ABSTRACT

OPTIMIZATION OF VERY EARLY STRENGTH CONCRETE MIXES USING MATURITY METHOD

by
Sun Punurai

Fast track concrete technology has been used over the past two decades for overnight highway repairs. Originally, fast track was concerned only with rapid strength gain and workability but little concern was paid to cost and durability of the concrete.

The purpose of this research study is to develop a current improved very early strength (VES) concrete mix, which was accepted by NJDOT in 1996. More than 50 concrete mixes were evaluated for strength, shrinkage, and curing rate (maturity), eventually leading to the development of an optimized VES concrete mix with improved performance characteristics.

The study has developed the new four mix designs, which can achieve the compressive strength in a range of 2,200-2,300 psi and the flexural strength (modulus of rupture) in a range of 350-365 psi at six and half hours with an improved shrinkage behavior. The study has found that an initial temperature of 27°C (81F) to 29°C (85F) is the influence factor to achieve a required strength in six and half hours. The test results also show that the required flexural and compressive strengths can be obtained with a maturity of 175 °C-hours. Using the non-destructive maturity test, this value can be used as an indicator to predict the real-time strength of concrete. Moreover, the maturity test can reduce a certain cost of breaking cylinder and can decrease the amount of debris, which can save natural resources and protect the environment.

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by
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This thesis is dedicated to

" my beloved family "

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CHAPTER 1

INTRODUCTION

1.1 General

New Jersey is a heavily populated state and its roads are principal arteries for the New York City metropolitan area. Traffic congestion is intrinsic to New Jersey highways, and repair operations usually bring traffic to a halt. Over the years, the New Jersey Department of Transportation (NJDOT) has tried a variety of innovative methods to reduce the time associated with repair of concrete pavement. In 1966, NJDOT approved the use of VES (Very Early Strength) concrete in pavement repair and construction, and the NJDOT's requirements for the fast track concrete included (Ansari and Luke 1996, Ansari, Luke, Vitillo, Blank and Turham 1997, Ansari and Luke 1998, Ansari Luke and Dong 1999):

- Achieving a compressive strength of about 2,500 psi (14 MPa) and a Modulus of rupture of about 350 psi (2 MPa) in six to seven hours after placement operation.
- Use of locally available materials and normal aggregate gradations, i.e., Type I portland cement.
- Use of accelerators limited to nonchloride based admixtures.
- Workability for placement and finishing operations.

The VES concrete approved for use in 1996 was originally intended as a joint patching material, which was soon to be overlaid with asphalt. However, within a year, several miles of Interstate 295 were paved with VES for regular pavement construction on a NJDOT project.

Two major technical problems were observed on this VES pavement project. First, transverse cracking was observed in the 78–foot long slabs at intervals of 25 feet. Second, the concrete curing temperatures reached a relatively high level. Subsequent to the I-295 project, "fast-track" concrete has also been applied to construction of exit ramps, streets and parking lots, and its use can only be expected to multiply. Thus, there is a need for a VES mix with a reduced potential for cracking.

The overall objective of this research study was to develop a modified VES concrete mix with reduced shrinkage and cracking potential, while maintaining or improving the rapid strength gain associated with VES. Shrinkage cracking is primarily the result of volume changes associated with water loss, and, generally, the less water there is in a mix, the lower the tendency for cracking. By reducing the amount of cement in the mix while keeping the water-cement ratio constant, the overall effect is to reduce the overall amount of water in the mix. Another way to reduce the shrinkage is to use a larger aggregate since a larger aggregate reduces the need for cement paste, thus reducing the possibility for shrinkage. The principal question was whether concrete with a significantly reduced cement factor could still achieve the required strength in the allotted time.

Optimization of a concrete mix is a process of balancing conflicting demands. Consideration must be given to its end use, placeability and workability, ultimate strength and durability, weight, color, and cost. For the purposes of this research, optimization of "fast-track" concrete means controlling shrinkage cracking without sacrificing rapid strength gain, while still maintaining satisfactory workability.

In view of the urgency to reopen traffic, an additional objective of the research study was to produce recommendations for a simple test method to quickly and reliably assess concrete strength. An in-place test method was desired since standard test cylinders do not include the effects of placing, compacting, and curing. The maturity method was employed for the determination of in-place strength since it is able to take these effects into account. The technique is based on the measured temperature history of concrete during the curing period. The combined effects of time and temperature lead to determination of a single parameter, the maturity index. This index is then used to correlate strengths of samples of the same concrete. The assertion is made that for a particular concrete at the same maturity, whether in cylinders, beams or in the structure, the strength will be approximately the same. More details can be found in ASTM C1074-98.

1.2 Objectives

In 1996, a cooperative research effort involving NJIT, NJDOT, Rutgers University, Sika Cooperation, Essroc Cement Co., concrete batch plants and contractors developed a "Fast-Track" concrete mix for very early strength concrete highway batches. This mix, containing a hardener accelerator, consistently produces a compressive strength of 2,500 psi and flexural strength of 350 psi at 6.5 hours, which was sufficient for unrestricted interstate highway use. Considerable effort was made to develop a field usable mix in terms of workability, finishability and, working time as well as rapid strength gain but little consideration was given to cost or durability issues.

The objective of this proposed research is to optimize the currently approved Very Early Strength (VES) concrete mix without significantly reducing its strength gain performance. Additionally, for durability consideration, this research will explore ways to reduce concrete shrinkage and transverse cracking in concrete. Finally, a procedure will be developed for using concrete maturity determinations of in-situ strength of VES concrete as criteria for early opening to traffic.

Develop the Optimized Mix: A series of concrete mixes evaluated the effects of cement brand, cement content, accelerator dosage rate, aggregate gradation were conducted. The current 799 lb./yd^3 (8.5 bag/yd^3) mixes served as the control mixes.

Not only its strength gain and maturity parameters will be reviewed but also its shrinkage behavior will be studied. Essentially, The mixes with cement contents running from 611 to 799 lb./yd^3 (6.5-8.5 cement-bag/yd^3) at increments of half bags of cement will be used. In the original study, the cement type will be keep constant. Working with a local batch plant, use the gradations from some currently approved DOT mix as a starting point. After a cost optimized mix has been identified, another mix utilizing #467 stone (1½") will be produced to investigate further reductions to the shrinkage potential. Based on the research work, the Impact of Coarse Aggregates on Transverse Crack Performance in Jointed Concrete Pavements (Neeraj, Michael, Frabizzio, and Jacob 2000), showed that the large stones as coarse aggregates appeared to be effective in providing improved aggregate interlock at transverse cracks.

Working up the strength-maturity relationship for "fast-track" concrete is a hectic process. It is in the nature of the hardening accelerator that it does not affect the set time of mix. It is expected that the final setting time will be four to five hours. At that time, the temperature spikes changing perhaps 45-degree slope or more in couple of hours. Strength gain of 2,000 psi in an hour can sometimes be seen. A compression test will indicate that the beams may be ready to test. A successful beam test requires determination of the current compressive strength. A dozen cylinders, or more, and three or four beams will be tested within that time.

Determine the Maturity Parameters of the Optimized Mix: In order to use maturity parameters for quality control of the Optimized Mix based on the concept that the same typical concrete reaches the same strength at the same maturity. The temperature measurement of concrete and the series of compressive strength of mortar cubes for datum temperature calculation have to be performed.

Instructions for implementing the maturity method for estimating the strength of concrete are found in ASTM 1074-98. Initially, the datum temperature is found through a series of experiments and calculations on data from tests of mortar cubes of the proposed mix, cured at three different temperatures. The reciprocal of the strength is plotted against the reciprocal of the time since final set. Linear regression analysis of the three series is applied to determine the equations of the best-fit lines. To determine the K values, each interception of the lines is divided by each slope. Once the K values are known they are plotted against the curing temperature. The intercept with x-axis is the graphical solution for the datum temperature. The analytical solution is the division, again, of the y-intercept of the line by the slope. Next, the compressive strengths of concrete cylinders are plotted with the maturity at the time of the test to specify the strength maturity relationship. Finally, that relationship is used to predict the strength of future applications of that mix. An example to complete the datum temperature can be found in Appendix B.

The maturity parameter from the laboratory will be used to estimate the strength of the same mix design of concrete in the field. It has been noticed that the flexural strength of the VES concrete calculated from the compressive strength is higher than that would be expected from the ACI empirical equations. Some thought will be given to expressing the strength-maturity equation in terms of flexural strength rather than compressive strength.

Determine the Shrinkage and Length Change of the Optimized Mix: A crack free surface is a prerequisite for durable concrete. The free shrinkage characteristic of the control mix and the most promising proposed mixes will be evaluated.

The series of concrete bars are produced to measure the length change compare with the control mix. It is expected that the reduction of cement content and use of larger aggregate will reduce the percent of shrinkage value. Shrinkage reducing admixture will further reduce the cracking. The comparison curve will be produced to show the amount of volume change in concrete mix design after casting.

Dry shrinkage testing is conducted over a 90-day period. In this test, a 2½-in thick concrete ring is cast around a 12-in. steel ring. Companion prisms are also cast to measure free shrinkage. After eight hours, the outside mold is removed to expose the outside of the concrete ring while keeping it restrained against the steel ring. When the shrinkage of the concrete exceeds the tensile strain capacity of the concrete, a crack will occur. This crack will widen as shrinkage continues. The number and width of cracks are measured as the test proceeds. This test measures not only the ability of the concrete to sustain tensile strain but also incorporates the effect of tension creep. It is expected that the reduction of cement content and use of larger aggregate will reduce the number and width of cracks. Shrinkage reducing admixture will further reduce the crack widths.

CHAPTER 2

LITERATURE REVIEW AND BACKGROUND

2.1 Literature Review

In 1980s, the early opening of concrete pavement to traffic has been given much attention, which is called fast track concrete. Fast track concrete was employed in order to minimize the repair time of the pavement when it was fixing. The fast track concrete paving is generating a great deal of interest in the concrete industry because it eliminates the one advantage of asphalt over concrete, the ability to get traffic onto the pavement a relatively short time after construction. This new concrete paving technique has been made possible by the development of economical concrete mixes that provide high strength in less than 24 hours.

On the outset of fast track concrete, the concrete gains strength faster than normal concrete. However, most of the recent development is aimed at workability and rapid strength gain. It has little concern about the cost and durability in order to produce fast track concrete. There are several kinds of fast track concrete (Pearson 1998, Peng 1993, and Rapid Set 1993) available in the market. Some types gain compressive strength 2,000-4,000 psi in three hours while other types gain compressive strength 2,000-4,000 psi after three hours. Normally, the three-hour fast track concrete is used when the structures needed to be put into service very quickly. Some examples of this type of fast track concrete are *polymer concrete* (Peng 1993) and *rapid-hardening hydraulic concrete* (Al-Manaseer 1999,and Pearson 1998).

Polymer concrete is a very expensive material, but the development of mixing polymer in concrete provides the new material, called Polymer Modified Concrete. The constituents retain their identity, it can be mixed with cement and aggregates to obtain a Polymer Modified Concrete (PMC). Therefore, PMC properties are higher in compressive and tensile strengths than those of normal concrete. Also, it provides good chemical resistance and its cost is relatively lower. The PMC provides three types of PMC; Polymer-impregnated (PIC), Polymer concrete (PC) and Poly-Portland-cement Concrete (PPCC). It has shown that Polymer concrete can gain the final setting time in a short time, 10-15 minutes after placing (Peng 1993). Polymer concrete is able to attain the compressive strength 5,623 psi (38.55 MPa) in a hour, 6,833 psi (46.85 MPa) in four hours, 7,607 psi (52.16 MPa) in a day, and 9,976 psi (68.40 MPa) in 28 days. For the 2"x2"x12"-beam test, the flexural strength reaches 236.4 psi (1.6 MPa) at a day and 2,380.3 psi (16.32 MPa) at 28 days.

Rapid hardening hydraulic concrete is made of ultimax cement (Al-Manaseer, Aquino, and Kumbargi 1999). This type of concrete also gains the final setting time within two to three hours after placing the concrete. Although it is quite expensive for use as one of materials for fast track, it is still widely used for repairing propose. With the concrete mixtures containing 674 lb/yd^3 (400 kg/m^3) of cement content and water cement ratio 0.38, the fresh concrete with six-inch to eight-inch slump can reach the compressive strength of 3,500 psi (24 MPa) in three-hours using the on-site nondestructive test.

In 1996, CTS Cement Manufacture Company produced a *rapid set concrete* mix. It is expensive which costs $20 for a 50-lb.bag of concrete mix. But with a five-in slump, the compressive strength can arrive at 2,650 (18.3 MPa), and 3,700 psi (25.5 MPa), respectively, in a hour and three hours. At the same time, the flexural strength reaches 650 psi (4.5 MPa) in five hours.

In addition to previously mentioned polymer concrete, rapid-hardening hydraulic concrete and rapid set concrete, there are other kinds of fast track concrete that can gain required compressive strength under 12 hours. This type of fast track concrete is more popular than the previous ones because of its price, its workability and its placeability. The first use of this type of fast track was in April 1986.

In April 1986, a fast track concrete paving developed by the Iowa-Department of Transportation (IDOT) in cooperation with members of the Iowa Concrete Paving Association, was first used at the new Des Moine, Iowa terminal of the Dundee concrete company. It was used to construct a 1,500-ft (457 m.) long, 10-in (254-mm.) thick, 20-ft (6.1 m.) wide driveway onto the terminal. At that time, the mixed design was 640 lb/yd^3 (380 kg/m^3) of Type III cement plus 70 lb/yd^3 (41 kg/m^3) of class C fly ash. The water-cement ratio was 0.43 to 0.45, air entrainment was 6.5 percent, and slump was 1.5 in (38 mm.). At the time of mixing, the ambient temperature exceeded 90°F (32°C). It reached specified flexural strength 350 psi (2.4 MPa), under eight hours and specified compressive strength, at least 2,500 psi (17.2 MPa), under 12 hours. At 24 hours the flexural strength was 604 psi (4.2 MPa) and the compressive strength was 3,467 psi (23.9 MPa) and at 28 days was 830 psi (5.7 MPa) and 5,900 psi (40.7 MPa), respectively.

In May 1987, 1.03 miles of an existing roadway (County Route P50, Dallas County, Iowa) were paved using six-in (152-mm.) deep and 22-ft (6.7 m.) wide concrete slab. The mix contained 640 lb/yd^3 (380 kg/m^3) of Type III cement and 70 lb/yd^3 (41 kg/m^3) of class C fly ash, 1,713 lb/yd^3 (1,016 kg/m^3) for aggregates and a water reducer. Air entrainment was five to six percent and slump was 1.25 to 1.5 in (32 to 38 mm). The flexural strength of the concrete was 333 psi (2.3 MPa) at eight hours, 415 psi (2.9 MPa) at 12 hours, 780 psi (5.4 MPa) at 26 hours, 978 psi (6.7 MPa) at seven days. At the same time, fast track technique was also applied at the Osceola (Iowa) airport. This fast track allowed adjacent areas to be paved after only three days of curing instead of seven days normal curing. It was a 4,000 ft × 75 ft (1,219 m. × 22.9 m.) concrete runway, a 3,500 yd^2 (2,926 m^2) apron, and a connecting taxiway. This pavement was five-in deep (127-mm.) concrete slab on a four-in (102-mm.) thick crushed aggregate base. This Pavement required a minimum flexural strength of 550 psi (3.8 MPa) before any heavy structures operated on it. The minimum flexural strength required 620 psi (4.3 MPa) at 28 day. This mixed design contained 640 lb/yd^3 (380 kg/m^3) of Type III cement, and 530 lb/yd^3 (314 kg/m^3) of Type I cement. Both mixes used fly ash, water reducer and air-entraining agents. In three days, the flexural strength reached 590 psi (4.1 MPa), 640 psi (4.4 MPa) in five days and 725 psi (5.0 MPa) in 28 days.

In May 1988, The concrete was also used to construct a pavement of six–in thickness (152–mm.) in Michigan. The design mix contained 710 lb/yd^3 (421 kg/m^3) of Type III cement, 1,358 lb/yd^3 (806 kg/m^3) of fine aggregate, 1,528 lb/yd^3 (907 kg/m^3) of coarse aggregate, and 265 lb/yd^3 (157 kg/m^3) of water, plus water-reducing and air-entraining agents. It reached the required flexural strength of 400 psi (2.8 MPa) at 12 hours. Fast track concrete has been successfully used for new paving. Its ability to carry heavy traffic within twelve hours of placement has been demonstrated. However, fast track concrete has not been fast enough.

In March 1996, New Jersey Institute of Technology, Rutgers University, the New Jersey Department of Transportation along with the cement and admixture industry had cooperated to develop a new "Fast Track" or "Very Early Strength" (VES) concrete for a full-depth repair on Interstate-295 in Southern New Jersey. The goal was to develop a mix that would produce a flexural strength of 300 to 350 psi (2 to 2.4 MPa) and a compressive strength of approximately 2,500 psi (17.2 MPa) within six to seven hours after placement. Also the maturity method was employed to determine the onsite concrete strength. The mix proportion contained 799 lb/yd^3 (475 kg/m^3) of Type I cement, 1,800 lb/yd^3 (1069 kg/m^3) of coarse aggregate, 1,200 lb/yd^3 (712.5 kg/m^3) of fine aggregate, 325 lb/yd^3 (192.97 kg/m^3) of water, air-entraining admixture, high range water reducer and hardening accelerating admixture. The fast track concrete reached the target compressive strength and flexural strength in about six and a half hours. The compressive strength and flexural strength at 24 hours was 3,865 psi and 380 psi (26.5 MPa and 2.6 MPa), respectively. The compressive strength at 28 days was 5,269 psi (36.22 MPa). The maturity method was first employed to relate the strength of the concrete and its time-temperature product to the curing process. At maturity value of 130-150°C-hrs, a compressive strength of 2,500 psi (17.2 MPa) was observed (Ansari and Luke 1996).

The development of the above VES concrete mix was the culmination of more than 40-trial batches, hundreds of cylinders and many beams. The data was presented in the various reports to NJDOT, additional information from laboratory notebooks and conversations with batch plant operations will be used for the optimization of the proposed mix in this study.

The March 29, 1996 report, "High Early Strength Concrete for Fast Track Construction and Repair", established the viability of the current mix. An additional mix utilizing 705 lb./yd^3 of cement was also presented. The utility of using the maturity method for predicting the strength of VES concrete was also demonstrated. In spite of a weak correlation between compressive and flexural strength, the observation was made in this report that the flexural strength of very young concrete was higher than that expected from accepted ACI equations. For this reason, it might be better to relate the maturity to the flexural strength instead of the compressive strength. These results were subsequently published in the May 1997 issue of *Concrete International* under the title, "Developing Fast Track Concrete for Pavement Repair" (Ansari, Luke, Vitillo, Bank and Turhan 1997).

