hour EOT concrete specimens. Paste homogeneity of laboratory-prepared specimens, as assessed using the relatively low magnification of the petrographic optical microscope, was better in the 20- to 24-hour mixtures. It appeared that the mixtures made with the Type III cement were slightly less homogenous than those made with Type I cement. Better cement grain dispersion was even observed when the Type F HRWR was used with Type I cement. Severe microcracking of the paste was observed in all of the laboratory-prepared, 6- to 8-hour concrete mixtures. Less severe microcracking was observed in the 20- to 24-hour EOT concrete mixtures, indicating a more consistent and less stressed paste.
- In the field study, although attempts were made to avoid EOT concrete with known alkali-aggregate reactivity problems, alkali-silica reactivity (ASR) was observed in both types of

materials obtained from Ohio. However, it was far less prevalent in the 20- to 24-hour EOT concrete repairs, which contained microsilica as a supplementary cementitious material.

- In the laboratory study, all of the 20- to 24-hour EOT concrete mixtures performed well in the cyclic freeze-thaw testing (AASHTO T 161), whereas some of the 6- to 8-hour mixtures performed poorly, exhibiting relatively high dilation values. Some of these 6- to 8-hour EOT concrete mixtures were made with Type III cement and Type F HRWR and had low air contents. In cases where significant dilation occurred, the dilation correlated mildly with the spacing factor as determined by ASTM C 457. There was some correlation between the air content measured in the hardened concrete and that measured in the fresh concrete, although the air content measured in the fresh concrete was higher than that measured in the hardened concrete when the air content was less than 6 percent. Also, there was little correlation between the air content of fresh concrete and the spacing factor as measured by ASTM C 457. These findings indicate that the air content measured in fresh concrete is not always a good predictor of the suitability of the air-void system to protect the concrete against freeze-thaw damage. The spacing factor was generally found to be a fairly reliable predictor of potential freeze-thaw performance, although in one instance a mixture with a spacing factor below the recommended maximum of 0.200 mm had relatively high dilations.
- In the laboratory study, the scaling results were variable, although the degree of scaling was significantly higher for the 6- to 8-hour EOT concrete mixtures than for the 20- to 24-hour EOT concrete mixtures. The mixtures with Type III cement and Type F HRWR suffered high degrees of deicer scaling, but there was little correlation between the two batches. These results illustrate the importance of using multiple batches and replicates when conducting durability testing. Further, analysis using the x-ray microscope revealed significantly less

overall penetration of chloride ions into the 6- to 8-hour EOT concrete mixtures than into the 20- to 24-hour EOT concrete mixtures, probably due to the reduced *w/c* ratio.

- The majority of the 6- to 8-hour EOT concrete mixtures did not meet the compressive and flexural strength criteria at 6 hours, although most gained sufficient strength by 8 hours. Almost all of the 20- to 24-hour concrete mixtures met the strength criteria by 20 hours. The 6- to 8-hour EOT concrete mixtures had higher 24-hour strengths than the 20- to 24-hour concrete mixtures. Test results did not show a consistent relationship between compressive strength and flexural strength, indicating that if such a correlation is required for construction monitoring, it should be determined on a mix-by-mix basis.

The results of this study were used to develop the guidelines to help SHA personnel to better understand mixture design, proportioning, and construction practices that affect EOT concrete durability.

CHAPTER 1

INTRODUCTION AND RESEARCH APPROACH

1.1 PROBLEM STATEMENT

With increasing traffic in urban areas, motorists are becoming less tolerant of delays during pavement rehabilitation. To minimize delays, state highway agencies (SHAs) use "early-opening-to-traffic" (EOT) rehabilitation strategies that allow work to be completed at night or during periods of low traffic. Generally, portland cement concrete (PCC) used in these applications is expected to become strong enough to carry traffic within 6 to 24 hours after placement. Rigorous requirements for mix design and strength development have usually been stipulated for EOT concrete applications, often with limited consideration given to materials and construction aspects that influence long-term performance and durability. Much of the recent research on EOT concrete investigated the mechanical properties, but not the durability aspect. In the absence of this information, the durability of the concrete used in these applications cannot be predicted, the long-term performance of the rehabilitated pavement cannot be ensured, and the cost-effectiveness of the rehabilitation strategy cannot be adequately assessed.

Research was needed to address durability issues associated with PCC for EOT pavement rehabilitation and to develop guidelines on its use. These guidelines will help engineers to select the materials, mixtures, and construction techniques that will ensure long-term performance, durability, and cost-effectiveness.

1.2 OBJECTIVE AND RESEARCH APPROACH

The objective of the research was to develop guidelines for materials, mixtures, and construction techniques of EOT concrete to obtain long-term durability of pavement rehabilitation. The research dealt with concrete suited for opening to traffic within (a) 6 to 8 hours and (b) 20 to 24 hours after placement and was limited to full-depth rehabilitation such as a full-depth repair and slab replacement. The project was accomplished by executing the following tasks:

1. Conduct a literature review.
2. Establish variables and ranges.
3. Develop a work plan.
4. Conduct the field evaluation, specimen preparation, and laboratory testing of 6- to 8-hour and 20- to 24-hour EOT PCC materials.
5. Develop guidelines.
6. Produce a final report.

This report presents a summary of the research effort. The guidelines developed in this project present the state of the practice for EOT concrete repairs; discuss performance issues related to EOT concrete mixtures, materials, and mixture design considerations; and provide specific recommendations for testing of fresh and hardened EOT concrete. These guidelines are designed for use by transportation agencies to improve the durability of EOT concrete pavement repairs.

CHAPTER 2

BACKGROUND INFORMATION

The study focused on the durability of EOT concrete mixtures used for full-depth repair of concrete pavements. This chapter briefly discusses materials and mixture designs used in EOT concrete and how these relate to specific concerns regarding durability. It concludes with a description of test methods that can be used to assess EOT concrete durability. Detailed background information can be found in Appendix A. (Note: Appendixes can be downloaded for free at http://www.trb.org/news/blurb_detail.asp?id=5203.)

2.1 FULL-DEPTH REPAIR OF CONCRETE PAVEMENTS

Full-depth repair is a commonly used concrete pavement restoration technique that restores both structural integrity and ride quality to concrete pavements. By definition, full-depth repairs are made through the entire depth of the pavement slab, are almost always full-width across the lane, and have minimum specified lengths dependent upon pavement type. After demarcating the repair boundaries with full-depth saw cuts, the deteriorated concrete is removed, the base is repaired, load-transfer devices are installed, and the repair area is filled with new PCC. In situations where lane closure time must be kept to a minimum, the use of EOT concrete is used to expedite reopening to traffic.

There are a number of sources of information regarding full-depth repairs, including publications by the Federal Highway Administration (FHWA 2003), the American Concrete

Pavement Association (ACPA 1995), and the National Highway Institute (NHI 2003). These publications discuss the selection of candidate projects, the sizing of repairs, the installation of load transfer, material selection, construction procedures, and to some degree performance and cost considerations. Only limited discussions regarding material selection are contained in these documents, and little if any guidance is provided for assessing the durability characteristics of the recommended EOT concrete mixtures. This chapter details the state of the practice for selecting mixture constituents and proportioning, construction practices, durability concerns, and commonly applied testing protocols to ensure durability.

2.2 MIXTURE CONSTITUENTS AND PROPORTIONS FOR EOT CONCRETE

The main difference between EOT concrete and normal paving concrete is that strength gain must occur much more rapidly in the former, thus more cement, less water, admixtures, and aids to retain heat are commonly used. This section discusses the various constituents and mixture proportioning of EOT concrete.

2.2.1 Constituent Materials

EOT concrete is composed of the same constituents as normal paving concrete. Coarse and fine aggregates are blended with portland cement, water, and admixtures to produce a stiff but moldable mass that hardens through a chemical process referred to as hydration. In the resulting stone-like mass, the aggregates have been bound together by the hydration products formed through chemical reactions between the water and cement. Air is also entrapped and/or entrained, typically making up 5 to 7 percent of the total volume.

Aggregates

Aggregates make up 70 to 80 percent of the total volume of hardened concrete (Folliard and Smith 2003). As such, they have a large impact on the behavior of the composite, and many characteristics and test methods are used to assess aggregates. This study did not deal with the influence of aggregate properties. Aggregate reactivity, whether alkali-silica reactivity (ASR) or alkali-carbonate reactivity (ACR), and aggregate freeze-thaw durability were not considered in this project. Information on the former is available in the Portland Cement Association (Farney and Kosmatka 1997) and the American Concrete Institute (ACI 2003C) publications. Aggregate freeze-thaw deterioration is described in papers by Schwartz (1987) and Stark (1976). Another aggregate property that may impact EOT concrete durability is the coefficient of thermal expansion (CTE) of the aggregate, which strongly influences the CTE of the concrete.

The CTE of a material is defined as the change in unit length per degree of temperature change (Mehta and Monteiro 1993). Because concrete is a composite material, the CTE of its major constituents could influence concrete performance. Aggregate and cement paste have different CTEs, and the former possesses a lower value (Lea 1971, Neville 1996). This difference in coefficients can lead to the development of thermal stresses within the concrete (Mindess et al. 2003) and in some cases may cause a separation between the aggregate and the paste (Neville 1996). Lea (1971) stated that differential thermal movements, either between the aggregate and cement paste or between different aggregates in the same concrete, could lead to deterioration. In addition, because the CTE influences the thermal shrinkage strain of concrete, it is a particularly important consideration in EOT concrete pavement materials selection. The CTE of concrete can be determined using an AASHTO provisional test method (AASHTO TP 60-00).

The CTE of concrete is strongly related to the proportion and type of coarse aggregate present in the concrete mixture. The CTE for the aggregates are generally lower than the CTE for cement paste. For example, the CTE is approximately 11–13 x $10^{-6}/^{o}C$ (6.1–7.2 x $10^{-6}/^{o}F$) for quartzite aggregate and is approximately 6 x $10^{-6}/^{o}C$ (3.3 x $10^{-6}/^{o}F$) for a limestone aggregate (Mindess et al. 2003). In contrast, the CTE ranges from 18–20 x $10^{-6}/^{o}C$ (10–11 x $10^{-6}/^{o}F$) for the cement paste (Mindess et al. 2003). Aggregate CTEs are influenced strongly by temperature and are almost unaffected by changes in moisture content, whereas the opposite is true for the cement paste (Mindess et al. 2003).

Portland Cement

The selection of cement is an extremely important element in designing durable EOT concrete mixtures. In many applications, the use of a standard AASHTO M 85 Type I cement can provide satisfactory results. However, Type III cement is commonly used in EOT concrete materials because of its high early strength gain. Regardless of the cement type used, the engineer should carefully evaluate the properties of the cement in the context of the long-term physical and chemical stability. Although numerous hydraulic cements are available that can be used to achieve high early strength gain (e.g., calcium sulfoaluminate cements), this study deals only with mixtures made with portland cements. Supplementary cementitious materials—including fly ash, ground granulated blast furnace slag, and silica fume—that are commonly used in pavement and structural concrete are not discussed in this report because of their limited use in EOT concrete pavement repair.

Specifications for portland cements used in the United States are presented in AASHTO M 85 (ASTM C 150). Type I is the most common cement type employed in pavement construction, including the construction of EOT concrete repairs. The chemical properties of Type III cements are similar to Type I, but Type III cements are ground more finely to promote the development of higher early strength. Type III cements are gaining more widespread use for EOT concrete repairs. Type II cements have been used in some cases. Type IV and V cements are not used because they are not conducive to early strength gains. References that provide detailed descriptions of the physical and chemical characteristics of cements are available (e.g., *Design and Control of Concrete Mixtures* [Kosmatka et al. 2002]).

Variations in the physical and chemical characteristics of cement can affect concrete durability. For example, the use of fine cements (e.g., Type III) increases early strength, yet increases water demand affecting flow characteristics (Mindess et al. 2003) and increases the amount of air-entraining agent needed (Whiting and Nagi 1998).

The chemical composition of cement may also affect concrete performance. For example, modern cements have greater amounts of C_3S (3 to 10 percent more) and less C_2S (5 to 14 percent less) than earlier cements. This contributes to faster early strength gain, which is desired for EOT concrete, but may adversely affect the long-term strength development and durability. Although the Portland Cement Association (PCA) report (Kosmatkta et al. 2002) does not show dramatic changes in cement properties, there is concern that the combination of small changes in cement characteristics may negatively affect the durability of concrete.

Chemical Admixtures

A number of chemical admixtures can be added to concrete during proportioning or mixing to enhance the properties of fresh and/or hardened concrete. Admixtures commonly used in EOT concrete mixtures include air entrainers, accelerators, and water reducers. Descriptions of these and other chemical admixtures can be found in a number of sources (Kosmatkta et al. 2002, Mehta and Monteiro 1993, Mindess et al. 2003). The following brief discussion focuses on the impact of various admixtures on EOT concrete durability.

The specification for chemical admixtures used in the United States is presented in AASHTO M 194 (ASTM C 494). Air-entraining admixtures are specified under AASHTO M 154. Cement/admixture interactions are not well understood, and compatibility problems can result in non-durable concrete (Kosmatka et al. 2002, Mindess et al. 2003).

Air-Entraining Admixtures. Air-entraining admixtures are specified and tested under AASHTO M 154 and T 157 (ASTM C 260 and C 233), respectively. Air-entraining admixtures are added just prior to or during concrete mixing. When a high-range water reducer is used, the air entrainer is added first in order to ensure that a stable air-void system is formed (Mindess et al. 2003). The entrained air voids protect the hardened concrete against freeze-thaw damage and deicer scaling. They also improve the workability of the fresh concrete, significantly reducing segregation and bleeding.

Air-entraining admixtures function by forming stable bubbles in the fresh paste during mixing. This is accomplished through surface-active agents that concentrate at the interface between air and water, thereby reducing the surface tension so that stable bubbles can form. These surface-active agents are composed of molecules that are hydrophilic (i.e., water loving) at one end and hydrophobic (i.e., water hating) at the other. These molecules align at the interface with their hydrophilic ends in the water and the hydrophobic ends in the air (Mindess et al. 2003). Typical compounds used as air entrainers include salts of wood resins (i.e., vinsol resins), salts of sulfonated lignin, salts of petroleum acids, alkylbenzene sulfonates, and salts of sulfonated hydrocarbons.

The dosage rate for air-entraining admixtures is usually very small, on the order of 0.005 to 0.05 percent active ingredients by weight of cement, normally diluted to assist in accurate batching (Mindess et al. 2003). The amount of entrained air required to protect normal concrete depends on the exposure level and the nominal maximum aggregate size. Recommended air contents for frost-resistant concrete from ACI (2003b) are reproduced in Table 1. Concrete pavements subject to deicer application experience severe exposure conditions.

The air content of fresh concrete can be determined using AASHTO T 152 or T 196 (ASTM C 173 or C 231). Air content alone does not ensure the adequacy of the air-void system, but relatively good correlations exist between air content and frost resistance for air-entrained concrete. The Air Void Analyzer (AVA) is starting to be used for air-void system characterization in fresh concrete, with some SHAs using it during concrete pavement construction. The complete air-void system in hardened concrete can be assessed microscopically using procedures described in ASTM C 457. Some mixtures that contain inadequate air-void system parameters, particularly high-strength mixtures containing water-reducing admixtures, have been shown to exhibit adequate freeze-thaw durability.

Accelerating Admixtures. Accelerating admixtures are commonly used in EOT concrete. ACI 212.3R-91, "Chemical Admixtures for Concrete," defines an accelerating admixture as "a material added to concrete for the purpose of reducing the time of setting and accelerating early strength development" (ACI 2003A). Accelerating admixtures are commonly categorized in one of four basic groups: soluble inorganic salts, soluble organic compounds, quick-setting admixtures, and miscellaneous solid admixtures. Concrete accelerating admixtures should meet the requirements of Type C or E in AASHTO M 194 (ASTM C 494).

TABLE 1 Recommended air contents for freeze-thaw distress resistant concrete (ACI 2003B)

Nominal Maximum Aggregate Size, mm (in.)	Average Air Content, Percent[1]	
	Moderate Exposure[2]	Severe Exposure[3]
9.5 (⅜)	6.0	7.5
12.5 (½)	5.5	7.0
19.0 (¾)	5.0	6.0
25.0 (1)	5.0	6.0
37.5 (1½)	4.5[4]	5.5[4]
75.0 (3)	3.5[4]	4.5[4]
150.0 (6)	3.0	4.0

[1] A reasonable tolerance for air content in field construction is ±1.5 percent.
[2] Exposure is outdoor in a cold climate where the concrete will be only occasionally exposed to moisture prior to freezing and where no deicing salts will be used. Examples are certain exterior walls, beams, girders, and slabs not in direct contact with soil.
[3] Exposure is outdoor in a cold climate where the concrete may be in almost continuous contact with moisture prior to freezing or where deicing salts are used. Examples are pavements, bridge decks, sidewalks, and water tanks.
[4] These air contents apply to the whole as for the preceding aggregate sizes. When testing these concretes, however, aggregate larger than 37.5 mm (1½ in.) is removed by handpicking or sieving, and the air content is determined on the -37.5 mm (1½ in.) fraction of the mixture. (The field tolerance applies to this value.) From this, the air content of the whole mixture is computed.

Soluble inorganic salts include chlorides, bromides, fluorides, carbonates, thiocyanates, nitrites, nitrates, thiosulfates, silicates, aluminates, and alkali hydroxides (ACI 2003A), with calcium salts generally being the most effective (Mindess et al. 2003). This group of accelerators functions primarily by increasing the rate of hydration of the tricalcium silicate phase in the cement. Calcium chloride, which falls into this group, is the most commonly used accelerating admixture. It is relatively inexpensive and readily available. However, it promotes corrosion of embedded steel and may lead to unwanted alteration of the concrete microstructure. Calcium chloride is an inorganic salt that is commercially available in an anhydrous and a dihydrate form. Commercial anhydrous calcium chloride is typically 94 to 97 percent calcium chloride by weight, whereas commercial flake products, which are close to the dehydrate, typically are composed of 77 to 80 percent calcium chloride by weight (ACI 2003A). The optimum dosage recommended is typically 2 percent for Type I calcium chloride (88 percent pure), and 1.5 percent for anhydrous calcium chloride (ACI 2003A, Mindess et al. 2003).

The hydration of tricalcium silicate is responsible for the majority of heat liberated in concrete mixtures, especially at early ages. Calcium chloride has been found to affect only the timeline of this release, providing for earlier heat release in the hydration process of cement, with little or no effect on the overall heat liberated by hydration (ACI 2003A). It has been speculated that the calcium chloride will combine with the tricalcium aluminate phase to form calcium chloroaluminate.

The addition of calcium chloride has a significant impact on the shrinkage and porosity of concrete mixtures. Research has shown that the addition of calcium chloride increases the amount of drying shrinkage while reducing the time frame in which it occurs (Lackey 1992) and thus affects the generation of internal stress. Also, studies have found that calcium chloride increases the number of fine pores present in the cement paste, while decreasing the number of coarse pores present (Suryavanshi et al. 1995). It is believed that this leads to a geometrically discontinuous network of pores, thereby lowering the resistance of the concrete to several types of physical and chemical attack. Furthermore, Wang and Gillott (1990) found that the addition of calcium chloride accelerator increased the air-void content and total porosity of mortar samples.

In many mixtures, less air-entraining admixture may be required to produce the required air content when a calcium chloride accelerator is used. However, in some instances, larger bubble sizes and higher spacing factors are produced, possibly reducing the effectiveness of air-entraining admixtures (ACI 2003A). These characteristics are believed to be at least partly responsible for reduced freeze-thaw resistance exhibited by mixtures containing calcium chloride accelerators. Rezansoff and Stott (1990) and Neville (1996) concluded that freeze-thaw resistance is severely diminished for mixtures containing chloride accelerators, even when an adequate air-void system (as measured by ASTM C 457) is present in the hardened cement paste.

Many studies have documented a slightly decreased compressive strength at 28 days for mixtures containing calcium chloride accelerators. Increases in compressive strength are typically larger than increases for flexural strength of concrete when calcium chloride accelerators are used (Lackey 1992). Research has also shown that for similar compressive strengths, mixtures containing chloride admixture show a reduced tensile strength compared with concrete without the admixture (Rezansoff and Stott 1990). The ability of calcium chloride to accelerate hydration varies with cement type and content. It can significantly accelerate Type I and Type II cement hydration, but it has been found to have little effect on Type III cement hydration (Ramachandran 1984).

Several commercial non-chloride accelerators conforming to ASTM C 494 Type C requirements for accelerating admixtures include calcium nitrate and ammonium calcium nitrate as their active ingredients. Others conform to ASTM C 494 Type E requirements for water-reducing and set-accelerating admixtures in addition to Type C requirements that incorporate calcium nitrate and calcium nitrite as their principal active ingredients. Research has shown that calcium nitrite is useful as a corrosion inhibitor in addition to its accelerating capabilities (Neville 1996).

Calcium nitrate is most effective for portland cements high in belite and low in alkalies (Justness and Nygaard 1997) and produces an increased compressive strength at 28 days for most additions below 1 percent by mass of cement (Rosskopf et al. 1975). Other non-chloride, non-corrosive accelerators are available that contain compounds such as triethanolamine, sodium thiocyanate, and calcium formate. Their impact on concrete durability is not well documented, but it is very unlikely that durability will be improved. It has been generally observed that concrete microstructure produced in rapidly setting concrete is coarser and composed of more soluble hydration products, which in turn are more prone to physical and chemical attack. Therefore, accelerators should only be used when necessary as their use may have a negative impact on the long-term durability of the concrete.

Water-Reducing Admixtures. Water-reducing admixtures are added to reduce the quantity of mixing water required to produce concrete of a given consistency. This allows for a reduction in the *w/c* ratio while maintaining a desired slump, thus having the beneficial effect of increasing strength and reducing permeability. A reduction in water content by 5 to 10 percent is obtainable through the use of conventional water reducers that are specified under AASHTO M 194-94 Type A. This class of water reducer typically will retard set, so accelerators are often added to offset this effect in EOT concrete mixtures. Water reducers that also act as accelerators are specified under AASHTO M 194-94 Type E. They are typically composed of lignosulfates, hydroxylated carboxylic acids, or carbohydrates.

The effect of water reducers on the fresh concrete properties varies with the chemical composition of the admixture, the concrete temperature, cement composition and fineness, cement content, and the presence of other admixtures (Kosmatka et al. 2002). The effect of Type A and Type E water reducers on the air-void structure is unclear because some sources report either no effect or an improvement (Kosmatka et al. 2002) while others report possible adverse effects (Pigeon and Plateau 1995). Thus, the fresh and hardened concrete properties of mixtures containing water reducers should be thoroughly evaluated during design to determine the extent of detrimental interactions that may occur.

High-range water reducers (HRWRs) specified under AASHTO M 194 Type F and Type G (also called superplasticizers) can reduce water content by 12 to 30 percent. In some instances, they have been used in EOT concrete mixtures where high cement contents and a low *w/c* ratio are desired, particularly if Type III cement is used. However, air voids produced in concrete made with HRWRs are often large, thereby increasing the spacing factor, and instability in the air-void system may occur (Kosmatka et al. 2002, Whiting and Nagi 1998, Pigeon and Plateau 1995).

2.2.2 EOT Concrete Mixture Proportioning

This section presents a brief summary of EOT concrete mixture proportioning as presented in documents published by the FHWA (2003), the ACPA (1995), and the NHI (2003). In these publications, EOT concrete repair materials use similar constituents and proportioning as normal paving concrete, except that higher cement contents and lower *w/c* ratios are common.

In addition, Type III (instead of Type I) cement, chemical accelerators, and water-reducing admixtures are used to accelerate strength gain. Specifically, the following items were reported in these publications:

- Type I or III cements are commonly used in EOT concrete. Additional water may be required to enhance workability with Type III cements. The use of a water reducer can reduce this need for extra water (ACPA 1995).
- Cement contents for mixtures that are to be opened to traffic within 24 hours commonly range from 385 to 530 kg/m^3 (650 to 890 lb/yd^3), with more cement being added for earlier opening times. For 24-hour, accelerated-strength concrete, a minimum cement content of 446 kg/m^3 (750 lb/yd^3) should be used (FHWA 2003).
- The *w/c* ratio in EOT concrete is typically between 0.40 and 0.48. In a draft specification for 24-hour accelerated strength concrete, a maximum *w/c* ratio of 0.45 is required (FHWA 2003).
- An accelerator is commonly employed and is almost a necessity for mixtures that are to be opened in 6- to 8-hours. The most common accelerator is calcium chloride, which is commonly added at 1 percent by weight of cement when the air temperature exceeds 27^{o}C (80^{o}F) and up to 2 percent by weight of cement at lower air temperatures.

As part of this study, standard specifications and special provisions for EOT concrete repair materials from a number of SHAs were reviewed in 2000. Details on the mixtures used by several SHAs are presented in Appendix A. A brief summary of the 6- to 8-hour and 20- to 24-hour mixtures is present below.

Six- to 8-hour EOT Concrete

Although time to opening was frequently stipulated in the specifications, it was also commonly linked to strength requirements. For example, the opening criterion for the various SHAs varied from as early as 4 hours (Kansas and Ohio) to as late as 12 hours (Maryland and Minnesota). In some cases, only a time to opening criterion was established, but in others, strength was used as the sole criterion for opening, and some SHAs even used a strength criterion in addition to time to opening. Florida, for example, specified that the compressive strength must exceed 21 MPa (3,000 psi) in 24 hours while allowing a 6-hour time to opening, and New York allowed openening the repair to traffic once the surface temperature of the repair reached 65^{o}C (150^{o}F).

Because the time at which the strength requirements are applied varies from state to state, it was difficult to extract the specific strength requirements during the initial 6 to 8 hours. Even so, the required compressive strength at opening ranged from 8.3 to 24.0 MPa (1,200 to 3,500 psi). The required flexural strength (as measured by third-point loading) ranged from 1.8 to 2.8 MPa (260 to 400 psi).

Research conducted under the Strategic Highway Research Project (SHRP) found that an opening criterion using a third-point modulus of rupture of 2.1 MPa (300 psi) or a compressive strength of 13.8 MPa (2000 psi) to be reasonable (Whiting et al. 1994). Recommendations made by the ACPA are not based on strength, but instead opening time (ACPA 1995). The most recent version of the NHI's *PCC Pavement Evaluation and Rehabilitation* recognizes both minimum strength requirements and time to opening requirements as being acceptable (NHI 2001). It stipulates a flexural strength requirement of 1.7 MPa (250 psi) for third-point loading and 2.1 MPa (300 psi) for center-point loading. The NHI document also states that having a strength requirement is preferable and that maturity meters or pulse-velocity devices may be useful for monitoring the strength development of very high early strength materials (e.g., 4 hours or less curing time).

All of the 16 states except California, which specified use of CSA cement, used either Type I or Type III portland cement in their 6- to 8-hour EOT concrete materials. When specified, the minimum cement contents varied from state to state, ranging from 440 to 534 kg/m^3 (740 to 900 lb/yd^3) for Type I and 390 to 490 kg/m^3 (660 to 825 lb/yd^3) for Type III. In addition, the use of accelerators was always specified for mixtures containing Type I cement, with the most common accelerator being calcium chloride, although New Jersey and Pennsylvania prohibited the use of a chloride-based accelerator. Some of the mixtures proportioned with Type III cement did not specify the use of an accelerator, relying on the early strength gain of the cement. The *w/c* ratios for the 6- to 8-hour EOT concrete mixtures vary widely, with maximum values ranging from 0.33 to 0.49. In general, a higher *w/c* ratio was allowed for mixtures containing Type III cement. In no instance is a supplementary cementitious material (e.g., fly ash, ground granulated blast furnace slag [GGBFS], or silica fume) specified for use in 6- to 8-hour EOT concrete mixtures.

Admixtures commonly specified for use in 6- to 8-hour EOT concrete mixtures included air entrainers, accelerators, and water reducers. The type of air-entraining agent was not specified, but instead air content was specified either directly or through reference to the SHA's standards for normal paving concrete. The most commonly specified accelerator was calcium chloride, either in solution or in flakes. Addition rates range from 1 to 2 percent, and in many cases the recommended rate was based on ambient conditions, with cooler temperatures requiring an increase in the calcium chloride added. The other accelerators allowed included non–chloride-based admixtures meeting AASHTO M 194 Type C or Type E requirements. The Type E admixture also acts as a water reducer, having the added benefit of being able to reduce the *w/c* ratio while maintaining the same workability.

In many cases, the use of water reducers was not specified. Where water reducers were specified for 6- to 8-hour EOT concrete materials, AASHTO M 194 Type A, Type D, Type E, or Type F were permitted. The main concern is that the water reducer, whether low range or high range, may result in retardation of the strength gain. In Ohio—the one state specifying a Type D admixture—the retardation effect would be likely offset by the specified high cement content (534 kg/m^3 [900 lb/yd^3]) and relatively low *w/c* ratio (< 0.40) of the mixtures. As noted previously, there are concerns that the use of Type F admixtures (high-range water reducers) may result in instability of the entrained air-void system.

The use of water reducers was not often specified in SHA specifications for 6- to 8-hour EOT concrete materials. In specifications for 6- to 8-hour EOT concrete materials where water reducers were included, AASHTO M 194 Type A, Type D, or Type F were permitted, with Type A and D being the most common.

Twenty- to 24-hour EOT PCC

Similar to the 6- to 8-hour EOT concrete mixtures, time to opening is frequently stipulated in the SHA specifications, most often being linked to strength requirements. The opening varied from as early as 12 hours to as late as 48 hours. In two cases, only a time to opening criterion is provided, but in all other cases, a strength criterion exists in addition to time to opening, or strength is used as the sole criterion for opening. Similar to the 6- to 8-hour EOT concrete, the time at which the strength requirements are applied varies from state to state, making it difficult to extract the specific strength requirements for the initial 20 to 24 hours.

