THE INFLUENCE OF SURFACE AREA-TO-VOLUME AND ACCELERATED CURING ON THE QUALITY CONTROL OF HIGH STRENGTH CONCRETE

John J. Myers, Ph.D.

and

Ji Shen

University of Missouri-Rolla

ABSTRACT

This research study investigates the surface area-to-volume (SA/V) ratio on the early and later-age strength development of High Strength Concrete (HSC). Match curing technology is used to assess the adequacy of current quality control procedures in predicting in-situ strength of HSC members. In addition, the effects of accelerated curing, water to cementitious ratio and fly ash replacement were also studied.

The research program was sub-divided into four phases of study: Phase I measured the temperature development profiles of eight (maximum) high strength concrete cubes that are representative of different SA/V ratios. The temperature development profiles of these cubes under 3 curing regimes, 2 w/cm ratios and 2 fly ash replacement levels, were also recorded; Phase II established the equations of relationship between peak hydration temperature and SA/V ratio and modeled the temperature development profiles of each condition from the data obtained in phase I; Phase III match-cured groups of 4 x 8-inch (100 x 200 mm) cylinders following the programmed temperature profiles developed in Phase II. Release (24 hours) and 56-days compressive strengths and modulus of elasticity of cylinders were measured respectively during this phase; Phase IV measured the temperature development profiles and in-situ strengths at different locations in a Missouri Type 2 girder fabricated at Egyptian Concrete Co. in Bonne Terre, Missouri. Eight cubes similar to Phase I were fabricated using the same concrete as the girders. These were produced to monitor their temperature profiles and in-situ strengths by using match curing technology.

This study provides recommendations for the use of match curing technology for application in the precast industry using high strength concrete and high performance concrete.

ACKNOWLEDGMENTS

The authors would like to acknowledge the financial support of the American Concrete Institute (ACI) and the University Transportation Center (UTC) at University of Missouri – Rolla, as well as Holnam, Inc., ISG Resources, Inc., and Degussa Construction Operation, Inc. for donating the cement, fly ash and HRWR.

TABLE OF CONTENTS

LIST OF ILLUSTRATIONS

Figure Page

LIST OF TABLES

ABBREVIATIONS

Symbol	Description
SA/V	Surface Area-to-Volume
MOE	Modulus of Elasticity
HSC	High Strength Concrete
NSC	Normal Strength Concrete
HPC	High Performance Concrete
w/c(m)	Water to Cement (Cementitious) Material
HRWR	High Range Water Reducer
AASHOTO	American Association of State Highway and Transportation
ASTM	American Society for Testing and Materials
ACI	American Concrete Institute
FHwA	Federal Highway Administration
DOT	Department of Transportation
MoDOT	Missouri Department of Transportation

1. INTRODUCTION

1.1. RESEARCH BACKGOUND

Increasing demand for faster and accelerated construction along with developments in structural material and design technologies require the structural engineer to determine the actual in-situ strength of concrete rather than relying on indications of potential strengths from traditional quality control test specimens. As more High Strength Concrete (HSC) is utilized, especially in the precast industry, realization of the full strength potential of the concrete becomes more dependent on curing and placement conditions, including moisture condition, hydration temperature and member dimensions. Previous researchers have shown that the strength of traditional quality control specimens may not be representative of actual strength in HSC members unless match cured specimens are utilized [Myers et al., (1997)]. Match cured specimens are cured under the actual hydration temperature within the member monitored by thermocouples embedded in the concrete. With the availability of match curing technology, researchers are now able to study the adequacy of traditional quality control methods for advanced materials. However, to date, no studies have investigated the use of match curing technology to adequately estimate the mechanical properties of HSC at both early and later-age under the combined interaction of member dimension and curing regime.

1.2. RESEARCH SIGNIFIANCE

The research study results in a better understanding of when traditional quality control specimens are adequate for the precaster and when match curing technology becomes not only necessary, but recommended as the primary quality control method for use in the precast industry based on mix proportion of HSC and curing / placement conditions. To date, specifications have not directly addressed the applicability of match curing technology as a quality control tool in the precast industry, nor are any guidelines provided for when the system should be specified. This study directly investigates these issues enhancing the current state-of-the-art regarding match curing technology and precast quality control methods. Results from this study will be shared with the precast industry, professional design community, and technical societies.

1.3. OBJECTIVES OF THE RESEARCH

There are three main objectives of this research. The first objective is to find how the surface area-to-volume (SA/V) ratio of the member and accelerated curing affects the early and later-age properties of HSC members. The second objective is to determine at what level conventional quality control specimens no longer adequately represent the mechanical properties of the prestressed / precast concrete within the member. The third objective is to investigate the feasibility of match curing technology as the primary quality control process and provide guidelines for its use in the prestressed / precast industry.