"Development of Fast Track Concrete-2" (Ansari and Luke 1998), submitted on July 20, 1998 to NJDOT, showed that VES concrete could be produced from all the cements approved for use in New Jersey, except one. It also concluded that no advantage was to be had using Type III cement with the hardening accelerator. It also considered the use of other accelerators and concluded that only Sika's hardening accelerator could produce the required 350-psi flexural strength in the five to seven-hour window.

Further investigations were presented in a July 27,1999 to NJDOT, "Development of Maturity Protocol for Construction of NJDOT Concrete Structure" (Ansari, Luke, and Dong 1999). This report showed that the value of the datum temperature, 6.5°C, used for computing the maturity of the fast track mixes maybe somewhat conservative. This report determined a datum temperature value of 5.8°C for a standard concrete mix. It also showed very definitively that different structures gained and held the heat of hydration quite differently. A column and footing cast at the same time recorded radically different temperature histories under freezing conditions. Clearly, this is an indication that structures will strengthen at different rates depending to some extent on the geometry of the structure and its surface to volume ratio. The maturity method can account for subtle differences in heat content. It is very usefully accounting for small variations in the conditions to which a given concrete will be subjected.

2.2 Accelerated Techniques for Concrete Paving

2.2.1 Concrete Mixture Proportioning

One of the primary ways to decrease facility closure time is to use a concrete mixture that develops its concrete strength rapidly. Rapid strength gain is not limited to use of special blended cements or sophisticated construction methods. It is usually possible to develop such a mixture using locally available cements, admixtures and aggregates.

When proportioning concrete mixtures for accelerated paving, concrete technologist also should be aware of the additional influences of heat of hydration, aggregates size distribution, entrained air, concrete temperature, curing provisions and ambient temperature. These factors may influence early and long-term concrete strengths. Many different combinations of materials will result in rapid strength gain. Table 2.1 shows acceptable materials and proportions to achieve rapid early strength gain. A complete list and discussion of admixture is provided in ASTM C494.

A thorough laboratory investigation is important before specifying an accelerated paving admixture. The lab work should determine plastic and hardened concrete properties using project materials and should verify the compatibility of all chemically active ingredients in the mixture. Table 2.2 shows some factors that influence mixture properties and may aid mixture proportioning.

Table 2.1 Example of Concrete Mixture Components for Accelerated Pavements (Ferragut 1990)

Material	Type	Quantity
Cement	ASTM C150 Type I	600-800 lb/yd^3
	ASTM C150 Type III	600-800 lb/yd^3
Fly ash	ASTM C618	10%-20% of cement by weight
Water-cementitious material ratio		0.37-0.43
Air entraining admixture	ASTM C260	As necessary
Accelerating admixture	ASTM C494	As necessary
Water-reducing admixture	ASTM C494	As necessary

Generally, accelerated-concrete pavement will provide good durability. Most accelerated paving mixtures have entrained air and a relatively low water content that improves strength and decreases chloride permeability (Kosmatka and Panarese 1998). Freeze-thaw deterioration can occur if water freezes and expands within a concrete binder with a poor-air-void distribution or if the concrete contains poor-quality aggregates. Properly cured concrete with an adequate air-void distribution resists water penetration and relieves pressures that develop in the binder. Air-entrained concrete pavement is resistant to freeze thaw deterioration even in the presence of de-icing chemicals.

Table 2.2 Some Factors that Influence Fresh and Hardened Mixture Properties (Shilstone 1990)

Fresh hardened Mixture property	Mixture proportioning or placement factor
Long-term strength	• Low water-cementitious material ratio • Cement (composition and fineness) • Aggregate Type & Entrained air content • Presence and type of admixture • Concrete temperature • Curing method and duration
Early strength gain rate	• Cement type • Water- cementitious material ratio • Concrete temperature • Mixture material temperature • Presence and type of admixture • Curing method
Freeze-thaw durability	• Aggregate quality and grading • Entrained air (bubble size and spacing) • Water-cementitious material ratio • Curing method and duration
Workability	• Aggregate particle shape • Combined aggregate grading • Total water content • Entrained air content • Presence and type of admixture • Presence of pozzolans
Abrasion resistance	• Aggregate hardness • Compressive strength • Curing method and duration

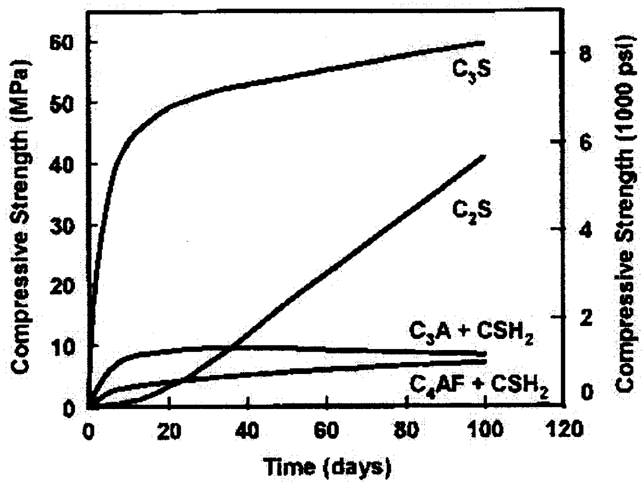


Figure 2.1 Contribution of cement compounds to strength development (Young and Mindess 1981).

2.2.2 Cement

ASTM C150 Types I, II or III Portland Cement can produce successful accelerated paving mixtures (Riley and Knutson 1987). Certain ASTM C 595 Portland/pozzolan cements and several proprietary cements that develop high early strengths may also be useful for accelerated pavement application (Jones 1998). Not every Portland cement will gain strength rapidly, however testing is necessary to confirm the applicability of each cement.

The speed of strength development is a result of the hydration and heat generation characteristics of a particular combination of cement, pozzolan and admixtures. Cements play major role in both strength and heat development, and these properties depend on the interaction of the individual compound constitute of cement. High levels of tricalcium silicate (C_3S) and finely ground cement particles will usually result in rapid strength gain (Young and Mindess 1981). Tricalcium aluminate (C_3A) does not contribute much to long term strength, and in general, C_3S is the major chemical contributor to both early and long-term strengths.

Finely ground cement increases surface area and allows more cement contact with mixing water and consequently the cement hydrates faster. Type III cement, which is much finer than other types of Portland cement, usually develops strength quickly. Blaine fineness values for Type III cement range from about 500 to 600 m^2/kg. Blaine fineness values for Type I cement usually do not exceed 300 to 400 m^2/kg (Kosmatka and Panarese 1998).

Although the greater fineness of Type III cement provides a much greater surface area for the hydration reaction, it requires more water to coat the particles. Because Type III cement is ground finer than other cements, there is more potential for problems that may result from overheating the cement during the grinding phase of manufacture, including false set. False set is a rapid stiffening of the concrete shortly after mixing. This is not a major problem, and it is possible to restore workability without damaging the normal set of the concrete through further mixing in a transit mixer (Young and Mindess 1981). The materials engineer and contractor should be aware of these phenomena when testing mixtures and trial batches. Tests should be conducted using the same cement that the contractor will use in construction.

A low water-cementituious material ratio (w/cm) contributes to low permeability and good durability (Young and Mindess 1981). A w/cm between 0.4-0.5 provides moderate chloride permeability for concrete made from conventional materials. A w/cm below 0.4 typically provides low chloride permeability (Whiting 1981). Some accelerated-paving mixtures have a ratio less then 0.43 and consequently, provide moderate to low permeability. It is important to remember that durability is not a function of early strength but is a function of long term strength, w/cm permeability, a proper air void system, and aggregate quality. Mixtures using these materials may appear to meet the quick strength development necessary for accelerated concrete paving but may not provide adequate durability. Because of this inconsistency, a mixture should be evaluated at various ages to ensure it meets both early strength and long-term durability requirements. With proper proportioning, concrete using Type I and II portland cements also can produce adequate characteristics for accelerated-concrete paving. To develop adequate early strength, concrete made from these cements will usually require chemical admixtures.

2.2.3 Water

The sooner the temperature of a mixture rises, the faster the mixture will develop strength. One way to raise the temperature of plastic concrete is to heat the mixing water; however, this is more practical for small projects that do not require a large quantity of concrete, such as intersection reconstruction.

Several factors influence the water temperature needed to produce a desirable mixture temperature at placement. The critical factors are ambient air temperature, aggregate temperature, and aggregate free moisture content. When necessary, ready-mixed concrete producers heat water to 60°C to 66°C (140F to 150F) to elevate mixture temperature sufficiently for cool weather constructions. In such conditions, the use of blanket insulation is advised. To avoid a flash set of the cement, the hot water and aggregate should be combined before adding the cement when mixing batches (Kosmatka and Panarese 1998).

Hot water only facilitates early hydration, and its benefits are generally short-lived. Several hours of heat containment through insulation may be necessary for rapid strength gain to continue particularly when cool conditions prevail.

2.2.4 Aggregate

Aggregates that comply with ASTM C33 specifications are acceptable for use in accelerated-concrete pavements. Existing accelerated-paving projects made with concrete containing these aggregates have met their early-strength requirements and are providing good service. Further consideration of grading and aggregate particle shape may optimize early and long-term concrete strengths. These factors also can have a significant influence on the plastic and hardened mixture properties and may warrant consideration for accelerated concrete pavements.

Grading: Grading data indicate the relative composition of aggregate be particle size. Sieve analyses of source stockpiles are necessary to characterize the materials. The best use of such data is to calculate the individual proportion of each aggregate stockpile in the mixture to obtain the designed combined aggregate grading. Well-graded mixtures generally have a uniform distribution of aggregates on each sieve. Gap-grade mixtures have a deficiency of aggregates retained on the 2.36 mm through 600 μm (No.8 through 30) sieves.

An optimum combined aggregate grading efficiently uses locally available materials to fill the major voids in the concrete to reduce the need for mortar. Particle shape and texture are important to the response of the concrete to vibration, especially in the intermediate sizes. A well-consolidated concrete mixture with an optimum aggregate grading will produce dense and durable concrete without edge slump.

One approach to evaluate the combined-aggregate grading is to access the percentage of aggregates retained on each sieve (Shilstone 1990). A grading that approach the shape of a bell curve on a standard grading chart indicates an optimal distribution. Blends that leave a deficiency in the 2.3 mm through 600 um (No.8 through No. 30) sieves are partially gap graded.

There is a definite relationship between aggregate grading and concrete strength, workability and long term durability (Kosmatka and Panarese 1998). Intermediate-size aggregates fill voids typically occupied density. Increasing concrete density in this manner will reduce mixing water demand and will improve strength because mortar is necessary to fill space between aggregate. It will increase durability through reduced avenues for water penetration in the hardened concrete, improve workability and mobility because large aggregate particles do not bind in contact with other large particles under the dynamics of finishing and vibration and less edge slump because of increased particle to particle contact.

Concrete with a well-graded aggregate often will be much more workable at low slump than a gap-graded mixture at a higher slump. A well-graded aggregate may change concrete slump by 3½ in. over similar gap-graded mixture. This is because approximately 540-650 lb/ft^3 less water is necessary to maintain mixture consistency than is necessary with gap grading (Chase et al. 1989).

Particle shape and texture: The shape and texture of aggregate particles impact concrete properties. Sharp and rough particles generally produce less workable mixtures than rounded and smooth particles at the same w/cm. The bond strength between aggregate and cement mortar improves as aggregate texture become rougher. The improved bond will improve concrete flexural strength (Kosmatka and Panarese 1998). Cube-shaped crushed aggregate is also more mobile under than flat or elongated aggregate. The good mobility allows concrete to flow easily around the baskets, chairs, and reinforcing bars, and is ideal for pavements. Flat or elongated intermediate and large aggregates can cause mixture problem. These shapes generally require more mixing water or fine aggregate for workability and, consequently, result in a lower concrete flexural strength. Allowing no more than 15 percent flat or elongated aggregate by weight of total aggregate is advisable.

2.2.5 Accelerating Admixture

Accelerating admixtures aid strength development and reduce initial setting time by increasing the reaction rate of C_3A. Accelerating admixtures generally consist of soluble inorganic salts or soluble organic compounds and should meet requirements of ASTM C494, Type C or Type E.

A common accelerator is calcium chloride ($CaCl_2$). Many agencies use $CaCl_2$ for full depth and partial-depth concrete pavement patching when quick curing and opening to traffic is needed. The optimum dose is about 2% by weight of cement. This dose will approximately double the one-day strength of normal concrete (Grove 1989). It is very important to test both fresh and hardened concrete properties before specifying a mixture containing an accelerating admixture. With some aggregate, concrete will be susceptible to early freeze-thaw damage and scaling in the presence of $CaCl_2$. Another drawback of $CaCl_2$ is its corrosive effects on reinforcing steel. If the pavement requires any steel, it is advisable to select a nonchloride accelerator or an alternative method of achieving early strength.

2.2.6 Water Reducing Admixtures

Water reducing reduces the quantity of water necessary in a concrete mixture or improves workability at given water content (Kosmatka and Panarese 1998). Water reducing admixture increases early strength in accelerated-concrete paving mixture by lowering the quantity of water required for appropriate concrete placement and finishing techniques. Water reducers disperse the cement, reducing the number of cement agglomerations (Young and Mindess 1981). More efficient and effective cement hydration occurs, thus increasing strength at all ages. Water reducers can be used to increase early concrete strength with any cement but are especially useful when using Type I cement in an accelerator concrete paving mixture.

There are five water reducing admixtures covered by ASTM C494. Water reducing admixtures (types A, E and F) generally provide the necessary properties for accelerated concrete paving and also classified certain high range water reducing admixtures as superplasticizers. Many available high range water-reducing admixtures meet both ASTM C494 and ASTM C 1017 requirements. While most water reducing admixtures will work well with different Portland cements, laboratory testing is essential to determine if concrete containing the admixture will develop the desired properties. Excessive dosage of high range water reducing admixtures may lead to retardation of setting.

An ASTM C494 type A admixtures are common in accelerated concrete paving. Generally, concrete containing a type A water-reducing admixture will require from 5 % to 10% less water than a similar mixture without the admixture. A type D water reducing, set-retarding admixture may be desirable when very high mixture temperatures induce an early set that preempts placing and finishing operations. A type D water reducer slightly retards the initial set to extend the period of good workability for placing and finishing. This retardation can also affect early strength gain, particularly during the first 12 hours. After 12 hours, the strength gain is similar to concrete containing a type A water reducer. Concrete made with type E, F or G admixtures requires thorough laboratory evaluation to determine if the concrete properties are acceptable for anticipated environmental conditions and placement methods. types F and G admixtures may be more appropriate for high slump mixtures or when a lower w/cm is desired.

Table 2.3 Water Reducing Admixtures Specified in ASTM C 494 (ACI325.11R-01 Accelerated Techniques for Concrete paving)

Type and classification	Effect
Water reducer (Type A)	Reduces water demand by at least 5%. Increases early and later age strength.
Water reducer and retarder (Type D)	Reduces water demand by at least 5%. Retard set. Reduces early age (12h strength) Increase later age strength
Water reducer and accelerator (Type E)	Reduces water demand by at least 5%. Accelerates set. Increase early and later age strength
High range water reducer (Type F)	Reduces water demand by at least 12%. Increase early and later age strength
High range water reducer and retarder (Type G)	Reduces water demand by at least 12%. Retard set Reduces early age (12h strength) Increase later age strength

2.2.7 Air Entraining Admixtures

Air entraining admixtures meeting ASTM C260 requirements are used to entrain microscopic air bubbles in concrete. Entrained air improves concrete durability by reducing the adverse effects of freezing and thawing. The volume of entrained air necessary for good durability varies according to the severity of the environment and the concrete's maximum aggregate size. Mixtures with larger coarse aggregates usually have less mortar and require less air than those with smaller maximum aggregate sizes. Typically, concrete mixtures have 4.5 to 7.5% total air content (Young and Mindess 1981).

Air entrainment is as necessary for accelerated concrete mixtures as for normal setting mixtures in freeze thaw environments. During field mixing, it is important to use the appropriated air-entraining admixture dosage rate so that the air content is adequate after placement. Higher percentages of entrained air can reduce the early and long-term strength of the admixture, while lower percentages may reduce the concrete durability. Therefore, close of air content is necessary for successful projects.

2.2.8 Curing and Temperature Management

Curing provisions are necessary to maintain a satisfactory moisture and temperature condition in concrete for a sufficient time to ensure proper hydration (Kosmatka and Panarese 1998). Internal concrete temperature and moisture directly influence both early and ultimate concrete properties. Therefore, applying curing provision immediately after placing and finishing activities is important. Even more so than with standard concrete, curing is necessary to retain the moisture and heat necessary for hydration during the early strength gain of accelerated-concrete pavement. Accelerated pavement required especially through curing protection in environmental conditions of high temperature, low humidity, high winds, or combinations of these.

Air temperature, wind, relative humidity, and sunlight influence concrete hydration and shrinkage. These factors may heat or cool concrete or draw moisture from exposed concrete surfaces. The subbase can be a heat sink that draws energy from the concrete in cold weather or a heat source that adds heat to the bottom of the slab during hot, sunny weather.

Monitoring heat development in the concrete enables the contractor to adjust curing measures to influence the rate of strength development, the window for sawing and the potential for uncontrolled cracking. Monitoring temperature when environmental or curing conditions are unusual or weather changes are imminent is particularly important (Shilstone 1988). Maturity testing allows field measurement of concrete temperature a correlation to concrete strength.

2.2.8.1 Curing Compounds. Liquid membrane-forming curing compounds should meet ASTM C309 material requirements. Typically, white-pigmented compound (type 2, class A) is applied to the surface and exposed edges of the concrete pavement. The materials create a seal that limits evaporation of mixing water and contributes to thorough cement hydration. The white color also reflects solar radiation during bright days to prevent excessive heat build up in the concrete surface. The Class A-liquid curing compounds are sufficient for accelerated-concrete paving under normal placement conditions when the application rate is sufficient.

Agencies that build concrete pavements in mountainous and arid climates often specify a slightly heavier dosage rate of resin-based curing compound meeting ASTM C309, type 2, class B requirements. The hasher climate causes dramatic daily temperature changes, often at low humidity labels. As a result, the concrete is often more susceptible to plastic-shrinkage cracking and has a shorter window for joint sawing.