The required compressive strength at opening ranged from 17 to 24 MPa (2,500 to 3,500 psi). The required minimum flexural strength (as measured by third-point loading) ranged from 2.1 to 4.2 MPa (300 to 600 psi). Although some states (Maryland, Missouri, and Ohio) specified the same strength criterion for both the 6- to 8-hour and the 20- to 24-hour EOT concrete, others (Michigan and Texas) reported lower strength requirements for the 6- to 8-hour EOT concrete.

All of the 11 states required the use of Type I, Type II, or Type III portland cement for the 20- to 24-hour EOT concrete. The specified minimum cement content ranged from 335 to 502 kg/m^3 (564 to 846 lb/yd^3), with only one state, Texas, specifying different minimum cement contents for Type I and Type III (390 and 335 kg/m^3 [658 and 564 lb/yd^3], respectively). The use of accelerators was either not specified or optional for many of these mixtures, except in Michigan, where the use of calcium chloride was required if ambient temperatures fell below 18^{o}C (65^{o}F), and in Georgia and Illinois, where the use of an accelerator was specified. The *w/c* ratios for the 20- to 24-hour EOT concrete mixtures were typically higher than those specified for 6- to 8-hour EOT mixtures, ranging from 0.42 to 0.53. None of the states except Indiana approved the use of a supplementary cementitious material (fly ash, GGBFS, or silica fume) for use in 20- to 24-hour EOT concrete. Indiana allowed a 10-percent fly ash or 15-percent GGBFS addition.

2.3 CONSTRUCTION CONSIDERATIONS

In addition to the selection and proportioning of constituent materials, specialized construction aspects need to be considered when conducting pavement repairs with EOT concrete. Construction of EOT concrete repairs consists of five basic operations: repair boundary identification and material removal, load transfer installation, batching, finishing, and curing. Although many ways exist to accomplish each task, generally accepted guidelines and practices are presented in a number of publications (ACPA 1994, NHI 2003, FHWA 2003). The following is a brief summary of the sequence used to construct full-depth pavement repairs with a particular emphasis on EOT installations.

The first step in the repair process is to identify the extent of deterioration and establish the repair boundaries. Guidance is provided in several publications (ACPA 1995, NHI 2001, FHWA 2003), but the critical task is to ensure that the entire area of deterioration is removed and that minimum repair lengths (2 m [6 ft]) are obtained when the repair is dowelled. The patch should always be a full-lane width for jointed concrete pavements. Saw cutting is most often done full depth, and a diamond saw blade is recommended. Upon completion of the saw cutting, the concrete is removed by either (1) breaking it up into small pieces and removing the pieces via hand tools or construction equipment or (2) removing the existing slab section in one or more large pieces, which induces less damage to the subbase than the first method. After removal of the concrete, the subbase must be carefully prepared to ensure uniformity of support.

The restoration of load transfer is an important consideration. A variety of techniques exist for ensuring that the repair does not fail because of improper load transfer installation. Most of these techniques focus on ensuring that spalling of the concrete and damage to the subbase do not occur because of rotation or movement of the patch. This is accomplished through the installation of dowel bars in jointed concrete pavement (FHWA 2003). Current practice is to drill multiple holes for dowel bars simultaneously using gang-mounted drill bits. Grout is inserted into the hole, the dowel bar is inserted with a twist, and a grout retention disk is used to prevent outflow. Proper dowel bar alignment is critical and must be ensured.

Regardless of whether the EOT concrete is batched at the job site or at a batching facility, it is important that the concrete produced be uniform in consistency and that the constituent materials be intimately blended. This can be ensured by adhering to a proper mixing sequence and timing. Also, admixtures must be added to fresh concrete in an appropriate dosage and order to avoid potential harmful effects. Concrete containing air-entraining admixtures must be sufficiently mixed to ensure the development of an adequate air-void system. Delays in placing the concrete, especially after the accelerator has been added, must be avoided because early setting may negatively impact consolidation of the repair.

Finishing operations should be performed in a timely fashion, and the surface must not be overworked. Because the high early strength materials set rapidly, this step is critical. Surfaces that are overworked often become brittle, are more susceptible to abrasion and/or freeze-thaw damage, and may exhibit a lowered resistance to chemical attack. Trapping of bleed water must also be avoided.

Internal concrete temperature and moisture directly influence early and ultimate concrete properties; thus, curing takes on special importance in EOT concrete installations. Proper curing provisions are necessary to maintain a satisfactory moisture and temperature condition for a sufficient time to ensure proper hydration (FHWA 1994). Protection against moisture loss becomes critical for EOT concrete repairs if a high temperature, a low humidity, high winds, or a combination of these environmental conditions exist because these conditions can heat or cool concrete and draw moisture from exposed concrete surface.

Many SHAs use AASHTO M 148 Class A liquid curing compounds for accelerated concrete paving under normal placement conditions. White pigmented compound (Type II Class A) is the most commonly used. This material has the potential to create a seal that minimizes evaporation of mixing water when it is applied to the surface and exposed edges of concrete. The white color also assists in reflecting solar radiation during bright days to prevent excessive heat development on the concrete surface. This outcome might not be desirable for EOT concrete repairs where heat generated by solar radiation accelerates hydration and thus early strength gain. Concrete repairs located in mountainous and arid climates may require heavier dosage rates of resin-based curing compound meeting AASHTO M 148 Type II Class B requirements. This is largely because concrete in harsher climatic condition is more susceptible to plastic-shrinkage cracking.

In addition to curing compounds, insulating blankets are often used in conjunction with EOT concrete to assist in holding in heat produced by the rapidly hydrating cement paste, thereby aiding in early strength development of the concrete. These blankets are often essential when cool ambient temperatures are present. Insulating blankets do not reduce the need for a curing compound, as blankets typically do not decrease the likelihood that plastic shrinkage cracking will occur. It is not recommended to place blankets too soon after applying a curing compound. In warm conditions, waiting several hours and placing the blankets as the work progresses is acceptable. Concrete exposed to temperature below 4^{o}C (40^{o}F) may need additional blankets.

2.4 EOT CONCRETE MATERIAL CHARACTERIZATION AND DURABILITY

This section briefly summarizes various material characterization parameters that have indirect or direct relevance to the durability of EOT concrete materials. It also describes the various test methods that were used in this study to evaluate the materials produced. The topics covered are concrete strength, shrinkage, durability, microstructural characterization, and absorbtion and permeability.

2.4.1 Concrete Strength

Although concrete strength is not directly related to durability, the strength criterion is an important consideration in deciding when a pavement can be opened to traffic. This is especially true where strength is not assessed in days or weeks, but in hours. Early strength is readily attainable in most EOT concrete mix designs; however, if any concrete is loaded prematurely, the long-term performance of the structure will be compromised. Therefore, EOT concrete repair materials must meet or exceed the criterion set for opening strength.

Compressive Strength. The compressive strength test is the most common strength test made on concrete. It typically employs calculation of a maximum failure stress based on the load at failure to cross-sectional area of a cylindrical test specimen (100 mm x 200 mm or 150 mm x 300 mm [4 in. x 8 in. or 6 in. x 12 in.]). The exact test method is described in AASHTO T 22 (ASTM C 39). Compression test results are typically correlated with other measures of strength, the most common being flexural strength.

Flexural Strength. Flexural strength testing is typically conducted in accordance with AASHTO T 97 (ASTM C 78 [third-point loading test]) or AASHTO T 177 (ASTM C 293 [center-point loading test]). AASHTO T 97 tests a beam at two loading points that divide the beam into thirds and thus creates a region of constant moment and zero shear over the central third of the beam. As a result, the value for flexural strength is typically less than that obtained through single-point loading.

2.4.2 Shrinkage

Total shrinkage that occurs in a concrete mixture is composed of several types of shrinkage that occur at different ages in the life of the material. Although shrinkage of concrete cannot be totally eliminated (excluding the use of expansive cements), it can be reduced or controlled by the use of an appropriate mix design and proper construction techniques. Controlling shrinkage contributes to crack prevention, which helps in preventing physical and chemical attack of concrete. Shrinkage cracking occurs in concrete structures when the tensile strength of the paste is exceeded by the tensile stresses generated by restraint as the concrete shrinks. Shrinkage is controlled during construction by the application of proper curing procedures; however, restraint within the concrete from either aggregate or steel reinforcement may also contribute to shrinkage cracking. Three types of shrinkage—plastic, drying, and autogenous—can affect EOT concrete mixtures.

Plastic shrinkage is the result of free water, or "bleed water," evaporating from the surface of concrete faster than the water appears during finishing operations (Kosmatka et al. 2002). Generally, an evaporation rate of 0.5 $kg/m^2/hour$ (0.1 $lb/ft^2/hour$) or more is considered critical, where evaporation may exceed the rate at which bleed water reaches the surface (Mindess et al. 2003). Plastic shrinkage, although commonly associated with water evaporating at the top of the slab, may also result if water is removed at the bottom of concrete slabs through a dry subbase or formwork (Kosmatka et al. 2002, Mindess et al. 2003). Evaporation leads to an increase in surface tension in the capillaries between cement particles due to the formation of a complex network of menisci. As plastic shrinkage is not uniform throughout the mass, the differential volume changes that occur may result in cracking if the induced tensile stresses exceed the strength of the fresh paste (Mindess et al. 2003). If the amount of evaporation is significant, small irregular cracks can form over the entire surface of the concrete. These cracks, although at first isolated to the slab surface, can develop into full-depth cracks under the influence of additional shrinkage and/or traffic loading. The cracks also provide pathways for chemical attack by destroying the water tightness of the concrete.

Generally, the potential for plastic shrinkage is increased by elevated temperatures (both concrete and air), low relative humidity, high wind velocity, low *w/c* ratio, and a high cement content (Mindess et al. 2003). Many of these factors are associated with EOT concrete installations; thus, plastic shrinkage is particularly important. Any factor that either increases the rate of evaporation or decreases the rate of bleed water rising to the surface makes the concrete more susceptible to plastic shrinkage cracking.

Drying shrinkage occurs after the paste has hardened and results from the strain produced by the loss of water from the hardened material (Mindess et al. 2003). The factors that influence drying shrinkage that are most relevant to EOT concrete materials are the aggregate volume/cement content, the *w/c* ratio, admixtures, aggregate characteristics, and curing. According to Neville (1996), the most important influence on shrinkage is the restraint provided by the aggregate. The amount of restraint provided directly relates to the aggregate volume; as the aggregate volume decreases (with a commensurate increase in paste volume), the amount of shrinkage increases. The use of larger coarse aggregate has the advantage of decreasing paste volume, but will also increase stress along the paste-aggregate interface due to shrinkage and may lead to cracking in this interfacial zone (Mindess et al. 2003).

The *w/c* ratio also directly and significantly affects drying shrinkage, with lower *w/c* ratio mixtures having reduced shrinkage (Neville 1996, Mindess et al. 2003). Thus, from the perspective of drying shrinkage, EOT concrete mixtures will benefit from the low *w/c* ratio that they commonly employ. For a given aggregate source and volume, the *w/c* ratio of concrete is one of the most important parameters for limiting drying shrinkage. Holding all other factors equal, using a low *w/c* ratio minimizes the amount of evaporable water available to cause drying shrinkage of concrete mixtures (Neville 1996). Kosmatka et al. (2002) approach this issue of drying shrinkage a little differently, stating that the most important factor affecting drying shrinkage is the amount of water added per unit volume of concrete and that shrinkage can be minimized by keeping the amount of water added low. Obviously, the aggregate volume, the *w/c* ratio, and the water added all relate to each other, but the main objective is to minimize the amount of evaporable water in the mixture.

There is an AASHTO provisional test method for assessing the potential for cracking due to drying shrinkage. The test is specified in AASHTO PP 34-99, "Standard Practice for Estimating the Crack Tendency of Concrete." In this test, ring specimens are molded in two layers. After 24 hours, the outer mold is removed, exposing the concrete surface. The top and bottom faces of the rings are covered with silicone rubber sealer to prevent moisture loss other than through the outside circumferential area. A steel ring inside the concrete specimen restrains the concrete specimen as it shrinks. This restraint will develop internal tangential tensile stresses, which will cause the concrete to crack once its tensile strength is exceeded (Kraai 1985). The time to cracking and the width and length of these cracks represent the damaging effect.

Concrete with low *w/c* ratio can undergo a process of self-desiccation that can lead to autogenous shrinkage (Mindess et al. 2003). This process is characterized by the removal of water from the capillary pores through the internal use of water in the formation of hydration products. Autogenous shrinkage is most evident as bulk shrinkage of concrete when a reactive pozzolan is used in conjunction with a *w/c* ratio below 0.30 (Mindess et al. 2003). Autogenous shrinkage is relevant to EOT concrete materials because it seems to increase at higher temperatures, in mixtures with higher cement contents, and in concrete made with finer cements possessing a higher C_3A and C_4AF content (Neville 1996).

Similar to drying shrinkage, autogenous shrinkage only occurs in the paste fraction of the concrete. Thus, the relative volume of aggregate to paste can directly impact the magnitude of the measured autogenous shrinkage. Because concrete made with higher volumes of aggregate has less measured autogenous shrinkage than concrete made with lower volumes of aggregate, increased cement contents generally result in increased autogenous shrinkage. However, the restraint created by the aggregate increases shrinkage stresses in the paste and may lead to increased microcracking.

There is no test method that can be easily applied to measure autogenous shrinkage in concrete. However, the impact on the concrete microstructure can be assessed through observations using the petrographic techniques described in ASTM C 856.

2.4.3 Durability

The performance of EOT concrete repairs can be adversely affected by the concrete's lack of durability (i.e., ability to maintain its integrity in the environment in which it is placed). In general, durability problems can be attributed to either physical or chemical mechanisms, although the two types of mechanism often act together to bring about the development of distress. Furthermore, problems with completely different causes may develop simultaneously, thereby complicating the determination of the exact cause(s) of material failure. The work presented in this section is based on that conducted for the FHWA under Contract No. DTFH61-96-C-00073, "Detection, Analysis, and Treatment of Materials-Related Distress in Concrete Pavements" (Van Dam et al. 2002a, Van Dam et al. 2002b). Only materials-related distress that can be directly attributed to the unique properties of EOT concrete will be discussed, including freeze-thaw deterioration, deicer scaling/deterioration, and sulfate attack. Certain types of materials-related distress, such as alkali-aggregate reactivity and corrosion of embedded steel, can be significantly impacted by some characteristics of EOT concrete mixtures (e.g., high–cement-content and chloride-based accelerators), but these topics were not addressed in this project.

Freeze-Thaw Deterioration

Freeze-thaw deterioration is caused by the deterioration of saturated cement paste under repeated freeze-thaw cycles. The mechanisms responsible for internal damage resulting from freeze-thaw actions are not fully understood, but the most widely accepted theories stipulate the development of internal stress in concrete as a result of hydraulic or osmotic pressures caused by freezing. A review of the literature related to these phenomena is provided by Marchand et al. (1994). Deterioration of the cement paste due to freeze-thaw damage manifests itself in the form of scaling, map cracking, or severe cracking and deterioration, most commonly occurring at joints where moisture is more readily available. The addition of an air-entraining agent (an admixture that introduces a system of dispersed, microscopic bubbles in the concrete) could effectively prevent this deterioration.

During construction, measurements of the total air content of fresh concrete are made in an attempt to ensure that adequate entrained air is available to protect the concrete. Three AASHTO test methods (AASHTO T 152, AASHTO T 196, and AASHTO T121) are available for measuring the air content of fresh concrete during construction. These methods, however, do not determine whether the air is truly entrained or entrapped or whether an adequate air-void system has been developed to protect the concrete against freeze-thaw damage. A test method that has been under investigation for a number of years provides a means for measuring the air-void system parameters for fresh concrete. The test equipment, known as the Air-Void Analyzer (AVA), has received mixed reviews (Price 1996, Magura 1996). The only currently acceptable method to assess whether the air-void system in the hardened concrete is adequate is through microscopic analysis in accordance with ASTM C 457, "Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete."

The freeze-thaw resistance of hardened concrete is often tested using AASHTO T 161, "Resistance of Concrete to Rapid Freezing and Thawing," to assess the resistance of concrete specimens to rapidly repeated cycles of freezing and thawing. Only Procedure A in the standard, in which the specimens are frozen and thawed in water, should be used (TRB 1999). Many SHAs have modified this procedure to address their specific needs and experiences.

Deicer Scaling/Deterioration

Deicer scaling/deterioration is typically characterized by scaling or crazing of the slab surface due to the repeated application of deicing chemicals. Although the exact causes of salt scaling are not known, this scaling is commonly believed to be primarily a form of physical attack that is similar to paste freeze-thaw deterioration. Deicer scaling/deterioration is more likely to occur on concrete that has been overvibrated or improperly finished—actions that create a weak layer of paste or mortar that is more susceptible to hydraulic pressures either at or just below the finished surface (Mindess et al. 2003). Even adequately air-entrained concrete can be susceptible to the development of salt scaling. Recommendations for the prevention of salt scaling include providing a minimum cement content of 335 kg/m^3 (564 lb/yd^3) and limiting the *w/c* ratio to 0.45, both of which are common in EOT concrete mixtures. Providing adequate curing and a minimum of 30 days of concrete "drying" before applying deicing chemicals is also recommended (ACPA 1992). ASTM C 672, "Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals," is the most commonly used test to investigate the scaling potential of concrete.

External Sulfate Attack

External sulfate attack results from the penetration of external sulfate ions (present in groundwater, soil, deicing chemicals, etc.) into the concrete. Although its mechanism is complex, sulfate attack is thought to be caused by two chemical reactions: (1) the formation of gypsum through the combination of sulfate and calcium ions and (2) the formation of expansive ettringite through the combination of sulfate ions and hydrated calcium aluminate (ACI 2003b). In either case, the formation of the reaction product(s) leads to an increase in solid volume that can be very destructive to the hardened paste.

Performance testing using ASTM C 452 and C 1012 should be considered to examine the sulfate resistance of portland cements and combinations of cements and pozzolans/slag, respectively. These tests only evaluate the cementitious materials, not the actual job mix formula. Standard tests to determine the sulfate resistance of the concrete are not currently available.

Internal Sulfate Attack

Internal sulfate attack is similar in many ways to external sulfate attack, except that the source of the sulfate ions is internal. Potential internal sources of sulfate include (1) the slowly soluble sulfate contained in clinker, aggregate, and admixtures (such as fly ash) and (2) the decomposition of primary ettringite due to high curing temperatures. Although internal sulfate attack is sometimes called delayed ettringite formation (DEF) or secondary ettringite formation (SEF), the term “internal sulfate attack” clearly reflects the source of sulfate ions.

It is believed that both SEF and DEF are forms of internal sulfate attack that result for different reasons. SEF is commonly a product of concrete degradation, characterized by the dissolution and subsequent precipitation of ettringite into available void space and into preexisting microcracks. SEF is possible if the concrete is sufficiently permeable and saturated, allowing the dissolution and precipitation process to occur. Although most experts agree that SEF will not generate sufficient expansive pressures to cause concrete fracture, the presence of SEF in the air void structure may limit the ability of the paste to resist freeze-thaw deterioration (Ouyang and Lane 1999). Thus, concrete that appears to be suffering paste freeze-thaw deterioration may have originally had a sufficient air-void system, but this system was compromised by infilling with SEF.

DEF, on the other hand, can lead to destructive expansion within the paste, resulting in microcracking and separation of the paste from aggregate particles. DEF is most often associated with steam curing because primary ettringite will not properly form at elevated temperatures (Thaulow et al. 1996, Klemm and Miller 1997). After the concrete has cured and temperatures are reduced to ambient conditions, sulfates and aluminate phases in the paste may react to form expansive ettringite, thereby disrupting the concrete matrix. It is still speculative, however, whether cast-in-place concrete, including EOT concrete, can experience DEF. But there is little doubt that under the right conditions (e.g., thick slab, high cement content, and high ambient temperature), EOT concrete may experience temperatures in excess of that required for DEF during curing, especially if curing blankets are used during summer placements. The manifestation of internal sulfate attack in many concrete structures is characterized by a series of closely spaced, tight map cracks with wide cracks appearing at regular intervals. DEF can be identified only through petrographic microscopic analysis in accordance with ASTM C 856. Paste expansion due to DEF can be identified by a uniform separation of the paste from the aggregate particles of similar size along the interfacial zone.

2.4.4 Microstructural Characterization

For the most part, concrete mechanical properties and durability are controlled by the paste microstructure. More complete discussions of concrete microstructure can be found in Mindess et al. (2003) and Mehta and Monteiro (1993). This section provides a brief background on concrete microstructure and describes experimental methods used to characterize it.

The hydrated cement paste microstructure consists of solid phases and a pore system. The solid phases consist of both unhydrated cement grains and the related hydration products. The number of unhydrated cement grains increases markedly in high–cement-content, low–*w/c*-ratio EOT concrete. While a variety of hydration products exist in cement paste, the primary phases of interest in determining the behavior of concrete are calcium-silicate-hydrate (C-S-H), calcium hydroxide (CH), and calcium sulfoaluminates (ettringite [AF_t] and monosulfate [AF_m]). The most abundant hydration product is C-S-H (sometimes called gel), which occupies more than half the solid volume of the hydrated paste (Mindess et al. 2003). C-S-H is amorphous, or poorly crystallized, and is considered the building block of a hydrated cement paste system. The next most abundant hydration product is CH, which typically forms as distinctive hexagonal crystals of various sizes and shapes. CH occupies about 20 to 25 percent of the solid phases (Mindess et al. 2003). It is easily fractured and is more soluble than C-S-H. The quantity of CH can be significantly reduced with the addition of a pozzolan and is increased through the use of some admixtures such as calcium chloride. Calcium sulfoaluminate phases make up 10 to 15 percent of the solid paste volume (Mindess et al. 2003).

The cement paste pore structure can generally be classified into three distinct groups: cement gel pores, capillary pores, and air voids (Neville 1996). The paste/aggregate interfacial transition zone and microcracking represent additional elements of the concrete pore structure. The pores within the C-S-H, referred to as gel pores or interlayer hydration space, make up the smallest individual elements of the total cement paste porosity. The gel pores are an intrinsic part of the C-S-H, representing the space between the distorted and randomly arranged calcium silicate sheets (Mindess et al. 2003). Gel pores are typically smaller than 10 nm and make up about 28 percent of the total porosity in the hydrated cement paste (Neville 1996). Characteristics of the interlayer spaces cannot be changed by changing mix design parameters.

In contrast, capillary porosity can be significantly modified by altering mixture properties, especially the *w/c* ratio. The capillary pore system is the remnant of the water-filled space that initially existed between anhydrous cement grains that did not get filled with hydration products. This system is typically irregular in both shape and spatial distribution, with the pore sizes and connectivity dependent on the size of the initial water-filled space (a direct function of the *w/c* ratio) and the degree of hydration. In well-hydrated, low–*w/c*-ratio systems such as is common in EOT concrete mixtures, capillary pores will typically range in size from 10 to 50 nm. In high–*w/c*-ratio systems at early stages of hydration, capillary pores can range in size from 3 to

5 mm (Mehta and Monteiro 1993). The use of admixtures that disperse cement grains, such as water reducers, typically result in smaller, more uniformly distributed capillary pores.

By far the largest pores are the air voids. The air voids are generally classified into two groups, those that are intentionally entrained and those that unintentionally entrapped. Entrained air voids are essentially spherical and tend to be randomly distributed throughout the cement paste. They are introduced into the matrix through the addition of admixtures that are specifically designed to produce large quantities of microscopic air bubbles when mixed into fresh concrete. Numbers of entrained air voids range up to tens of billions of bubbles per cubic yard of concrete, depending on the particular air-entraining agent used. The size of entrained air voids ranges from about 10 µm to over 1 mm. Unintentionally entrapped air voids can range in size from microscopic to over 3 mm. While it is virtually impossible to make a clear distinction between entrained and entrapped air voids, quite often voids larger than 1 mm in diameter, and/or irregular in shape, are labeled as entrapped (ASTM C 125). These larger air voids contribute significantly to the total air content of concrete but not to frost resistance of concrete.

Microcracking of hydrated cement paste may occur relatively early in the hydration process (before the paste has gained significant strength) when internal stress exceeded the strength of the paste. Shrinkage and/or thermal strain could produce such stress when restrained by the aggregates. Both autogenous shrinkage and rapid changes in temperature of EOT concrete mixtures could lead to microcracking of the paste.

As has been described previously, the microstructure of concrete can be altered through the addition of chemical admixtures and high curing temperature. The use of low–*w/c*-ratio and fine cements also has an impact on the resulting microstructure. Many of these factors are common in EOT concrete, and it is therefore important to understand these alterations and their impact on durability.

Concrete microstructure is studied through application of ASTM C 856. Guidelines have been developed to assist in this process for the evaluation of deteriorated pavement concrete (Van Dam et al. 2002b). With techniques such as staining, stereo and optical microscopy, and scanning electron microscopy, a trained petrographer can develop a great understanding of the concrete microstructure and the mechanisms that may have led to distress.

2.4.5 Absorption and Permeability

The absorption characteristics and permeability of concrete directly influence concrete durability. Concrete that is permeable to air, water, or other substances is more likely to suffer some kind of durability distress. The ingress of gases and liquids will lead to solubility of some components in the hardened paste, can result in expansive reactions, and in general provides a medium through which ions can be transported. For this reason, changes in mixture design that decrease porosity/permeability often lead to an increase in durability.

Currently, there is no readily available method to measure concrete permeability. In a study for the FHWA, Hooton et al. (2001) summarized the effectiveness of various methods that can be used to assess chloride penetration into concrete. In general, the tests that accurately modeled chloride ingress were long-term tests that were not suitable for design or construction quality control. The rapid tests exhibited a number of limitations of which the most relevant to EOT concrete were that the results were affected by the presence of ions in the concrete such as occurs when common accelerators are used.

Absorption is a measure of the volume of pore space in concrete irrespective of the interconnectivity of the pores (Neville 1996). Thus, although absorption and permeability are commonly correlated, they are not necessarily related. A variety of techniques are used for determining the absorption rate of concrete. One common test is ASTM C 642, "Test Method for Specific Gravity, Absorption, and Voids in Hardened Concrete," which entails drying a concrete specimen at 100 to 110^{o}C (212 to 230^{o}F) and then immersing it in water at 21^{o}C (70^{o}F) for at least 48 hours. This test is commonly used as a quality control test for precast members (Neville 1996).

Because of the shortcomings of absorption-testing procedures, there has been interest in sorptivity testing that measures the rate of absorption by capillary suction of water into the concrete (Neville 1996). Generally, it is too difficult to mathematically model this flow in any but a single direction, and thus sorptivity tests are configured to establish one-directional flow into the specimen (Hooton et al. 2001). The benefits of sorptivity testing are reduced testing time, low equipment cost, and simplicity of procedure. The proposed ASTM standard test for sorptivity requires only a scale, a stopwatch, and a shallow pan of water (Stannish et al. 1997).

CHAPTER 3

EXPERIMENTAL PROGRAM AND RESULTS

The previous chapter explained that premature deterioration in EOT concrete mixtures result from the altered microstructure and/or microcracking in the hydrated cement paste caused by rapid hydration. The altered or cracked microstructure increases permeability, paste porosity, and solubility of paste constituents, thus negatively affecting the concrete durability. Also, the altered or cracked microstructure may result in paste expansion from delayed ettringite formation, as the temperatures achieved during hydration can exceed 70^{o}C. The factors that have the greatest influence on EOT concrete durability are those that impact shrinkage, thermal stress, altered microstructure, and chemical attack. The selection of the constituent materials, mix design/proportioning, construction, curing, and age at opening-to-traffic will influence these properties and the durability of the resulting EOT concrete. Earlier research has shown that achieving early-age mechanical properties (strength, abrasion, etc.) with EOT concrete materials is not difficult. Therefore, this project focused on durability issues related to the interaction between the EOT concrete materials and the environment in which they serve and not issues related to the effect of traffic loading.

Addressing durability issues is a difficult task because no universally accepted tests that “measure” durability are available. Durability is a function of the material properties and the environment in which the material serves. For example, a non–air-entrained concrete might make a durable indoor floor, but will likely not be durable if exposed to freezing and thawing in the presence of deicer applications. Similarly, concrete with poor sulfate resistance might be extremely durable in a non-sulfate environment, but might perform poorly if exposed to an external source of sulfate ions.

The research included a limited field evaluation and a laboratory evaluation of EOT concrete mixtures. The results of the laboratory research were used to develop guidelines to assist engineers and contractors in selecting appropriate materials, mixtures, and construction techniques for use in EOT concrete repairs. This chapter discusses the field and laboratory phase of this study.

3.1 FIELD EVALUATION

It was not possible to develop a statistically valid experiment for the field evaluation because the data for many repair installations were sparse and of questionable quality and because researchers could not locate repairs that met all the requirements specified in a factorial experimental plan. For this reason, an alternative approach was followed for the field evaluations that recognized these limitations while yielding information useful for this study. Various SHAs were contacted to identify those agencies that had broad material and climatic representation. The following general conclusions were drawn from discussions with these agencies:

- Most SHAs had both 6- to 8-hour and 20- to 24-hour EOT PCC mixtures.
- Other than the experimental sites constructed as part of the SHRP studies (Whiting et al. 1994), none of the SHAs had any extant experimental sites that could be evaluated in the

course of this project. Further, although all SHAs had mixture design information available, few maintained detailed construction records on repair installations.

- None of the SHAs tracked performance of their full-depth repairs, thus no performance data were available. However, the SHAs reported their general feelings about full-depth repairs. Most SHAs felt that they had both “good” performing and “poorly” performing 6- to 8-hour EOT concrete repairs, but felt that the 20- to 24-hour repairs were performing satisfactorily.
- In all cases, SHAs offered to provide assistance to this research effort by providing traffic control and coring.