1.4. PROJECT DESCRIPTION

This research was composed of two parts: laboratory study and field study. The laboratory study was funded by the American Concrete Institute (ACI) Concrete Research Council (CRC) and the University Transportation Center (UTC) at University of Missouri – Rolla. The field study was funded by the Missouri Department of Transportation (MoDOT) and Federal Highway Administration (FHwA) under research study entitled "HPC for Bridge A6130 – Route 412 Pemiscot County". The laboratory experiments utilized a commercially available match curing system and data acquisition system to model the hydration temperature profiles generated from various surface area-to-volume (SA/V) ratios of several standard prestressed / precast shapes including American Association of State Highway and Transportation (AASHTO). In conjunction with various SA/V ratios, several combinations of mix designs with different degrees of cementitious contents were also investigated. The field experiments were conducted at Egyptian Concrete Co. in Bonne Terre, Missouri. Different locations in these high strength concrete girders were monitored to investigate the applicability and reliability of the match curing system for field use in Missouri.

1.5. RESEARCH PLAN

The research study was sub-divided into four phases. Phase I, II and III are associated with the laboratory study, while Phase IV is associated with the field study. Phase I measured the temperature development profiles of eight high strength concrete cubes which represent eight different SA/V ratios ranging from 0.25 in.^{2}/in.^{3} (1/inch) to 1.0 1/inch. These cubes were subjected to 3 ambient temperatures of 70^{o}F, 130^{o}F, and

160^{o}F (21.1^{o}C, 54.4^{o}C, and 71.1^{o}C), 2 water to cement ratios (w/cm=0.25, w/cm=0.30) and 2 fly ash replacement levels (with / without 35% fly ash); Phase II established the equations of relationship between peak hydration temperature and SA/V ratio and modeled the temperature development profiles of each condition from the data obtained in phase I. Four values of SA/V ratio: 0.1, 0.2, 0.4, 0.6 1/inch, were selected as the standard key points for match curing in the next phase; Phase III match-cured groups of 4 x 8-in. (100 x 200 mm) cylinders following the programmed temperature profiles developed in Phase II. Release (24 hours) and 56-days compressive strength and modulus of elasticity of cylinders were measured respectively during this phase; Phase IV measured the temperature development profiles and in-situ strengths at different locations in a Missouri Type 2 girder fabricated at Egyptian Concrete Co. in Bonne Terre, Missouri. Eight cubes similar to Phase I were fabricated using the same concrete as the girders. These were produced to monitor their temperature profiles and in-situ strengths as a supplement to the results in the laboratory.

1.6. REPORT STRUCTURE

This report is divided into six chapters. A review of quality control in HSC and match curing technology is presented in Section Two. The test procedures, equipment and materials used in the study are described in Section Three. Test results are presented in Section Four and discussed in Section Five. Section Six summarizes the research and presents several conclusions and recommendations. Additional detailed material information, operation of matching curing system as well as additional hydration temperature development profiles and test data are included in the Appendices.

2. LITERATURE REVIEW

2.1. INTRODUCTION

Utilizing High Strength Concrete (HSC) in the structural members has greatly increased due to the growing demand for taller structures, longer spans and smaller member cross-section. Researchers showed continuous interests in the effects of the mechanical properties of HSC, compared to the Normal Strength Concrete (NSC). It has been reported [Cetin (1995), Balendran et al., (2000), Lura et al., (2001)] that HSC may lead to a higher hydration temperature which results in higher early-age strength but lower later-age strength of the concrete. Numerous researchers have reported the significant differences between the strength of current standard quality control specimens and the in-situ strength in the member [Urpani et al., (1991), Myers et al., (1997), Yang et al., (2002)]. In addition to the cementitious material content in the mix design and the curing condition of the concrete, the surface area-to-volume (SA/V) ratio of the structural member plays an important role of the concrete hydration temperature [Harrison (1983)].

This section presents a review of the factors that affect temperature rise and are relevant to the research topic at hand. These factors include surface area-to-volume (SA/V) ratio, High Strength Concrete (HSC), Quality Control Meteorology and Match Curing Technology.

2.2. SURFACE AREA-TO-VOLUME (SA/V) RATIO

2.2.1 Description of SA/V Ratio. The SA/V ratio is an indicator of how massive an object is. As the SA/V ratio increases, the massiveness of the object decreases.

Conversely, as the SA/V ratio decreases, the massiveness of the object increases. Therefore, the SA/V ratio reflects the shape and size of the cross-section of the concrete.