Most conventional paving specifications require an application rate around 5.0 m^2/L (200 $ft^2/gal.$). Accelerated concrete pavement mixtures rapidly use mixing water during early hydration and this may lead to a larger potential for plastic shrinkage at the surface. Therefore, increasing the application of curing compound for accelerated paving projects to about $3.75 m^2/L$ (150 $ft^2/gal.$) is advisable. Because deep tinning increase surface area, the higher application rate also is important where surface texture tine depth exceeds about 3 mm (1/8 in). Bond overlays less than 150 mm (6 in) thick require an application rate of 2.5 m^2/L (100 ft^2/gal). The thin overlay slabs have a large ratio of surface area to concrete volume so evaporation consumes proportionately more mixing water than with typical slab.

The first few hours, while the concrete is still semi-plastic, are the most critical for good curing. Therefore, the contractor should apply the curing compound as soon as possible after final finishing. Construction and public vehicle tires may wear some of the compound off of the surface after opening, but this does not pose a problem because the concrete should have reasonable strength and durability by that time. Curing compound should apply in two passes at 90 degrees to each other. This will ensure complete coverage and offset wind effects, especially for tined surfaces.

2.2.8.2 Blanket Insulation. Insulating blankets provide a uniform temperature environment for the concrete. Insulating blankets reduce heat loss and dampen the effect of both temperature and solar radiation on the pavement, but do not neglect the need for curing compound (Grove 1989). The purpose of blanket insulation is to aid early strength gain in cool ambient temperature. Table 2.4 indicates when insulation is recommended.

Table 2.4 Blanket Use Recommendations (Federal Highway Administration, Washington D.C.)

Minimum ambient air temperature in period	**Opening time (hours)**				
	8	**16**	**24**	**36**	**48**
Less than 10°C (50F)	Yes	Yes	Yes	Yes	Yes
10°C to 18°C (50F to 65F)	Yes	Yes	Yes	Yes	No
18°C to 27°C (65F to 80F)	Yes	Yes	No	No	No
More than 27°C (80F)	Yes	No	No	No	No

Care should be taken not to place blankets too soon after applying a curing compound. In warm condition, waiting several hours and placing the blankets as the joint sawing progresses may be acceptable. In any case, it is inadvisable to wait until after finishing all joint sawing to start placing insulating blankets. Blankets are in maintaining the temperature of concrete compared to an exposed surface of the same mixture. Experience indicates that an insulating blanket with a minimum thermal resistance (R) rating of 0.035 m^2.K/W (0.5 h ft^2. F/Btu) is adequate for most conditions (Grove 1989). The blanket should consist of a layer of closed-cell polystyrene foam with another protection layer of plastic film. Additional blanket may be necessary for temperature below about 4°C (40°F).

Thermal shock may occur within a few hours after removing curing blankets from a new slab. It may be necessary to remove only the blankets needed to allow joint sawing. Blanket should not be completely removed until after completion of all sawing to eliminate uncontrolled cracking from thermal shock.

2.2.8.3 Plastic Shrinkage. The temperatures of accelerated paving mixtures often exceed air temperature and require special attention to avoid plastic-shrinkage cracking. Plastic-shrinkage crack can form during and after concrete placement when certain prevailing environmental conditions exist. The principal cause of plastic shrinkage cracking is rapid evaporation of water from the slab surface. When this occurs while concrete is in a plastic or simplistic state, it will result in shrinkage at the surface. Air temperature, relative humidity, wind velocity, and concrete temperature influence the rate of evaporation. The tendency for rapid evaporation increases when concrete temperature exceeds air temperature. Among the ways to moderate the environment and cool concrete components to slow evaporation are pave during the evening or nighttime, water-mist aggregate stockpiles and subbases before paving and use and evaporative retardant (monomolecular compound) on the surface. When the evaporation rate exceed 1.0 $kg/m^2/h$ (0.2 $lb/ft^2/h$), plastic-shrinkage cracking is likely. As a precaution, closely monitor and adjust the field curing practice if the evaporation rate exceed 0.5 $kg/m^2/h$ (0.1 $lb/ft^2/h$). For misting immediately after placement may be needed to prevent plastic-shrinkage cracking (Kosmatka and Panarese 1998).

2.2.8.4 Sawing and Sealing. The sawing window is a short period of time after placement when the concrete can be cut successfully before it cracks. The window opens when concrete strength is acceptable for joint cutting without excessive raveling along the cut. The window closes when significant concrete shrinkage occurs and induces uncontrolled cracking, unless sawing is done in time.

Sawing must be completed before the concrete shrinks and significant restraint stress develops. Drying shrinkage occurs partly from moisture consumption through hydration and moisture loss to the environment (Okamoto et al. 1994). Thermal contraction and curing restraint stress occur as the concrete temperature begins to fall and the top of the slab cools more rapidly than bottom. For accelerated concrete paving, it is preferable to complete sawing before the concrete surface temperature begins to drop after initial set.

After the concrete sets, uncontrolled cracking might occur when conditions induce differential concrete shrinkage and contraction. Differential shrinkage is a result of temperature differences throughout the pavement depth. Normally, the concrete surface temperature drops before the temperature at middepth or bottom. The temperature at middepth usually remains warm for the longest period. The temperature differential may be enough to causes cracking. A drop in surface temperature more than 9.5°C (15 F) can result in excessive surface shrinkage and induce cracking if sawing has not been completed (Okamoto et al. 1994).

Early age saws allow cutting very early during the initial concrete set stage. Cutting is feasible after compressive strength reach about 150 psi (1.0 MPa). All cutting should be done before final set of the concrete. Most currently available early age saws provide only a shallow initial cut of about ¾ to 1 ¼ in and require a second cut using a standard saw for a sealant reservoir or to meet typical specifications of saw cut depth of 0.33 or 0.25 of the slab thickness.

2.3 The Maturity Concept

The maturity method accounts for the combined effects of time and temperature on concrete strength development. The strength of a given concrete mixture that has been properly placed, consolidated, and cured, is a function of its age and temperature history.

The maturity concept has been around for many years and has proven to a useful tool in several specialties within the concrete industry. This is especially true among fabricators of prestressed and other prefabricated concrete products where it is essential to obtain strength information at an early date and with minimal cost.

Early research was conducted by Saul (1951) who introduced and defined the term "maturity" as follows during his investigation on steam curing of concrete: "the maturity of concrete may be defined as its age multiplied by the average temperature above freezing that it has maintained." Over the years, Saul's work has been confirmed and refined by other researches (Carino and Tank 1992), (Nurse 1949).

Maturity testing provides strength evaluation by the monitoring of internal concrete temperature in the field. The basis of maturity is that each concrete mixture has a unique strength-time/temperature relationship. Therefore, for concrete of a specific mixture, the same strength will develop at a given maturity value.

The maturity value is the sum of the degree-hours from initial concrete placement to given time during the curing process. The application of computer introduces a low cost method of analyzing the strength gain that is taking place in concrete structures or pavements. This is especially of interest for pavements, where it is essential to evaluate strength at early ages in order to determine the time at which they may opened to traffic.

2.3.1 Nurse-Saul Equation

The Nurse-Saul equation calculates the time – temperature factor (TTF) using following equation (ASTM C1074, 1998):

$$M(t) = \Sigma (T_a - T_0) \Delta t \tag{2.1}$$

Where

$M(t)$ = temperature-time factor (TTF) at time t, degree-days or degree-hours

Δt = time interval, days or hours

T_a = average concrete temperature during time interval, °C or °F

T_0 = datum temperature at which it is assumed that concrete ceases to gain strength with time; the value of –10°C(14F) is most commonly used for normal concrete and 6.5°C for VES concrete or laboratory result.

This equation (Nurse-Saul) is the most popular procedure in use by state DOTs. When this equation is used, the concrete strength is related to the logarithm of temperature time factor (TTF).

2.3.2 Arrhenius Equation

The Arrhenius equation is used to calculate the "equivalent age" maturity index. Equivalent age represents the equivalent duration of curing at the reference temperature that would result in the same value of maturity as the curing period at a given average temperature (ASTM C1074, 1998).

$$t_e = \Sigma e^{-Q}\left(\frac{1}{T_a} - \frac{1}{T_s}\right) \Delta t \tag{2.2}$$

Where

t_e = equivalent age at standard or reference temperature, days or hours

e = 2.718

Q = apparent activation energy divided by the gas constant, (E/R) , $^{\circ}$K

E = apparent activation energy, J/mole

R = universal gas constant = 8.314 J/mole $^{\circ}$K

K = absolute temperature, Kelvin, K = $^{\circ}$C + 273.15

Ta = average concrete temperature during time interval, Δt, K

Δt = Time interval, days or hours

With this equation, a different function (hyperbolic rather than logarithm) is used to relate concrete strength to equivalent age. The Arrhenius equation is used less commonly for concrete pavement work in the United State. According to Carino (ASTM C1074, 1998), the Arrhenius relationship may be more appropriate when a wide variation in concrete temperature is expected.

2.3.3 Maturity Testing Procedure

The maturity method is a two-step process. First, a relationship is established between the maturity values and the concrete strength as measured by test of beams or cylinders. The development of the maturity-strength curve is done at the beginning of construction using project materials. The application covers only one mixture. If there are changes in material source, mix proportions, or mixing equipment, another correlation must be run.

Preliminary testing is necessary before technician can accurately analyze concrete in the field. Using the actual job mixture concrete materials, test specimens are prepared with thermocouples or microprocessors embedded in them. The temperature is monitored and beams or cylinders are broken to develop a relationship between the strength values and the temperature-time factor (TTF). The strength-maturity equation is developed by performing strength test at various ages, computing the corresponding temperature-time factor at the test ages, and plotting the strength as a function of the logarithm of the temperature-time factor. A best-fit line is then plotted through the data. Test data from one field project indicate that the maturity curves may be more reproducible when using compressive strengths rather than flexural strengths.

The second step is instrumenting and monitoring the concrete pavement. Temperature probes or microprocessors are embedded in the concrete and the temperature is measured periodically. Thermocouple wires are inserted to the desired depth in fresh concrete, shortly after placement.

2.3.4 Maturity Test Equipment

Maturity meters automatically monitor and record concrete temperature as a function of time. Acceptable devices include thermocouples or thermostats connected to strip chart recorders or digital data loggers. Commercially available devices automatically compute and display information that can be used as an index to strength, either flexural or compressive. Several maturity devices are available which continuously measure the concrete temperature and calculate the maturity automatically at least once every hour. The meter can also display the maturity value digitally at any point in time. Depending on the meter used, several different locations can be monitored simultaneously. Some maturity meters can be set to use either the Nurse-Saul or the Arrhenius equation (see Figures 2.2, 2.3 and 2.4).

Figure 2.2 Spectrum SP-1700 thermocouple temperature data logger.

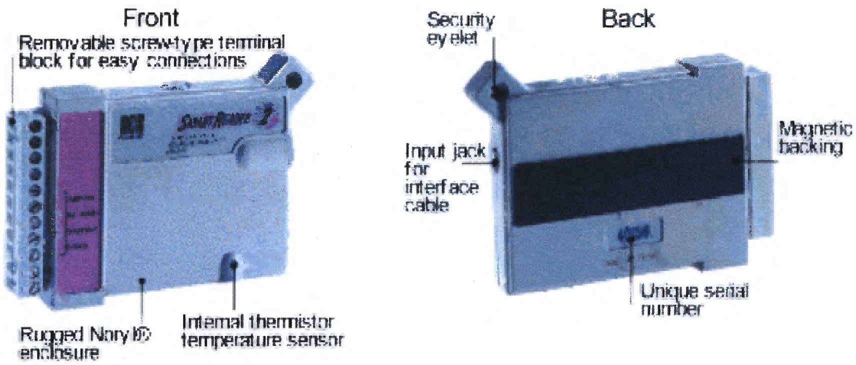


Figure 2.3 Data loggers ACR SmartReader 6 by ACR company.

Figure 2.4 Maturity system.

Figure 2.4 shows one of the popular maturity meters. This and similar meter are microprocessor-based, battery operated data collection systems. They typically have several channels for temperature measurement, and calculate the maturity value for each channel for many hours of elapsed time. The cost of maturity meter kit are around $1,300-$2,000.

There are six types available for thermocouple wire. Each type of wire will have different specification and cost. Choosing a proper thermocouple type for your application depends primarily on the temperature range in which its use is intended. The most widely used and available thermocouples are types "K "and "J", however there are several other types that you should consider. To determine which thermocouple type best suits your application's temperature range, refer to the individual data pages for each thermocouple type (J, K, T, E, R or S). These pages include a temperature range and accuracy graph you can use for comparison with your measurement requirements.

Table 2.5 Temperature Accuracy for Each Thermocouple Type by Veriteq Company SP-1700 Narrow range

Thermocouple	**Type K**	**Type J**	**Type T**	**Type E**	**Type R**	**Type S**
Temperature Measure range (°C)	-220-240	-130-180	-240-200	-110-150	-50-870	-50-900
Instrument Temp. accuracy	0.35	0.27	0.38	0.23	1.4	1.5
Resolution (°C)	0.10	0.08	0.11	0.07	0.40	0.43

2.3.5 Location of Test Probes

The location and depth of the thermocouple wire (or embedded microprocessor) are dependent on the use of the data. Cable has made the following recommendations (Cable 1998).

- The thermocouple should always be placed at least 300 mm (1 ft) from the edge of the pavement.
- Decision regarding the timing of joint sawing should be based on data from a thermocouples placed with in 25 mm (1 in) of the concrete surface.
- Measurements taken at mid-depth of the concrete are useful in the determination of he average strength of the slab and should be used in the determination of pavement opening times.
- Thermocouples can be mounted easily on wood dowels and inserted to the required depth in the concrete. They are left in the slab after measurements have been completed.
- The thermocouples should be inserted at longitudinal interval of 150 mm to 300 mm (500 ft to 1,000 ft) to account for variations in placement time along the project and to provide estimates of the best time to saw joints in each interval or section.

2.4 Concrete Cracking

2.4.1 Plastic shrinkage

Loss of water from fresh concrete, if not prevented, can cause cracking. The most common situation is surface cracking due to evaporation of water from the surface, but suction of water from the concrete by the subbase or by formwork materials also can cause cracking or can aggravate the effects of surface evaporation. In fresh concrete, the space between particles is completely filled with water. When water is removed from the paste by the exterior influences, such as evaporation at the surface, a complex series of menisci are formed. These generate negative capillary pressures, which will cause the volume of the paste to contact. Capillary pressures continue to rise within the paste until a critical "breakthrough" pressure at which point the water is no longer evenly dispersed through the paste and rearranges to form discrete zones of water with voids between. The maximum rate of plastic shrinkage occurs just prior to the breakthrough pressure and little shrinkage occurs afterwards.

Plastic shrinkage cracking is most common on horizontal surfaces of pavements and slabs were rapid evaporation is possible, and its occurrence will destroy the integrity of the surface and reduce its durability. It is aggravate by a combination of high wind velocity, low relative humidity, high air temperature, and high concrete temperature. If the rate of surface evaporation exceeds 0.5 $kg/m^2/h$ (0.1 $lb/ft^2/h$), loss of moisture may exceed the rate at which bleed water reaches the surfaces, creating negative capillary pressure, which cause plastic shrinkage. Concrete containing admixtures that reduce the rate of bleeding are particularly susceptible to plastic shrinkage cracking.

2.4.2 Drying Shrinkage

The term drying shrinkage is generally reserved for hardened concrete. It represents the strain caused by the loss of water from the hardened material. Autogenous shrinkage, which occurs when concrete can self desiccate during hydration. Carbonation shrinkage, which occurs when hydrated cement reacts with atmospheric carbon dioxide, also can be considered as a special case of drying shrinkage. Shrinkage is a paste property; in concrete, the aggregate has a restraining influence on the volume change that will take place within the paste. **Autogenous Shrinkage** (chemical shrinkage): If no additional water added during curing, concrete will begin to dry internally, even if no moisture is lost to the surroundings as water is consumed by hydration. However, bulk shrinkage is only observably in concrete with low w/c ratio (less than 0.3) and is increased by the addition of reactive pozzolans (silica fume).

The phenomenon is known as self-desiccation and is manifested as autogenous shrinkage. In extreme cases, the internal relative humidity can drop to 75%-80% relative humidity. Thus, this shrinkage is a special case of drying shrinkage, since it is immaterial whether the water is removed by physical or chemical process. Autogenous shrinkage will only occur even if the concrete is sealed or in dense concrete, low W/C and the addition of silicafume. Self-desiccation may occur to some extent, even if water is supplied during the curing process, because external water can not easily penetrate the concrete. Otherwise, any effects of self-desiccation are usually masked by expansion associated with the formation of ettringite or the hydration of free MgO.

Carbonation Shrinkage: Hardened cement paste will react chemically with carbon dioxide. The amount present in the atmosphere (0.04%) is sufficient to cause considerable reaction with cement paste only over a long period of time. However, this is accompanied by irreversible shrinkage, and hence it is called carbonation shrinkage. The extent to which cement paste can react with carbon dioxide, and hence undergo carbonation shrinkage, is a function of relative humidity and is greater around 50% relative humidity. At high humidity, carbonation is low because the poses are mostly filled with water and CO_2 can not penetrate into paste very well. At very low humidity, an absence of water films is believed to lower the rate of carbonation. Carbonation shrinkage is greatest when carbonation occurs after drying, rather than during drying, except at low humidity.

Concrete exposed to carbonation loses water and behaves as though it has been dried to a much lower relative humidity than that to which it is actually exposed. The shrinkage-water loss relationship is similar to that observed for normal drying. Furthermore, carbonation shrinkage is wholly irreversible. It is believed that CO_2 reacts with C-S-H inducing a decrease in its C/S ratio and a concomitant loss of water. Carbonation of C-S-H is known to change the bonding characteristics of the material, which could account for the irreversible nature of accompanying shrinkage. Thus, carbonation can views as promoting changes in C-S-H that normally only occur at much lower relative humidity. Calcium hydroxide also will form calcium carbonate by reacting with atmospheric CO_2.

Carbonation shrinkage can be important from a practical point of view. Advantage can be taken of it irreversible nature for precast concrete. For example, by exposing concrete block to CO_2-rich air (to hasten the carbonation process), the block can be made much more dimensionally stable to subsequent wetting and drying. On the other hand, carbonation can be detrimental to cast-in-place concrete. Since it is much less porous than block, carbonation can occur only near the outside, precisely where the maximum rate of drying is also occurring. Thus, carbonation shrinkage is likely to be maximized and, when added to drying shrinkage, may cause severe shrinkage cracking.

2.4.3 Control of Cracking

Cracking is best controlled during the design and construction phases. In many instance cracking may be avoided by proper selection of materials, provided that the potential problem has been anticipated through a careful assessment of the expected environment.