On the basis of this information, it was decided to seek assistance from one SHA in each of the four long-term pavement performance (LTPP) climatic zones. A brief description of each test site is presented in the following section. Photographs from each site and the results of the visual assessment are presented in Appendix B. Table 2 lists the EOT concrete mixture design parameters for the mixtures used at the selected sites.

3.1.1 Experimental Sites

Interstate 20, West of Augusta, Georgia

Eastbound Interstate 20, a four-lane divided highway, contained full-depth EOT concrete repairs placed as part of the Strategic Highway Research Program studies (Whiting et al. 1994). Two mixtures placed during the SHRP study selected for inclusion as part of the field investigation in this experiment are known as the Fast Track or FT1 (4 to 8 hour) and GA DOT mixtures (20 to 24 hour). Both mixtures were placed in July 1992, making the repairs 9 years old at the time of this investigation.

Georgia Department of Transportation (GA DOT) personnel indicated that a number of patches had been removed and replaced because of poor performance, making it difficult to locate “poor” performing repairs for sampling operations for this study. In addition, the pavement had been diamond ground, making it difficult to visually assess surface defects although GA DOT officials indicated that surface cracking of the patches was present prior to the diamond grinding of the pavement. Of the four patches chosen for sampling in this project, only one patch was found to be performing poorly, containing a longitudinal crack across the full length of the patch. There was no staining evident at the crack location; however, the crack appeared to be beginning to spall at some locations. The remainder of the patches sampled appeared to be performing satisfactorily.

TABLE 2 Specifications for field mixtures included in study

SHA	Mixture	Year Const.	Cement Type	Cement Factor	*w/c* ratio	Coarse Aggregate	Fine Aggregate	Accelerator	Air	Water Reducer
GA	FT1	1992	III	439 kg/m^3 (740 lb/yd^3)	0.35	842 kg/m^3 (1420 lb/yd^3)	783 kg/m^3 (1320 lb/yd^3)	None	5.6% Darex	Type A
	GA DOT	1992	I	446 kg/m^3 (750 lb/yd^3)	0.38	1071kg/m^3 (1805 lb/yd^3)	608 kg/m^3 (1025 lb/yd^3)	CC[2] (0.5%)	3.7% Darex	No
NY	18502	1993	III	490 kg/m^3 (825 lb/yd^3)	0.39	849 kg/m^3 (1430 lb/yd^3)	758 kg/m^3 (1280 lb/yd^3)	CC (2.5%)	4.5 to 6.0% Daravair	No
	Modified Class F	2000	I	418 kg/m^3 (705 lb/yd^3)	0.39	1124 kg/m^3 (1895 lb/yd^3)	602 kg/m^3 (1015 lb/yd^3)	NS	6.5%	Type A
OH	Class FS	1992	III	534 kg/m^3 (900 lb/yd^3)	0.41	843 kg/m^3 (1420 lb/yd^3)	593 kg/m^3 (1000 lb/yd^3)	CC (2%)	4.5%	Type D
	Class MS[1]	1993	I	456.5 kg/m^3 (770 lb/yd^3)	0.30	754 kg/m^3 (1270 lb/yd^3)	848 kg/m^3 (1430 lb/yd^3)	CC (0.5%)	7.5% AE-360	Type F & Type D
TX	Class K	1996	III	418 kg/m^3 (705 lb/yd^3)	0.39	1144 kg/m^3 (1930 lb/yd^3)	636 kg/m^3 (1070 lb/yd^3)	Type C NC	3.0 to 6.0% AE-90	Type A
	Class K Modified	1998	I	390 kg/m^3 (658 lb/yd^3)	0.40	1137 kg/m^3 (1915 lb/yd^3)	597 kg/m^3 (1005 lb/yd^3)	Type C NC	3.0 to 6.0% Air 30	Type A

[1] This mixture includes 415-kg cement and 41.5-kg AXIM microsilica per cubic meter (700 lb cement and 70 lb microsilica per cubic yard).
[2] CC: calcium chloride; NC: non-chloride; NS: not specified.

Long Island Expressway (US 454) in Suffolk County, Long Island, New York

The New York Department of Transportation (NY DOT) performed full-depth EOT concrete repairs for 4- to 8-hour opening on a six-lane divided highway (Long Island Expressway [US 454] in Suffolk County). Repairs in this section were placed during the summer of 1993 using a concrete mix specified under the NY DOT specifications as Item 18502.6027. Information on the fresh concrete properties are not available; however, all mixtures placed were reported to have met specification tolerances.

One unique aspect of this mixture is that the temperatures of the aggregate are taken prior to batching to determine the temperature of the heated mixing water to be added. The heated mixing water is added to a tank on the truck, where it remains until the truck arrives on site. This water and the calcium chloride solution are then added just prior to placement in order to achieve a concrete temperature of 32 to 38°C (90 to 100°F) at time of placement. The opening criteria for these repairs was based on temperature, being opened to traffic when the surface temperature reaches 65°C (150°F), which has been found to correspond to a compressive strength of approximately 13.8 MPa (2,000 psi). Peak hydration temperatures for these mixes can reach 82°C (180°F).

The visual assessment of the repairs and of the surrounding pavement prior to sampling operations found the repairs to be in good to excellent condition, with only one of the patches exhibiting a transverse crack at mid-panel. Almost all of the patches surveyed displayed slight map cracking on their surface. The surrounding pavement was in generally good condition, with some spalling occurring at joint locations in the pavement structure. Based on these observations, two repairs were selected for coring operations, one of which was performing satisfactorily, whereas the other exhibited mid-panel transverse cracking with some brown staining present.

Interstate 390, Monroe County, South of Rochester, New York

The NY DOT Modified Class F EOT concrete materials were obtained from a four-lane divided Interstate (Northbound Interstate 390). The repairs in this section were placed during August of 2000. Fresh and hardened concrete properties were recorded by representatives of the NY DOT.

Selection of repairs and assessment of patch condition were conducted by representatives of the NY DOT. Records indicate that one satisfactorily performing repair and one unsatisfactorily performing repair were sampled. The unsatisfactorily performing repair contained a transverse crack across the entire slab width. This crack is believed to be a thermal crack resulting from the overall patch length of 6 m (20 ft).

State Route 2, West of Cleveland, Ohio

The Ohio DOT Class FS test site was on Westbound State Route 2, a four-lane divided highway west of Cleveland, Ohio. This site was constructed as part of the SHRP studies during September of 1992. The Class MS site was constructed on the same section of State Route 2 in September of 1993. Although not part of the SHRP study, it was part of an Ohio Department of Transportation (ODOT) project; thus, detailed construction data were available.

The opening strength for the Class FS mixtures ranged from 7.6 to 36.5 MPa (1,100 to 5,300 psi) as determined from both insulated and uninsulated cylinder specimens, and the peak measured hydration temperature for the ODOT mix was nearly 74^{o}C (165^{o}F) (Whiting et al. 1994). Ohio specifications stipulate that the MS mixtures may be opened to traffic after 24 hours, provided that a flexural strength of 2.76 MPa (400 psi) is met.

Surveys of the Class FS repairs were conducted 2 months after placement. Results from the inspection revealed that nearly all the FS patches had cracked longitudinally at this time. SHRP studies found that the higher the peak curing temperature of a repair, the more likely for that repair to exhibit longitudinal cracking in the 2-month survey (Whiting et al. 1994). Additional surveys of the Class FS repairs were conducted annually from 1994 to 1998. All of the repairs eventually contained longitudinal cracking, whereas two out of nine repairs had transverse cracking at the end of 1998.

A visual assessment was made of the patches prior to core sampling; however, little information was available on the existing condition of the concrete because the pavement had been overlaid with asphalt. Despite these challenges, patches were located through the use of milepost stations and reflective cracking visible at the joints. Although surface deterioration of the concrete could not be directly assessed, in many cases manifestation of joint deterioration and cracking could be seen in the surface of the overlay.

Northbound US Interstate 81, Wise County, Texas

The 6- to 8-hour EOT concrete repairs on northbound US Interstate 81 in Wise County, Texas (a four-lane divided highway), were evaluated. The repairs, constructed in 1996, were made on the continuously reinforced concrete pavement with a Texas DOT Class K mix. Fresh concrete properties were documented by the Texas DOT; however, information for the individual patches included in this study was not available.

A visual distress survey was conducted of the patches and surrounding pavement prior to selection of individual patches for core sampling operations. As is typical with continuously reinforced concrete pavements, the surrounding pavement surface contained hairline transverse cracks at regularly spaced intervals. Several of the patches also contained transverse cracks at mid-panel, and many showed signs of slight joint faulting and moderate-severity joint spalling. In addition, one of the patches also contained a corner break.

Frontage Road, Texas 2871, Tarrant County, Texas

This 20- to 24-hour EOT concrete test site was a frontage road connecting US Interstate 20 to Texas 2871 in Tarrant County, Texas. The repairs on this continuously reinforced concrete pavement were constructed during April and May of 1998 using a Texas DOT Class K modified mix. Although the fresh concrete properties were documented for the project, information for the individual repairs was not available.

A visual distress survey was conducted of the patches and surrounding pavement prior to selection of individual patches for core-sampling operations. As is typical with continuously reinforced concrete pavements, the surrounding pavement surface contained small transverse cracks at short, regularly spaced intervals. All patches surveyed appeared to be performing satisfactorily, which made it impossible to select a good and bad patch for this location. As a result, two satisfactorily performing patches were selected for coring operations.

3.1.2 Sampling of Test Sites

Using the results from the visual survey, two repairs were selected from each test site, with few exceptions, one rated as “satisfactory” and the second as “unsatisfactory.” Four

150-mm (6-in.)-diameter cores were then obtained from each repair, for a total of eight cores for each mixture type being evaluated. Coring locations were based on whether the repair was rated as “satisfactory” or “unsatisfactory.” The only difference between the two sampling locations is that one core was taken in a cracked/deteriorated area in the “unsatisfactory” repair.

Core Sample A was obtained at roughly the geometric center of the patch. This location was selected to evaluate the influence of hydration temperatures on the concrete, as this location is where the highest temperature within the repair would be expected to occur. Core Sample B was obtained on the inside edge of the repair at the transverse joint between the dowels. This location was selected to determine the effects of moisture and chemical ingress at the joints. Core Sample C was taken at the approximate location of the outside wheel path, which was determined in the field by examining the concrete surface for evidence of wear or by selecting a location 0.6 m (24 in.) in from the shoulder line. The final sample, Core Sample D, was obtained at an interior location. In the case of an “unsatisfactory” repair, the sample was taken in a cracked or deteriorated location, representing an area of the typical distress present for that repair. Details regarding the core locations for each repair are provided in Appendix B.

3.1.3 Approach to Laboratory Testing of Field Specimens

The analysis of the field-collected specimens included pulse velocity testing, microstructural characterization, absorption/sorptivity testing, and the determination of the CTE, as shown in Table 3. Of the four cores collected from each repair, two were used for microstructural characterization and two were used in the pulse velocity testing, in the absorption/sorptivity testing, and to determine the CTE in accordance with AASHTO TP 60-00.

Pulse Velocity Testing

Two specimens from each material/condition combination were tested using pulse velocity. The procedure for conducting pulse velocity testing required providing a smooth surface at each end of the core sample. Pulse velocity measurements were conducted on 100-mm (4-in.)-diameter specimens that were cored from the 150-mm (6-in.)-diameter specimens obtained from EOT concrete repairs. A petroleum-based jelly was applied to the ends of the cores to improve contact between the transmitter/receiver and the test specimen. Three tests were conducted on each sample using a Jones Instruments NDT 3000 apparatus that generated a compression wave (P wave) from a transmitter with an energizing pulse of 500 volts. To minimize error produced by vibration or movement of the testing apparatus transmitter and receiver, a bracket was developed to hold these instruments in place at the ends of the sample.

Microstructural Characterization

Two specimens from each material/condition combination were evaluated microstructurally using four separate test methods: staining, stereo microscopy, petrographic microscopy, and limited scanning electron microscopy (SEM). Staining techniques were performed to identify the depth of carbonation and the degree of voids/cracks infilling with ASR gel and/or sulfate minerals. Such observations help to assess the failure mechanisms. Stereo optical microscopy was used to make a general assessment of the concrete's condition and to determine the characteristics of the air-void system using ASTM C 457 procedures. An analysis of the amount of air void infilling with secondary deposits (ettringite, calcite, ASR gel, etc.) was also made to determine the contribution of paste freeze-thaw damage and other concrete durability problems.

TABLE 3 Field evaluation tests (number of samples per material/condition combination)

Test Attribute	Test Name/ Equipment	Measured Property	No. of Specimens	Specification and Laboratory[1]
Pulse Velocity	Pulse Velocity	Non-destructive estimation of relative stiffness	2	Non-Standard (MTU)
Microstructural Characterization	Staining Techniques	Staining of carbonated paste, ASR gel and sulfate minerals	2	No Standard Test (MTU)
	Stereo Optical Microscopy	Air-void system parameters	2[2]	ASTM C 457 (MTU)
	Petrographic Optical Microscopy with UV dye impregnation	Microstructure analysis, reaction products, and evidence of deterioration	2[2]	ASTM C 856 (MTU)
	Scanning Electron Microscopy (SEM)	Microstructure analysis, elemental mapping	2[2]	No Standard Test (MTU)
Volume Change	CTE	CTE	2[3]	AASHTO TP 60-00 (MSU)
Total Specimens per Treatment			4	

[1] The agency responsible for running each test is shown in parentheses.
[2] Single cores were cut in such a way that specimens were obtained for staining, stereo optical microscope, petrographic optical microscope, and SEM.
[3] Tests were performed on the same specimens used for pulse velocity testing.

Petrographic microscopy was used to examine the concrete microstructure. Mineralogical identification, evidence of paste expansion, and microcracking were characterized to identify causes of deterioration. The petrographic analysis was also used to assess paste porosity. The

SEM was applied in limited cases to identify microstructural features that were not easily resolved using optical microscopy.

Determination of CTE

The CTE was determined on the field-obtained core specimens using the provisional AASHTO Standard Test Method for the Coefficient of Thermal Expansion of Hydraulic Cement Concrete (TP 60-00). The test was performed on the same 100-mm (4-in.)-diameter trimmed cores used for the pulse velocity testing.

3.1.4 Results and Discussion of Field Specimens

This section presents and discusses the results of tests conducted on the field-obtained specimens, including the pulse velocity testing, microstructural evaluation, and CTE values. The discussion focuses on how the analysis of these results provides insight into typical EOT concrete mixtures as actually constructed.

Pulse Velocity Testing Results

Tests conducted on four core specimens from each mixture indicated a good consistency within each mixture for most repairs sampled, as evidenced by the low coefficients of variation values. Detailed results are presented in Appendix B.

Pulse velocity values are known to be highly correlated with density of the concrete, the amount of air present in the concrete (which is also related to the concrete density), and the length of the signal path. The relationship between density and pulse velocity for all specimens, plotted in Figure 1, shows a strong correlation exists.

Statistical analysis using a t-test conducted on satisfactory and poorly performing samples found statistically significant differences in the density values for satisfactory and poorly performing NY Modified Class F repairs and velocity values of Texas Class K repairs; however, neither mixture contained statistically significant differences in both density and velocity data. Therefore, existence of trends between the density, velocity, and repair condition for mixtures studied has not been demonstrated.

Depth-of-Carbonation Testing

Depth of carbonation was assessed on two cores from each repair sampled, for a total of four samples per mixture. Carbonation measurements were taken at 10 points across the face of the sample at the surface of the repair and were then averaged to obtain an average depth of carbonation for each repair. Test results revealed no significant carbonation to be present in any of the repairs sampled. Nearly all repairs exhibited some depth of carbonation, with most samples carbonated to a depth of less than 1 mm (0.04 in.) uniformly across the surface. Many samples also revealed localized areas of significant carbonation depth. ODOT repairs were found to exhibit the greatest depth of carbonation, with carbonation extending to a depth in excess of 1 mm (0.04 in.) for at least part of the surface. Samples obtained from repairs that had been diamond ground might not adequately reflect the depth of carbonation because part of the carbonated layer was removed. Detailed test results are presented in Appendix B.

Results of t-tests conducted on satisfactory and poorly performing repairs indicate that a significant statistical difference was found only for the NY Modified Class F mixture, with the “poor” performing repairs having less carbonation than repairs performing in a “satisfactory” manner. These results indicate that the depth of carbonation is not a predictor of repair performance.

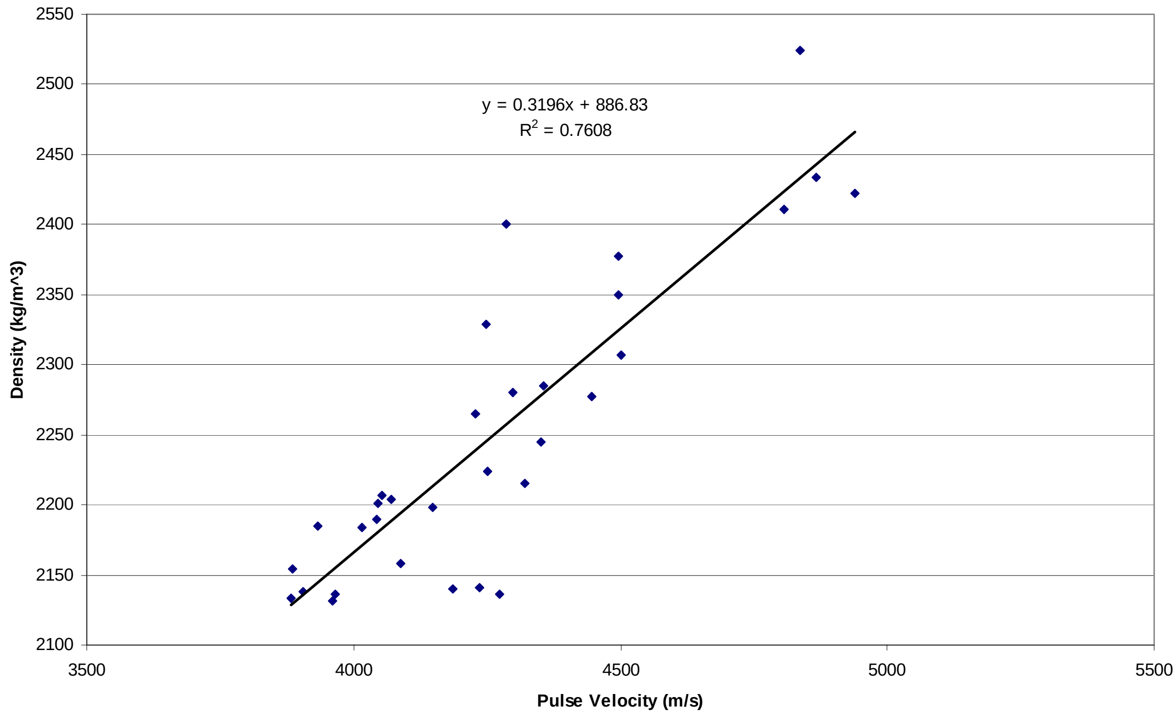


Figure 1. Density versus pulse velocity for all samples tested.

Air-Void Analysis

Polished samples that had been stained with barium chloride and potassium permanganate were used for air-void system characterization conducted in accordance with ASTM C 457 modified point count method. Samples were placed on an automated stage and viewed at a magnification of 70 times through a stereo optical microscope. Several air-void system parameters were determined and compared with accepted values for a satisfactory air-void system as presented in ASTM C 457. Further analysis of data obtained from these tests was conducted using t-testing to determine statistical significance of differences between key air-void system parameters for satisfactory and poorly performing repairs.

Over time, air voids in some specimens have filled with secondary deposits, ettringite being the most common. Through the use of the barium chloride/potassium permanganate (BCPP) staining technique, concentrations of sulfate-bearing phases such as ettringite can be identified in the course of the ASTM C 457 testing. With computation of the existing air content, the BCPP staining technique allows for both the estimation of the original air-void system parameters and the assessment of the degree of air-void infilling that may have occurred. Figures 2 through 5 present the impact of infilling on the air-void system in the field specimens. Significant infilling has occurred in the two mixtures from Georgia and in the Ohio Class FS mixture. Negligible infilling was observed in mixtures from New York and Texas.

Figure 2. Percent reduction in air content due to secondary ettringite infilling.

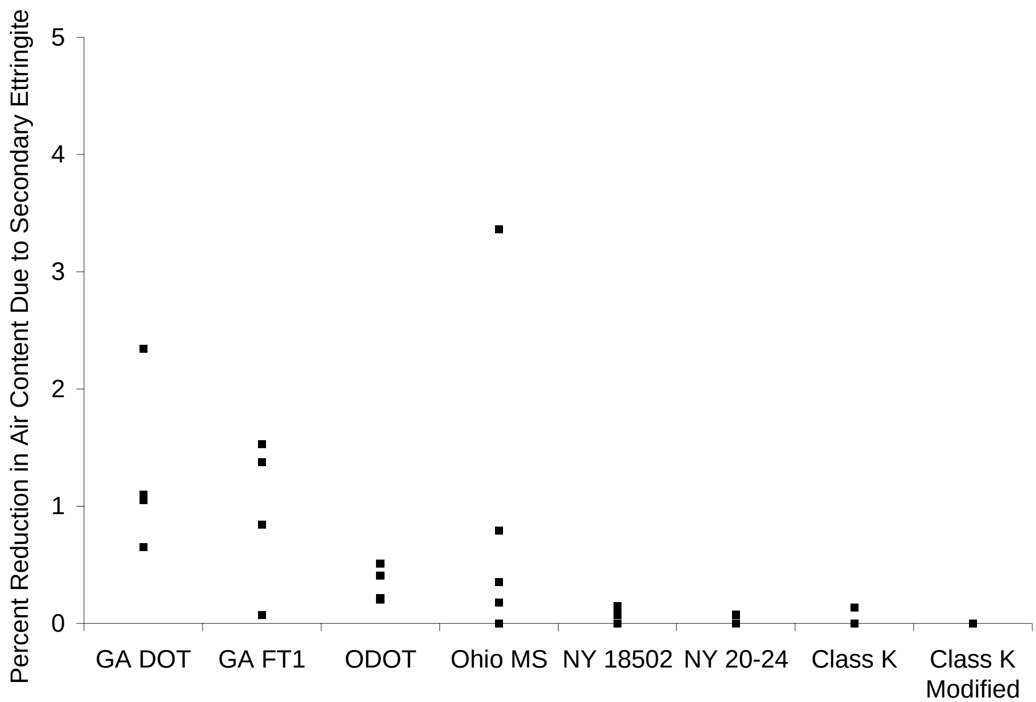


Figure 3. Original spacing factors for various field mixtures.

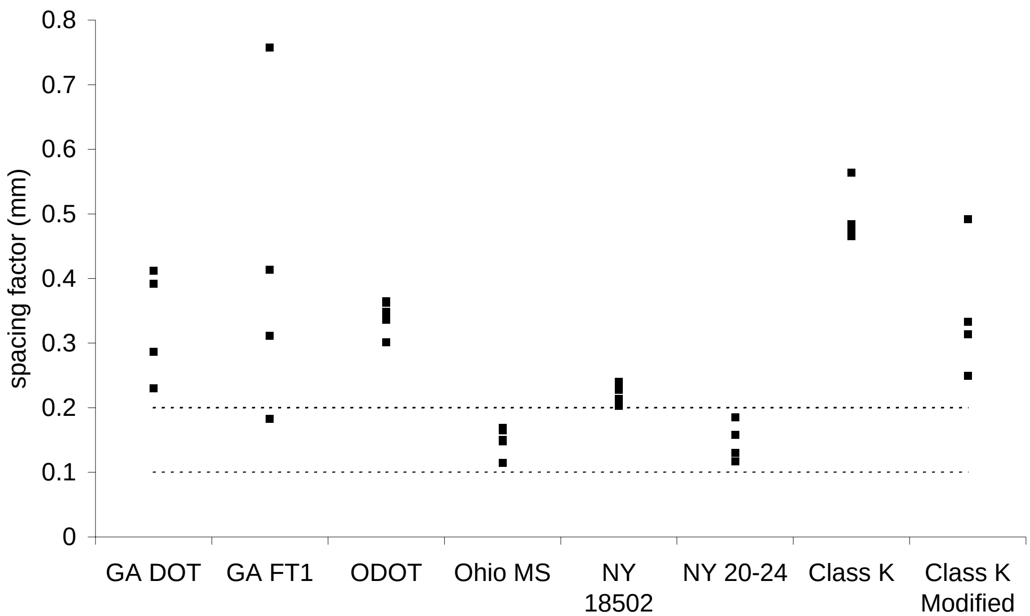


Figure 4. Existing spacing factor for various field mixtures.

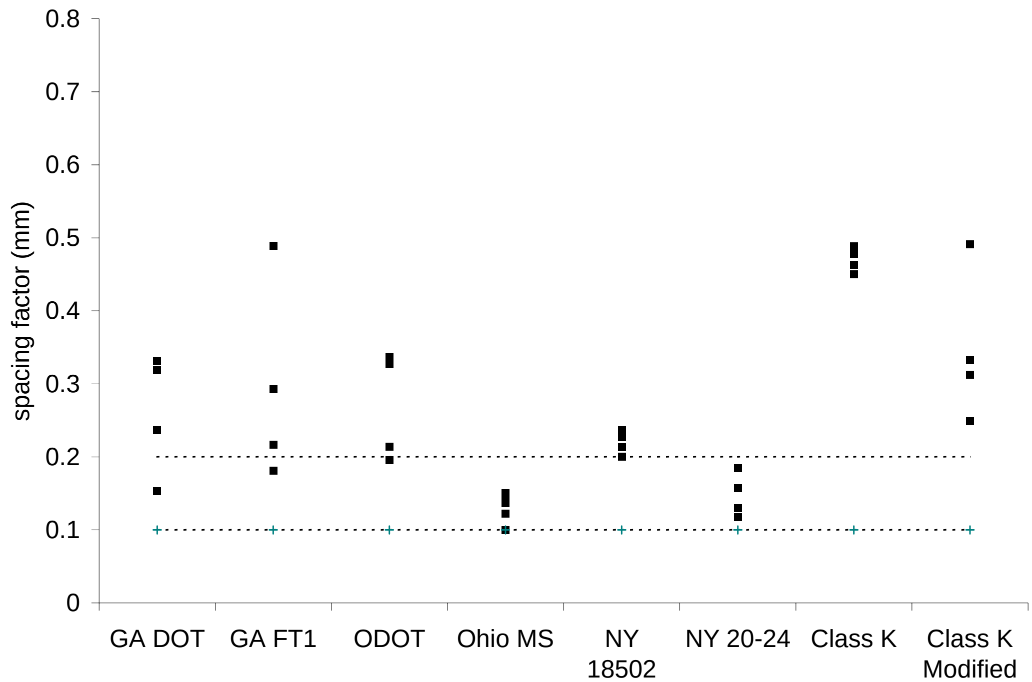


Figure 5. Percent increase in spacing factor for field specimens due to infilling.

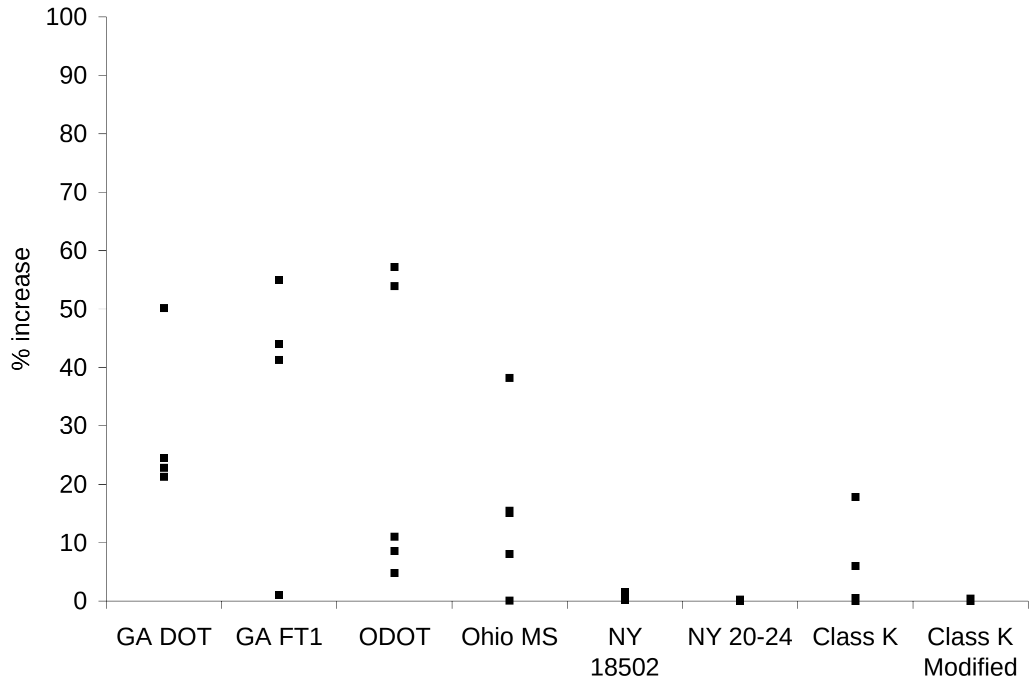


Figure 3 presents the estimate of the as-constructed, original spacing factors for the various mixtures. The dashed horizontal lines illustrate the normally accepted range in values for this parameter, with 0.200 mm (0.008 in.) commonly considered as the upper limit for protecting the paste against freeze-thaw damage. As can be seen, the mixtures from the two southern states (Georgia and Texas) did not generally meet the recommended limit. This finding is not of great of concern for these sites, which are located in “non-freeze” zones, although these sites are subjected to a few freeze-thaw cycles during the year. Of greater concern are the relatively poor spacing factors for the Ohio Class FS and New York 18502 mixtures, which are located in severe freeze-thaw environments. As shown in Figures 4 and 5, the infilling of the air voids with sulfate-bearing phases such as ettringite increased the existing spacing factor for mixtures from Georgia and Ohio, but not from New York and Texas. The increase did not appear to compromise the air-void systems of mixtures that started with sufficient spacing factors.

Statistical analysis of the air-void system parameters for satisfactory and poorly performing concrete found, in general, little difference for a given mixture, with two exceptions: the GA DOT and New York Modified Class F. Significant differences between satisfactory and poorly performing repairs were observed for the GA DOT mixtures, with poorly performing repairs having more infilling. It is not possible to determine from the test results whether the infilling of voids in GA DOT repairs with ettringite caused the poor performance or whether the infilling is the result of the poor performance. The New York Modified Class F mixtures had similar results, with the satisfactory repairs having more favorable air-void system parameters than poorly performing repairs. The observed distress (a single crack), however, is not consistent with paste freeze-thaw deterioration; therefore, it is unlikely that the air-void system is responsible for the poor performance.

Stereo Optical Microscope Observations

In addition to characterizing the air-void system, stereo optical microscopy was used to make general observations of the concrete, particularly the existence of ASR. ASR was observed in the two mixtures from Ohio. It was isolated to the dark shale constituent of the fine aggregate and was most severe in the Ohio Class FS mixture, but was also identified in the Ohio Class MS mixture. It is believed that the high cement content aggravated the ASR in the FS mixture by increasing the total alkalinity of the mixture, whereas the use of silica fume in the MS mixture helped suppress ASR. This observation suggests the need for considering ASR potential when constructing EOT concrete repairs.

Petrographic Microscope Observations

Petrographic observations were made of thin sections prepared from the field concrete. To provide a means of comparison, each thin section was rated for paste homogeneity and degree of microcracking on a scale of 1 to 3, with 1 being best (i.e., homogenous and free of microcracking) and 3 being worst (i.e., highly inhomogeneous with significant microcracking). The nature of calcium hydroxide was also recorded. The results of this evaluation are presented in Table 4. Micrographs of each thin section are presented in Appendix B. No observable differences in microstructure were observed between satisfactorily and poorly rated repairs.

TABLE 4 Summary of petrographic evaluation of thin sections from field concrete

Site	Paste Homogeneity	Microcracking	Condition of Calcium Hydroxide
GA FT1	1	1	Normal
GA DOT	1	1	Normal
NY 18502	2	2	Large and patchy
NY Mod. Class F	1	1	Normal
OH Class FS	2	3	Normal
OH Class MS	1	2	Very small, less abundant
TX Class K	1	1	Normal
TX Class K Modified	1	2	Large and patchy

The results from the petrographic analysis indicate that four of the repair materials (Georgia DOT, Georgia FT1, New York Modified Class F, and Texas Class K) had microstructure with good homogeneity, little microcracking, and normal calcium hydroxide. Two of the mixtures (Ohio Class MS and Texas Class K Modified) had good homogeneity, but moderate microcracking. The calcium hydroxide was very small and less abundant in the Ohio Class MS mixture, whereas it was large and patchy in the Texas Class K Modified mixture. The finding for the Ohio Class MS material is not surprising given that the mixture contained microsilica, which is a very active pozzolan that reduces calcium hydroxide in the concrete.

The fast-setting materials (Ohio Class FS and New York 18502 mixtures) have moderate paste inhomogeneity and moderate microcracking, with the calcium hydroxide being large and patchy in the New York 18502 mixture. These observations suggest that on occasion, difficulties in achieving adequate dispersion of cement may occur in these high–cement-content mixtures (these two mixtures had the highest cement content of the field mixtures). Further, both these mixtures had marginal air-void systems, as indicated by the spacing factors. It is impossible to determine whether the observed microcracking resulted from construction, freeze-thaw damage to the poorly protected paste, or ASR as observed in the case of the Ohio Class FS mixture.

3.1.5 Summary of Field Evaluation

The following observations were made from the information collected in the field study:

- No single cause of distress was observed in the field concrete. In general, the concrete was of good quality, and it was difficult to find "distressed" repairs for use in this study. This finding indicates that although some durability problems have been observed in EOT concrete repairs, it is clearly possible to construct durable, long-lasting repairs.
- One problem observed in the field concrete was poorly formed air-void systems that may have been inadequate for protecting the hydrated cement paste against freeze-thaw damage. In some cases, the original spacing factors were inadequate even though the air content was sufficient. In other cases, infilling of the air voids with secondary deposits, such as ettringite, increased the spacing factor to unacceptable levels.
- Although attempts were made to avoid EOT concrete with known alkali-aggregate reactivity problems, ASR was observed in both types of materials obtained from Ohio, but was far less prevalent in the Ohio Class MS mixture, which contained a microsilica supplementary cementitious material.
- In general, less homogeneous paste, increased microcracking, and large patchy calcium hydroxide was observed in the faster-setting materials. These properties and the poor air-void

systems observed in many of these mixtures indicate that thorough blending of the constituents in these high–cement-content mixtures may not have occurred.

3.2 LABORATORY EVALUATION

Because of the complexity of EOT concrete mixtures and the variability of the various constituent materials, information obtained during the initial phases of this study was used to select material combinations that broadly represent the type of EOT concrete mixtures being constructed nationwide. A factorial experimental design that allows for more than two levels to be set for each variable was then adopted. These levels were set based on knowledge of concrete technology applicable to EOT concrete mixtures, for variables of greatest relevance. Each of these material combinations is discussed in detail below.

3.2.1 Material Combinations for 6- to 8-hour EOT Concrete Materials

The material combinations used for the 6- to 8-hour EOT concrete materials are presented in Table 5. A number designation (1 through 7) has been placed above each independent variable, and an alpha-numeric designation (A-1 through N-14) is used for each mixture. The initially proposed strength criterion at 6 hours was 13.8 MPa (2,000 psi) for compressive strength or 2.1 MPa (300 psi) for third-point flexural strength.

Each of the seven independent variables considered in the study of the 6- to 8-hour EOT concrete mixtures has either two or three levels. This consideration results in a possible 288 material combinations, of which 14 material/curing combinations were evaluated. Two batches were made for each mixture; thus, 28 batches of were made for tests. A discussion of the seven independent variables follows.

Independent Variables for Laboratory Experiment

The seven independent variables were selected after considering the findings of the NCHRP Project 18-04A and information contained in published articles and specifications used by various SHAs in the construction of EOT concrete repairs. Each of the first six variables (cement type, cement factor, *w/c* ratio, coarse aggregate type, accelerator type, and water reducer type) is common in the mix design/proportioning process; they are included in most SHA specifications. Further, of these variables seems to directly affect the shrinkage and durability of concrete.

TABLE 5 Summary of 6- to 8-hour EOT concrete mixture designs used in laboratory study

Combination	1 Cement Type	2 Cement Factor	3 *w/c* Ratio	4 Coarse Aggregate Type	5 Accelerator Type	6 Water Reducer Type	7 Curing Temperature
A-1	Type I	425 kg/m^3 (716 lb/yd^3)	0.40	Carbonate	Non-Chloride	No	23°C (73°F)
B-2	Type I	525 kg/m^3 (885 lb/yd^3)	0.40	Carbonate	Non-Chloride	No	23°C (73°F)
C-3	Type I	525 kg/m^3 (885 lb/yd^3)	0.40	Carbonate	Non-Chloride	No	Heated blanket
D-4	Type I	525 kg/m^3 (885 lb/yd^3)	0.40	Siliceous	Non-Chloride	No	Heated blanket
E-5	Type I	425 kg/m^3 (716 lb/yd^3)	0.40	Siliceous	Non-Chloride	No	23°C (73°F)
F-6	Type I	525 kg/m^3 (885 lb/yd^3)	0.36	Carbonate	Non-Chloride	No	23°C (73°F)
G-7	Type I	525 kg/m^3 (885 lb/yd^3)	0.40	Carbonate	Non-Chloride	Type F	23°C (73°F)
H-8	Type I	525 kg/m^3 (885 lb/yd^3)	0.36	Carbonate	No	Type E	23°C (73°F)
I-9	Type I	425 kg/m^3 (716 lb/yd^3)	0.40	Carbonate	Calcium Chloride	No	23°C (73°F)
J-10	Type I	525 kg/m^3 (885 lb/yd^3)	0.40	Carbonate	Calcium Chloride	No	23°C (73°F)
K-11	Type III	425 kg/m^3 (716 lb/yd^3)	0.40	Carbonate	Non-Chloride	Type F	23°C (73°F)
L-12	Type III	525 kg/m^3 (885 lb/yd^3)	0.40	Carbonate	Non-Chloride	Type F	23°C (73°F)
M-13	Type III	525 kg/m^3 (885 lb/yd^3)	0.40	Carbonate	Non-Chloride	Type F	Heated blanket
N-14	Type III	425 kg/m^3 (716 lb/yd^3)	0.40	Carbonate	Calcium Chloride	Type F	23°C (73°F)

The seventh variable, curing temperature, was also thought to have a potentially large impact on the durability of EOT concrete repairs. Field and laboratory studies have shown that many of these 6- to 8-hour EOT concrete mixtures obtain relatively high temperatures (in excess of 65°C [150°F]) in the first few hours after placement because of the high heat of hydration that occurs. These high temperatures may negatively affect the durability of the EOT concrete through adverse alteration of the hydration products (e.g., DEF), increased thermal stresses, and increased rate of evaporation. It is very difficult to produce this phenomenon in the laboratory using heat generated through hydration alone, since the test specimens are of inadequate size to contain the heat. Since SHA laboratories commonly conduct only standard tests that stipulate ambient laboratory temperatures during curing, the effect of high curing temperatures may never be observed in laboratory tests. For this reason, a limited study of the effect of increased curing temperature was included in this study. Below is a detailed description of each of the proposed variables.

Cement Type. Based on the review of SHA specifications and relevant literature, only Type I and III portland cements were selected for evaluation. The Type I cement was produced by Lafarge in the Alpena Plant and the Type III by Lafarge at the Woodstock Plant. Chemical and physical properties of the cements are presented in Table 6. Sufficient quantities of both cements were purchased prior to making specimens so that the same lots were used throughout the project.

TABLE 6 Chemical and physical properties of the cements used in this study

Property	Type I	Type III
Silicon Dioxide	20.7%	20.5%
Aluminum Oxide	4.2%	4.7%
Ferric Oxide	2.3%	2.7%
Calcium Oxide	65.3%	63.2%
Magnesium Oxide	2.0%	2.1%
Sulphur Trioxide	2.5%	4.1%
Tricalcium Silicate	70.0%	55.0%
Tricalcium Aluminate	7.0%	8.0%
Alkalies (Na_2O + 0.685 K_2O)	0.5%	0.4%
Loss on Ignition	2.0%	2.2%
Fineness (Blaine m^2/kg)	394	608
Compressive Strength (1 day)	15.1 MPa (2,190 psi)	27.2 MPa (3,945 psi)

Cement Factor. Cement factor is a commonly specified variable in SHA specification. Mixtures with a higher cement factor have higher hydrated cement paste content and are thus more susceptible to shrinkage and paste deterioration problems. Two levels (425 and 525 kg/m^3 [716 and 885 lb/yd^3]) were selected for the cement factor variable reflecting the general range in values used by SHAs.

w/c Ratio. The *w/c* ratio is commonly recognized to be the single most important mixture variable with regards to concrete strength and durability. Two levels of *w/c* (0.36 and 0.40) were evaluated. Although the difference between these two levels is somewhat small, the levels fall within the ranges specified by most SHAs.

Coarse Aggregate Type. Recognizing the variability inherent in aggregates, it was decided to investigate two coarse aggregate types: a quarried carbonate and a quarried silicate aggregate. The use of quarried (or fully crushed) materials allows for a direct comparison to be made based on aggregate type alone, since a gradation meeting AASHTO No. 57 requirements was used for both aggregate types. The carbonate aggregate selected was a high-quality limestone from the Presque Isle Corporation (MDOT Pit No. 71-47) located in northern Michigan. The siliceous aggregate was a high-quality quarried gabbro from Bruce Mines, Ontario, Canada (MDOT Pit No. 95-10).

The fine aggregate used was from the Doctors Pit (MDOT No. 34-86), which is primarily a natural siliceous sand deposit known not to be alkali-silica reactive.

Accelerator Type. Concrete mixtures made with no accelerator (although a Type E water-reducing and water-accelerating admixture [Degussa Lubricon NCA] was used) and mixtures made from different types of accelerator (calcium chloride [Dow Flake] and Grace PolarSet [a non-chloride accelerator meeting AASHTO M 194 Type C requirements]) were evaluated.

Water Reducer Type. Concrete mixtures made with two types of levels of water reducer or with no water reducer, reflecting SHA specifications that in general do not specify a water reducer, were evaluated. Yet, it is known that water reducers can be advantageous, because they allow less water to be added while maintaining workability and also help disperse the cement grains more uniformly, which is important in high–cement-content mixtures. The types of water reducer selected were an AASHTO M 194 Type E (Degussa Lubricon NCA) and a Type F

(Grace ADVA Flow), respectively. Both Type E and Type F (a high-range water reducer) are specified in a limited number of SHA specifications.

Curing Temperature. Two curing temperatures were evaluated in this study. The first temperature—the laboratory ambient temperature of 23^{o}C (73^{o}F)—was used for curing 11 different mixtures. For the second temperature, three mixtures ("C," "D," and "M") were cured under electric heated blankets to achieve a temperature of approximately 65^{o}C (150^{o}F) to evaluate the effects that higher-temperature curing has on strength development, shrinkage, durability, microstructure, and sorptivity. Previous studies have found that 6- to 8-hour EOT concrete materials can be placed during summer months, and the New York Department of Transportation's specifications even use a 65^{o}C (150^{o}F) surface temperature as an opening criterion. The higher-temperature curing was applied after the final set had occurred by grouping the relevant specimens together in a frame covered with an electric blanket for 6 hours after casting.

Other Comments Regarding 6- to 8-hour EOT Concrete Mixture Designs

It was obvious that dosage rates for the various admixtures would need to be adjusted from mixture to mixture once specific constituents were identified and experience was gained in the laboratory. For example, it was necessary to adjust the accelerator dosage rate depending on the cement type, cement content, and the *w/c* ratio. The volume of the aggregate also had to be varied as the cement factor changed, but the ratio of coarse to fine aggregate was held constant.

Further, certain fresh mixture properties were not specifically controlled and thus were allowed to vary within a specified range from mixture to mixture. For example, the consistency of the mixture, as measured by slump, varied from mixture to mixture, with a desired range of 50 to 150 mm (2 to 6 in.). All mixtures were also air-entrained with a desired fresh concrete air content of 6 ± 1.5 percent. Table 7 summarizes the 6- to 8-hour EOT concrete mixture design parameters as batched for this study. A detailed summary of the mixture design information is presented in Appendix C.

TABLE 7 Summary of 6- to 8-hour EOT concrete mixture design parameters as batched per cubic meter (cubic yard)

Mix Number	Batch	Cement		w/c Ratio	Coarse Agg.	Accelerator		Water Reducer		Cure Temp
		kg (lb)	Type			Type	Amount	Type	Amount ml (oz)	
A-1	A	425 (716)	I	0.40	71-47	NC	5,868 ml (198 oz)	No		23°C (73°F)
	B	425 (716)	I	0.40	71-47	NC	5,868 ml (198 oz)	No		23°C (73°F)
B-2	A	525 (885)	I	0.40	71-47	NC	3,912 ml (132 oz)	No		23°C (73°F)
	B	525 (885)	I	0.40	71-47	NC	3,912 ml (132 oz)	No		23°C (73°F)
C-3	A	525 (885)	I	0.40	71-47	NC	3,912 ml (132 oz)	No		blanket
	B	525 (885)	I	0.40	71-47	NC	3,912 ml (132 oz)	No		blanket
D-4	A	525 (885)	I	0.40	95-10	NC	3,912 ml (132 oz)	No		blanket
	B	525 (885)	I	0.40	95-10	NC	3,912 ml (132 oz)	No		blanket
E-5	A	425 (716)	I	0.40	95-10	NC	5,868 ml (198 oz)	No		23°C (73°F)
	B	425 (716)	I	0.40	95-10	NC	5,868 ml (198 oz)	No		23°C (73°F)
F-6	A	525 (885)	I	0.36	71-47	NC	3,920 ml (132 oz)	No		23°C (73°F)
	B	525 (885)	I	0.36	71-47	NC	3,920 ml (132 oz)	No		23°C (73°F)
G-7	A	525 (885)	I	0.40	71-47	NC	3,920 ml (132 oz)	F	130 (4.4)	23°C (73°F)
	B	525 (885)	I	0.40	71-47	NC	3,920 ml (132 oz)	F	87 (2.9)	23°C (73°F)
H-8	A	525 (885)	I	0.36	71-47	No		E	2,608 (88.2)	23°C (73°F)
	B	525 (885)	I	0.36	71-47	No		E	1,630 (55.1)	23°C (73°F)
I-9	A	425 (716)	I	0.40	71-47	CC	1.08 kg (2.38 lb)	No		23°C (73°F)
	B	425 (716)	I	0.40	71-47	CC	1.08 kg (2.38 lb)	No		23°C (73°F)
J-10	A	525 (885)	I	0.40	71-47	CC	0.67 kg (1.48 lb)	No		23°C (73°F)
	B	525 (885)	I	0.40	71-47	CC	0.67 kg (1.48 lb)	No		23°C (73°F)
K-11	A	425 (716)	III	0.40	71-47	NC	5,868 ml (198 oz)	F	211 (7.1)	23°C (73°F)
	B	425 (716)	III	0.40	71-47	NC	5,868 ml (198 oz)	F	157 (5.3)	23°C (73°F)
L-12	A	525 (885)	III	0.40	71-47	NC	3,912 ml (132 oz)	F	130 (4.4)	23°C (73°F)
	B	525 (885)	III	0.40	71-47	NC	3,912 ml (132 oz)	F	130 (4.4)	23°C (73°F)
M-13	A	525 (885)	III	0.40	71-47	NC	3,912 ml (132 oz)	F	130 (4.4)	blanket
	B	525 (885)	III	0.40	71-47	NC	3,912 ml (132 oz)	F	154 (5.2)	blanket
N-14	A	425 (716)	III	0.40	71-47	CC	1.08 kg (2.38 lb)	F	157 (5.3)	23°C (73°F)
	B	425 (716)	III	0.40	71-47	CC	1.08 kg (2.38 lb)	F	211 (7.1)	23°C (73°F)

NC: non-chloride.
CC: calcium chloride.

3.2.2 Material Combinations for 20- to 24-hour EOT Concrete Mixtures

The material combinations used for the 20- to 24-hour EOT concrete materials are presented in Table 8. A numerical designation (1 through 6) has been place above each independent variable and an alpha-numeric designation (A-15 through N-28) is alongside each combination.

TABLE 8 Summary of proposed 20- to 24-hour EOT concrete mixture designs for laboratory study

Combination	1 Cement Type	2 Cement Factor	3 *w/c* Ratio	4 Coarse Aggregate Type	5 Accelerator Type	6 Water Reducer Type
A-15	Type I	400 kg/m^3 (674 lb/yd^3)	0.43	Carbonate	Non-Chloride	No
B-16	Type I	400 kg/m^3 (674 lb/yd^3)	0.43	Carbonate	Calcium Chloride	No
C-17	Type I	400 kg/m^3 (674 lb/yd^3)	0.43	Siliceous	Non-Chloride	No
D-18	Type I	400 kg/m^3 (674 lb/yd^3)	0.43	Gravel	Non-Chloride	No
E -19	Type I	400 kg/m^3 (674 lb/yd^3)	0.40	Carbonate	Non-Chloride	No
F-20	Type I	400 kg/m^3 (674 lb/yd^3)	0.40	Carbonate	Calcium Chloride	No
G-21	Type I	475 kg/m^3 (800 lb/yd^3)	0.43	Carbonate	Non-Chloride	No
H-22	Type I	475 kg/m^3 (800 lb/yd^3)	0.43	Carbonate	No	Type E
I-23	Type I	475 kg/m^3 (800 lb/yd^3)	0.43	Carbonate	No	Type A
J-24	Type I	475 kg/m^3 (800 lb/yd^3)	0.43	Carbonate	No	No
K-25	Type I	475 kg/m^3 (800 lb/yd^3)	0.43	Siliceous	No	No
L-26	Type I	475 kg/m^3 (800 lb/yd^3)	0.43	Gravel	No	No
M-27	Type III	400 kg/m^3 (674 lb/yd^3)	0.40	Carbonate	Non-Chloride	No
N-28	Type III	400 kg/m^3 (674 lb/yd^3)	0.40	Carbonate	Calcium Chloride	No

Each of the six independent variables considered in the study of 20- to 24-hour EOT concrete mixtures has either two or three levels. This consideration results in a possible 216 material combinations, of which 14 combinations were evaluated. Two batches of each material combination were produced and test specimens were cured at 23°C. High curing temperature was not considered. A discussion of the six independent variables follows.

Independent Variables for Laboratory Experiment

The six variables considered in the 20- to 24-hour EOT concrete experiment are the same first six variables in the 6- to 8-hour EOT concrete experiment. However, the range of each variable and the selected levels differed from those used for the 6- to 8-hour EOT concrete experiment to consider the slower strength gain requirement of the 20- to 24-hour EOT concrete mixtures. Below is a description of each of these variables.

Cement Type. The same Type I and III portland cements (Lafarge Alpena Type I and Lafarge Woodstock Type III) were used in both the 20- to 24-hour and the 6- to 8-hour EOT concrete experiments. (See Table 6 for the properties of the cement.) Both Type I and Type III cements have been used by SHAs and in studies on 20- to 24-hour EOT concrete materials.

Cement Factor. Two levels (400 and 475 kg/m^3 [674 and 800 lb/yd^3]) were selected for evaluation. This range is lower than that considered for the 6- to 8-hour EOT concrete experiment to account for the slower rate of strength gain required for 20- to 24-hour EOT concrete materials and the range stipulated in SHA specifications. It is noted that the lowest cement factors were used only for Type III cement.

w/c Ratio. Two levels of *w/c* ratio (0.40 and 0.43) were evaluated. This range is slightly higher than that proposed for the 6- to 8-hour EOT concrete mixtures to consider the slower rate of strength gain stipulated in SHA specifications.

Coarse Aggregate Type. In addition to the two quarried coarse aggregate types included in the 6- to 8-hour EOT concrete experiment, a processed gravel was included in the 20- to 24-hour EOT concrete experiment as a third coarse aggregate type because many SHAs use this type of material in their 20- to 24-hour EOT concrete repairs. The gravel aggregate was obtained from the same source of the natural sand being used as the fine aggregate (Doctors Pit [MDOT No. 34-86]). This coarse aggregate is primarily siliceous in nature and known not to be alkali-silica reactive.

Accelerator Type. The accelerators used in the 20- to 24-hour EOT concrete experiment were the same as those used in the 6- to 8-hour experiment, although the addition rates were altered to reflect differences in mixture components and early strength gain requirements. These accelerators included calcium chloride (Dow Flake), a non-chloride accelerator (Grace PolarSet) specified under AASHTO M 194 as Type C, and an AASHTO M 194 Type E water-reducing and accelerating admixture (Degussa Lubricon NCA). Also, mixtures containing no accelerator were made.

Water Reducer Type. In addition to mixtures containing no water reducer, mixtures were made with two types of water reducers: AASHTO M 194 Type A (Grace WRDA 20) and Type E (Degussa Lubricon NCA). The Type A water reducer is a low-range water reducer that is specified by a number of SHAs for 20- to 24-hour EOT concrete materials.

Notes on Mixture Proportioning

As is true with the 6- to 8-hour EOT concrete mixtures, dosage rates for the various admixtures were varied once specific components were identified and experience was gained in the laboratory. Certain fresh mixture properties were not specifically controlled, but were allowed to change within a specified range from mixture to mixture. For example, the consistency of the mixture, as measured by slump, was targeted to vary from 50 to 100 mm (2 to 4 in.) as other mixture parameters were changed. All mixtures were air-entrained using a vinsol resin–based admixture (Axim Catexol VR), as is common in most SHAs, with a desired fresh concrete air content of 6 ± 1.5 percent. Table 9 summarizes the as-batched parameters for the 20- to 24-hour EOT concrete mixtures. A detailed summary is presented in Appendix C.

TABLE 9 Summary of 20- to 24-hour EOT mixture design parameters as batched per cubic meter (cubic yard)

Mix Number	Batch	Cement		w/c Ratio	Coarse Agg. Type	Accelerator		Water Reducer		Curing Temperature
		kg/m³ (lb/yd³)	Type			Type	Amount	Type	ml (oz)	
A-15	A	400 (674)	I	0.43	71-47	NC	978 ml (33.1 oz)	No		23°C (73°F)
	B	400 (674)	I	0.43	71-47	NC	978 ml (33.1 oz)	No		23°C (73°F)
B-16	A	400 (674)	I	0.43	71-47	CC	0.51kg (1.12 lb)	No		23°C (73°F)
	B	400 (674)	I	0.43	71-47	CC	0.51kg (1.12 lb)	No		23°C (73°F)
C-17	A	400 (674)	I	0.43	95-10	NC	978 ml (33.1 oz)	No		23°C (73°F)
	B	400 (674)	I	0.43	95-10	NC	978 ml (33.1 oz)	No		23°C (73°F)
D-18	A	400 (674)	I	0.43	34-86	NC	1,300 ml (44.0 oz)	No		23°C (73°F)
	B	400 (674)	I	0.43	34-86	NC	1,300 ml (44.0 oz)	No		23°C (73°F)
E-19	A	400 (674)	I	0.40	71-47	NC	978 ml (33.1 oz)	No		23°C (73°F)
	B	400 (674)	I	0.40	71-47	NC	978 ml (33.1 oz)	No		23°C (73°F)
F-20	A	400 (674)	I	0.40	71-47	CC	0.51kg (1.12 lb)	No		23°C (73°F)
	B	400 (674)	I	0.40	71-47	CC	0.51kg (1.12 lb)	No		23°C (73°F)
G-21	A	475 (800)	I	0.43	71-47	NC	520 ml (17.6 oz)	No		23°C (73°F)
	B	475 (800)	I	0.43	71-47	NC	520 ml (17.6 oz)	No		23°C (73°F)
H-22	A	475 (800)	I	0.43	71-47	None		E	1,300 (44.0)	23°C (73°F)
	B	475 (800)	I	0.43	71-47	None		E	1,300 (44.0)	23°C (73°F)
I-23	A	475 (800)	I	0.43	71-47	None		A	79 (2.7)	23°C (73°F)
	B	475 (800)	I	0.43	71-47	None		A	79 (2.7)	23°C (73°F)
J-24	A	475 (800)	I	0.43	71-47	None		No		23°C (73°F)
	B	475 (800)	I	0.43	71-47	None		No		23°C (73°F)
K-25	A	475 (800)	I	0.43	95-10	None		No		23°C (73°F)
	B	475 (800)	I	0.43	95-10	None		No		23°C (73°F)
L-26	A	475 (800)	I	0.43	34-86	None		No		23°C (73°F)
	B	475 (800)	I	0.43	34-86	None		No		23°C (73°F)
M-27	A	400 (674)	III	0.40	71-47	NC	520 ml (17.6 oz)	F	199 (6.7)	23°C (73°F)
	B	400 (674)	III	0.40	71-47	NC	520 ml (17.6 oz)	F	199 (6.7)	23°C (73°F)
N-28	A	400 (674)	III	0.40	71-47	CC	0.51kg (1.12 lb)	F	199 (6.7)	23°C (73°F)
	B	400 (674)	III	0.40	71-47	CC	0.51kg (1.12 lb)	F	199 (6.7)	23°C (73°F)

NC: non-chloride.
CC: calcium chloride.

3.2.3 Specimen Preparation

Laboratory specimens were prepared in strict accordance with AASHTO T 126 at the Michigan DOT's Construction and Technology Laboratory in Lansing, Michigan. A 0.4-m^3 (14-ft^3) capacity Lancaster mixer was used to ensure that all specimens needed for a given mixture were produced in a single batch, reducing experimental variability. As noted, two batches were made of each material combination. Furthermore, the order in which batches were prepared was randomized to avoid systematic error. Figure 6 lists the specimens that were made from each batch, how they were cured, and how they were ultimately tested. As can be seen, 30 specimens were made from each batch, including cylinders, beams, prisms, and rings.

All coarse aggregates were sieved into standard-size fractions and then combined to the desired grading. The aggregate volume fraction was changed for the various mixtures as cement factor and *w/c* ratio varied, but the proportion of coarse aggregate to fine aggregate was held constant. Details on the volume fraction of coarse and fine aggregate for all mixtures are provided in Appendix C. The aggregates were brought to SSD condition prior to mixing to ensure accuracy in the resulting *w/c* ratio. Molds were stripped from the specimens 6 and 20 hours after casting for the 6- to 8-hour and 20- to 24-hour EOT concrete mixtures, respectively. Specimens were cured as specified in the applicable test method.

3.2.4 Testing Laboratory-Prepared Specimens

Tests were conducted on all the 6- to 8-hour and 20- to 24-hour EOT concrete materials using the same testing procedures. As presented in Table 10, 28 specimens were needed per batch for each 6- to 8-hour EOT concrete mixture, and 26 specimens were needed per batch for the 20- to 24-hour EOT concrete mixtures. In total, 1,512 specimens were made and tested in the course of this study.

The tests dealt with the following five concrete attributes:

- Properties of fresh concrete,
- Volume change,
- Freeze-thaw durability,
- Microstructural characterization, and
- Absorption/sorptivity.

Figure 6. Specimen production and disposition for each batch.