Chemical attack of concrete involves ingress of moisture, either as a carrier for aggressive agents or as a participant in destructive reactions. Thus, precautions in mix design and construction practices that prevent the entry of water into and passage through concrete should improve durability. Concrete of low permeability can be assured by the use of sufficient (but not excessive) quantities of cementitious and adequate moist curing. Provisions for effective drainage and the use of watertight construction joint can be vary beneficial in some situations.

Two types of cracking are often observed in plastic concrete. A plastic shrinkage cracks occurs when moisture is removed more rapidly from the surface of fresh concrete than it is replaced by bleedwater from below. When it faces with high temperatures, high winds, and low relative humidity, construction procedures and protection for the surface of slab must be adjusted to limit the potential for rapid evaporation. Settlement cracks, which occur above and parallel to reinforcing bar near the surface of concrete, can provide a ready path for corrosive chemicals to reach the reinforcement. The extent of settlement cracking will be decreased with reduced concrete slump, increased cover, and reduced reinforcing bar size.

Cracking due to drying shrinkage and thermal expansion is caused by tensile stress that are created by differential strain that occur under nonuniform drying, temperature rise, or uneven restrain. Thus, shrinkage and thermal cracking resembles flexural cracking and can be controlled by suitable location of reinforcement, which will reduce the amount of cracking and will cause several fine cracks rather than a single large crack. The finer the crack, the less likely it is to contribute to durability problems. Crack widths less than 0.1 mm (0.004 in) are desirable in cases where severe exposure is anticipated.

The use of properly located isolation and contraction joints helps reduce stress due to expansion and contraction of concrete. Rigid joints should be adequately designed to accommodate additional stress that may be caused by moisture or temperature changes. Shrinkage cracking in walls and slabs on ground usually can not be completely avoided, necessitating the use of contraction joints. These are grooves cast into the concrete or sawn soon after hardening that provide planes of weakness where the crack will form preferentially. In this way, random cracking is avoided. The cracks are located in a manner that allows them to be sealed against moisture. More contraction joints must be cut than are actually needed because the amount of restraint is not known. Cracked joints must be regularly maintained. For this reason, strategies to eliminate contraction joints are pavements have been sought; the use of expansive cements, continuously reinforced or prestressed pavements, or fiber reinforced concrete is possible solution.

Many cracks in concrete structures are due to poor construction practices or errors in design and detailing. Examples of the former are the use of high w/c ratios and inadequate curing. An example of the latter is the use if reentrant corners in structure walls and slabs-on-grade without the provision of reinforcement to limit the width of the crack or proper joint detailing to control crack location.

Cracking may be controlled if the magnitude or rate of environmental changes can be reduced. In this way, tensile stresses will be lowered and may be further reduced by tensile creep and, in the early life of a structure, by an increase in tensile strength. Temporary protection of concrete or the use of reflective or impermeable coating can make changes.

The cracking is a complex process that depends on a large number of factors. The major parameters are: i) magnitude of drying shrinkage, (ii) tensile strain capacity of concrete, (iii) restraint that prevents free movement to accommodate shrinkage and, (iv) loading conditions that cause tension fatigue. A number of test setups have been developed to measure the cracking potential of concrete (Balaguru and Shah 1993).

2.4.4 Cracking Measurement

The measuring cracking potential are three different shapes (linear, plate, and ring) have been tried for measuring the contribution of fibers to shrinkage crack reduction. Ring specimens seem to have better potential because they can be provided with good restraint. The arrangement is also conductive to the development of mathematical models

Linear Specimens: The test specimens consist of long prisms with flared ends. The lateral dimensions 0.2 in x 4.8 in. (5 mm x 120 mm) are small compared to the longitudinal dimension 60 in. (1500 mm). Restraints are applied at the flared ends, inducing cracks in the middle (uniform) section. The method was successfully used for studying the contribution of steel fibers.

Another way to provide restraint is to use reinforcing bars in the center of the matrix. The center part of the bar was debonded using a rubber tube. Hence the midsection of the specimen is subjected to tension created by the shrinkage. The specimen size was 2 x 2 x 12 in. (50 x 50 x 300 mm).

In another study, prismatic concrete specimens were glued to a stiff steel frame. The specimen dimensions were 500 x 80 x 20 mm (20.0 x 3.2 x 0.8 in.). The ends were held in position by the rigid frame. When the composite started to shrink, the reduction in length created the tensile stresses.

In these three types of specimens, stress distribution at the restrained ends is rather complex. Therefore, it is difficult to formulate an analytical model for the prediction of tensile stresses and crack widths that develop because of shrinkage strains.

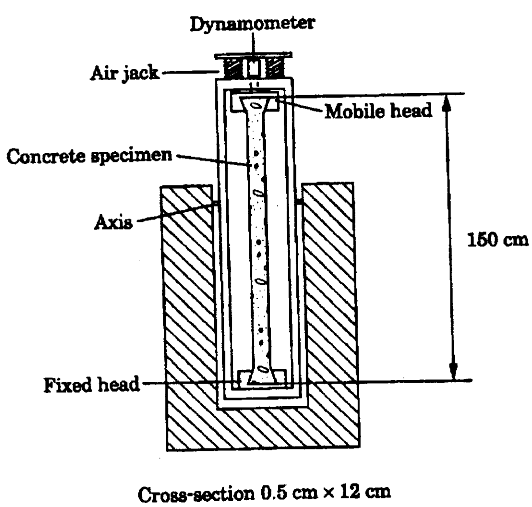
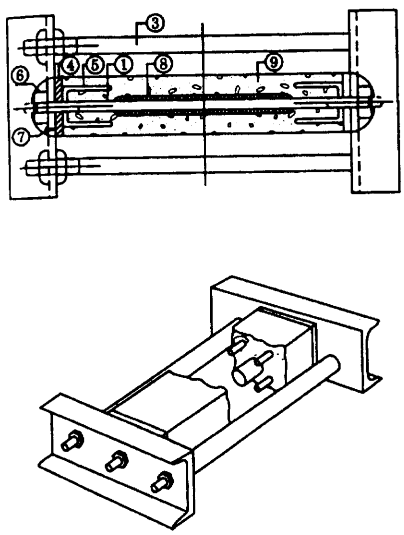


Figure 2.5 Restrained shrinkage test: Restraints applied using external grips (Paillere et al. 1989).



1	Center reinforcing bar, 3/8"Ø or ¼"Ø
2	Channel
3	Rigid bar of the frame, 3/4"Ø
4	End plate
5	Bonding bar, 3/16"Ø
6	Fasten screw

Figure 2.6 Restrained shrinkage test: Restraints applied using bar embedded in the specimen (Pan et al. 1987).

Plate Specimens: Plate specimens, similar to the ones used for plastic shrinkage, have also been tried for measuring cracks caused by drying shrinkage. The restraints were provided by means of stirrups attached to rigid steel frames. The primary difficulty with this kind of setup is to estimate the actual extent of restraint provided by the stirrups. Hence, the tests could be used only to make qualitative judgments among the various types of fibers.

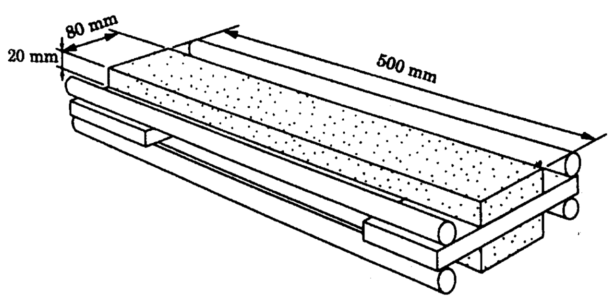


Figure 2.7 Test setup for linearly restrained shrinkage (Pihlajavaara et al. 1978).

Ring specimens: Ring specimens were used by a number of investigators for evaluating fiber-reinforced cement composites under restrained drying shrinkage. Essentially, a ring of concrete is cast around a stiff steel ring. As the composite shrinks, it induces stresses on the steel ring. Since the steel ring is stiff and undergoes very little deformation, the outer cement composite ring is subjected to tension. If the concrete ring is thin in relation to the internal diameter, then the stresses across the thickness can be considered uniform. The compressive stress developed at the interface between the steel ring and the concrete ring is also negligible. The researchers used various external diameters for steel rings.

The thickness of the cement composite was also varied depending on the composition of the matrix. Typically, thicker sections were used with concrete containing coarse aggregates.

As mentioned earlier, this setup shows the most promise because of the uniform restraint provided by the steel ring. The restraining force is imposed by the steel ring across the perimeter of the concrete, instead of two or four locations as with linear and plate specimens. The method is described in details in the following paragraphs.

The variation of stresses across the thickness of the concrete ring depends on the internal diameter of the ring. For the dimensions shown in Chapter 3, the difference between the values of tensile hoop stress on the outer and inner surface is only 10% (Grzybowski 1990). In addition to hoop stress, the concrete ring is also subjected to radial compressive stress when the steel ring exerts radial pressure. Since the diameter of the ring is relatively large, this radial compressive stress is only 20% of maximum hoop stress. Since cement composites are an order of magnitude stronger in compression, the maximum compressive stress in the ring is only about 2% of the compressive strength. Hence, the effect of compressive stresses can be neglected.

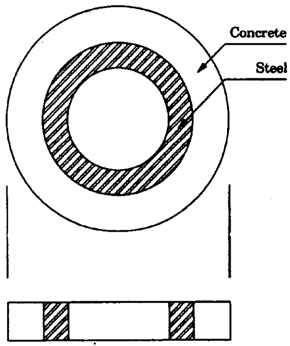


Figure 2.8 Schematic view of a restrained ring shrinkage setup (Grzybowski, 1989).

The shrinkage of the unrestrained specimen, known as free shrinkage, should also be measured for estimating the amount of stresses and crack widths. Measurements taken using an unrestrained ring (in which the inner steel tube was removed) and prismatic Specimens show very little difference (Figure 2.9) Bar specimens recorded slightly larger strains, which should be expected, because of larger exposed areas.

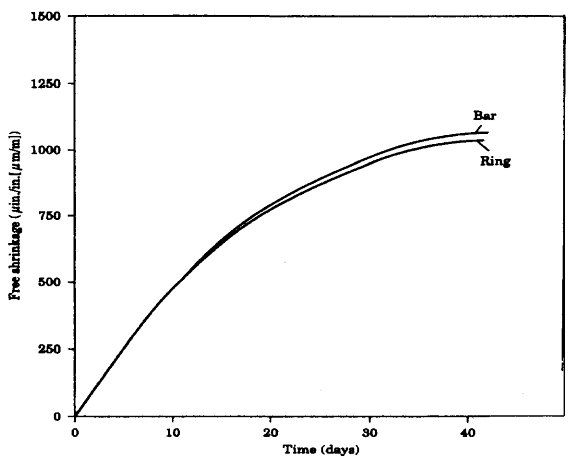


Figure 2.9 Comparison of the results between ring and prismatic bar specimens for plain concrete (Grzybowski 1989 and 1990).

CHAPTER 3

EXPERIMENTAL PROGRAMS

3.1 Development of Optimized Mix and Its Maturity Parameters

The NJDOT's approved mix in 1996 was used to be a control mix. The properties of mix design are shown in Table 3.1. According to the objectives, the optimization will improve the Very Early Strength (VES) concrete mix without significantly reducing its strength gain performance. Trial and error mix designs were applied based on the acceleration technique in order to decrease the construction cost and cracking problem due to drying shrinkage. The maturity parameter were developed for the developing mixs.

Table 3.1 The Approved Mix 799 lb/yd^3 of Cement in 1996 (Ansari and Luke 1996)

Cement	799	Lb/yd^3
Coarse aggregate (3/4")	1,800	Lb/yd^3
Fine aggregate	1,200	Lb/yd^3
Water	325	Lb/yd^3
Water reducer	16	oz/c.wt
Air Entraining	1.0	oz/c.wt
Rapid Hardening	32	oz/c.wt

3.1.1 Test Specimen and Instrument

3.1.1.1 Materials. From the research experiment by Ansari and Luke in 1996 report to NJDOT, they found that only two types of cement in New Jersey can reach the target strength. Two brands of type I cement, namely "Essroc cement" and "Lafarge cement" were used for this experiment. The coarse aggregate used in this study was composed of crushed granite. There were two sets of coarse aggregate. The maximum nominal size of the first aggregate was 1½ in. and the other was ¾ in. The fine aggregate came from the river sand and had a fineness modulus ranging from 2.45 to 2.55. Three types of admixtures, High range water reducer Type F, Hardening Accelerator Type C and Air entraining admixture, were used in this study.

Cement:

- ESSROC Type I (provided by ESSROC Company)
- LAFARGE TYPE I (provided by Lafarge Company)

Chemical Admixtures:

- Sika Set NC (Non-Chloride Set Accelerator) Recommended dosage : 15-45 Oz/Cwt
- Sika Rapid-1(Non-Chloride Hardening Accelerator) Recommended dosage : 15-45 Oz/Cwt
- Sikament 86 (High Range Water Reducer)
- Sika AER (Air Entraining Agent)

Fine Aggregate: Local siliceous sand (river sand) was used. The percentage of absorption is 0.4 and specific Gravity is 2.66. Dry rodded unit weight is 106 lb/ft^3.

Coarse Aggregate: Artificially crushed rock ¾-in and 1½-in sizes were used. The percentage of absorption is 0.8 and specific gravity is 2.86.

3.1.1.2 Cylinder and Beam Test. 4 in x 8 in and 6 in x 6 in x18 in were used for conducting compressive and flexural strength tests. Figure 3.1 shows the MTS machine, which is used for measuring the compressive strength (ASTM C 39) and flexural strength (ASTM C 78). Figure 3.2 shows a device for measuring the set time of concrete following the ASTM C 405.

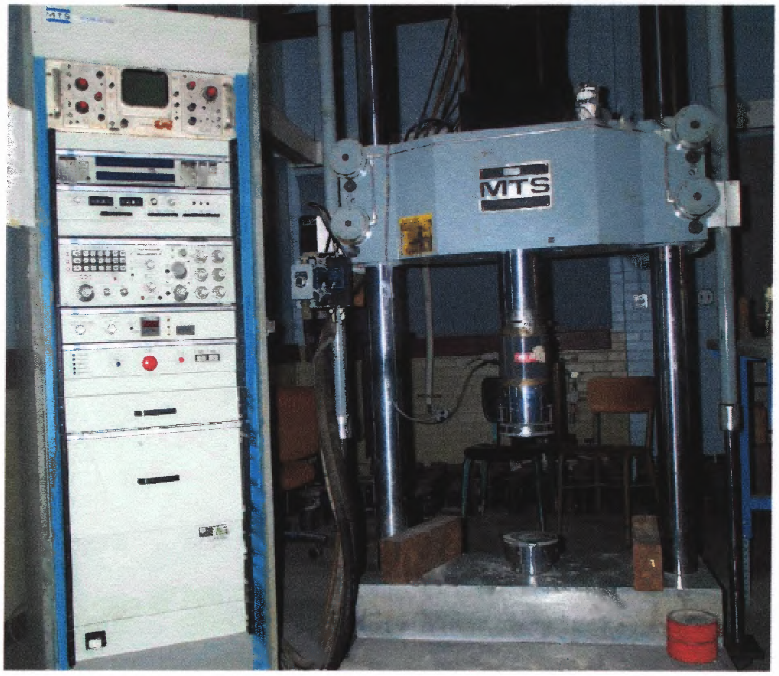


Figure 3.1 The MTS machine.

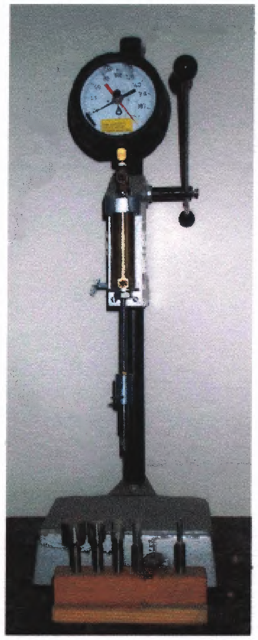


Figure 3.2 Set time equipment.

3.1.1.3 Maturity Apparatus. The maturity measurement tool composed of four importance things. The data-logger was used to automatically collect and record the temperature from specimens. The thermocouple was used to be a conductor. The computer software and hardware was used to calculate the raw signal data to maturity value.

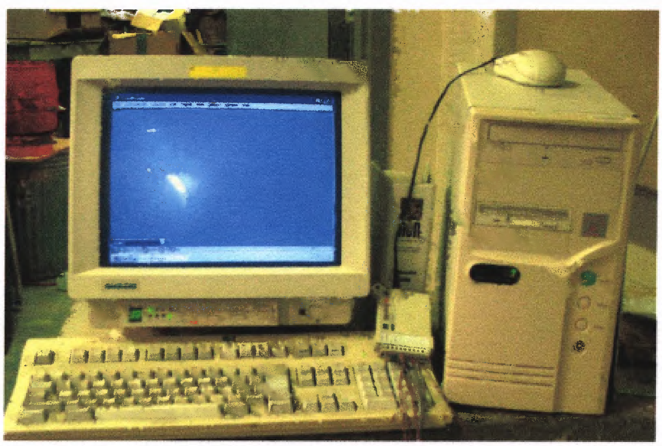


Figure 3.3 ACR Model 6 Data-logger connecting with T-type thermocouples.

The Humbolt H-2680 maturity meter: ACR Model 6 Data-loggers conforming to the requirements of ASTM C 1074 is used at present investigation. This device can measure temperatures within the range of -20°C to 100°C, with an accuracy of ±1°C. Measurements need to be recorded at least every half-hour for the first 48 hours, and hourly thereafter. The data file must be available for analysis and archiving. It should be rugged enough for the field use or be carefully protected.

T-type thermocouples : These temperature sensors are accurate to ±1°C, compatible with the chosen apparatus.

A computer for programming : A computer program is needed for recovering, saving, and analyzing the data. Devices that display maturity values directly are permitted only as long as the temperature history upon which the values are computed, and notes on maturity parameters used, are available.