TABLE 10 Details of the laboratory evaluation

Concrete Attribute	Test Name/Equipment	Measured Property	No. of Specimens per batch	Specification
Properties of Fresh Concrete	Slump	Concrete workability	One per batch	AASHTO T 119-93 *MDOT*[1]
	Air Content	Total air content of fresh concrete	One per batch	AASHTO T 152-93 *MDOT*
	Maturity and Time of Setting	Time of curing and temperature of strength gain/time and depth	2	ASTM C 1074 AASHTO T131-93 *MDOT*
Volume Change	Coefficient of Thermal Expansion	Coefficient of Thermal Expansion	2	AASHTO TP 60-00 *MSU*
	Restrained Drying Shrinkage Test Ring	Susceptibility to drying shrinkage cracking	2	AASHTO PP 34-99 *MSU/MDOT*
Freeze-Thaw Durability	Resistance of Concrete to Freezing and Thawing	Loss of concrete stiffness due to freeze-thaw damage	2	AASHTO T 161-93 *MDOT*
	Exposure to Deicers	Scaling resistance under deicer application	2	ASTM C 672-92 *MDOT*
Microstructural Characterization	Compressive Strength	Concrete stiffness and strength	8[2]	AASHTO T 22-92 *MDOT*
	Flexural Strength	Concrete Flexural Strength	4[3]	AASHTO T 97-86 *MDOT*
	Stereo Optical Microscopy	Air-void system parameters	2 originals others[4]	ASTM C 457 and C 856 *MTU*
	Petrographic Optical Microscopy with UV Dye Impregnation	Microstructure analysis, reaction products, and evidence of deterioration	Same specimens as stereo microscopy	ASTM C 856 *MTU*
	Scanning Electron Microscopy (SEM)	Microstructure analysis, elemental analysis	Same specimens as stereo microscopy	ASTM C 856 *MTU*
	X-Ray Microscopy (XRM)	Chloride profiling and elemental analysis	Deicer test specimens	No standard test *MTU*
Absorption/Porosity	Specific Gravity, Absorption, and Voids	Determines specific gravity, percent adsorption, and voids	2	ASTM C 642-90 *MDOT*
	Sorptivity	Assessment of concrete permeability	2	Proposed ASTM Sorptivity Test *MSU*
Total Specimens			28	

[1] The agency responsible for running each test is identified in Italics in the last column of the table.
[2] For the 6- to 8-hour EOT concrete mixtures, two cores were tested at 6, 8, and 24 hours and two at 28 days. For the 20- to 24-hour EOT concrete mixtures, two cores were tested at 20 and 24 hours, and two at 28 days.
[3] Tested either at 6 hours or 20 hours.
[4] Microscopy was done on two original specimens, one from AASHTO T161 and one from ASTM C 672.

Properties of Fresh Concrete

Testing of fresh concrete was conducted in accordance with the relevant test methods, and the results were used to verify workability and air content and to establish maturity trends. This testing was necessary to ensure that the mixtures produced and tested could be constructed and thus have practical application. The measured air contents were also compared with the air-void system parameters obtained from the hardened concrete as determined by ASTM C 457. The established maturity trends help provide a basis for an early opening criterion, as well as an understanding of the heat of hydration characteristics for the various mixtures.

Volume Change

Two tests were used to assess the volume change characteristics of the concrete mixtures: the CTE and the restrained drying shrinkage cracking tests. The CTE is an important parameter with regards to thermal stress development. It is largely influenced by aggregate type. Concrete made with siliceous aggregates typically has higher coefficients of thermal expansion. Variations due to other mixture parameters were also investigated, including paste volume, *w/c* ratio, and cement type. The results of the restrained drying shrinkage cracking test (AASHTO PP 34-99) provide a direct indication of the propensity for the various mixtures to undergo potentially damaging volume change due to drying.

Freeze-Thaw Durability

Two freeze-thaw durability tests were conducted on specimens that were cured for 28 days. AASHTO T 161 Procedure A was used to assess the durability of concrete subjected to cyclic freezing and thawing. Specimens were tested to 300 cycles in accordance with the temperature cycling regime specified in the test method and the dilation measured (only dilation was measured in accordance with the MDOT standard operating protocol). ASTM C 672 was performed to evaluate the concrete's resistance to scaling from deicer applications.

Microstructure of the test specimens was also characterized after testing. The degree of microcracking in the specimens after the freezing and thawing tests (AASHTO T 161) and the scaling resistance tests (ASTM C 672) was assessed to determine if significant alterations resulted from the environmental conditioning. The microstructural characterization observations verified that the distress was indeed due to freezing and thawing and/or deicer application.

Microstructural Characterization

Seven separate test methods were conducted under the microstructural characterization test attribute. Two of these tests are standard compressive strength (AASHTO T 22) and flexural strength (AASHTO T 97) tests. Compressive strength tests were conducted at 6 hours, 8 hours, 24 hours, and 28 days for the 6- to 8-hour EOT concrete mixtures and at 20 hours, 24 hours, and 28 days for the 20- to 24-hour EOT concrete mixtures. Flexural strength tests were only conducted at 6 hours and 8 hours for the 6- to 8-hour EOT concrete and at 20 hours and 24 hours for the 20- to 24-hour EOT concrete to provide a comparison between the two modes of testing.

The other microstructural characterization tests include the same staining and microscopy techniques previously described under the field evaluation. Specimens from all 28 material combinations were evaluated with these techniques. The purpose of the intensive microstructural characterization was to gain a good understanding of how the microstructure varied among mixtures and to determine the impact of microstructural characteristics on EOT concrete performance. The results of the microstructural characterization were also correlated, to the degree possible, with the volume change, freeze-thaw durability, and sorptivity test data.

ASTM C 457 was used to determine the air-void system parameters from polished slabs using the stereo optical microscope. Reported factors used in the statistical analysis included the air content, spacing factor, specific surface, and paste-to-air ratio. Petrographic evaluation in accordance with ASTM C 856 was also conducted on thin sections. Because of the subjective nature of petrographic analysis, a system was developed to rate the degree of paste inhomogeniety on a scale of 1 to 3 (with 1 being homogenous and 3 being highly inhomogenous) and the degree of microcracking also on a scale of 1 to 3 (with 1 being free of microcracks and 3 being a high degree of microcracking). Beams that had been subjected to AASHTO T 161, the freezing and thawing test, were sectioned. The degree of microcracking (reported as microns per square millimeter) was measured using a high-resolution flatbed scanner. The degree of chloride ion ingress into the ASTM C 672 specimens was assessed using the x-ray microscope.

Absorption/Sorptivity

Two 100-mm (4-in.)-diameter specimens from each material/condition combination were evaluated for absorption and sorptivity in accordance with ASTM C 642 and the proposed ASTM sorptivity test to provide an indication of the porosity and permeability.

3.2.5 Results and Discussion of the Laboratory Evaluation

This section presents the results of the laboratory experiment. It describes the individual results for the following test attributes: properties of fresh concrete, volume change, freeze-thaw durability, microstructural characterization, and adsorption/porosity. Summary plots, tables and detailed data for both the 6- to 8-hour and 20- to 24-hour EOT concrete mixtures are presented in Appendix C and are briefly discussed in this chapter. Statistical analysis of the results is also presented.

Properties of Fresh Concrete

The properties of fresh concrete measured for each mixture include the slump (AASHTO T 119), air content (AASHTO T 152), unit weight (AASHTO T 121), and maturity (ASTM C 1074). As would be expected, the results varied between the 6- to 8-hour and the 20- to 24-hour EOT concrete and also within each category of mixture. The average slump was slightly higher for the 20- to 24-hour mixtures than for the 6- to 8-hour mixtures, although one 6- to 8-hour mixture (G-7) exhibited slump values in excess of 250 mm (10 in.). This mixture had a high cement content, a high *w/c* ratio, a calcium chloride accelerator, and a Type F HRWR, making it difficult to obtain the desired properties with a lower slump.

The results of the laboratory-measured air content of the fresh concrete, shown in Figures 7 and 8, for the most part were within the desired range, with a few exceptions for the 6- to 8-hour EOT concrete mixtures category. One mixture, E-5, exhibited an air content slightly higher than desired. This mixture contained a low cement content and was made with a high *w/c* ratio and a non-chloride accelerator. Three of the eight mixtures made with Type III cement had low measured fresh air content. All mixtures containing Type III cement were made using a Type F HRWR to facilitate mixing. An overall higher variability was observed in the 6- to 8-hour mixtures than for the 20- to 24-hour mixtures, as reflected in the coefficient of variability.

The maturity results followed expected trends, with the 6- to 8-hour mixtures having much higher maturity, especially at the 8-hour test time. These numbers are a little misleading because some of the mixtures (C-3, D-4, and M-13) from the 6- to 8-hour category were cured at higher ambient temperatures, contributing to a noticeable increase in maturity. But even accounting for these mixtures, the 8-hour maturity levels were higher for the 6- to 8-hour mixtures than for the 20- to 24-hour mixtures. Within a category, there is little noticeable difference due to different cement content, *w/c* ratio, accelerator type, or cement type.

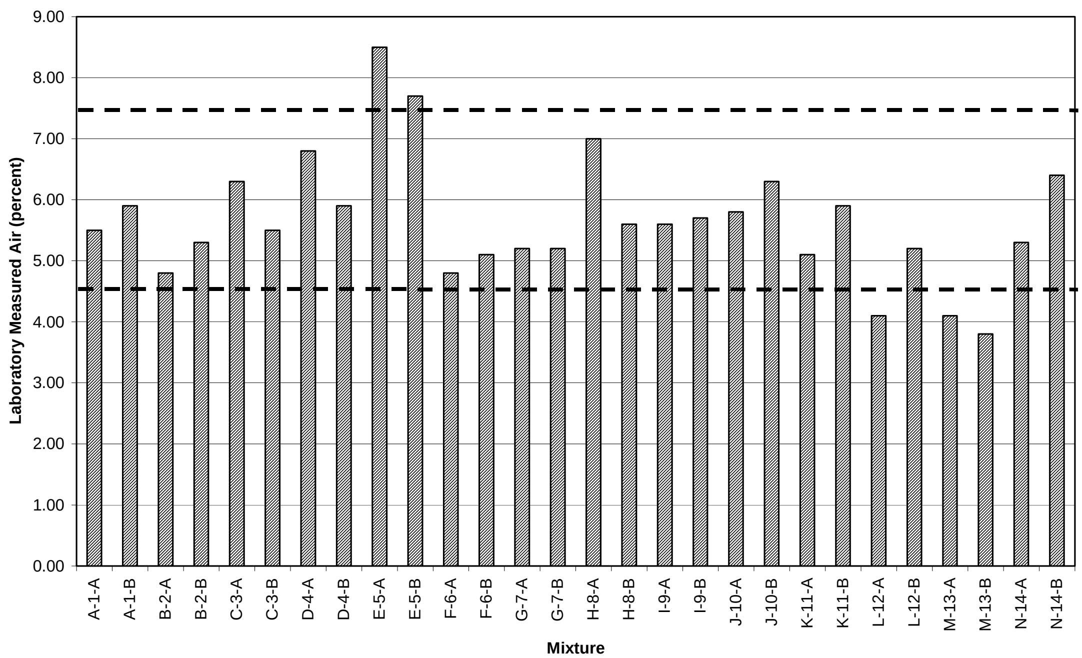


Figure 7. Air content for 6- to 8-hour mixtures (dashed lines indicate desired target range).

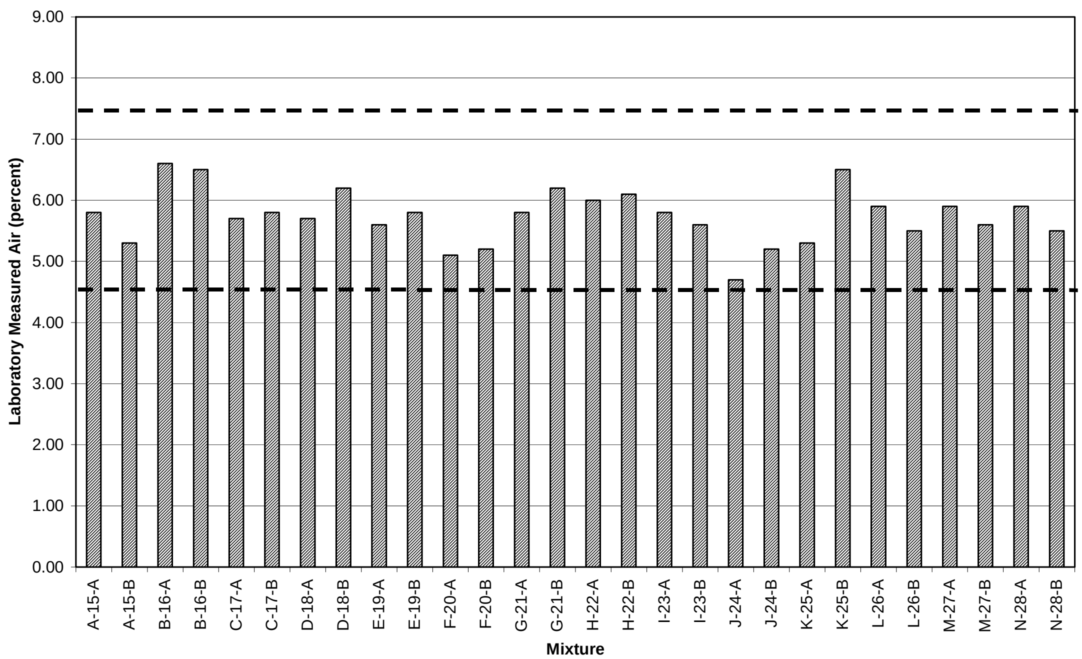


Figure 8. Air content for 20- to 24-hour mixtures (dashed lines indicate desired target range).

Volume Change

Assessment of volume change was made based on the CTE (AASHTO TP 60) and restrained drying shrinkage (AASHTO PP 34) values. The CTE values varied from mixture to mixture, with the type of coarse aggregate having the biggest influence. Consistent results were obtained for each mixture that exhibited higher-than-expected CTE values except for the second replicates for C-3-B and L-12-B. Overall, there was slightly more variability observed between replicates in the 6- to 8-hour mixtures than in the 20- to 24-hour mixtures, reflecting a higher inherent variability in the higher early strength mixtures, although all the reported values are within the expected range for the materials used. Over the range of variables studied, the cement content did not correlate with the CTE.

The time to first crack in days and the total number of cracks that occurred over the duration of the test were determined from the restrained shrinkage ring tests. Evaluating the data for time to first crack revealed a statistically significant difference ($\alpha = 0.05$) between the 6- to 8-hour and the 20- to 24-hour mixtures, indicating that the higher early strength materials will crack earlier. No such difference could be statistically shown for the total number of cracks, suggesting that higher early strength mixtures do not necessarily crack more often.

It is also observed that although most mixtures cracked at some point, some did not. The most notable observation made was that curing temperature had a large influence on the shrinkage/cracking behavior of the concrete, with the high-temperature curing resulting in a reduced propensity to crack (of the 12 shrinkage ring specimens cured at the elevated temperature, only 1 cracked). The diverse factors contributing to this observation (e.g., early strength development, shrinkage at elevated temperatures, and expansion/contraction of the steel ring) and the limited tests made it impossible to determine the role of higher-temperature curing on the propensity for cracking in the concrete.

The restrained shrinkage test did not always provide repeatable results between batches or even between replicates made from the same batch. In some cases, excellent repeatability was observed, such as for mixture B-2. But as often, poor repeatability was observed, such as in the case of mixture I-9. The use of the restrained shrinkage test in this study indicates that the test might provide useful information regarding the shrinkage and cracking propensity of high early strength materials.

Freeze-Thaw Durability

The resistance of concrete to freezing and thawing test (AASHTO T161) results indicate that the dilation values varied greatly from mixture to mixture. This variation can be seen graphically in Figures 9 and 10. Statistically, a significant difference ($\alpha = 0.05$) exists between the dilation values for the 6- to 8-hour and the 20- to 24-hour mixtures, with the 6- to 8-hour mixtures having higher dilation values. In general, none of the 20- to 24-hour EOT concrete mixtures had unacceptable dilation values, whereas approximately 20 percent of the 6- to 8-hour mixture specimens exceeded 0.01-mm/mm dilation. Most of these specimens were made with Type III cement and Type F HRWR, which would initially suggest that the cement type and/or water reducer might play a role. Further, one of the 6- to 8-hour mixtures (N-14) made with Type III cement and calcium chloride accelerator did not show exceptionally high dilations.

Figure 9. Dilation for 6- to 8-hour mixtures.

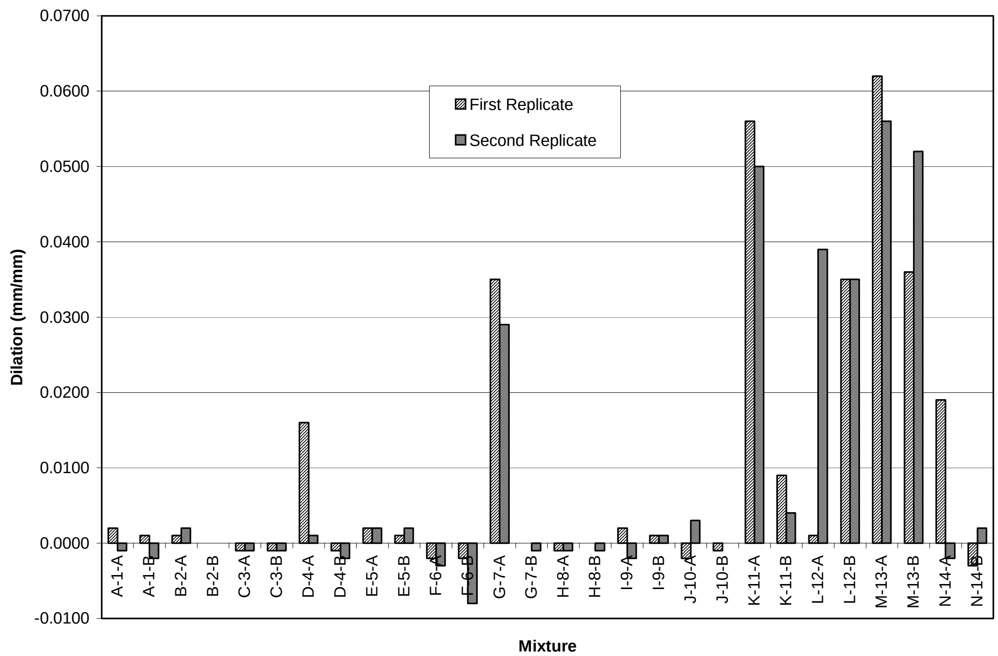
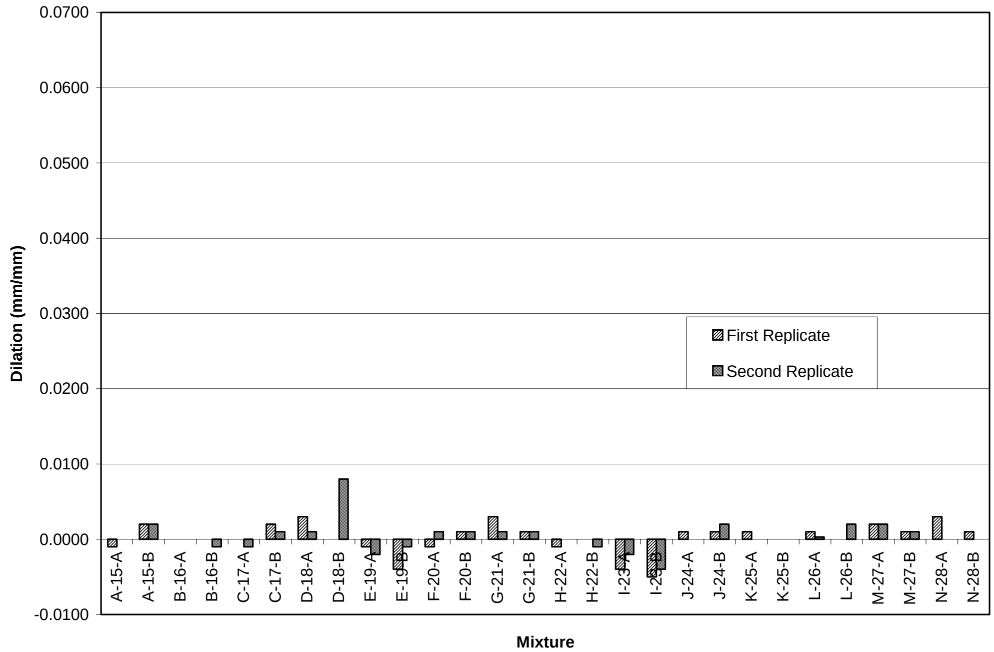


Figure 10. Dilation for 20- to 24-hour mixtures.



Batch G-7-A, which is the only batch made with Type I cement that contained both the Type F HRWR and the non-chloride accelerator had high dilations. The dosage of the Type F admixture was higher for G-7-A than for G-7-B (the admixture dosage was reduced to correct the high slump), whereas the dosage of air entrainer was increased (both G-7-A and G-7-B had measured air contents of 5.2 percent). Similar results were observed for one of the Type III mixtures, where the first replicate (K-11-A) was made with a higher HRWR dosage and lower air entrainer dosage than the second replicate (K-11-B). These results suggest that a possible interaction between the various admixtures may negatively affect the air-void system in high–cement-content, low–*w/c*-ratio mixtures.

The deicer scaling results were even more variable than the freeze-thaw results. Statistically, the amount of scaling present was higher for the 6- to 8-hour mixtures than for the 20- to 24-hour mixtures. Many of the 6- to 8-hour Type III mixtures (K-11, L-12, M-13, N-14) that had relatively high dilation values also had a high degree of scaling. But there is little direct correlation between scaling rating and dilation (e.g., M-13-A had the highest dilation, but the degree of scaling rating was less than 1, whereas K-11-B and N-14-A had the highest severity of scaling, but both had relatively low dilation values).

Microstructural Characterization

The microstructural characterization of the mixtures included compressive and flexural strength tests, stereo optical microscopy, petrographic microscopy, electron microscopy, x-ray microscopy to assess chloride penetration into scaling specimens, and crack length in specimens that were subjected to freeze-thaw testing.

A summary of the strength data for the 6- to 8- hour and 20- to 24-hour EOT concrete mixtures is presented in Tables 11 and 12, respectively. The data follow predicted trends, with the 6- to 8-hour mixtures gaining strength quickly, having higher average 24-hour and 28-day compressive strength than the 20- to 24-hour mixtures. The initial criteria were to achieve a compressive strength of 13.8 MPa (3,000 psi) and a flexural strength of 2.1 MPa (300 psi) within the opening time criterion. The gray shading in Tables 11 and 12 identifies the strength values that were below the desired values. Of the twenty-eight 6- to 8-hour EOT concrete mixtures, nineteen did not achieve the desired 6-hour compressive strength, and five of the remaining nine mixtures that achieved the desired early strength were cured at a high temperature, demonstrating the importance of increased temperature. The four batches cured at laboratory ambient temperatures that met the minimum compressive strength criterion were from two mixtures made with Type I cement: one mixture had a low cement content and high *w/c* ratio, and the other mixture had high cement content and low *w/c* ratio. Within 8 hours, only five batches from three mixtures (all made with Type I cement) did not meet the minimum compressive strength criterion. Of the two mixtures that failed to meet the minimum compressive strength criterion within 8 hours, one (G-

7) used a Type E admixture, and the other (H-8) used a Type F admixture. The fifth batch (J-10-B) had unexpectedly low strengths.

TABLE 11 Compressive and flexural strength for 6- to 8-hour EOT concrete mixtures

Mixture	Compressive Strength, MPa (psi)				Flexural Strength, MPa (psi)	
	6 hour	8 hour	24 hour	28 day	6 hour	8 hour
A-1-A	14.6 (2,120)	20.4 (2,960)	36.7 (5,325)	46.7 (6,775)	2.2 (320)	2.7 (390)
A-1-B	14.5 (2,105)	19.9 (2,885)	33.0 (4,785)	47.9 (6,950)	2.6 (375)	2.8 (405)
B-2-A	13.3 (1,930)	16.5 (2,395)	29.3 (4,250)	43.7 (6,340)	1.9 (275)	2.4 (350)
B-2-B	11.0 (1,595)	16.3 (2,365)	28.7 (4,165)	44.2 (6,410)	2.0 (290)	2.4 (350)
C-3-A	9.4 (1,365)	14.8 (2,145)	27.0 (3,915)	37.0 (5,365)	1.7 (245)	2.3 (335)
C-3-B	14.4 (2,090)	20.2 (2,930)	31.1 (4,510)	38.1 (5,525)	2.1 (305)	2.5 (365)
D-4-A	14.3 (2,075)	19.5 (2,830)	28.1 (4,075)	37.1 (5,380)	2.1 (305)	2.2 (320)
D-4-B	16.9 (2,450)	21.3 (3,090)	28.5 (4,135)	38.9 (5,640)	2.6 (375)	2.6 (375)
E-5-A	9.4 (1,365)	14.1 (2,045)	25.2 (3,655)	43.4 (6,295)	1.9 (275)	2.4 (350)
E-5-B	8.4 (1,220)	14.0 (2,030)	24.3 (3,525)	41.5 (6,020)	2.0 (290)	2.5 (365)
F-6-A	15.3 (2,220)	20.9 (3,030)	35.0 (5,075)	52.9 (7,670)	2.6 (375)	2.9 (420)
F-6-B	13.8 (2,000)	19.8 (2,870)	35.1 (5,090)	54.3 (7,875)	2.5 (365)	3.0 (435)
G-7-A	5.5 (800)	10.2 (1,480)	28.5 (4,135	46.0 (6,670)	1.4 (205)	2.1 (305)
G-7-B	4.9 (710)	9.6 (1,390)	24.1 (3,495)	43.8 (6,350)	1.4 (205)	2.0 (290)
H-8-A	1.1 (160)	4.3 (625)	26.2 (3,800)	47.3 (6,860)	0.3 (45)	0.9 (130)
H-8-B	1.5 (220)	6.1 (885)	23.1 (3,350)	48.9 (7,090)	0.6 (90)	1.2 (175)
I-9-A	10.8 (1,565)	17.9 (2,595)	33.4 (4,845)	52.6 (7,630)	1.7 (245)	2.4 (350)
I-9-B	10.3 (1,495)	16.1 (2,335)	35.2 (5,105)	55.1 (7,990)	1.8 (260)	2.5 (365)
J-10-A	7.8 (1,130)	15.2 (2,205)	31.0 (5,715)	47.0 (6,815)	1.0 (145)	2.2 (320)
J-10-B	2.2 (320)	8.7 (1,260)	29.1 (4,220)	46.8 (6,790)	0.8 (115)	1.6 (230)
K-11-A	12.4 (1,800)	20.8 (3,015)	39.0 (5,655)	54.7 (7,935)	2.3 (335)	3.1 (450)
K-11-B	11.0 (1,595)	17.4 (2,525)	35.2 (5,105)	49.8 (7,225)	2.4 (350)	2.6 (375)
L-12-A	11.5 (1,670)	23.9 (3,465)	39.4 (5,715)	53.0 (7,685)	2.2 (320)	3.0 (435)
L-12-B	8.2 (1,190)	21.4 (3,105)	36.4 (5,280)	49.8 (7,225)	1.8 (260)	2.8 (405)
M-13-A	22.9 (3,320)	29.5 (4,280)	39.9 (5,785)	51.0 (7,395)	2.4 (350)	3.1 (450)
M-13-B	22.5 (3,265)	31.4 (4,555)	39.8 (5,770)	48.3 (7,005)	2.5 (365)	2.8 (405)
N-14-A	4.8 (695)	19.4 (2,815)	47.4 (6,875)	62.7 (9,095)	1.5 (220)	2.9 (420)
N-14-B	11.3 (1,640)	24.5 (3,555)	49.7 (7,210)	67.6 (9,805)	1.9 (275)	3.1 (450)
Average	10.86 (1,575)	17.65 (2,560)	32.83 (4,760)	48.22 (6,995)	1.86 (270)	2.46 (355)
St. Dev	5.415 (785)	6.240 (905)	6.709 (975)	7.107 (1,030)	0.612 (89)	0.543 (79)
COV	49.9%	35.4%	20.4%	14.7%	32.8%	22.0%

COV = coefficient of variation.
St. Dev = standard deviation.
Gray shading indicates that the strength values are below the desired criterion.