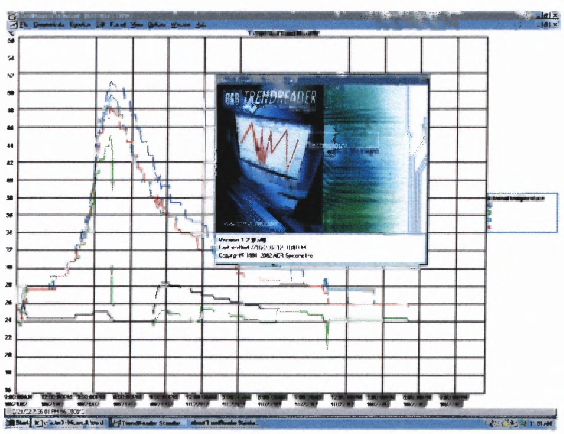


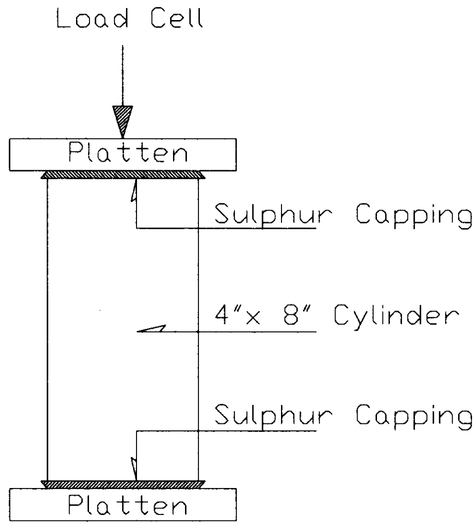
Figure 3.4 ACR Trend Reader software.

Microsoft's Excel, ACR's Trend Reader: The maturity and strength value were calculated using Microsoft's Excel. The collecting temperature data of specimens from ACR Trend Reader Software was transfer to Microsoft's Excel work sheet.

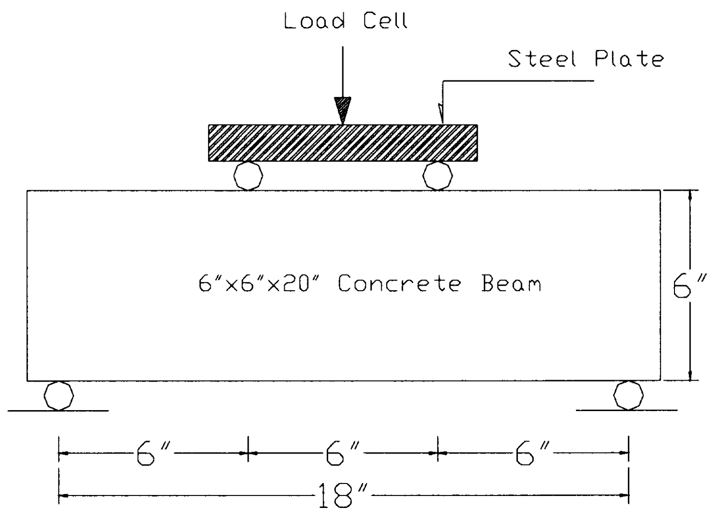
3.1.2 Test Setup and Experimental Procedure

Control Mix was conducted to study the mix properties, maturity parameter and the shrinkage value. The shrinkage issues will be further discussing. The control mix (Table 3.1) was adjusted to Mix No. I, which maintained the amount of cement at 799 lb/yd^3 by decreasing the w/c and others as seen in Chapter 4. The Mix No. II (658 lb/yd^3) was proposed by decreasing the amount of cement from 799 lb/yd^3 to 658 lb/yd^3. The Mix No. III (658 lb/yd^3) was used to improve the cracking problem by increasing the size of coarse aggregate. The mixed design was to be adjusted accordingly. They also can be found in Chapter 4.

Compressive and flexural strengths: For each mix, at the beginning of the test, the cone slump test and the air content were performed in accordance with ASTM C 143 and ASTM C 231, respectively. The mortar specimens using the container specified in the ASTM Penetration Method C 403 were performed for the set time test. A maximum of sixteen 4 in ×8 in. cylinders and four 6 in x 6 in x 18 in. beams were cast. The cylinders were cast in three layers and rodded with a rodded as described in ASTM C 192. The beams were cast in two layers and rodded as illustrated in ASTM C 192. After casting, all specimens were moved to curing room and covered with insulation blanket. The cylinders were tested on their compressive strengths at ages of 6, 6.5, 7, 7.5, 8, 24 and 672 hours. The beams were performed by the test as described in ASTM C 78 at the level of compressive strength reaching 2,500 psi or seven hours after mixing operation and at the age of 24 and 672 hours. Details of the compressive and flexural strength tests are shown in Figures 3.5 and 3.6 respectively.



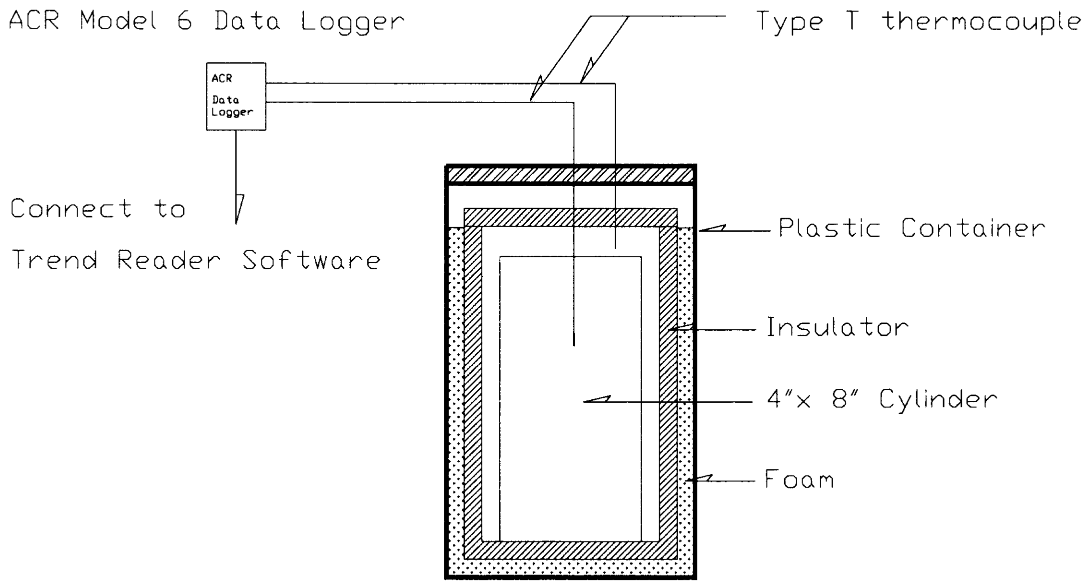
Figures 3.5 Compressive strength test setup.



Figures 3.6 Flexural strength test setup.

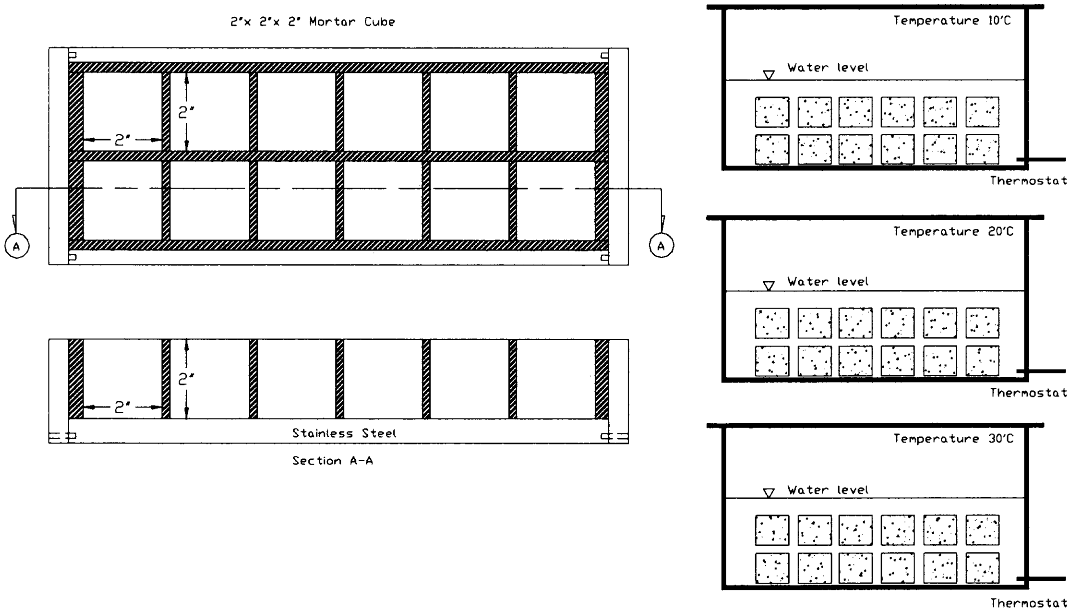
Maturity: Two of cylinders were embedded with thermocouples in order to conduct the maturity parameter in the concrete sample as described in ASTM C 1074, the Maturity Method. All of the cylinders were cured in a heat protection chamber. The data aquarsition and ACR system were used to collect the temperature of concrete.

The Nurse-Saul equation was used to calculate the time – temperature factor (TTF) using $M(t) = \Sigma (T_a - T_0) \Delta t$. Detail of the maturity test setup is shown in the Figure 3.7. The concrete maturity versus compressive strength for each mix was developed. Each of them represents the properties of each mix that will be used to predict the onsite compressive strength.



Figures 3.7 Maturity test setup.

Datum temperature: The six mortar containers specified in the Test Method ASTM C109 were poured with the mortar. Two of them were cured in the 33±2°C curing tank, other two tanks were cured in 22±2°C curing tank and the rest were cured in 10±2°C curing tank in order to determine the datum temperature. All of them were tested at 1, 2, 4, 7, 14, and 28 days. Details of the datum temperature test setup is shown in Figures 3.8. However, for convenience, 6.5°C will be used as the datum temperature in this study.



Figures 3.8 Datum temperature test setup.

3.2 Effect of The Local Cement Brand on Early Strength of Concrete

Ansari and Luke report to NJDOT in 1996 stated that only two types of cement in New Jersey were studied for their target strengths. Two brands, namely Type I "Essroc cement" and "Lafarge cement" were studied at present experiment. The mix designs in Table 3.2 were used to compare the results of flexural strength between two local cement brands. The mix design in Table 3.2 was the optimized mix which was developed from the mixed design as seen in Table 3.1.

Three batches were prepared for 6 in x 6 in x 18 in. beams. The first batch was set up using the "Essroc cement". The second batch was set up using the "Lafarge cement". The first and second batch specimens were cured in the Moisture room (73°F).

The third batch was set up in a different condition curing using the "Lafarge cement" and cured in chamber, which had a controlled temperature of 92°F. The flexural strengths of those batches were attained at 6.5 and 7.0 hours. The result are shown and discussed in Chapter 4.

Table 3.2 The Optimized (658 lb/yd^3) Concrete Mix Design

Cement	658	Lb/yd^3
Coarse aggregate (3/4")	1,850	Lb/yd^3
Fine aggregate	1,350	Lb/yd^3
Water	230	Lb/yd^3
Water reducer	12	oz/c.wt
Air Entraining	0.5	oz/c.wt
Rapid Hardening	40	oz/c.wt

3.3 Early Strength Development in Insulated Specimens

Objective of this part studies the effect of the insulation curing on early compressive and flexural strengths of concrete. This batch was conducted to study the effect of heat protection on very early strength concrete. This batch consisted of twelve– 4 in x 8 in cylinders and two - 6 in x 6 in x 18 in beams. They were mixed following the ASTM C192 and curing in the moist room, which had the same condition as in the ASTM procedure. The insulator was applied to prevent the heat loss from specimens. The cylinder and beam were stored inside the Styrofoam insulation up to the testing period. The compressive strength (ASTM C39) was attained at 6, 6.5, 7.0 and 24 hours.

The flexural strength (ASTM C78) was obtained at six and half hours and seven hours. Details of insulated specimens test setup are shown in Figure 3.9.

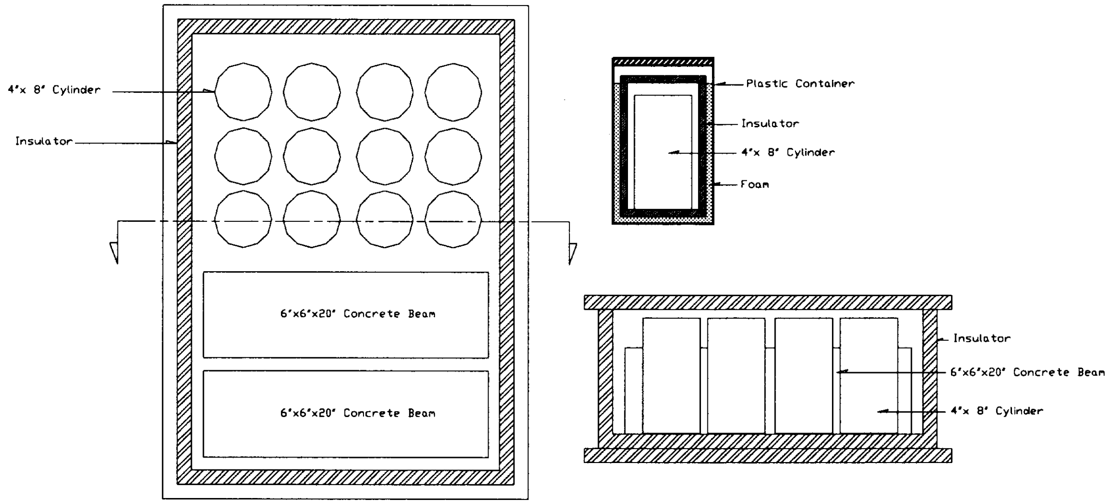


Figure 3.9 Detail of insulated specimens test setup.

3.4 Effect of Initial Mixing Temperature on Early Strength of Concrete

The same mix designs shown in Table 3.2 were used to compare the early strength of concrete using three different initial temperature conditions. The Essroc cement was used in this experiment. Three batches were set up. The setting time test and compressive strength test were performed. The 6 in x 6 in x 6 in mold was prepared for the setting time test and eight-cylinder each was conducted to determine their compressive strength at 6.5, 7.0, and 8.0 hours, respectively. The thermocouple and data acquisitions were set up to monitor the temperature of specimens. The chamber temperature was controlled by a thermostat, which was connected to the water in the chamber. Mixing water was used to control the initial temperature of concrete. The initial temperature was inspected by a thermometer during mixing process and after casting.

The first batch was set up to control initial temperature at 81°F, 85°F, 90°F, respectively. The flexural strength of those batches was determined at 6.5 and 7.0 hours, respectively. The result of this study can be found in Chapter 4.

3.5 Effect of Mixing Method on Early Strength Concrete

The object of this study is to study the early compressive strength and flexural strength using the ASTM C192 "Making and Curing Concrete Test Specimens in the Laboratory" and Non-ASTM. The initial mixing temperature at 81ºF (27ºC) was controlled.

ASTM C192 mixing concrete: The coarse aggregate, some of the mixing water and the solution of the admixture were added into mixer prior to start rotation of mixer. The admixture was dispersed into the mixing water before addition. After starting the mixer, the fine aggregate, cement, and water were added while the mixer was running. These components may be added to the stopped mixer after permitting it to turn a few revolutions following changing with coarse aggregate and some of the water. The mixer was continuous mixing after all ingredients were in the mixer for three minutes, followed by three-min rest and followed by two-min. final mixing. The mixer was covered at the open end or top of the mixer to prevent evaporation during the rest period.

Non ASTM C192 mixing concrete: The fine aggregate, Air Entraining admixture (AER) and cement were added into mixer prior to start rotation of mixer. The mixer was rotated for one to two minutes. The admixture was dispersed into the mixing water before addition. Some of water was added while the mixer was running. The coarse aggregate and some water were added while the mixer was running.

The mixer was rotated after all ingredients were in the mixer for three to five minutes or until clearly a uniform concrete was achieved.

The twenty-four of 4 in x 8 in cylinders and four of 6 in x 6 in x 18 in beams were prepared for this study. The mix design from Table 3.2 and Essroc cement were used. Two different concrete mixing methods mentioned above were used to study the effect of mixing methods on early compressive strength and flexural strengths. Each batch of specimens consisted of 12–4 in x 8 in cylinders and 2–6 in x 6 in beams. The first batch was mixed following the ASTM C192 while the second batch was mixed following the non-ASTM C192.

The fresh concrete was controlled at the initial mixing temperature of 81°F (27°C) using the 81°F- mixing water. The ambient temperature in mixing room was 23°C. The second batch was conducted testing within one-hour after the first batch.

After placing, all specimens were put in the curing room which had temperature of 73.4+3°F or 23+1.7°C and 100% Relative humidity. All specimens were cured until testing period. According to ASTM C192, the concrete specimens were not covered with blanket or any insulator in the curing room.

In addition, another batch was conducted to study the effect of heat protection on very early strength concrete. This batch was composed of 12 – 4 in x 8 in cylinders and 2- 6 in x 6 in x 18 in beams. This batch was mixed following the ASTM C192 and curing in curing room, which had the same condition as the first and second batches. The insulators were performed to prevent the heat loss from the specimens. The compressive strength (ASTM C39) was attained at 6, 6.5, 7.0 and 24 hours. The flexural strength

(ASTM C78) was tested at 6.5 and 7.0 hours, respectively. The test result are shown in Chapter 4.

3.6 Determination of Unrestrained Free Shrinkage and Length Change

The shrinkage of the unrestrained specimen, known as free shrinkage, should also be measured for estimating the amount of stresses and crack widths. All developed mixes were performed and tested their volume changes which represented the free shrinkage of the mixed design. Each mix was set out the initial mixing temperature at 27°C. The procedure of mixing concrete can be found in the ASTM C192. The testing method for the determination of length change is described in the ASTM C490.

3.6.1 Test Specimens

Concrete and Mortar Bar: Four concrete bars 2 in x 2 in x 11 ¼ in were produced. The mortar bar was produced by sieving from the fresh concrete. Details of concrete and mortar prism molds are shown in Figure 3.10.

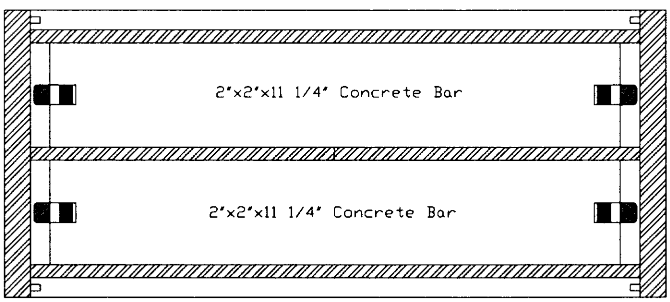


Figure 3.10 Details of concrete and mortar prism mold.

Length Change Instrument: Details of instrument are shown in Figure 3.11 (ASTM C490 Standard Practice for Use of apparatus for the Determination of Length Change of Hardened Cement Paste, Mortar and Concrete).