TABLE 12 Compressive and flexural strength for 20- to 24-hour EOT concrete mixtures

Mixture	Compressive Strength, MPa (psi)			Flexural Strength, MPa (psi)	
	20 hour	24 hour	28 day	20 hour	24 hour
A-15-A	20.4 (2,960)	23.2 (3,365)	37.5 (5,440)	3.3 (480)	3.6 (520)
A-15-B	21.1 (3,060)	24.8 (3,590)	37.4 (5,425)	3.6 (520)	3.6 (520)
B-16-A	25.1 (3,640)	27.5 (3,990)	44.8 (6,500)	3.3 (480)	3.6 (520)
B-16-B	24.0 (3,480)	29.2 (4,235)	47.3 (6,860)	3.5 (510)	3.9 (565)
C-17-A	19.5 (2,830)	21.1 (3,060)	38.6 (5,600)	3.8 (550)	3.8 (550)
C-17-B	17.7 (2,565)	20.9 (3,030)	36.1 (5,235)	3.5 (510)	3.5 (510)
D-18-A	18.1 (2,625)	21.1 (3,060)	35.7 (5,180)	3.3 (480)	3.3 (480)
D-18-B	20.5 (2,975)	22.0 (3,190)	36.5 (5,295)	3.5 (510)	3.7 (535)
E-19-A	20.8 (3,015)	23.0 (3,335)	39.9 (5,785)	3.8 (550)	4.1 (595)
E-19-B	19.0 (2,755)	23.3 (3,380)	41.2 (5,975)	3.9 (565)	4.1 (595)
F-20-A	32.2 (4,670)	35.1 (5,090)	52.0 (7,540)	3.5 (510)	4.3 (625)
F-20-B	33.4 (4,545)	36.7 (5,325)	49.9 (7,235)	4.2 (610)	3.9 (565)
G-21-A	21.9 (3,175)	23.6 (3,425)	37.5 (5,440)	3.5 (510)	3.3 (480)
G-21-B	18.0 (2,610)	19.6 (2,845)	35.5 (5,150)	3.3 (480)	3.8 (550)
H-22-A	14.2 (2,060)	16.1 (2,335)	38.7 (5,615)	2.9 (420)	3.0 (435)
H-22-B	14.5 (2,105)	16.9 (2,450)	39.0 (5,655)	2.7 (390)	3.2 (465)
I-23-A	13.9 (2,015)	16.1 (2,335)	37.9 (5,495)	2.3 (335)	2.8 (405)
I-23-B	13.3 (1,930)	16.3 (2,365)	38.3 (5,555)	2.3 (335)	3.0 (435)
J-24-A	20.9 (3,030)	24.2 (3,510)	33.8 (4,900)	3.6 (520)	3.6 (520)
J-24-B	19.6 (2,845)	21.7 (3,145)	37.8 (5,480)	3.2 (465)	3.6 (520)
K-25-A	19.4 (2,815)	21.2 (3,075)	39.8 (5,775)	3.8 (550)	3.7 (535)
K-25-B	16.2 (2,350)	18.9 (2,740)	38.8 (5,625)	3.3 (480)	3.3 (480)
L-26-A	15.5 (2,250)	20.6 (2,990)	40.4 (5,860)	2.8 (405)	3.1 (450)
L-26-B	15.8 (2,290)	18.8 (2,725)	38.7 (5,615)	2.7 (390)	3.2 (465)
M-27-A	34.1 (4,945)	38.0 (5,510)	54.6 (7,920)	4.8 (695)	4.7 (680)
M-27-B	33.7 (4,890)	37.9 (5,495)	55.4 (8,035)	4.2 (610)	4.8 (695)
N-28-A	43.1 (6,250)	47.7 (6,920)	64.2 (9,310)	4.8 (695)	5.0 (725)
N-28-B	45.7 (6,630)	49.4 (7,165)	66.5 (9,645)	4.5 (655)	4.6 (665)
Average	22.56 (3,270)	25.53 (3,705)	42.64 (6,185)	3.50 (510)	3.72 (540)
St. Dev	8.580 (1,244)	9.066 (1,315)	8.623 (1,251)	0.641 (93)	0.568 (82)
COV	38.0%	35.5%	20.2%	18.3%	15.3%

COV = coefficient of variation.
St. Dev = standard deviation.
Gray shading indicates that the strength values are below the desired criterion.

The flexural strength results were slightly better, with 12 of the 28 batches meeting the 6-hour criterion of 2.1 MPa (300 psi). These batches included those from the same mixtures that met the compressive strength criterion and those from mixtures K-11 and L-12. The latter two mixtures were made with Type III cement. Only four batches did not meet the flexural strength criterion at 8 hours: three of these batches were from mixtures G-7 and H-8, and one was from mixture J-10-B, which is the same batch that had low compressive strengths. Overall, meeting the strength criterion for the 20- to 24-hour mixtures was not a problem. All batches met the compressive strength criterion at 24 hours and the flexural strength criterion at 20 hours. Only one batch (I-23-B) did not meet the compressive strength criterion at 20 hours.

These results indicate a difficulty in meeting high early strength criterion for the fastest-setting mixtures, although the heat generated during hydration assists in this process. Also, admixtures can have a large effect on early strength gain, with the type of water reducer and accelerator playing a role. Difficulty in achieving the desired strength within 8 hours was encountered only for a few mixtures that were primarily made with Type I cement and a water reducer.

Figure 11 shows the relationship between compressive and flexural strength for all mixtures (the data shown are for 6- and 8-hour strengths for the 6- to 8-hour EOT concrete and for the 20- and 24-hour strengths for the 20- to 24-hour EOT concrete). These data indicate no unique relationship between these two measures of strength; therefore, if compressive strength is to be used during construction to estimate flexural strength, the relationship should be established for the actual job mix.

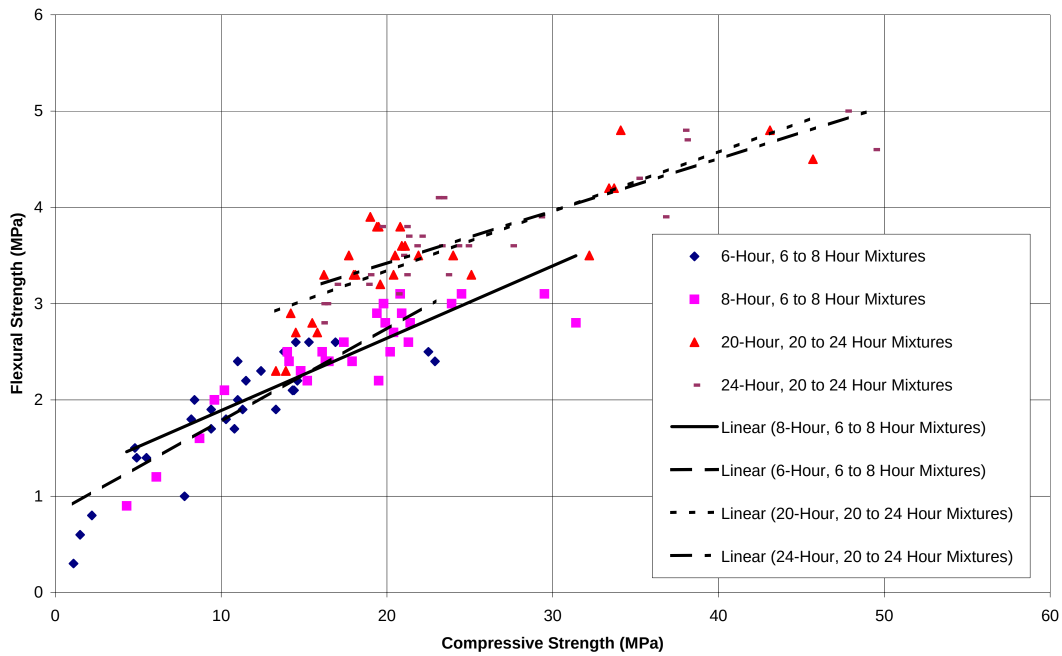


Figure 11. Compressive versus flexural strength for all mixtures.

The stereo optical microscope was used in accordance with ASTM C 457 to collect air-void system parameters. A detailed discussion is presented in Appendix C. The measured air content of the hardened concrete averaged 5.64 percent and 5.54 for the 6- to 8-hour and 20- to 24-hour mixtures, respectively (desired value was 6 ± 1.5 percent). Figures 12 and 13 show the measured values. In general, most mixtures had air contents within the desired range, with an obvious deviation for three of the four 6- to 8-hour mixtures made with Type III cement (L-12, M-13, and N-14), which had air contents significantly below the desired value. These mixtures also contained Type F HRWR, which in combination with fine cement can negatively influence the air-void system. Two of the 20- to 24-hour batches produced with Type III cement (K-25-A and N-28-B) had similarly low air contents. It is not known what factors may have contributed to the low air content in the other 20- to 24-hour mixture (F-20). Obviously, it is difficult to control air content in mixtures made with high cement contents, low *w/c* ratios, and multiple admixtures, especially if fine cement is used.

Figure 12. Air content measured using ASTM C 457 for 6- to 8-hour mixtures.

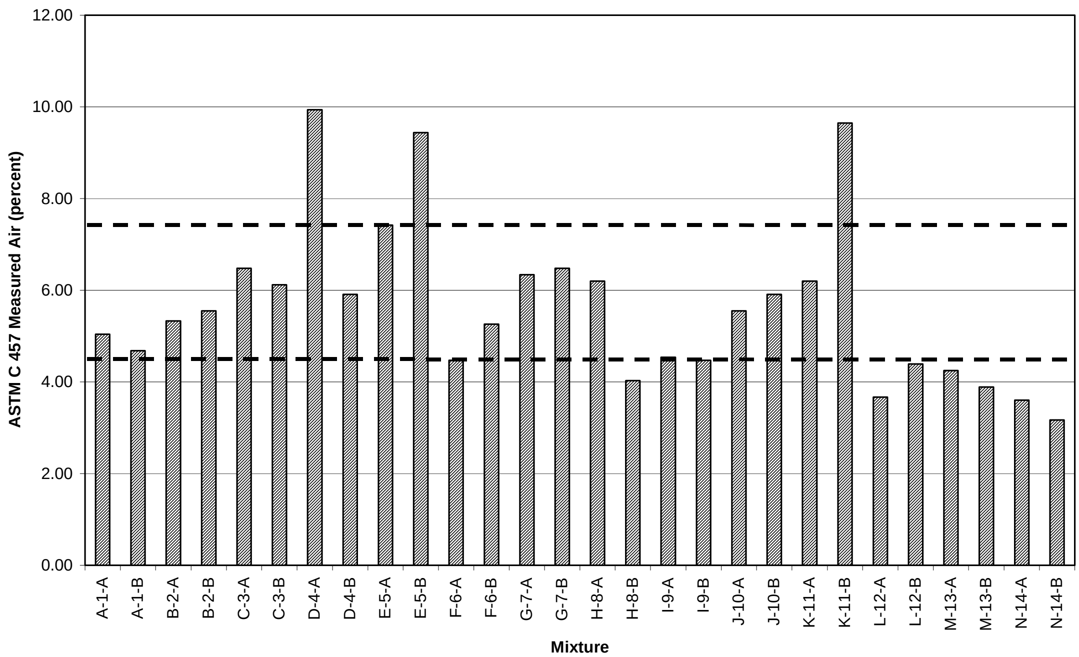
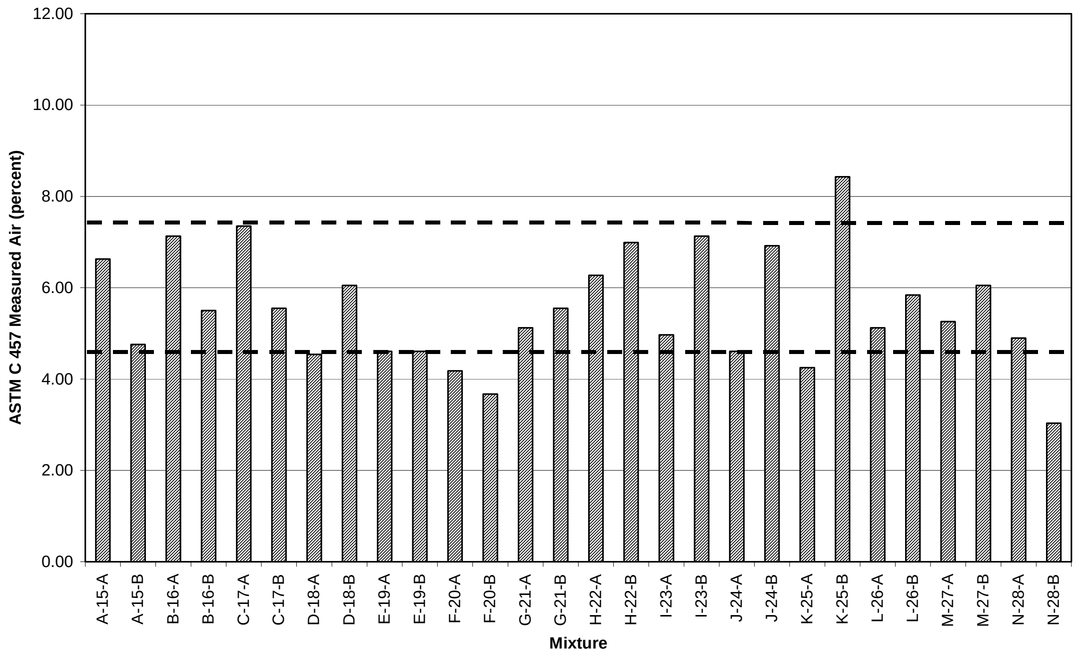


Figure 13. Air content measured using ASTM C 457 for 20- to 24-hour mixtures.



In general, the total air content of a mixture is not thought to be nearly as important in protecting the paste against freeze-thaw damage as the size and spacing of the air bubbles. The most common parameter used to assess the spacing of air bubbles is the spacing factor. The average spacing factors are 0.1591 mm (0.006263 in.) and 0.1273 mm (0.005010 in.) for the 6- to 8-hour and 20- to 24-hour mixtures, respectively. Individual values are shown in Figures 14 and 15. Although both average values are below the criterion set for protecting the paste against damage (0.200 mm [0.008 in.], which is shown as a dashed line in the figures), the value for the 6- to 8-hour mixtures is significantly higher than that for the 20- to 24-hour mixtures. This point is illustrated in Figure 16. Further, it can be seen in Figures 14 and 15 that all of the spacing factors exceeding the limiting criterion are in mixtures made with a Type III cement, even though some batches made with Type III cement had acceptable spacing factors (K-11-B, M-27-

A, and M-27-B). These findings further illustrate that the combination of fine cement and multiple admixtures made it difficult to create the desired air-void system. Interestingly, most (although not all) of these mixtures had acceptable fresh air contents, suggesting the need to better assess the air-void system for such mixtures.

After stereo optical microscope evaluation, thin sections were made to assess the microstructural characteristics of the concrete. Using the petrographic microscope, the paste homogeneity and microcracking were assessed on a scale of 1 to 3 for each batch. It was observed that the 6- to 8-hour mixtures had higher degrees of inhomogeneity and microcracking than the 20- to 24-hour mixtures, indicating variation in the density of the hydrated cement paste because of difficulties in uniformly dispersing the cement grains. The highest degree of inhomogeneity was found in mixtures made with Type III cement, although some batches made with Type III cement (e.g., N-14-A, M-27-A, and M-27-B) had good paste uniformity. In comparing 6- to 8-hour mixtures made with Type I cement, the effect of the Type F HRWR is observed in that mixture G-7 had greater paste uniformity than B-2. Similar benefits were not noted with the Type E water reducer, where moderate inhomogeneity was observed both in the 6- to 8-hour mixture (H-8) and in the 20- to 24-hour mixture (H-22).

The degree of microcracking was considerably higher in the 6- to 8-hour mixtures than in the 20- to 24-hour mixtures, although by no means was it absent in the latter. A small degree of microcracking is a common feature in concrete and thus is not of great consequence. A rating of “1” indicated an absence of microcracking. Moderate microcracking (i.e., rating of “2”), although not desirable, does not indicate a problem with the mixture and should not be assumed to have a great impact on the durability of the concrete. But the severe microcracking (i.e, rating of “3”) noted in many of the mixtures is out of the ordinary and may indicate potential problems, especially in the higher early strength materials.

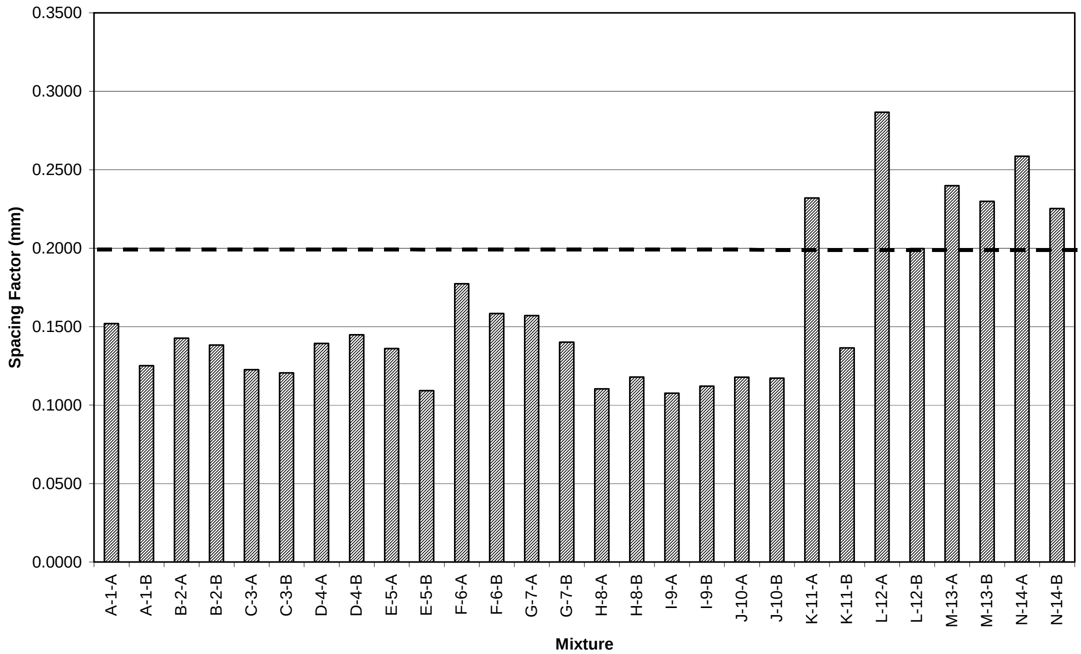


Figure 14. Spacing factors measured using ASTM C 457 for 6- to 8-hour mixtures.

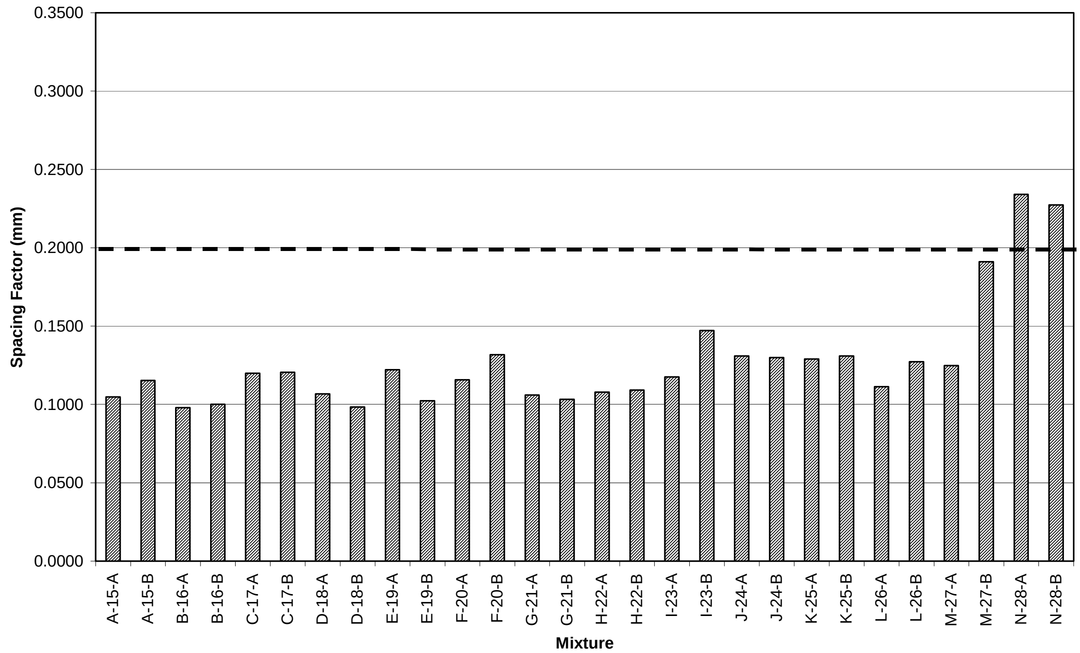


Figure 15. Spacing factors measured using ASTM C 457 for 20- to 24-hour mixtures.

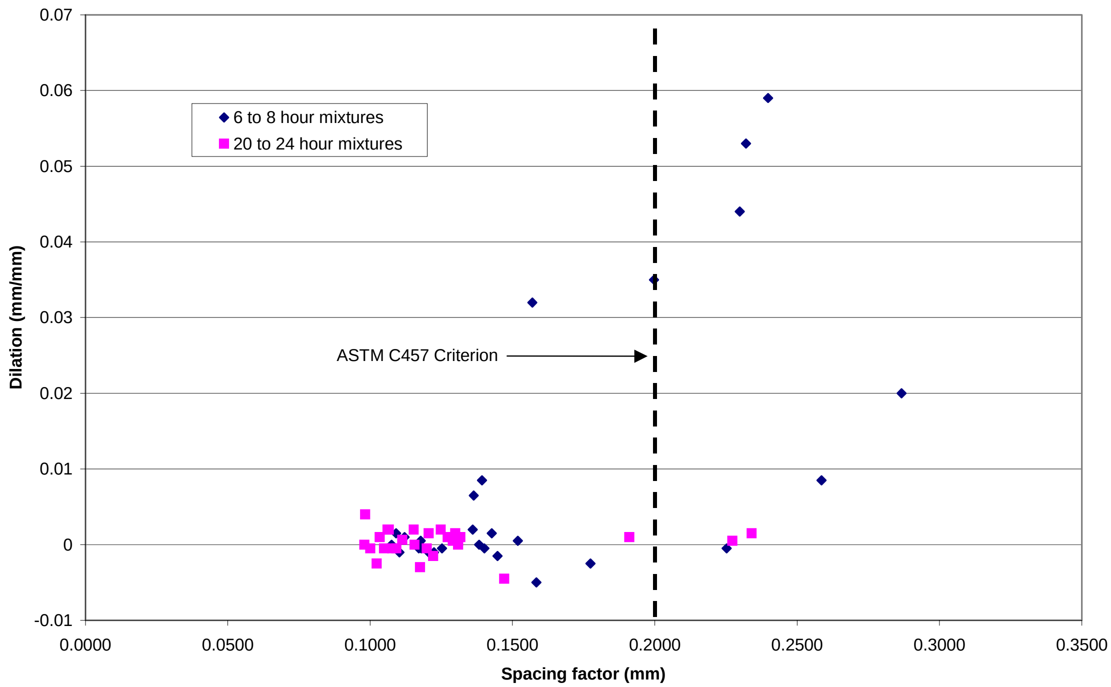


Figure 16. Relationship between spacing factor and dilation due to freezing and thawing.

These same thin sections were evaluated using the scanning electron microscope (SEM) in backscatter electron mode. As was seen using the stereo optical microscope, very different air-void systems were observed in the mixtures prepared using a Type I cement from those made with a Type III cement, with air bubbles being far more abundant in the Type I cement mixtures. This was true for both the 6- to 8-hour and 20- to 24-hour EOT concrete. In addition, in the 6- to 8-hour EOT concrete, the hydrated cement paste was more uniform in the Type III cement mixtures, primarily a result of the fine cement, which left fewer unhydrated cement grains. The SEM images also revealed that the hydrated cement paste in mixtures made with the non-chloride accelerator was distinctly more uniform than that in the mixtures made with calcium chloride. In the 20- to 24-hour EOT concrete, it was also observed that the hydrated cement paste in non-chloride accelerator mixtures was more uniform than that produced when no accelerator was used (without the use of accelerators, patches of higher porosity were observed). Similar observations were not made using the petrographic optical microscope because of a limitation in resolution. Only the 20- to 24-hour mixtures with the Type E water reducer showed a more uniform paste than those with Type A water reducer or those containing no water reducer at all. No other differences in the mixtures were readily observed.

The x-ray microscope was used to assess the degree of chloride ion ingress into the specimens that had undergone exposure to deicers in accordance with ASTM C 672. From the data collected, an effective diffusion coefficient and the area under the chloride ion profile were computed. Overall, the determination of the effective chloride ion diffusion coefficient was straightforward except when a calcium chloride accelerator was used in the mixture. Although steps were made to correct for this, in one case (batch N-14-B), the background chloride concentration was not zero, significantly affecting the computed coefficient. The diffusion coefficients varied widely between mixtures, and although the average effective diffusion coefficient for the 6- to 8-hour mixtures was less than for the 20- to 24-hour mixtures, the variability made it impossible to assess this difference. The area under the chloride profile also shows a great amount of variability, yet a statistically significant difference does exists, with less area observed for the 6- to 8-hour mixtures than for the 20- to 24-hour mixtures. This result would be expected because of the lower permeability of the lower–*w/c*-ratio, higher–cement-content, faster-setting mixtures. Figure 17 shows the effect of *w/c* ratio on the area under the chloride profile for all mixtures.

Another microstructural observation was the estimation of crack length per unit area of concrete subjected to freeze-thaw testing (AASHTO T 161). It is observed that significantly more microcracking was present in the 6- to 8-hour mixtures than in the 20- to 24-hour mixtures. This observation is similar to that observed using petrographic means. Microracking correlated in an imprecise way to the measured dilation values, but, as shown in Figure 18, more cracking is measured in higher–cement factor mixtures. This trend occurs because of the paste that is present in such mixtures or the high susceptiblity to microcracking of the high–cement-content mixtures.

Absorption/Porosity

The apparent density of the concrete varied slightly, with coefficients of variation of 1.52 and 1.97 percent for the 6- to 8-hour and 20- to 24-hour EOT concrete mixtures, respectively. As would be expected, mixtures with the highest apparent density (D-4, E-5, C-17, and K-25) were made using the siliceous coarse aggregate, which had the highest specific gravity (2.91) of the coarse aggregates used. The percent permeable void space varied from mix to mix, with no discernible difference between the 6- to 8-hour and the 20- to 24-hour EOT concrete. As shown in Figure 19, a clear trend exists between the cement factor and the percent permeable void space, with increasing cement content (and thus greater overall porosity) resulting in a greater percentage of permeable void space.

Sorptivity measurement plots are characterized by an initial phase with a relatively steep slope and a later phase with a much less steep slope, reflecting the continued uptake of water. The sorptivity test can be run with water ponded on the surface (top) or with the bottom of the sample barely suspended in water (bottom). The latter procedure provides more repeatable results. As with the percent permeable void spaces, sorptivity is related to the cement factor in 20- to 24-hour mixtures but not clearly for the 6- to 8-hour mixtures.

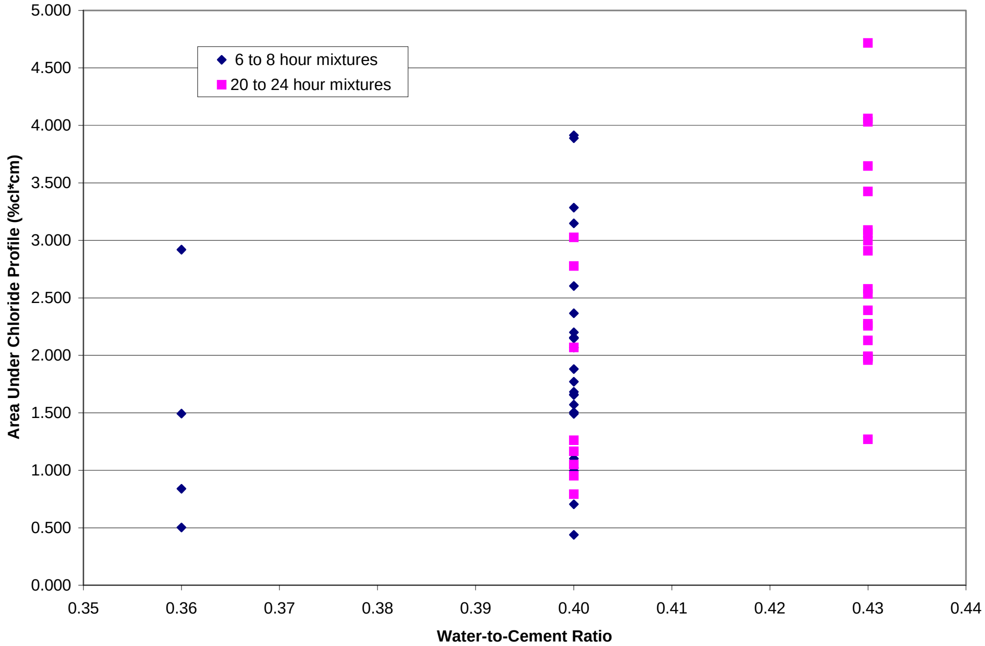


Figure 17. w/c ratio versus area under chloride profile for Type I cement.

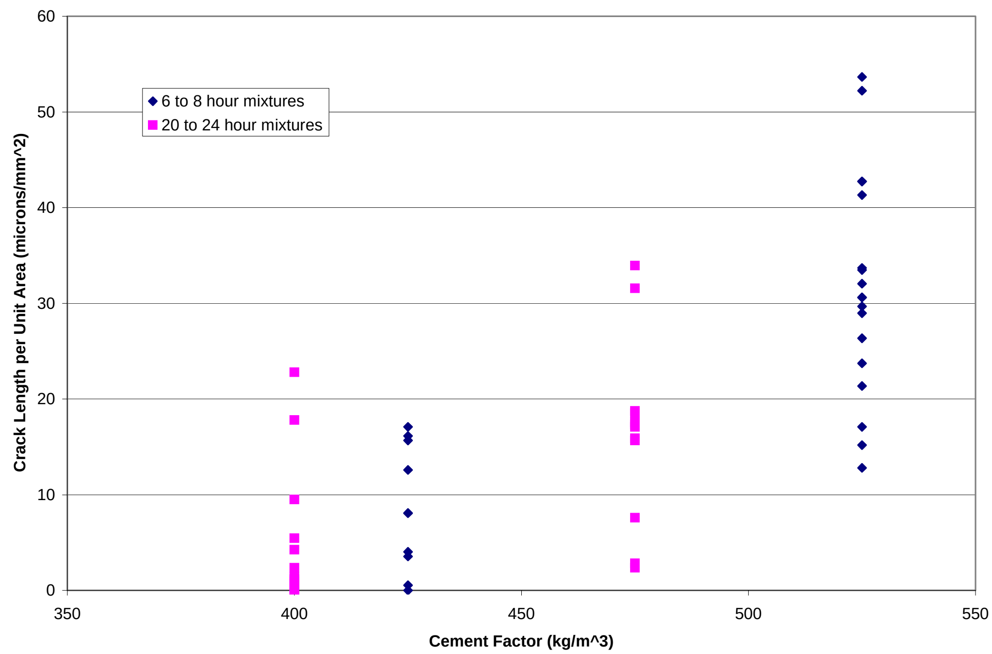


Figure 18. Cement factor versus crack length per unit area for all mixtures.