Figure 3.11 Detail of length change apparatus.

The gage length was considered as the nominal length between the inner most ends of the gage studs. The parts of the mold were tight fitting and firmly held together when assembled and their surfaces should be smooth and free of pits. The molds were made of steel not readily attacked by the cement paste mortar or concrete. The sides of molds were sufficiently rigid to prevent spreading or wrapping. Each end plate of the mold was equipped to hold properly in place during the setting period. The gage studs were made of the American Iron and steel Institute Type 316 stainless steel.

The instrument for determining the length changes of specimens produced in the molds can provide a dial micrometer (0.0001 in) unit, accurate within 0.0001 in. in any 0.0010 in. range and within 0.0002 in. in any 0.01 in range.

Procedure to measure the length change: According to ASTM C490, the Reference bar was used for setting Gage Zero. Take the reference bar out and then put the specimens in the same position each time a comparator reading is taken. Rotate specimens slowly in the measuring instrument while the comparator reading is being taken. Record the digital reading from the monitor of instrument. Clean the hold at the base of the comparator into which the gage stud on the lower end of the bar fits (hold ten to collect water and sand and clean after every reading). Calculate the length change at any age by using the $L = \frac{L_x - L_i}{G} \times 100$, where L_x is comparator reading of specimen at x age in inches, L_i is comparator reading of specimen at initial age in inches, and G is nominal gage length, 10 in. The length change values from calculation are in the nearest percentage of 0.001 and report the average value to the nearest percentage of 0.01.

3.6.2 Test Setup

The four concrete bars, 2 in x 2 in x 11¼ in. were cast in two layers and rodded in accordance with ASTM C 192. All specimens were put on the top of cylinder and beam in order to decrease the volume effect and covered with blanket insulator in a moist room until concrete specimen can reach 350 psi-flexural strength or 2,500 psi – compressive strength. Plate form of specimens were taken out, measured the first length of bar. Two concrete bars were moved to 16°C and 50% relative humidity room. The other two concrete bars were cured in 21 to 22 °C and 100% relative humidity room. All bars were investigated the length change at 8, 48, 96, 168, 336 and 672 hours after first measurement. Detail of test setup is shown in Figure 3.12.

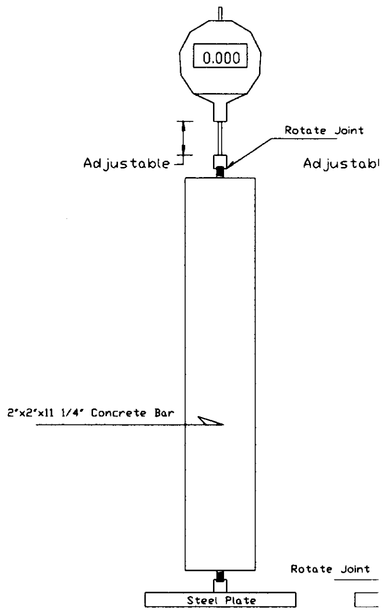


Figure 3.12 Length change test setup.

3.7 Determination of Drying Shrinkage Cracking Using Ring Test

This section deals with the cracking potential due to drying shrinkage. The cracking is a complex process that depends on a large number of factors. The major parameters are: i) magnitude of drying shrinkage, (ii) tensile strain capacity of concrete, (iii) restraint that prevents free movement to accommodate shrinkage and, (iv) loading conditions that cause tension fatigue. A number of test setups have been developed to measure the cracking potential of concrete. The ring setup was chosen to evaluate the rapid hardening concrete.

This section provides the details of the setup and results. The various setups are also described. The setups for measuring cracking potential are: Three different shapes (linear, plate, and ring) have been tried for measuring the contribution of fibers to shrinkage crack reduction. Ring specimens seem to have better potential because they can be provided with good restraint. The arrangement is also conducive to the development of mathematical models, as explained previously (Hsu et al, 2002).

3.7.1 Ring Specimens

Ring specimens were used by a number of investigators for evaluating fiber-reinforced cement composites under restrained drying shrinkage. Essentially, a ring of concrete is cast around a stiff steel ring, which is shown in Figure 3.13. As the composite shrinks, it induces stresses on the steel ring. Since the steel ring is stiff and undergoes very little deformation, the outer cement composite ring is subjected to tension. If the concrete ring is thin in relation to the internal diameter, then the stresses across the thickness can be considered uniform. The compressive stress developed at the interface between the steel ring and the concrete ring is also negligible. The researchers used various external diameters for steel rings.

The thickness of the cement composite was also varied depending on the composition of the matrix. Typically, thicker sections were used with concrete containing coarse aggregates.

As mentioned earlier, this setup shows the most promise because of the uniform restraint provided by the steel ring. The restraining force is imposed by the steel ring across the perimeter of the concrete, instead of two or four locations as with linear and plate specimens. The method is described in detail in the following paragraphs.

The variation of stresses across the thickness of the concrete ring depends on the internal diameter of the ring. For the dimensions shown in Figure 3.14, the difference between the values of tensile hoop stress on the outer and inner surface is only 10%. In addition to hoop stress, the concrete ring is also subjected to radial compressive stress when the steel ring exerts radial pressure. Since the diameter of the ring is relatively large, this radial compressive stress is only 20% of maximum hoop stress. Since cement composites are an order of magnitude stronger in compression, the maximum compressive stress in the ring is only about 2% of the compressive strength. Hence, the effect of compressive stresses can be neglected.

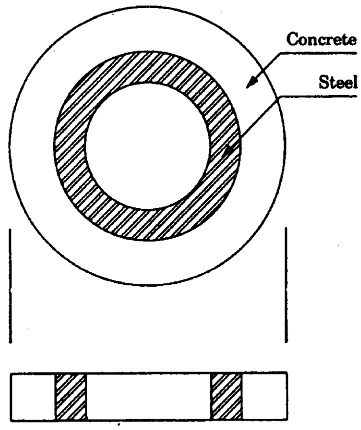


Figure 3.13 Schematic view of a restrained ring shrinkage setup.

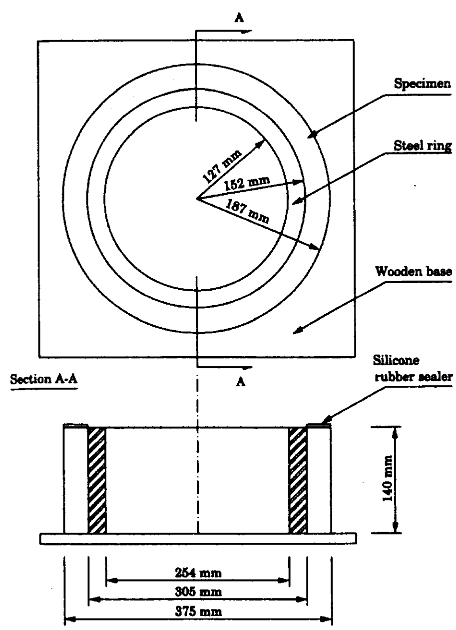


Figure 3.14 Dimensions of a ring specimen.

3.7.2 Test Setup

The cement composite can be cast between a steel ring and an annular outer mold. The outer mold can be made of cardboard or plastic. Provisions should be made to remove the outer mold without causing disturbance to the young cement composite. Care should also be taken to place the outer ring concentrically with the inner ring to avoid non-uniform thickness of the cement composite ring. The outer mold can be removed as soon as the concrete hardens. The drying should be done in a controlled environment at a chosen temperature and relative humidity. The concrete is sealed at the top using a silicone rubber sealer, allowing it to dry evenly only at the outer edge. A relatively large ratio of the width (exposed surface) to the thickness (four or higher) can provide uniform drying across the thickness.

Figure 3.15 shows the microscope setup for measuring the cracks that develop because of restrained shrinkage. The microscope is attached to the center of the steel ring and can rotate 360°. It can also travel up and down, facilitating the crack-width measurements across the 140 mm (5.5 in.) width of the exposed concrete surface.

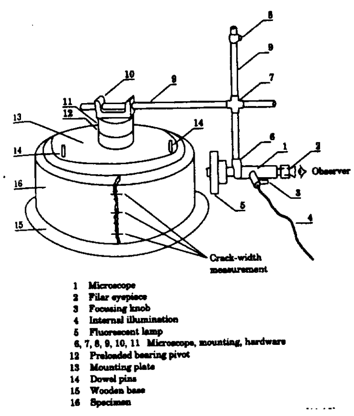


Figure 3.15 Test setup to measure crack width using a microscope.

CHAPTER 4

EXPERIMENTAL RESULTS AND DISCUSSIONS

4.1 Optimized Mix and Its Maturity Parameters

More than 20 trial mixes were casted and tried. The results of the control and three optimized mixes are shown in Table 4.1. The maturity parameters of optimized mixes and datum temperature can be found in Table 4.2. Their compressive strength and flexural strength are detailed in Table 4.3. Note that all specimens were controlled the initial mixing temperature and an insulation was provided.

Table 4.1 The Four Optimized Mixed Designs

Material (Lb/yd^3)	**Mix No. I (Lb/yd^3)**	**Mix No. II (Lb/yd^3)**	**Mix No. III (Lb/yd^3)**	**Control Mix (Lb/yd^3)**
Cement	799	658	658	799
Coarse aggregate	1,840	1,850	2,057	1,800
Fine aggregate	1,090	1,350	1,241	1,200
Water	282	230	230	325
Water Reducer	12 oz/cwt	12 oz/cwt	16 oz/cwt	16 oz/cwt
Air Entraining	0.5 oz/cwt	0.5 oz/cwt	0.65 oz/cwt	1 oz/cwt
Rapid Hardening	32 oz/cwt	38 oz/cwt	40 oz/cwt	32 oz/cwt

Table 4.2 Fresh Concrete Properties of Optimized Mixes

	Mix No. I	**Mix No. II**	**Mix No. III**	**Control Mix**
Mix volume	2.00 ft^3	2.25 ft^3	2.00 ft^3	2.00 ft^3
Initial mix temperature	28 ° C	28 ° C	28 ° C	28 ° C
Slump	5.0 inches	4.5 inches	3.0 inches	3.0 inches
% Air content	6.0 %	4.0 %	4.5 %	5.0 %
Final Setting time	4.5 hrs	4.5 hrs	4.5 hrs	5.0 hrs
Water/Cement	0.35	0.33	0.35	0.35
C.A./Cement	2.30	2.80	3.10	2.25
F.A./Cement	1.36	2.05	1.88	1.50
Datum temperature	6.70 ° C	6.20 ° C	7.00° C	6.00° C

Table 4.3 Maturity (°C-hr) and Compressive Strength of Laboratory Mix Designs

Time (hrs)	**Mix No. I**		**Mix No. II**		**Mix No. III**		**Control Mix**	
	Maturity (°C-hrs)	Strength f'c (psi)	Maturity (°C-hrs)	Strength f'c (psi)	Maturity (°C-hrs)	Strength f'c (psi)	Maturity (°C-hrs)	Strength f'c (psi)
5.50	60	560	-	-	-	-	-	-
6.00	-	-	-	-	22	216	-	-
6.50	110	2,221	99	2,260	146	861	-	-
7.00	134	2,440	118	2,607	-	-	149	2,222
7.50	148	2,566	-	-	177	2,640	166	2,976
7.45	-	-	-	-	-	-	-	-
8.15	-	-	-	-	-	-	-	-
8.45	-	-	-	-	-	-	-	-
10.15	-	-	237	3,375	-	-	-	-
22	-	-	-	-	-	-	-	-
24	615	4,017	608	4,548	612	3,598	605	4,417

The maturity values are calculated from $[(0.5 \times (T_0-T_x))-T_d] \times$ Duration.
T_0 is the beginning point of temperature rise up.
T_x is the temperature in any period
T_d is the datum temperature which shown in Table 4.2
Duration is the period of collecting temperature data

Table 4.4 Mechanical Properties of the Optimized Mixes

	Flexural strength (psi) (Modulus of Rupture)	**Compressive strength (psi)**
Mix No. I	353 psi (6.5 hrs) 390 psi (7.5 hrs) 875 psi (28 days)	2,221 psi (6.5 hrs) 2,566 psi (7.5 hrs) 4,017 psi (24 hrs) 4,138 psi (48 hrs)
Mix No. II	350 psi (6.5 hrs) 366 psi (7.0 hrs) 370 psi (7.5 hrs)	2,260 psi (6.5 hrs) 2,607 psi (7.0 hrs) 4,548 psi (24 hrs) 5,096 psi (7 days) 5,607 psi (14 days) 6,019 psi (21 days) 6,424 psi (28 days)
Mix No. III	365psi (6.5 hrs) 552 psi (4 days)	2,640 psi (7.40 hrs) 3,598 psi (24 hrs) 4,165 psi (4 days) 5,266 psi (14 days) 5,457psi (28 days)
Control Mix	338 psi (6.5 hrs) 485 psi (7.5 hrs) 539 psi (24 hrs) 998 psi (28 days)	2,222 psi (7.0 hrs) 2,976 psi (7.5 hrs) 4,417 psi (24 hrs) 5,796 psi (7 days) 5,914 psi (28 days)

4.2 Effect of Local Cement Brand on Early Strength of Concrete

The final set time and flexural strength result between the "Type I Essroc cement" and "Type I Lafarge cement" are shown in Table 4.6. The results in Table 4.5 show that the "Essroc cement" using the same mixed design can reach its final set time faster than the "Lafarge cement". However, Lafarge cement will set faster when it is cured in a higher temperature. It produces the flexural strength closed to Essroc cement at the same period of time.

Table 4.5 Concrete Flexural Strength of Two Local Cement Brands

Batch Number	**1**	**2**	**3**
Type I Cement Brand	**Essroc**	**Lafarge**	**Lafarge**
Initial temperature (°F)	81	81	81
Curing temperature (°F)	78	78	92
Final Set time (hours)	4.0	5.6	4.25
Flexural Strength (psi) at 6.5 hours	336	180	335
Flexural Strength (psi) at 7.0 hours	369	200	390

4.3 Early Strength Development in The Insulated Specimens

Based on present study, the compressive strength and flexural strength using the ASTM methods together with heat prevention during curing period improve their early concrete strength. The results are presented in Table 4.6. The improvements of compressive strength are 64.6 % at six hours, 47.3 % at six and half hours, 36.7% at seven hours and 3.3% at twenty four hours. The improvements of flexural strength are found to be in 33.6% at six and half hours and 25.6% at seven hours, respectively.

Table 4.6 Early Compressive Strength and Flexural Strength of Concrete Specimens after Using the ASTM Method with Blanket and Insulator

Time (hours)	Compressive Strength (psi)		% Diff	Flexural Strength (psi)		% Diff
	ASTM	ASTM & Heat Protection		ASTM	ASTM & Heat Protection	
6.0	697	1,970	64.6	-		
6.5	1,160	2,202	47.3	219	330	33.6
7.0	1,650	2,607	36.7	272	366	25.6
24	4,396	4,548	3.3	-		

4.4 Effect of Initial Mixing Temperature on Early Strength of Concrete

This study composed of the same concrete mix using five different degrees of initial temperature (77°F, 83°F, 86°F, 92°F, and 95°F) to study their effect on the early strength of concrete. The result in Table 4.7 shows the final set time of 5 batches. Additionally, Figure 4.1 presents the compressive strength versus time (hours). Note that the specimens used in this study were not provided by insulation.

Table 4.7 Final Set Time of Concrete at Different Initial Temperatures

Initial mixing temperature	**77 °F**	**83 °F**	**86 °F**	**92 °F**	**95 °F**
Time for final setting time (hour)	6.5	5.5	4.5	4.0	3.7

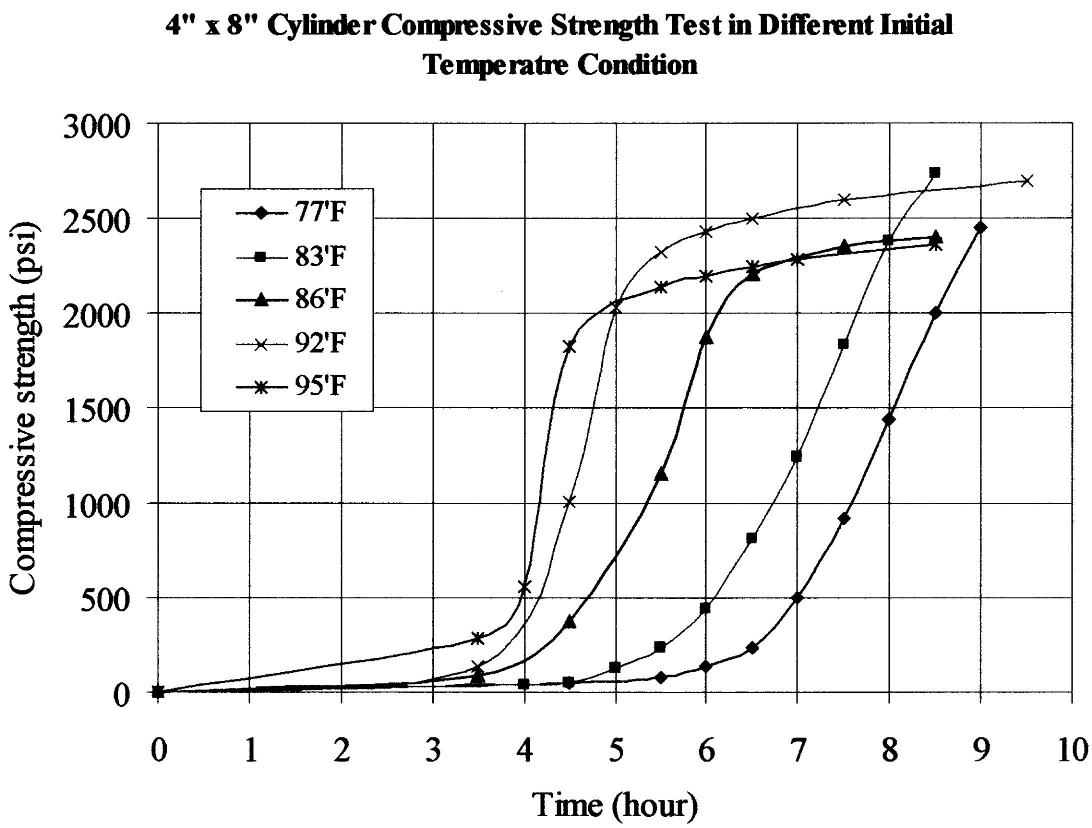


Figure 4.1 Compressive strength versus time at different initial mixing temperatures.

4.5 Effect of Mixing Method on Early Strength of Concrete

The present experimental results show that the mixing method used has influence on the early compressive and flexural strength of concrete. The ASTM mixing method shows greater compressive strength and flexural strength in early period. Comparison of compressive strength and flexural strength using two methods are presented in Table 4.8. The percent differences of compressive strength between these two methods are 47.3% at six hours, 42.2% at six and half hours, 35.8% at seven hours and 2.4% at twenty four hours. The differences in flexural strength of these two methods are found to be 27.9% at six and half hours and 19.1% at seven hours. However, the compressive strength of these two mixing methods shows little difference at twenty four hours.

Table 4.8 Early Compressive Strength and Flexural Strength of Concrete

Time (hours)	Compressive Strength (psi)		% Diff	Flexural Strength (psi)		% Diff
	ASTM	Non ASTM		ASTM	Non ASTM	
6.0	697	367	47.3	-	-	
6.5	1,160	670	42.2	219	158	27.9
7.0	1,650	1,060	35.8	272	220	19.1
24	4,396	4,291	2.4	-	-	

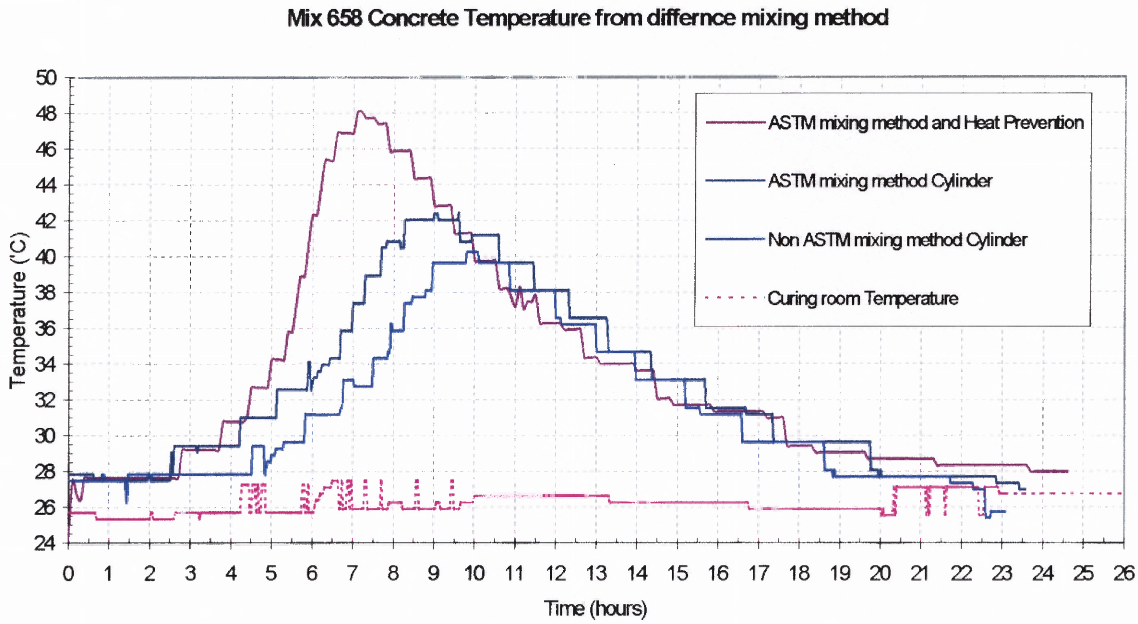


Figure 4.2 Concrete temperature of Mix No. II using different mixing and curing methods.

4.6 Unrestrained Free Shrinkage and Length Change

After concrete can provide the 2,500-psi compressive strength or 350-psi in flexural strength, all shrinkage bar specimens were decomposed from mold and put in the 50% humidity room. The free shrinkage occurred and measured by the micrometer instrument, which discussed in last Chapter. The free shrinkage values were measured for each optimized mixed. The result of each mix was present in Figure 4.3. Control mix showed shrinkage value of –0.108, Mix No. I provided the shrinkage value of –0.098, and the Mix No. III provided the shrinkage value of –0.0691 at 60 day.

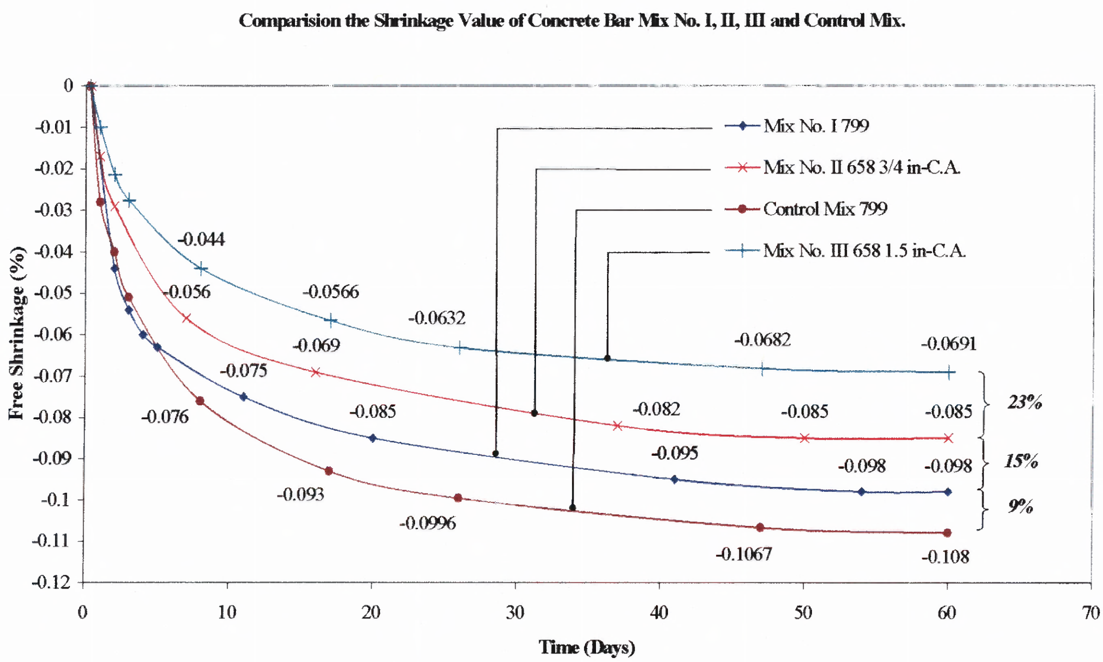


Figure 4.3 The percentage of length change for four optimized mixes.

4.7 Determination of Drying Shrinkage Cracking using Ring Test

The present test shows that the rings have not cracked. It is believed that the rings do not crack because the rapid hardening concrete gains sufficient tensile strength during the first five hours. This also results very little lose of water and hence less drying shrinkage.

Based on the performance of the rings, it is believed that, if proper care is taken during the first six hours to eliminate water loss from the concrete, the rapid hardening concrete can be proportioned to provide less shrinkage cracking than normal concrete.

The ring setup was chosen for the experimental investigation. The primary variable was the mix proportion as shown in Table 4.1. The following two mixes were evaluated (see Figure 4.4 and 4.5):

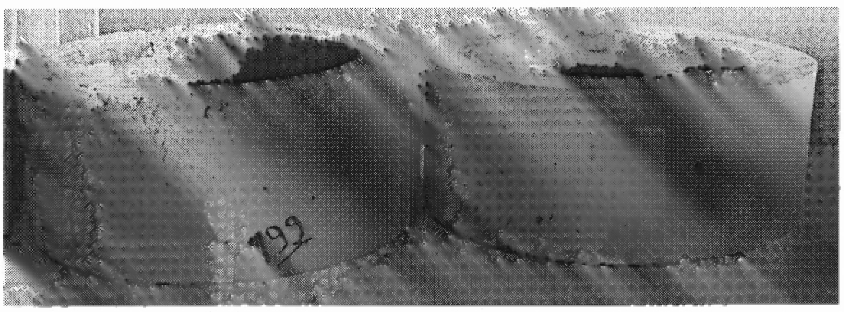


Figure 4.4 Ring specimens for Mix no. I (cement 799 lb/yd^3).

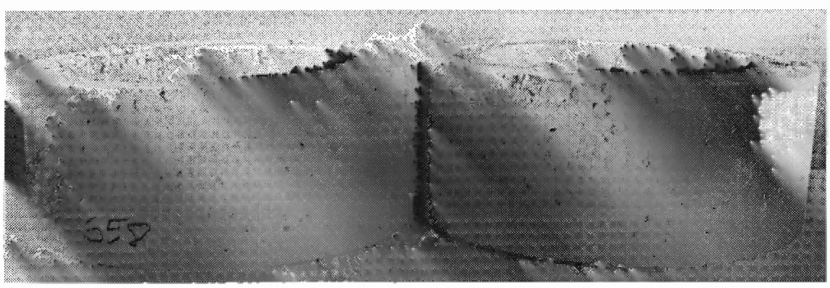


Figure 4.5 Ring specimens for Mix no. II (cement 658 lb/yd^3).

4.8 Discussion of Test Results

4.8.1 Discussion on Development of Optimized Mix

Mix. No. I, which was developed from the control mix was designed by increasing coarse aggregate and fine aggregate cement content in order to improve the durability characteristic for shrinkage proposes. The proportion of water cement ratio was maintained the same as control mix. The coarse aggregate content was increased from 2.25 to 2.30. The fine aggregate content was decreased from 1.50 to 1.36. The proportion of admixture were adjusted for workability, durability and sustaining the strength development. Although, the compressive strength of concrete specimens in laboratory gave 2,221 psi, which did not reach 2,500-psi required relative compressive strength but the flexural strength gave 353 psi, which reached the required strength of 350 psi. However, the concrete specimens produced the compressive strength 2,566 psi at seven and a half hours and 4,017 psi at twenty four hours. The accomplishment in maintaining the strength by adjusting the mix proportion leads to next step in developing the optimized mix.

Mix No. II was designed by decreasing the cement weight per cubic yard from eight and a half bags to seven bags per cubic yard, increasing the coarse and fine aggregate cement content to 2.8 and 2.5, respectively and decreasing the water cement content to 0.33. For this mix the one and a half bag per cubic yard can reduce the cost of construction. The cement was decreased from 799 lb/yd^3 to 658 lb/yd^3. At the present time, the cement price for 94-lb bag is \$8.00. However, the compressive strength was 2,260 psi, but the flexural strength arrived at the value of 350 psi at six and a half hours and 366 psi at seven hours. Note that, Mix no I and II used the same size of coarse aggregate (¾").

Mix no. III used the bigger size of coarse aggregate (1½ in) in order to reduce the shrinkage value of concrete mix and to protect the transverse cracking. Using the bigger size of coarse aggregate caused lower slump and decreased the results of compressive strength and flexural strength at early age. Because the bigger size of coarse aggregate decreased the cement paste, the chemical reaction, which generated heat that was slower than other mix so the rate of hydration was decreased. Even through the more rapid hardening was added to concrete mix, the result still could not improve. More than 40 oz/cwt, which is proposed in Table 4.1 was used in mix design but it reduceed the strength of concrete specimens instead of increasing the strength.

The present study finds that not only the special admixture such as Water Reducer, Air Entraining and Rapid Hardening, but also the initial temperature, curing procedure, mixing method and appropriate material must be used simultaneously in order to produce the optimized early strength concrete specimens.

The initial mixing temperature of the successful laboratory concrete is in range of 27 to 29°C (81 to 85°F). Using the insulator to prevent the heat lost from the concrete specimens gives an additional better early strength condition. The result in Table 4.7 shows that increasing of initial mixing temperature leads to decreasing final setting time of concrete. Note that the concrete specimens, which were used to study the effect of initial temperature on early strength concrete, were not provided the insulator to protect the heat loss. It can be concluded that that the higher the initial mixing temperature is, the higher the strength gain in a early strength concrete will be.

The curing temperature and heat protection are other issues, which affect the early strength of concrete. From the test results, it is found that the cylinder and beam specimens using the insulation blanket can improve the early compressive strength 47.3% at six and a half hours and can improve the early flexural strength 33.6% at six and a half hours. However, only 3.3% improvement of compressive strength has been found at twenty four hours.

To study the effect of mixing method on early strength concrete, there are two methods, namely ASTM and Non ASTM mixing method were used. They were discussed in Chapter 3. The ASTM mixing method stops mixing for a few minutes before the last mixing. In contrary, the Non ASTM method does not stop the mixer until the uniform concrete has been observed. The Non-ASTM method clearly shows that there are some mortars still remaining inside the mixer after casting. The Non ASTM method obviously requires more water in order to maintain a good slump.

The ASTM Mixing method has been found to be an outstandingly practical method for mixing concrete. The ASTM method gives the advantage in making a uniform concrete. From observation during mixing concrete had obviously developed the uniform fresh concrete and mixing machine had less the mortar left than Non ASTM method after placing.

The hydration of cement and water lost during mixing procedure reduced the compressive and flexural strengths of concrete using Non-ASTM method. The strength results shown in Table 4.8, the Non-ASTM method gives lower compressive strengths and flexural strengths than those of the ASTM method.

The initial mixing temperature of concrete plays an important role for a successful very early strength concrete. The ASTM method can prevent heat loss and thus leads to continuity of temperature development from the hydration reaction. By stopping the mixing machine for a few minutes and then starting a final mix before placing, it increases the temperature inside the concrete specimens.

The ASTM curing procedure has been recommended for normal strength concrete in a longer period. This method, however, does not prevent heat loss from the specimens at the beginning of hydration. As presented in Table 4.6, the ASTM curing method plus heat protection gives a better compressive and flexural strength at early age.

The volume of concrete made in the laboratory is smaller when compared with the volume of concrete on the construction field. In laboratory, it usually produces only four cubic feet (0.15 yd^3) while in the field manufactures more than two cubic yards. The amount of heat which has been generated by concrete itself are tremendous different between the laboratory and the field. The tremendous heat influences the strength development of concrete. Even the heat prevention is performed for concrete specimens in the laboratory, the amount of temperature is still lower than that on the construction site. The higher the mixture temperature is, the sooner the hydration will begin and therefore the faster of strength development will process. The rate of strength development in the laboratory will be slower than the concrete in the filed because of the difference volume. At the early age, the strength of same concrete mix made in laboratory will be less than the one using the same concrete mix produced in the field. In a future work, the effect of volume on temperature development in concrete has to be studied thoroughly.

In addition to the initial temperature, curing procedure and the mixing method which affect the strength of very early strength concrete, an appropriate cement material has also to be carefully chosen for a concrete mix. In an experimental comparison between the “Essroc” and “Lafarge” brands of Type I cement, it shows that only the Essroc cement reaches the target strength within six and half hours. For Lafarge cement, which reaches its final setting time later than the Essroc-cement mix for two hours. In order to accelerate the strength, an additional heat has to be applied to concrete made from Lafarge cement. The experimental result of Table 4.5 shows that the curing temperature of 92°F can accelerate the final set time of the Lafarge cement and thus achieves the same flexural strength as the Essroc cement. The Essroc cement has been founded to be a more appropriate for very early strength concrete than the Lafarge cement. The different cement brands would have different material compositions in manufacturing. This present study is intended to study the effect of cement brand on early strength development and therefore the quality of cement is disregarded. It may be possible that the Lafarge-cement will reach higher strength at 28 days strength, and may achieve more durable concrete in a long period of time.

The Essroc-cement company has recently was changed the matrix of cement composition. The change of composition affects the mixed design to maintain the required strength at six and half hours. The changes of composition in cement are presented in Table A.1. In addition, from Figure A.1 in Appendix A.3 shows that the uncleaned aggregates retard the hydration reaction of cement, which leads to decreasing the concrete strength at early age.

4.8.2 Discussion on Shrinkage

The shrinkage is primarily cause by the loss of water from the hardened material. To prevent loss of water by decreasing the water cement ratio and decreasing paste ratio in the mixed design can prevent some of volume change in concrete. The specimens were put in critical condition of humidity, 50% relative humidity, at six and a half hours after casting. With this condition specimen started to change the volume from loss of water because in fresh concrete, the space between particles is completely filled with water. When water is removed from the paste, these generate negative capillary pressures, which will cause the volume of the paste to contact. Capillary pressures continue to rise within the paste until a critical "breakthrough" pressure at which point the water is no longer evenly dispersed through the paste and rearranges to form discrete zones of water with voids between. The maximum rate of plastic shrinkage occurs just prior to the breakthrough pressure and little shrinkage occurs afterwards. Moreover, the shrinkage can occurred from self-desiccation and is manifested as autogenous shrinkage. Autogenous shrinkage will only occur even if the concrete is sealed or in dense concrete, low water-cement ratio and the addition of silicafume. Self-desiccation may occur to some extent, even if water is supplied during the curing process, because external water can not easily penetrate the concrete.

At 60 days the control mix gave the free shrinkage result –0.108. Mix No. I, which increased the coarse and fine aggregate cement content can reduce the free shrinkage in percentage of nine or -0.098. Not only increased the coarse and fine aggregate in Mix No II. but also decreased the cement content was applied. The free shrinkage was improved to –0.085 or fifteen percent improvement. Mix No. III was redesigned from Mix. No. II. The significant adjustment was the increasing the size of coarse aggregate from ¾ in to 1½ in. The adjustment can reduce the free shrinkage value for twenty three percent from shrinkage value of Mix. No. II or fifty six percent improvement from control mix.

The ring test result shows the corresponding result to volume change study, which the rings have not cracked. The rings did not crack because the rapid hardening concrete gained sufficient tensile strength during the first five hours. This also results very little lose of water and hence less drying shrinkage.

Based on the performance of the rings, if proper care is taken during the first six hours to eliminate water loss from the concrete, the rapid hardening concrete can be proportioned to provide less shrinkage cracking than normal concrete.

CHAPTER 5

SUMMARY AND CONCLUSIONS

The results obtained from all of the experimental tests conducted in this study lead to the following conclusions.

1. A new concrete mix design has been produced for fast track concrete at present study. It can achieve a flexural strength (Modulus of rupture) of about 350 psi (2 MPa) in six to seven hours after placement operation. But the compressive strength of a new concrete mix is 2,250 psi (15 MPa) in six to seven hours. It is important to note that the repair of highway pavement work is more interesting in flexural strength than compressive strength. Also the cost of construction can be reduced by less cement use.
2. The initial mixing temperature of the fast track concrete mix has to be controlled in range of 27°C to 29°C (81°F to 86°F) for all mixes. The heat and moisture prevention is a very important factor for producing such mixes, especially for laboratory specimens. The insulation blanket has to be applied to cover the concrete immediately after placement. In order to achieve the target strength during the winter time, the heater or heater blanket may require during curing process until the temperature inside concrete reaches the maximum temperature. The curing temperature should correspond to initial temperature. Note that in construction field, the concrete can produce the early strength faster than that in the laboratory because the huge volume of field concrete can accumulate heat from hydration reaction more than the concrete in the laboratory.