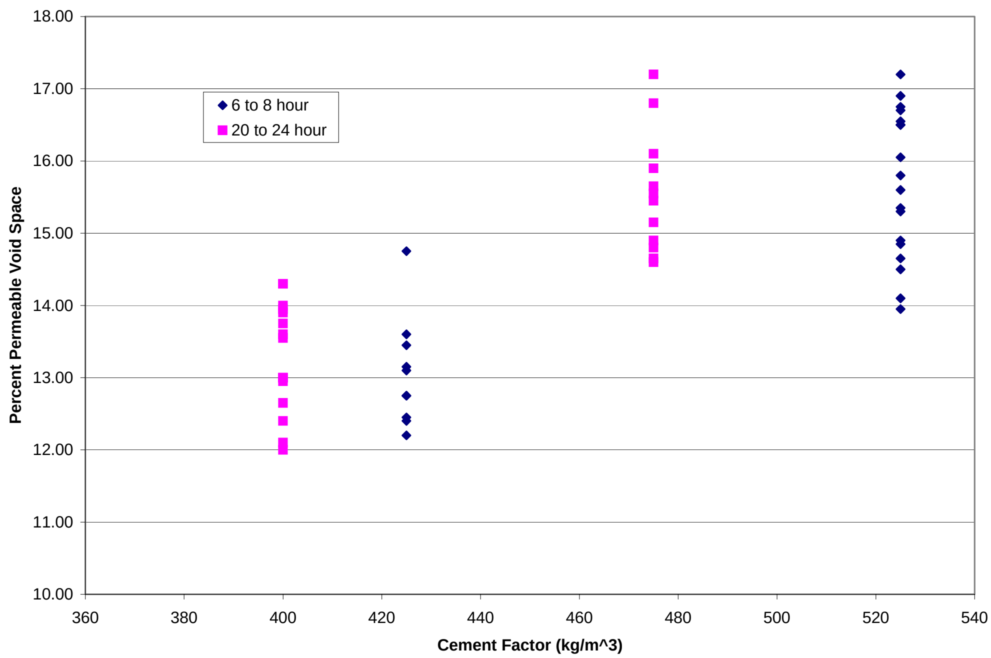


Figure 19. Cement factor versus percent permeable void space for all mixtures.

3.2.6 Summary of Laboratory Study Test Results

The following observations can be made from the information collected in the laboratory study:

- Because of the nature of this study, the slump of the mixtures was allowed to vary in order to maintain other experimental mixture design parameters. Type F HRWR was used with all mixtures made with the Type III cement; otherwise, mixtures were unworkable.
- Overall, the laboratory-measured air contents of the fresh concrete fell within the desired range, with a few exceptions in the 6- to 8-hour mixtures. Of the eight 6- to 8-hour Type III mixtures, three had fresh air contents lower than desired.
- CTE results were highly repeatable, with only a couple of exceptions. The variability was slightly higher for the 6- to 8-hour mixtures than for the 20- to 24-hour mixtures.
- Results from the restrained shrinkage ring test were highly variable, but a statistical difference in the time to first cracking was observed, with 6- to 8-hour mixtures cracking earlier than 20- to 24-hour mixtures. The curing temperature had a large impact on shrinkage cracking, with increased curing temperatures producing far less tendency to crack. In general, the repeatability of the shrinkage ring test as used in this study was not very good between batches or even between replicates from the same batch.
- The 6- to 8-hour mixtures statistically performed more poorly in freeze-thaw testing than did the 20- to 24-hour mixtures. None of the 20- to 24-hour mixtures performed poorly in freeze-thaw testing, whereas a number of the 6- to 8-hour mixtures had unacceptably high dilation values. Many of the poorly performing mixtures were those made with Type III cement and

Type F HRWR mixtures that had low air contents. There appears to be some type of indefinable interaction between the various constituents—including the cement type and content, water reducer, accelerator, and air-entraining admixture—that have influenced the freeze-thaw performance of some mixtures.

- The scaling results were variable with the degree of scaling being significantly higher in the 6- to 8-hour mixtures than in the 20- to 24-hour mixtures. The same Type III cement, Type F HRWR mixtures that had poor dilations also exhibited high degrees of deicer scaling, but there was little batch-to-batch correlation between the two. These results illustrate the importance of using multiple batches and replicates for durability testing.
- Strength relationships followed expected trends, with the 6- to 8-hour mixtures having higher 24-hour strengths than the 20- to 24-hour mixtures. The majority of the 6- to 8-hour mixtures did not meet the compressive and flexural strength criterion at 6 hours, although most gained sufficient strength by 8 hours. Almost all of the 20- to 24-hour mixtures met the strength criterion by 20 hours. The 6- to 8-hour EOT concrete mixtures with the lowest early strength were those made with Type I cement, Type E water reducer, and Type F HRWR. In particular, mixtures with the Type E water reducer had extremely low strengths both at 6 and 8 hours. The relationship between compressive strength and flexural strength was variable, indicating that this relationship should be determined on a mix-by-mix basis.
- Most mixtures had hardened air contents (as measured by ASTM C 457) that fell within the desired range, although there were some exceptions. Three of the four 6- to 8-hour mixtures (six of the eight batches) made with Type III cement and Type F HRWR and two of the 20- to 24-hour batches had insufficient air contents. Air content was difficult to control in

mixtures made with high cement contents, low *w/c* ratios, and multiple admixtures, especially when Type III cement was used with a Type F HRWR.

- The ASTM C 457 spacing factors were generally considered adequate to protect the paste against freeze-thaw damage, except for some of the mixtures made with Type III cement and Type F HRWR. Insufficient spacing factors were measured for some of the mixtures that had satisfactory air contents, as measured on the fresh concrete. It was also observed that the spacing factor criterion appears to be adequate to prevent freeze-thaw damage, as excessive dilation occurred in only a single batch meeting the spacing factor criterion.
- Paste homogeneity, as assessed using the relatively low magnification of the petrographic optical microscope, was considerably better in the 20- to 24-hour mixtures than in the 6- to 8-hour mixtures, indicating a better-blended, more uniform cement paste. Mixtures made with Type III cement were slightly less homogenous than those made with Type I cement, although better cement grain dispersion was observed when the Type F HRWR was used with Type I cement.
- Severe microcracking of the paste was observed in all of the 6- to 8-hour mixtures, but less severe microcracking was observed in the 20- to 24-hour mixtures, indicating a lower stress in the paste.
- A difference in the air-void system was observed in backscatter electron images from the SEM, with a far greater abundance of air in mixtures made with Type I cement than with Type III cement for both the 6- to 8-hour and 20- to 24-hour mixtures. Under the higher magnification of the SEM (1000x), the hydrated cement paste in the Type III mixtures appeared more uniform than that in the Type I mixtures because of more complete hydration

of the cement grains. The paste was also more uniform in mixtures made with the non-chloride accelerator than in mixtures made with calcium chloride or without any accelerator.

- Based on analysis using the x-ray microscope, there was significantly less overall penetration of chloride ions into the 6- to 8-hour mixtures than into the 20- to 24-hour mixtures, resulting from the reduced *w/c* ratio.
- The crack length per unit area measurements indicated that significantly more microcracking was present after freeze-thaw testing in the 6- to 8-hour mixtures than in the 20- to 24-hour mixtures. In general, the higher the cement content, the more microcracking was observed. It is unclear whether this observation means that the paste has more microcracking in the high-cement mixtures or whether the measurement simply reflects that there is more paste in the high-cement mixtures.
- The cement factor appeared to have the largest impact on the percent volume of permeable voids measured in ASTM C 642, with void volume increasing with increasing cement content. Sorptivity results were more variable and did not provide as clear a trend as the results from ASTM C 642. More repeatable results were obtained when the bottom of the specimen was barely immersed in water as opposed to water being ponded on the surface.

3.3 STATISTICAL ANALYSIS OF TEST RESULTS

This section presents a statistical analysis of the data. The data set and the output of the statistical analysis are provided in Appendix D. The results of the multiple single-factor experiments are presented to assess the significance of the various independent variables. Results of the statistical analyses were used to draw generalized conclusions on how mixture design parameters affect the properties of fresh concrete, strength gain, and potential durability as assessed through the test methods used in the experiment. Based on an initial analysis of the test results, results from the 47 individual tests presented in Table 13 were analyzed for the EOT concrete mixtures. A summary of the statistical analysis and the derived conclusions are presented in this section.

3.3.1 Six- to 8-hour EOT Concrete

Forty-two of the 47 tests (tests 1–5, 8–22, 24–27, and 30–47) listed in Table 13 were included in the analysis of the 6- to 8-hour EOT concrete. Both single-factor and two-factor experiments were analyzed.

Single-Factor Effects

Table 14 lists the mixture pairs used to study the single-factor effect of each mixture variable (cement type, cement factor, etc.). For example, the effect of cement type was studied using mixture pairs A-1 and B-2, I-9 and J-10, and K-11 and L-12. Each pair has similar composition and curing regime, differing only in the factor under study. For example, mixtures A-1 and B-2 both use Type I cement with no water reducer, while mixtures K-11 and L-12 both use Type III cement with a Type F HRWR.

TABLE 13 Tests used in the statistical analysis

01 – CTE	25 – Compressive Strength (28 Day)
02 – Shrinkage Ring Test (Days to 1st Crack)	26 – Flexural Strength (6 Hour)
03 – Shrinkage Ring Test (Initial Slope of 1st Crack)	27 – Flexural Strength (8 Hour)
04 – Shrinkage Ring Test (Days to 2nd Crack)	28 – Flexural Strength (20 Hour)
05 – Shrinkage Ring Test (Initial Slope of 2nd Crack)	29 – Flexural Strength (24 Hour)
06 – Shrinkage Ring Test (Days to 3rd Crack)	30 – Density (% Absorption after Immersion)
07 – Shrinkage Ring Test (Initial Slope of 3rd Crack)	31 – Density (% Absorption after Immersion and Boiling)
08 – Sorptivity Testing - Top of Sample (Initial Slope)	32 – Density (Bulk Dry Density)
09 – Sorptivity Testing - Top of Sample (Final Slope)	33 – Density (Bulk Density after Immersion)
10 – Sorptivity Testing - Bottom of Sample (Initial Slope)	34 – Density (Bulk Density after Immersion and Boiling)
11 – Sorptivity Testing - Bottom of Sample (Final Slope)	35 – Density (Apparent Density)
12 – ASTM C 672: Scaling Test Rating	36 – Density (Volume of Permeable Pore Spaces)
13 – ASTM C 457: Point Count; Air Volume %	37 – Air Measured by Field Method
14 – ASTM C 457: Point Count; Air Void Specific Surface	38 – Shrinkage Ring Test - Total Number of Cracks
15 – ASTM C 457: Point Count; Paste-to-air Ratio	39 – Initial Concentration (wt % Cl) at Surface
16 – ASTM C 457: Point Count; Air Void Spacing Factor	40 – Effective Diffusion Coefficient
17 – Maturity (8-Hour Average)	41 – R-Squared Value between Fick's Law and Data
18 – Maturity (24-Hour Average)	42 – Area under Curve (as Measure of Absorbed Cl)
19 – Freeze-Thaw Dilation	43 – Specific Gravity from Unit Weight Bucket
20 – Slump	44 – Specific Gravity from Point Count Data
21 – Compressive Strength (6 Hour)	45 – Homogeniety Rating
22 – Compressive Strength (8 Hour)	46 – Microcracking Rating
23 – Compressive Strength (20 Hour)	47 – Crack Length per Unit Area
24 – Compressive Strength (24 Hour)	

For each test conducted, the mean, variance, and 95-percent confidence intervals for each factor was calculated. The results for the 42 tests are presented in Appendix D. To determine if a mixture variable affects a certain test result, p-values were calculated. A p-value of less than 0.05 indicates that the mixture variable has a significant impact on that test result. The p-values for the tests on the 6- to 8-hour mixes are also presented in Appendix D. The significant test results for each of the seven mixture variables are summarized in the following text.

Cement Type. The statistical analysis showed that varying the cement type from Type I to Type III cement increased scaling (test 12), maturity (tests 17 and 18), and some of the early-age compressive and flexural strengths (tests 22, 24, and 27), but reduced air content that was determined from ASTM C 457 (test 13) tests. It is noted that only limited direct comparison between Type I and III cements could be conducted, since all mixtures made with Type III cement also contained Type F HRWR in order to achieve satisfactory consistency during mixing. This one comparison shows that although early strength was enhanced through the use of the Type III cement Type F HRWR, the use also resulted in increased scaling and a decrease in the total air content.

TABLE 14 Mixture pairs used in the 6- to 8-hour EOT concrete analysis

Factor	Mixes Used	Cement Type	Cement Factor kg/m^3 (lb/yd^3)	*w/c* Ratio	Coarse Aggregate Type	Accelerator Type	Water Reducer Type	Curing Temperature °C (°F)
Cement Type	G-7 and L-12	I/III	525 (885)	0.40	Carbonate	NC	F	23 (73)
Cement Factor	A-1 and B-2 I-9 and J-10 K-11 and L-12	I I III	425/525 (716/885)	0.40	Carbonate	NC CC NC	No No F	23 (73)
w/c ratio	B-2 and F-6	I	525 (885)	0.40/ 0.36	Carbonate	NC	No	23 (73)
Coarse Aggregate Type	A-1 and E-5 C-3 and D-4	I	425 (716) 525 (885)	0.40	Carbonate/ Siliceous	NC	No	23 (73) 65 (150)
Accelerator Type	A-1 and I-9 B-2 and J-10 K-11 and N-14	I I III	425 (716) 525 (885) 425 (716)	0.40	Carbonate	NC/CC	No No F	23 (73)
Water Reducer Type	B-2 and G-7	I	525 (885)	0.40	Carbonate	NC	No/F	23 (73)
Curing Temperature	B-2 and C-3 L-12 and M-13	I III	525 (885)	0.40	Carbonate	NC	No F	23/65 (73/150)

Cement Factor. In general, it appears that increasing the cement factor increased absorption, volume of permeable void space, and shrinkage potential, while not significantly improving strength (in some cases, strength was detrimentally affected). The only durability measure that was significantly improved through the use of higher cement contents was scaling.

w/c Ratio. Reducing the *w/c* ratio from 0.40 to 0.36 resulted in a significant increase in the CTE (test 1), various measures of strength (tests 22, 24, 25, 26, and 27), and density (tests 33 and 34), while reducing absorption (tests 30 and 31) and paste inhomogeniety (test 45). There was no significant adverse effect observed in reducing the *w/c* ratio.

Coarse Aggregate Type. Coarse aggregate type had a big effect on density, with the mixtures made with siliceous aggregate having higher concrete density than mixtures made with the carbonate aggregate. Yet the CTE was not significant, although the average was higher for the mixtures made with the siliceous coarse aggregate. The use of the siliceous aggregate produced mixtures that had slightly better durability, as measured by a lower effective chloride diffusion coefficient and decreased crack length per unit area after freeze-thaw testing.

Accelerator Type. In general, the non-chloride accelerator was very effective (at times more effective than calcium chloride) at increasing early strength, but long-term strength was either similar to or less than that achieved with calcium chloride. Other parameters were not notably affected by the change in accelerator.

Water Reducer Type. Using a Type F water reducer versus not using a water-reducing admixture decreased the air content as measured by ASTM C 457 (test 13), the air-void specific surface (test 14), the maturity (tests 17 and 18), and 6- to 8-hour strength (tests 21, 22, 26, and 27). It also increased the paste-to-air ratio (test 15). The mixture made without water-reducing admixture had a better-entrained air-void system, higher maturity, and significantly higher early strength values than the mixture made with the Type F HRWR. One of the four batches made with the Type F HRWR (G-7-A) had very high dilation values in cyclic freeze-thaw testing as a result of the poor air-void system. However, mixtures made with the Type F HRWR had better paste homogeneity (test 45).

Curing Temperature. Raising the curing temperature from 23^{o}C (73^{o}F) to 65^{o}C (150^{o}F) led to less shrinkage cracking, as assessed through the restrained shrinkage ring test. It was also observed that the air-void system parameters and freeze-thaw durability were improved by high-temperature curing, although the long-term compressive strength was reduced for the mixtures made with Type I cement.

Two-Factor Analysis

There was only one two-factor analysis possible with the data collected in the 6- to 8-hour experiment because of the need to use the Type F HRWR with the Type III cement mixtures. The mixtures used in the two-factor analysis are A-1, B-2, I-9, and J-10. These mixtures allowed for the interaction between cement factor and accelerator type.

It was found, in general, that the cement factor of a mixture is a more important factor then accelerator type. The only meaningful two-way interaction that existed was for the apparent density (test 35). In this test, both of the two factors are significant and the p-value of the interaction term is 0.0048. For mixtures with a cement factor of 425 kg/m^3 (716 lb/yd^3), a mix without accelerator has a higher apparent density, but for mixtures with a cement factor of 525 kg/m^3 (885 lb/yd^3), the mixture containing the non-chloride accelerator has a higher apparent density.

Relationships Among Tests

The relationships among the 47 individual tests were studied to see if the results from some of the complex, costly tests could be obtained from simpler, less costly tests. Details regarding this analysis are provided in Appendix D.

As expected, a number of strong correlations existed for different measurements made within the same test (e.g., the various measures of density and absorption reported under ASTM

C642 and the two reported maturity values made at different times) and between tests measuring similar properties (such as specific gravity measurements and density measurements and compressive and flexural strength at similar ages).

Very few correlations were observed between simple, easy-to-run tests and more complex, expensive tests. For example, no correlations exceeding 0.50 were observed for sorptivity (tests 8–11) to some of the more complex durability tests, such as scaling (test 12) or freeze-thaw dilation (test 19). Only one sorptivity test (test 10) had correlations exceeding 0.50 with any other important test, having some correlation with some elements (tests 30, 32, 33, and 34) of density (ASTM C 642) and specific gravity (test 43), as well as for the measure of cracking per unit area (test 47). It was also observed that various measurements collected for ASTM C 642 (tests 30, 32, 33, 34, and 36) correlated well with the crack length per unit area (test 47).

Dilation due to freezing and thawing correlated mildly with the spacing factor, as is shown in Figure 20. It appears the maximum spacing factor criterion of 0.200 mm (0.008 in.) is acceptable to minimize damage due to freezing and thawing. There is some correlation between the air content measured on fresh concrete and that measured using ASTM C 457 (see Figure 21). A similar trend is observed with the spacing factor, as shown in Figure 22. Although, in some instances, the air content was "acceptable," the spacing factor was not achieved.

The only correlation related to paste homogeneity (test 45) is with freeze-thaw dilation, whereas decreasing paste homogeneity resulted in increased dilation (see Figure 23).

3.3.2 Twenty- to 24-hour EOT Concrete

Forty-one of the 47 tests (tests 1–5, 8–20, 23–25, and 28–47) listed in Table 13 were included in the analysis of the 20- to 24-hour EOT concrete mixtures. Both single-factor and two-factor experiments were analyzed.

Single-Factor Effects

Table 15 lists the mixture pairs used to study the single-factor effect of each mixture variables (cement type, cement factor, etc.). For each test conducted, the mean, variance, and 95-percent confidence intervals for each factor were calculated. The p-value was calculated to determine if a mixture variable affects a certain test result, with a p-value of less than 0.05 indicating that the mixture variable has a significant impact on that test result. Details of the statistical analysis are presented in Appendix D. A summary follows.

Cement Type. It was found that changing the cement type from Type I to Type III increased the compressive strength (tests 23–25) regardless of the accelerator type used. When a non-chloride accelerator was used, changing the cement type from Type I to Type III increased the time to first cracking (test 2) and 24-hour flexural strength (test 29) and decreased microcracking (test 46). When calcium chloride accelerator was used, the air-void system (tests 14 and 16) was negatively affected by changing cement type from Type I to Type III. Changing to Type III also decreased maturity (tests 17 and 18) and absorption and percent permeable voids (tests 30, 31, and 36).

Cement Factor. Varying the cement factor from 400 kg/m^3 (674 lb/yd^3) to 475 kg/m^3 (800 lb/yd^3) increased the percent absorption (test 31), total number of shrinkage cracks (test 38), microcracking rating (test 46), and crack length per unit area (test 47). Further, the mixture with the lower cement factor had a higher bulk density (test 32).

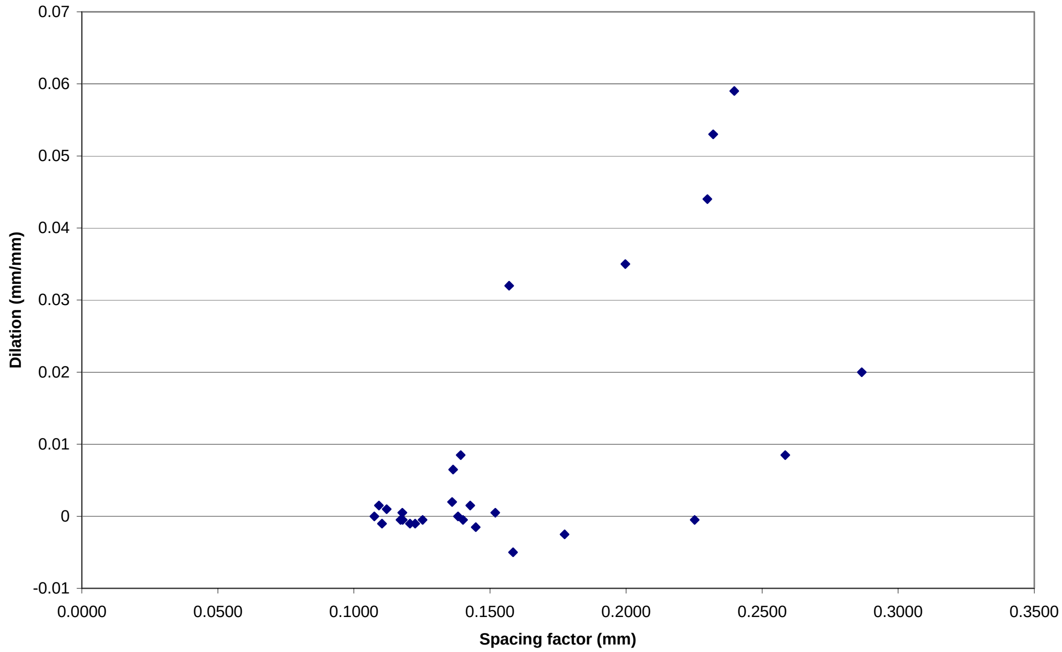


Figure 20. Spacing factor (test 16) versus freeze-thaw dilation (test 19).

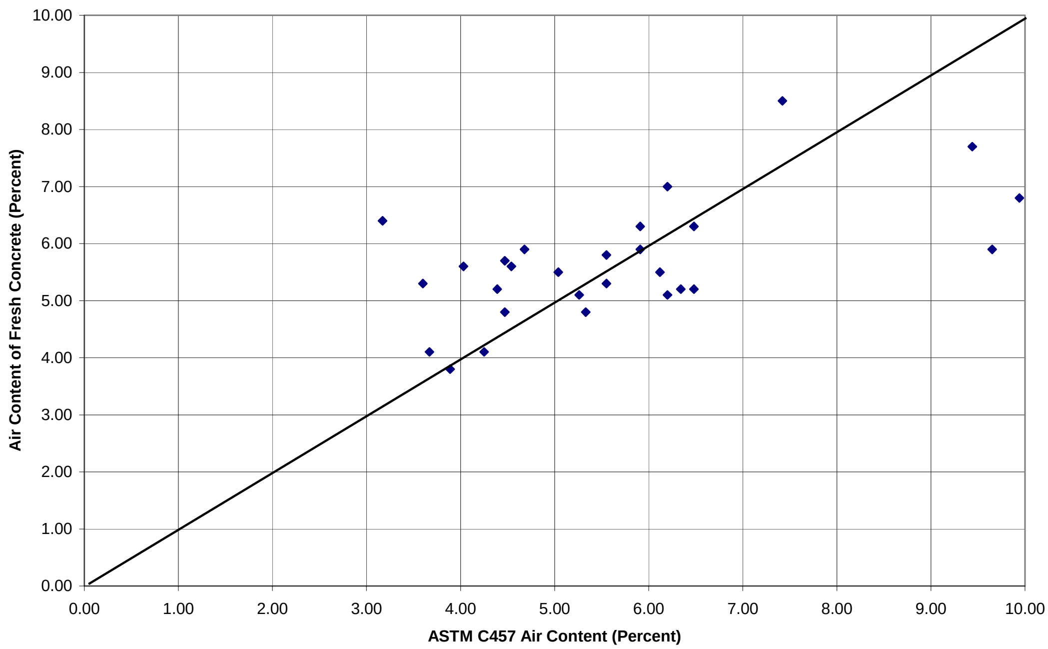


Figure 21. Air content measured using ASTM C 457 (test 13) versus air content of fresh concrete (test 37).

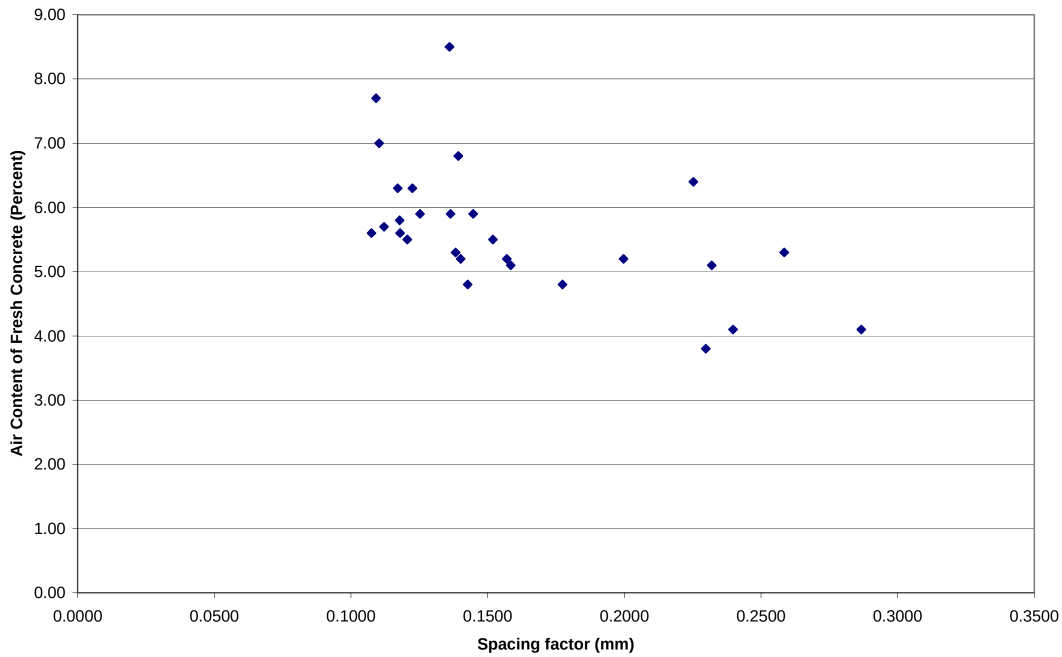


Figure 22. Spacing factor (test 16) versus air content of fresh concrete (test 37).

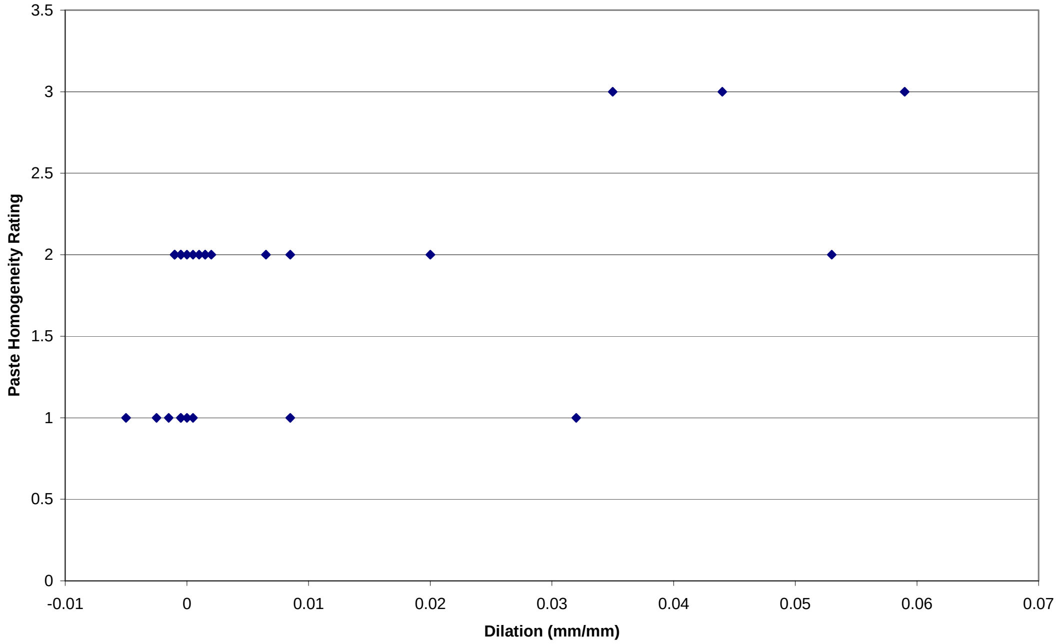


Figure 23. Paste homogeneity rating (test 45) versus freeze-thaw dilation (test 19).

TABLE 15 Mixture pairs used in 20- to 24-hour EOT concrete analysis

Factor	Mixes Used	Cement Type	Cement Factor kg/m^3 (lb/yd^3)	*w/c* Ratio	Coarse Aggregate Type	Accelerator Type	Water Reducer Type
Cement Type	E-19 and M-27 F-20 and N-28	I/III	400 (674)	0.40	Carbonate	NC CC	No
Cement Factor	A-15 and G-21	I	400/475 (674/800)	0.43	Carbonate	NC	No
w/c Ratio	A-15 and E-19 B-16 and F-20	I	400 (674)	0.43/0.40	Carbonate	NC CC	No
Coarse Aggregate Type	A-15 and C-17 A-15 and D-18 C-17 and D-18 J-24 and K-25 J-24 and L-26 K-25 and L-26	I	400 (674) 400 (674) 400 (674) 475 (800) 475 (800) 475 (800)	0.43	Carb/Silic Carb/Grav Silic/Grav Carb/Silic Carb/Grav Silic/Grav	NC NC NC No No No	No
Accelerator Type	A-15 and B-16 E-19 and F-20 G-21 and J-24 M-27 and N-28	I I I III	400 (674) 400 (674) 475 (800) 400 (674)	0.43 0.40 0.43 0.40	Carbonate	NC/CC NC/CC NC/No NC/CC	No
Water Reducer Type	H-22 and I-23 I-23 and J-24	I	475 (800)	0.43	Carbonate	No	E/A A/No

w/c Ratio. In the case where a non-chloride–based accelerator was used, it was found that decreasing the *w/c* ratio from 0.43 to 0.40 increased the 28-day compressive strength (test 25) and 24-hour flexural strength (test 29). Similarly, the 20-hour and 24-hour compressive strength (tests 23 and 24) increased when the *w/c* ratio was decreased from 0.43 to 0.40 in. mixtures using calcium chloride accelerator. Also, the CTE (test 1) and maturity (test 18) both increased with a decrease in the *w/c* ratio.

Coarse Aggregate Type. Aggregate type had a significant impact on the measured density (tests 30 through 35), with mixtures made with the carbonate coarse aggregate having the lowest density and those made with the siliceous coarse aggregate having the highest density. The CTE (test 1) was higher for the mixture made with the gravel coarse aggregate. Mixtures with gravel had lower 20-hour compressive strength (test 23) and 24-hour flexural strength (test 29) than the other mixtures. Also, mixtures made with a cement factor of 400 kg/m^3 (674 lb/yd^3)

and siliceous coarse aggregate had significantly higher scaling than the other mixtures. Using a higher cement factor (475 kg/m^3 [800 lb/yd^3]) and no accelerator changed the results slightly, with the gravel mixtures having some minor scaling (rating of 1), the siliceous mixtures having even less scaling (rating of 0.50), and the carbonate mixtures having no scaling.

Accelerator Type. Although many results were influenced by the specific mixture parameters, it was generally observed that the use of calcium chloride increased compressive strength at various ages. In some cases, the use of calcium chloride increased scaling, decreased paste homogeneity, and produced poorer air-void system parameters.

Water Reducer Type. The maturity test results (tests 17 and 18) were highest when no water reducer was used, followed by the mixture with the Type E water reducer. The mixture containing the Type A water reducer had the lowest maturity values. Similarly, the mixture without water reducer also had the highest 20-hour flexural strength (test 28) of the three, followed by the mixture made with the Type E water reducer. The mixture made with Type A water reducer had the lowest 20-hour flexural strength. The mixture made with the Type E water reducer had less paste homogeneity (test 45) and more microcracking (test 46) than either of the other two mixtures. The percent volume of permeable voids was highest with the mixture made with Type E water reducer, followed by the mixture made with the Type A water reducer. The mixture made without water reducer had the lowest volume of permeable voids. The mixture made with the Type A water reducer also had the smallest level of chloride penetration after the salt ponding test (test 42).

Two-Factor Models

A couple of different two-factor analyses were possible for the 20- to 24-hour mixtures. One analysis was conducted using mixtures A-15, B-16, E-19, and F-20, in which the *w/c* ratio (0.40 to 0.43) and the type of accelerator used (non-chloride versus calcium chloride) were varied. Another analysis was conducted using mixtures E-19, F-20, M-27, and N-28, in which cement type (Type I versus Type III) and accelerator type (non-chloride versus calcium chloride) were varied.

In the first analysis, the *w/c* ratio was generally more important than the accelerator type in affecting mixture properties, and meaningful two-way interactions existed for 20- and 24-hour compressive strength (tests 23 and 24) and chloride penetration (test 42). For the compressive strength tests, both *w/c* ratio and accelerator type were highly significant by themselves, and when analyzed together, the results were highly significant as well, with a lower *w/c* ratio and a calcium chloride accelerator resulting in the highest compressive strength values, whereas a high *w/c* ratio with a non-chloride–based accelerator resulted in the lowest compressive strength values. Chloride penetration was also significantly affected by the *w/c* ratio and accelerator type alone or when analyzed together. Chloride penetration was the lowest for mixtures with a low *w/c* ratio and a non-chloride accelerator and highest for mixtures with a high *w/c* ratio and a calcium chloride accelerator.

In the analysis, the type of cement used in a mixture was a more important factor than the accelerator type. A meaningful two-way interaction existed for paste homogeneity (test 45), where both the cement type and accelerator type were extremely significant factors individually and when combined. A combination of Type III cement and calcium chloride accelerator resulted in the lowest paste homogeneity, whereas mixtures made with a non–chloride-based accelerator and/or Type I cement had good paste homogeneity.

Relationships Among Tests

The relationships among the 47 individual tests were studied to see if the results from some of the complex, costly tests could be obtained from simpler, less costly tests. Details regarding this analysis are provided in Appendix D.

Similar to the 6- to 8-hour mixtures, a number of strong correlations existed for different measurements made within the same test (e.g., the various measures of density and absorption reported under ASTM C 642 and the two reported maturity values made at different times) and between tests measuring similar properties (such as specific gravity measurements and density measurements and compressive and flexural strength at similar ages).

The same trends observed in the 6- to 8-hour mixtures were also observed for the 20- to 24-hour mixtures in that very few correlations were found between simple, easy-to-run tests and more complex, expensive tests. For example, only a single sorptivity test (test 10) had some correlations to other tests, mainly measurements of voids made by ASTM C 642 (tests 30 and 36). Various measurements collected for ASTM C 642 had far more correlations with other tests. For example, measurements of density (tests 32–35) correlated with scaling (test 12), but this correlation most likely reflects simply the density of coarse aggregate used and is not indicative of a predictive relationship. As with the 6- to 8-hour mixtures, the volume of permeable voids (test 36) correlated to some measures of strength (tests 30, 32, 33, 34, and 36) as well as to the measure of crack length per unit area (test 47).

No strong correlations existed for dilation due to freezing and thawing for the 20- to 24-hour mixtures, as none of these mixtures suffered significant dilation. There was some correlation between the air content measured on fresh concrete and that measured using ASTM C 457 (see Figure 24). The air content measured in the fresh concrete was commonly higher than that measured using ASTM C 457.

3.3.3 Summary of Statistical Analysis

The statistical analysis provided some insights into both the influence of mixture parameters on mixture behavior and the correlations between various tests used to assess concrete mixtures. Based on this analysis, the following findings were obtained regarding the influence of mixture parameters on concrete test behavior:

- The cement type had a notable impact on various measures of early concrete strength, with Type III cements producing higher strengths at a given age than Type I cements. The increased rate of hydration was evidenced in the maturity readings for the 6- to 8-hour mixtures, which were also significantly higher. In many cases, the change from Type I to Type III cement negatively affected the air-void system parameters, most acutely when a calcium chloride accelerator was used. The resistance to deicing was also compromised in the 6- to 8-hour mixtures made with Type III cement.

- The cement factor had a large effect on measures of paste porosity, increased sorptivity, absorption, and percent volume of permeable void space was observed under many situations as cement content increased. Increases in paste content also increased the number of shrinkage cracks. Increasing the cement content in 6- to 8-hour mixtures decreased scaling, but also led to a reduction in some measures of early strength.
- As expected, decreasing the *w/c* ratio increased various measures of strength. It also increased the CTE. In some cases, absorption was reduced and paste homogeneity improved when the *w/c* ratio was lowered.

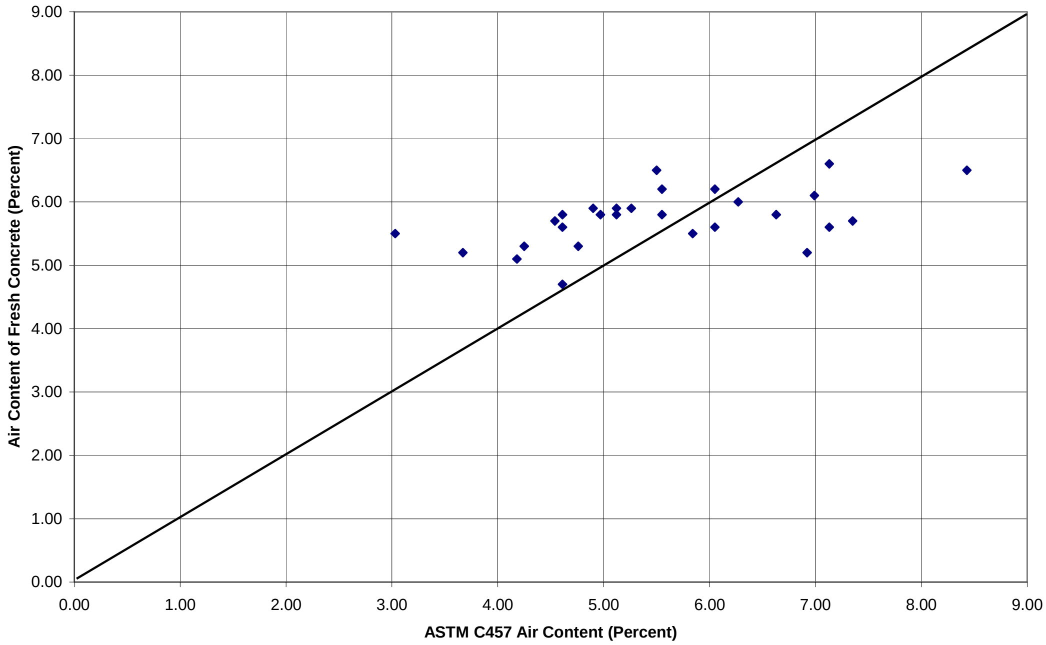


Figure 24. Air content measured using ASTM C 457 (test 13) versus air content of fresh concrete for 20- to 24-hour mixtures (test 37).

- As expected, the coarse aggregate type had a very significant impact on measures of density, with denser aggregate resulting in denser concrete. Mixtures made with the gravel coarse aggregate had the highest CTE, and mixtures made with the carbonate aggregate had the least scaling. Aggregate type also affected some of the strength properties of the mixtures, most notably compressive strength.
- In some instances, mixtures made with a calcium chloride accelerator as opposed to a non-chloride accelerator had lower early strengths, but increased long-term strengths. In some cases, using a calcium chloride accelerator significantly increased the CTE, although the increase was small. Results on the scaling test were variable. With the use of calcium chloride, the scaling resistance was improved in some cases but deteriorated in others.

Similar conflicting results were observed for some of the air-void system parameters and paste homogeneity.

- The use of the Type F HRWR in the 6- to 8-hour mixtures reduced the air content and negatively impacted a number of the air-void system parameters, as well as the early strength characteristics of the concrete. Further, a number of the specimens made with the Type F HRWR had high dilation values in cyclic freeze-thaw testing. The use of a Type F HRWR appeared to improve paste homogeneity. The 20- to 24-hour mixtures made without water reducer had the highest maturity values, followed by those made with a Type E water reducer. Mixtures made with a Type A water reducer had the lowest maturity values. Similar results were obtained for the 20-hour flexural strength test results. Mixtures made with the Type E water reducer had the lowest level of paste homogeneity and the highest volume of permeable voids.
- Raising the curing temperature had many unexpected positive results, including reduced shrinkage, improved air-void system parameters, reduced dilation due to cyclic freezing and thawing, and reduced scaling. Although some early strength measures were increased, 28-day strength was decreased in some cases.
- In the limited study of interactions between mixture parameters, it was found that the *w/c* ratio, cement factor, and cement type were more important parameters than the type of accelerator used. In one case, a strong interaction between cement type and accelerator type was observed, where mixtures made with Type III cement and calcium chloride accelerator produced a less homogenous paste than that observed in mixtures made with Type I cement and a non-chloride accelerator.

Based on the statistical analysis, the following findings were obtained regarding the relationships between the various concrete tests:

- As would be expected, certain tests are highly correlated one with another, such as the various measures of compressive strength, flexural strength, maturity, and density. However, few strong correlations were observed between tests that are relatively simple to conduct and more complex material characterization tests.
- One measure of sorptivity correlated mildly with some elements of ASTM C 642, but overall, the sorptivity test did not correlate well with other tests. Results from ASTM C 642, however, had good correlation with a number of other tests and therefore might be worth further evaluation as a possible routine test for high early strength materials.
- In cases where significant dilation occurred, dilation due to cyclic freezing and thawing correlated mildly with the spacing factor, as determined by ASTM C 457. Further, there was mild correlation between the air content measured with ASTM C 457 and that measured on the fresh concrete, but less correlation between the air content of fresh concrete and the spacing factor, as measured by ASTM C 457.
- There was fairly strong correlation between spacing factor and various measures of concrete compressive strength, with increasing the spacing factor resulting in higher strength. This correlation is not simply a reflection of higher air content, but an indication that larger air voids spaced further apart produce stronger concrete.

CHAPTER 4

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK

This study examined field- and laboratory-prepared EOT concrete to evaluate factors that influence durability and to develop guidance on constructing repairs with enhanced durability. An experiment was conducted using 6- to 8-hour and 20- to 24-hour EOT concrete mixtures obtained from four states (Ohio, Georgia, Texas, and New York) to determine typical EOT concrete mixture properties and performance characteristics. Also, a large laboratory study was undertaken in which 28 different EOT concrete mixtures (two replicates, or batches, were made for each mixture, for a total of 56 batches) were produced and tests were conducted to assess the properties of the fresh concrete, volume change, freeze-thaw durability, microstructural characterization, and the absorption/porosity of the concrete. The results were analyzed to draw conclusions regarding the durability of such mixtures. The following is a summary of the most relevant conclusions drawn from this study and recommendations for future work.

4.1 SUMMARY OF CONCLUSIONS

4.1.1 General Observations

General observations are as follows:

- In general, the concrete obtained in the field and that produced in the laboratory were of good quality, indicating that durable EOT concrete could be proportioned and constructed.

- Both the field concrete and the laboratory concrete exhibited poorly formed air-void systems, although the original air-void system parameters and air content were, in some cases, adequate. The inadequate air-void systems in the laboratory appeared to be linked to interactions between multiple mixture constituents, including the Type III cement and the Type F HRWR. Although a few mixtures had insufficient air contents as measured in the fresh concrete, a far greater number had insufficient air as measured in the hardened concrete by ASTM C 457. The most notable deficiency was that six of eight batches of the 6- to 8-hour mixtures and two batches of the 20- to 24-hour mixtures made with Type III cement and Type F HRWR had insufficient air contents in the hardened concrete. Air content was difficult to control in mixtures made with high cement contents, low *w/c* ratios, and multiple admixtures, especially if fine cement was used. Only single sources of Type III cement and Type F HRWR were investigated, and limited tests were conducted; thus, conclusions could not be drawn from this study regarding the effects of Type III cement or Type F HRWR on durability.
- In general, less homogeneous paste and increased microcracking was observed in the faster-setting mixtures. This observation was made both in the field concrete and in the laboratory-prepared specimens. Paste homogeneity of laboratory-prepared specimens, as assessed using the relatively low magnification of the petrographic optical microscope, was better in the 20- to 24-hour mixtures, indicating a more thoroughly blended, more uniform paste. It appeared that the mixtures made with the Type III cement and Type F HRWR were slightly less homogenous, and better cement grain dispersion was observed when the Type F HRWR was used with Type I cement. Severe microcracking of the paste was very obvious in all of the 6- to 8-hour mixtures. Less severe microcracking occurred in the 20- to 24-hour mixtures.

- Although attempts were made to avoid EOT concrete with known alkali-aggregate reactivity problems in the field study, ASR was observed in the Ohio materials. The ASR was observed in both types of mixtures, but was far less prevalent in the 20- to 24-hour repairs (MS mixture), which contained a microsilica supplementary cementitious material. Due to the high cement content of many EOT concrete mixtures, thorough screening of constituent materials must be conducted to minimize the occurrence of alkali-aggregate reactivity.
- None of the 20- to 24-hour mixtures evaluated in the laboratory study performed poorly in freeze-thaw testing (AASHTO T 161), but a number of the 6- to 8-hour mixtures had unacceptably high dilation values. Among the poorly performing mixtures were those made with Type III cement and Type F HRWR that had low air contents. The findings of this study indicate that the air content measured in fresh concrete is not always a good predictor of the sufficiency of the air-void system. Also, the spacing factor was found to be a fairly reliable predictor of potential freeze-thaw performance. Unexpectedly high dilation values were found in only one instance where the spacing factor was below the recommended maximum of 0.200 mm (0.008 in.).
- The results of laboratory scaling tests were variable. They showed a significantly higher degree of scaling in the 6- to 8-hour mixtures than in the 20- to 24-hour mixtures. The mixtures made with Type III cement and Type F HRWR also suffered high degrees of deicer scaling, but there is little batch-to-batch correlation between dilation and scaling. These results illustrate the importance of using multiple batches and replicates when conducting durability testing. The x-ray microscope analysis showed significantly less overall penetration of chloride ions into the 6- to 8-hour mixtures than into the 20- to 24-hour mixtures as a result of the reduced *w/c* ratio in these mixtures.

- The 6- to 8-hour mixtures had higher 24-hour strengths than the 20- to 24-hour mixtures. However, the majority of the 6- to 8-hour mixtures did not meet the compressive and flexural strength criteria at 6 hours, but most gained sufficient strength by 8 hours. Most all of the 20- to 24-hour mixtures met the strength criteria by 20 hours. The relationship between compressive strength and flexural strength is mixture specific. If such a relationship is to be used to monitor construction, it should be determined on a mix-by-mix basis.

4.1.2 Mixture-Specific Observations

The following mixture parameters impacted the behavior of EOT concrete:

- The cement type had a notable impact on various measures of early concrete strength, with Type III cements producing higher strengths at a given age than Type I cements. This increased rate of hydration was evidenced by the higher maturity readings for the 6- to 8-hour mixtures. In many cases, the use of the finer Type III cement negatively influenced the air-void system parameters, most acutely when the calcium chloride accelerator was used, and the resistance to deicer scaling was also affected in the 6- to 8-hour mixtures. Because only a single source of Type III cement was used and limited tests were conducted, conclusions regarding the effects of Type III cement on durability of EOT concrete mixture could not be made.
- The cement factor had a large impact on the percent volume of permeable voids. Void volume increased with increasing cement content. Increases in paste content also increased the number of shrinkage cracks as measured in the restrained shrinkage test. For 6- to 8-hour

mixtures, increasing the cement content decreased scaling, but also led to a reduction in some measures of early strength.

- Decreasing the *w/c* ratio increased various measures of strength. It also increased the CTE. In some cases, absorption was reduced and paste homogeneity improved when the *w/c* ratio was lowered.
- The coarse aggregate type had a very significant impact on measures of density, with denser aggregate resulting in denser concrete. Mixtures made with the gravel coarse aggregate had the highest CTE, and mixtures made with the carbonate aggregate had the least scaling. Aggregate type also affected some of the strength properties of the mixtures, particularly compressive strength.
- The hydrated cement paste was far more uniform in mixtures made with the non-chloride accelerator than those made with calcium chloride or without an accelerator. In some instances, mixtures made with calcium chloride accelerator as opposed to a non-chloride accelerator had lower early strengths, but long-term strengths were increased. In some cases, using a calcium chloride accelerator significantly increased the CTE, although the increase was small.
- The use of the Type F HRWR in the 6- to 8-hour mixtures reduced the air content and negatively impacted a number of the air-void system parameters and the early strength characteristics of the concrete. Mixtures containing Type E water reducer had poor paste homogeneity, the highest volume of permeable voids, and very low strengths, both at 6 and 8 hours.
- Raising the curing temperature had many positive effects, including reduced shrinkage, improved air-void system parameters, reduced dilation due to cyclic freezing and thawing,

and reduced scaling. Although some early strength measures were increased, the 28-day strength was decreased in some cases.

4.1.3 Testing-Specific Observations

Testing-specific observations are as follows:

- Certain tests are highly correlated one with another, such as the various measures of compressive strength, flexural strength, maturity, and density. However, few strong correlations were observed between tests that are relatively simple to conduct and more complex material characterization tests.
- CTE results were highly repeatable, with very few exceptions. The variability was slightly higher for the 6- to 8-hour mixtures.
- Results from the restrained shrinkage ring test were highly variable, but a statistical difference in the time to first cracking was observed, with 6- to 8-hour mixtures cracking earlier than 20- to 24-hour mixtures. Curing temperature had a large impact on shrinkage cracking, with increased curing temperatures producing far less tendency to crack. In general, the repeatability of the shrinkage ring test (as used in this study) was not very good, between batches of the same mixture or even between replicates from the same batch.
- Overall, the sorptivity test did not correlate well with other tests, although one measure of sorptivity correlated mildly with some elements of ASTM C 642. Results from ASTM C 642, however, had good correlation with the results from a number of other tests, and therefore this test method might be worth further evaluation as a possible routine test for high early strength materials.

4.2 RECOMMENDATIONS FOR FUTURE WORK

The following are recommendations for future work to help address some issues related to EOT concrete durability:

- Study of the role of cement fineness and chemistry on EOT concrete durability,
- Research to develop a more fundamental understanding of admixture interactions in EOT concrete,
- Evaluation of alternative methods for assessing the air-void system characteristics of both fresh and hardened concrete of EOT concrete, and
- Research to establish a set of tests for assessing multiple mixture characteristics that relate to EOT concrete.

REFERENCES

American Concrete Institute (ACI), "Chemical Admixtures for Concrete." *ACI Manual of Practice—Part 1*, ACI 212.3R-91. Farmington Hills, Michigan, ISSN 0065-7875 (2003a).

American Concrete Institute (ACI), "Guide to Durable Concrete." *ACI Manual of Practice—Part 1,* ACI 201.2R-92, Farmington Hills, Michigan, ISSN 0065-7875 (2003b).

American Concrete Institute (ACI), "State-of-the-Art Report on Alkali-Aggregate Reactivity." *ACI Manual of Practice—Part 1,* ACI 221.1R-98, Farmington Hills, Michigan, ISSN 0065-7875 (2003c).

American Concrete Pavement Association (ACPA), *Scale-Resistant Concrete Pavements.* Information Series IS117.02P, Skokie, Illinois (1992).

American Concrete Pavement Association (ACPA), *Concrete Paving Technology—Fast Track Concrete Paving*. TB004.02, Skokie, Illinois (1994).

American Concrete Pavement Association (ACPA), *Concrete Paving Technology—Guidelines for Full-Depth Repair*. TB002-02P, Skokie, Illinois (1995).

Farney, J. A., and S. H. Kosmatka, *Diagnosis and Control of Alkali-Aggregate Reactions in Concrete.* Concrete Information Series No. IS413.01T. Portland Cement Association, Skokie, Illinois, ISBN 0 89312 146 0 (1997).

Federal Highway Administration (FHWA), *Temperature Management of Slabs.* Final Report, Special Project 201, U.S. Department of Transportation, Washington D.C., June (1994).

Federal Highway Administration (FHWA), *Full-Depth Repairs*. U.S. Department of Transportation, Washington, D.C., http://fhwa.dot.gov/pavement/full.htm. (Accessed February 19, 2003).

Folliard, K. J., and K. D. Smith, *NCHRP Research Results Digest 281: Aggregate Tests for Portland Cement Concrete Pavements: Review and Recommendations.* Transportation Research Board of the National Academies, Washington, D.C., September (2003).

Hooton, R. D., M. D. A. Thomas, and K. Stannish, *Prediction of Chloride Penetration in Concrete.* FHWA-RD-00-142, Federal Highway Administration, McLean, Virginia. October (2001).

Justness, H., and E. C. Nygaard, “Changes in the Microstructure of Cement Paste and Concrete due to Calcium Nitrate Addition.” *5th Canmet/ACI International Conference on Superplasticizers and Other Chemical Admixtures in Concrete,* Rome, Italy, ACI SP 173 (1997).

Klemm, Waldemar A., and F. MacGregor Miller, *Internal Sulfate Attack: A Distress Mechanism at Ambient and Elevated Temperatures?* PCA R&D Serial No. 2125, Portland Cement Association, Skokie, Illinois (1997).

Kosmatka, S. H., B. Kerkhoff, and W. C. Panarese, *Design and Control of Concrete Mixtures*. Fourteenth Edition, Engineering Bullentin 001, Portland Cement Association, Skokie, Illinois, ISBN 0-89312-217-3 (2002).

Kraai, P. P., "A Proposed Test to Determine the Cracking Potential due to Drying Shrinkage of Concrete," *Concrete Construction*, Sept. (1985).

Lackey, H. B., "Factors Affecting Use of Calcium Chloride in Concrete." *Cement, Concrete, and Aggregates*, Vol. 14, No. 2 (Winter 1992).

Lea, F. M., *The Chemistry of Cement and Concrete*. Chemical Publishing Company (1971).

Magura, D., "Evaluation of the Air Void Analyzer." *Concrete International*, August (1996).

Marchand, J., E. J. Sellevold, and M. Pigeon, "The Deicer Salt Scaling Deterioration of Concrete—An Overview." *Durability of Concret,* Third International Conference, Nice, France, ACI SP-145 (1994).

Mehta, P. K., and P. J. M. Monteiro, *Concrete: Structure, Properties, and Materials*. Second Edition, Prentice Hall, Englewoods Cliffs, New Jersey (1993).

Mindess, S., J. F. Young, and D. Darwin, *Concrete,* Second Edition, Prentice-Hall Pearson Education, Inc., Upper Saddle River, New Jersey, ISBN 0-13-064632-6 (2003).

National Highway Institute (NHI). *PCC Pavement Evaluation and Rehabilitation*, Reference Manual, NHI Course Number 131062, Federal Highway Administration, U.S. Department of Transportation, October (2003).

Neville, A. M., *Properties of Concrete*, John Wiley and Sons, Inc., New York, New York, ISBN 0-470-23527-6 (1996).

Ouyang, C., and O. J. Lane, "Effect of Infilling of Air Voids by Ettringite on Resistance of Concretes to Freezing and Thawing." *Ettringite—The Sometimes Host of Destruction,* SP-177, American Concrete Institute, Farmington Hills, Michigan (1999).

Pigeon, M., and R. Plateau, *Durability of Concrete in Cold Climates.* E & FN Spon. ISBN 0-419-19260-3 (1995).

Price, B., "Measuring Air Voids in Fresh Concrete." *Concrete*, Vol. 30, No. 4 (1996).

Ramachandran, V. S., *Concrete Admixtures Handbook.* Noyes Publications (1984).

Rezansoff, T., and D. Stott, “Durability of Concrete Containing Chloride-Based Accelerating Admixtures.” *Canadian Journal of Civil Engineering*, Vol. 17 (1990).

Rosskopf, P. A., F. J. Linton, and R. B. Peppler, “Effect of Various Accelerating Chemical Admixtures on Setting and Strength Development of Concrete.” *Journal of Testing and Evaluation (JTEVA),* Vol. 3, No. 4 (July 1975).

Schwartz, D. R. *NCHRP Synthesis of Highway Practice 134: D-Cracking of Concrete Pavements.* Transportation Research Board, Washington, D.C. (1987).

Stanish, K. D., R. D. Hooton, and M. D. A Thomas, *Testing the Chloride Penetration Resistance of Concrete: A Literature Review.* FHWA Contract DTFH61-97-R-00022, University of Toronto, Toronto, Ontario, Canada, June (2000).

Stark, D. *Characterization and Utilization of Coarse Aggregate Associated with D-Cracking.* RD047.01p, Portland Cement Association, Skokie, Illinois (1976).

Suryavanshi, A. K., J. D. Scantlebury, and S. B. Lyon, “Pore Size Distribution of OPC and SRPC Mortars in Presence of Chlorides.” *Cement and Concrete Research*, Vol. 25, No. 5 (1995).

Thaulow, Niels, Vagn Johansen, and Ulla Hjorth Jakobsen. *What Causes Delayed Ettringite Formation?* Bulletin No. 60, presented to the Fall 1995 Meeting of the Material Research Society, Boston, Massachusetts (1996).

Transportation Research Board (TRB), *Transportation Research Circular 494: Durability of Concrete*. Washington, D.C., December (1999).

Van Dam, T. J., L. L. Sutter, K. D. Smith, M. J. Wade, and K. R. Peterson, *Guidelines for Detection, Analysis, and Treatment of Materials-Related Distress in Concrete Pavements, Volume 1: Final Report.* FHWA-RD-01-163, Federal Highway Administration, Turner-Fairbank Highway Research Center, McLean, Virginia, March (2002a).

Van Dam, T. J., L. L. Sutter, K. D. Smith, M. J. Wade, and K. R. Peterson, *Guidelines for Detection, Analysis, and Treatment of Materials-Related Distress in Concrete Pavements, Volume 2: Guidelines Description and Use.* FHWA-RD-01-164, Federal Highway Administration, Turner-Fairbank Highway Research Center, McLean, Virginia, March (2002b).

Wang, H., and J. E. Gillott, "The Effect of $CaCl_2$–Based Accelerator on Alkali Silica Reaction." *Cement and Concrete Research*, Vol. 20 (1990).

Whiting, D. A., and M. A. Nagi, *Manual on Control of Air Content in Concrete*. EB116, Portland Cement Association, Skokie, Illinois (1998).

Whiting, D., M. Nagi, P. Okamoto, T. Yu, D. Peshkin, K. Smith, M. Darter, J. Clifton, and L. Kaetzel, *Optimization of Highway Concrete Technology*. SHRP-C-373, Strategic Highway Research Program, Transportation Research Board, Washington, D.C. (1994).

APPENDICES

The following appendices are not are published herein, but can be downloaded for free at http://www.trb.org/news/blurb_detail.asp?id=5203:

- Appendix A: Detailed Background Information
- Appendix B: Data from Field Investigation
- Appendix C: Laboratory Test Results
- Appendix D: Results of Statistical Analysis
- Appendix E: Implementation Plan