3. In this study, only the Essroc cement can produce such as fast track concrete. It's because the final setting time of Essroc-cement is faster than Lafarge cement. The chemical analysis of cement has found that Essroc-cement has higher amount of C_3S than that of the Lafarge-cement. However, when the same mix design using different cement brand to produce the concrete specimens, the compressive strength in a long period (28 day) shows that of the Lafarge-cement is higher than that of the Essroc cement.
4. The control mix, which was an approved mix for NJDOT in 1996, can not produce the compressive and flexural strengths as much as it can in past years. The reason in that the cement composition of the Essroc-cement has been changed. It is therefore the mix design has to be updated when the cement composition have been changed.
5. At present study, fresh concrete have the slump from four inches to five inches and the air content is ranged from 4.5% to 5.0%. The datum temperature of all mixes are shown in Table 4.2, which is ranged from 6.00°C to 7.00°C. However, the datum temperature used for calculating the maturity in this study was 6.5°C.
6. At a compressive strength of 2,500 psi, the maturity reaches 150 °C-hr to 175°C-hr for this study. The construction works can use the same maturity value to estimate the compressive strength of the fast track concrete in the field when they produce concrete from the same mix in the laboratory. The concrete will arrive at a compressive strength of 2,500 psi or higher. The maturity value may result in different value depending on the time to start calculating the maturity and datum temperature value. Typically, the calculation will start when concrete reaches its final set or when the concrete temperature begin to change slope, perhaps 45 degree-slope.

7. All developed mixes have been compared the amount of length change after casting. Humidity has an influence on shrinkage properties. From this study, it is observed that the more the humidity is, the less shrinkage will be ensured. The reduction of cement content and use of larger aggregate can reduce the percent of shrinkage value. Shrinkage reducing admixture will further reduce the crack. The shrinkage of concrete containing 1½ " stone exhibits less shrinkage (Mix no. III) than concrete containing ¾" stone (Mix no. II), which is about 23% at the same curing condition and at age of 60 days. By using the bigger size of coarse aggregate, it can reduce the shrinkage effect on concrete.

8. To study drying shrinkage cracking, two ring tests each were performed for Mix no. I and Mix no. II, respectively. The test rings have shown no cracks after three-month testing periods. It is concluded that, if proper care is taken during the first six hours to eliminate water loss from the concrete, the very early strength (VES) concrete can be proportioned to provide less shrinkage cracking than normal concrete. It must be noted that Mix no. III was not experimented with ring test because it contained 1½ inches stone and was too large to fit into the ring.

APPENDIX A

PROPERTIES OF PROPOSED MIXES

In appendix A, Figure A.1 presents the time-temperature data of all mixes. Figure A.2 presents the relationship between flexural strength and compressive strength of Very Early Strength concrete. Figure A.3 shows the influence of clean sand to developing temperature of control mix. Table A.1 shows the changing of chemical proportion of Essroc cement in 1996 and 2001.

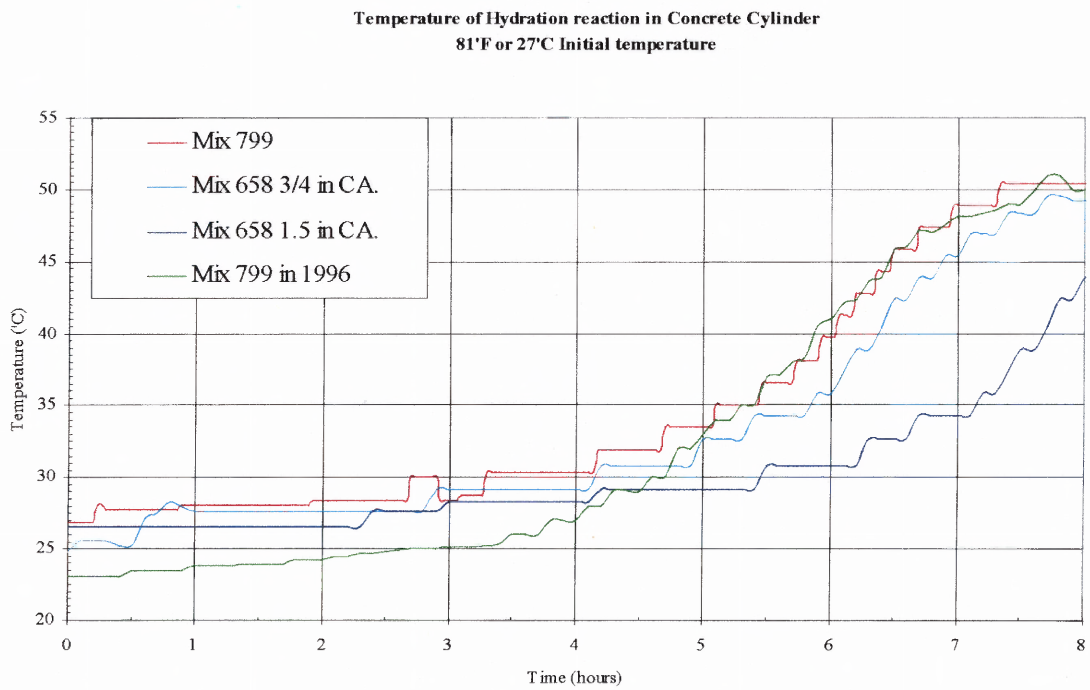


Figure A.1 Temperature data for all mix within 8 hours.

Table A.1 Laboratory Test Reports For ESSROC Type I Cement

Chemical Analysis			Physical Tests			
	13-Fed-1996	01-Jun-2001			13-Fed-1996	01-Jun-2001
SiO_2	20.0	19.8	Compressive Strength (psi)	1 day	2,520	2,970
Al_2O_3	5.4	5.2		3 day	4,226	4,151
Fe_2O_3	2.18	2.3		7 day	5,231	4,960
CaO	62.15	62.2				
MgO	3.56	3.4	Surface area	Blaine	399	373
SO_3	4.02	4.0		325 Mesh	95.1	94.1
LOI	1.26	1.3	Setting time(min)	Vicat Initial	126	115
C_3S	50	53		Vicat Final	280	240
C_3A	10.6	9.9				
Na_2O equiv.	0.90	0.93				

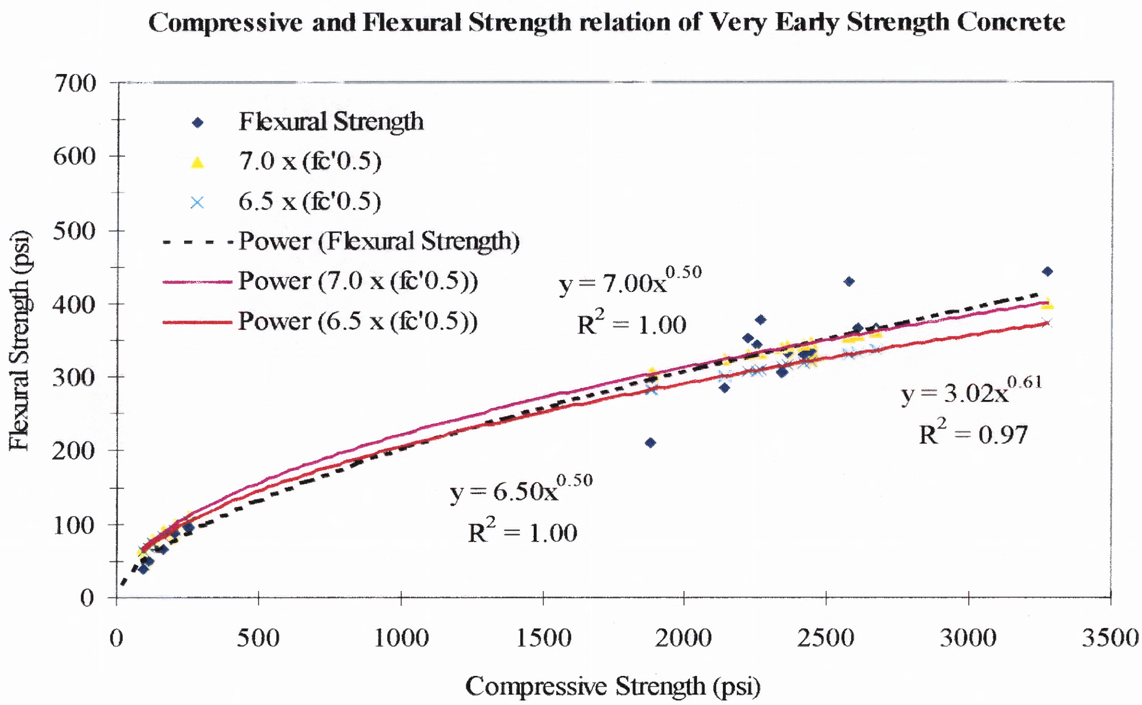


Figure A.2 Strength data from Mix No. II.

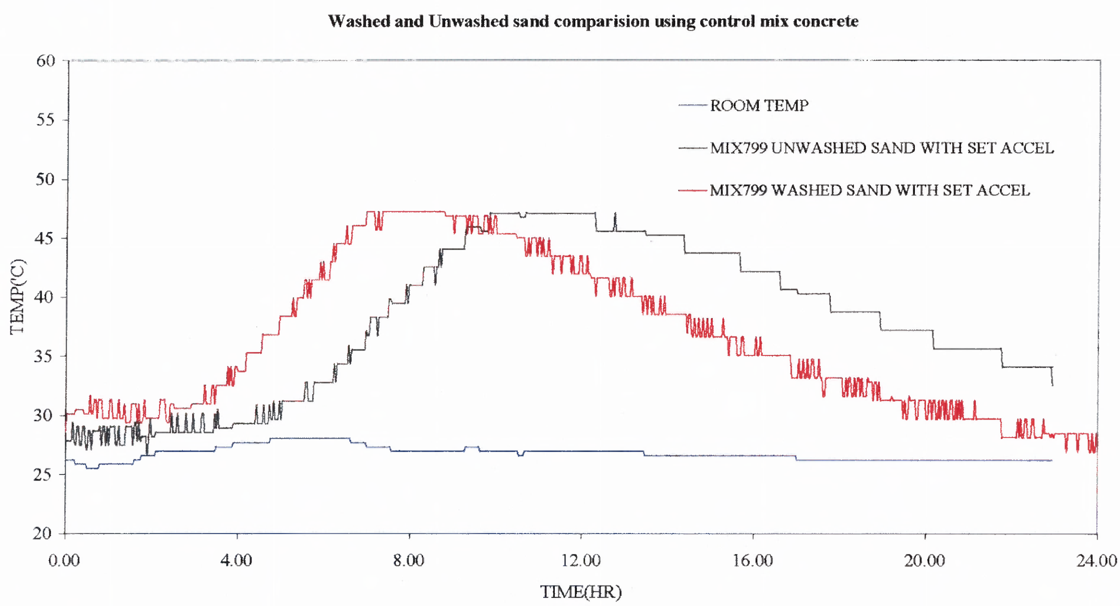


Figure A.3 Developing temperature of control mix using washed and unwashed sand.

APPENDIX B

SAMPLE OF DATUM TEMPERATURE CALCULATION

This section presents the example of collecting data and the procedure of calculating datum temperature. The procedure mainly consists of five steps which are shown in Table B.1, B.2, B.3, B.4 and B.5, respectively.

Table B.1 The Compressive Load of Mortar Cube at Three Different Curing Condition

Date	Age	Compressive load (lbf) at temperature T (°C)		
		10	20	30
08/14/2001	1	4,230	15,430	17,230
		4,030	16,250	17,520
		4,130	**15,840**	**17,375**
8/15/2001	2	13,460	20,100	15,780
		12,720	14,140	20,110
		12,630	x	20,440
		12,937	**17,120**	**18,777**
8/17/2001	4	14,650	19,530	19,120
		13,680	16,960	20,890
		14,165	**18,245**	**20,005**
8/20/2001	7	16,380	18,250	20,250
		18,250	19,210	x
		17,700	x	x
		17,443	**18,730**	**20,250**
8/27/2001	14	21,200	20,760	23,980
		24,900	20,410	x
		23,050	**20,585**	**23,980**
9/10/2001	28	31,030	30,590	26,010
		x	x	27,580
		31,030	**30,590**	**26,795**

Table B.2 The Compressive Strength of Mortar Cube at Three Different Curing condition

Final setting time (hours)		6.5	5.5	4.5
Date	Age	**Compressive Strength (psi) at Temperature**		
		T-10	T	T+10
08/14/01	1	1,033	3,960	4,344
08/15/01	2	3,234	4,280	4,694
08/17/01	4	3,541	4,561	5,001
08/20/01	7	4,361	4,683	5,063
08/27/01	14	5,763	5,146	5,995
9/10/2001	28	7,758	7,648	6,699

Table B.3 Reciprocal of Age, Compressive Strength and Age Minus Sitting Time

1/Age	**1 / Compressive Strength at Temperature**			**1/(Age-Setting Time)**		
	T-10	T	T+10	T-10	T	T+10
1.000	0.00097	0.00025	0.00023	1.37143	1.29730	1.23077
2.000	0.00031	0.00023	0.00021	0.57831	0.56471	0.55172
4.000	0.00028	0.00022	0.00020	0.26816	0.26519	0.26230
7.000	0.00023	0.00021	0.00020	0.14861	0.14769	0.14679
14.000	0.00017	0.00019	0.00017	0.07284	0.07262	0.07240
28.000	0.00013	0.00013	0.00015	0.03606	0.03601	0.03596

Table B.4 The Slope and Interception Value Between Reciprocal of Compressive Strength and Reciprocal of Age Minus Sitting Time

T	M	b	K
20.00	0.00006400	0.00018192	2.8423
10.00	0.00059566	0.00010290	0.1727
30.00	0.00005445	0.00017193	3.1574

Table B.5 The Slope and Interception Value between K-Value and Curing Temperature

m_{tk}	b_{tk}	Datum Temp
0.149232	-0.927149	6.213

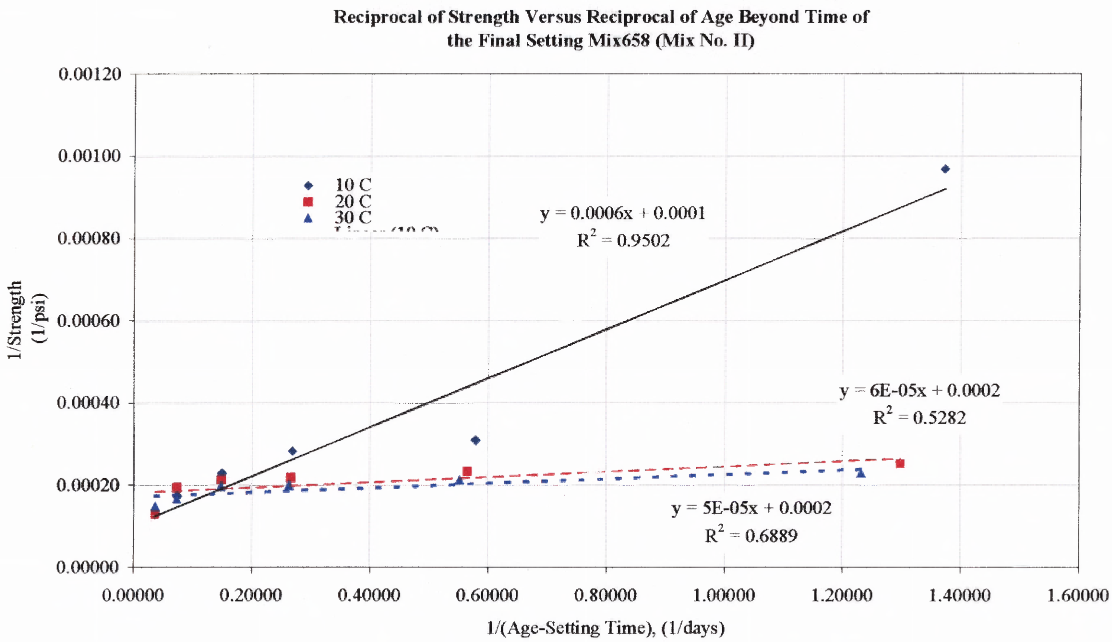


Figure B.1 Reciprocal of strength versus age beyond time of final set time.

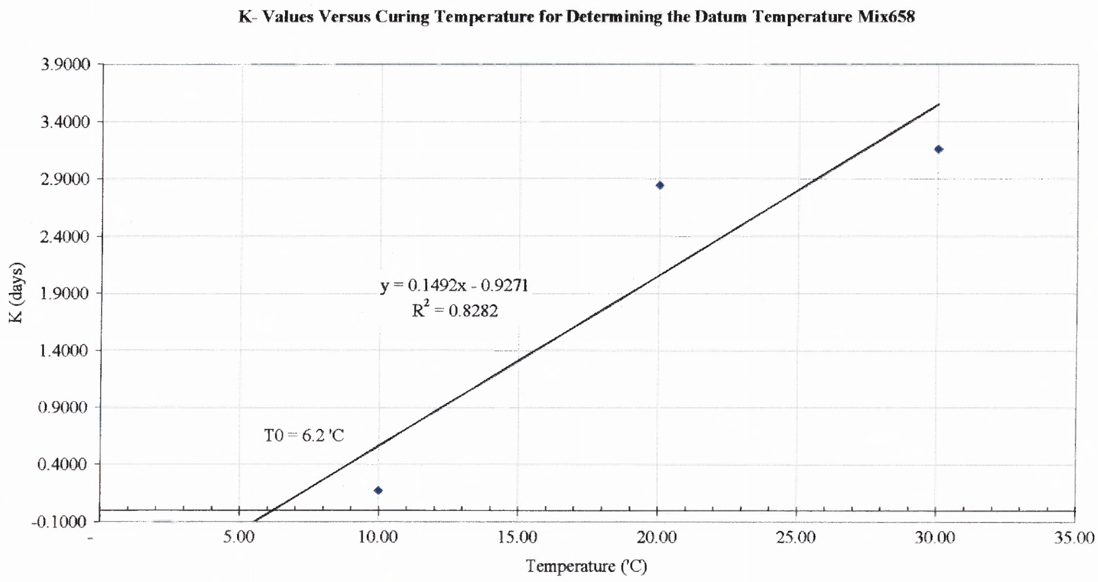


Figure B.2 The datum temperature determination.

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