Massive concrete with low SA/V ratio can result in high temperature rise in the concrete due to the accumulation of the heat generated by the hydration of the cement. In the past, research studies on massive concrete were limited within Mass Concrete, where thermal behavior and cracking are considered as primary concerns rather than the strength [ACI 207.1R-96 (1996)]. As HSC has been widely applied in recent years to structural members, especially in the Precast / Prestressed industry, the effects of Mass Concrete on strength has been noted [Myers (1998)].

2.2.2. Mass Concrete. “Mass Concrete” is defined in ACI 116R-90 (1990) as “any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of the cement and attendant volume change to minimize cracking.” From this definition, the “Mass Concrete” is concerned with heat generation as it relates to cracking rather than strength. It has been stated in ACI 207.1R-96 (1996) that the design of Mass Concrete structures is generally based principally on durability, economy, and thermal action, with strength often being a secondary rather than a primary concern.

Mass Concrete practices are developed largely for concrete dam construction, where temperature-related cracking is first identified. Other practices include mat foundations, pile caps, bridge piers, thick walls, and tunnel linings where SA/V ratios are very low with often poor heat dissipation properties.

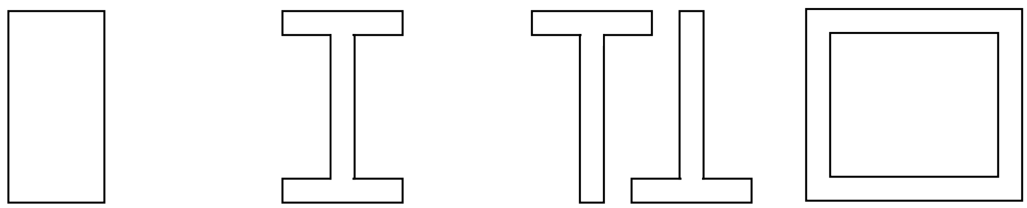
High compressive strengths are usually not required in Mass Concrete structures. These structures such as gravity dams, resist loads by virtue of their shape and mass, and only secondarily by their strength. Of greater importance are the durability and properties connected with temperature behavior and the tendency for cracking.

2.2.3. Thick-Section Concrete Member. Another application of massive concrete is thick-section structural member composed of HSC. Though these members have relatively higher SA/V ratio than “Mass Concrete” such as gravity dams, they often have high hydration temperature rise due to the combined affects of cementitious contents and low SA/V ratio. Such temperature rise will greatly affect the early-age and later-age compressive strength of the member.

Thick-section structural members, especially those used in precast / prestressed industry, are more concerned about strength development compared to the “Mass Concrete” industry. Up to date, few research studies have been carried out to characterize the relationship between the SA/V ratio and the compressive strength of HSC.

2.2.4. Survey of SA/V ratios of Current Standard Sections of the Girder. The shape of concrete member sections can be various. As illustrated in Figure 2.1, the sections used in structural applications include: (a) Rectangular Section, (b) “I” Section including Symmetrical and Unsymmetrical sections, (c) “T” and Inverted “T” Sections, (d) Box Section. In structural applications using concrete, the most commonly used sections include: Rectangular Section (End Blocks), I Section and Box Section are discussed.

Most often precast plants fabricate typical AASHTO-PCI standard sections. Other standards can be state DOT standard sections such as Missouri and Washington. Table 2.1 and Table 2.2 tabulate the SA/V ratios of standard girder sections and the corresponding end block ratios. From the table, it may be observed that, the SA/V ratio for I-sections range from 0.21 ~ 0.43 1/inch. For box-sections, the SA/V ratios range from 0.38 ~ 0.45 1/inch. The SA/V ratios for end blocks are rather small, ranging ratios of 0.09 ~ 0.20 1/inch.



(a) Rectangular (b) “I” Shape (c) T and Inverted T (d) Box

Figure 2.1. Shapes of Concrete Sections

Table 2.1. SA/V Ratios of Standard Section Beams

AASHTO-PCI Box Beam		AASHTO-PCI I Beam		Missouri I Beam		Washington I Beam	
Type	SA/V (1/in)	Type	SA/V (1/in)	Type	SA/V (1/in)	Type	SA/V (1/in)
BI-36	0.42	Type I	0.33	Type 2	0.38	40	0.43
BI-48	0.45	Type II	0.30	Type 3	0.32	60	0.40
BII-36	0.40	Type III	0.25	Type 4	0.23	80	0.35
BII-48	0.43	Type IV	0.21	Type 6	0.22	100	0.35
BIII-36	0.38	Type V	0.22			120	0.37
BIII-48	0.41	Type VI	0.23				
BIV-36	0.38	Quality Control Specimen Size		4"x8"	1.00		
BIV-48	0.40			6"x12"	0.67		

Conversion: 1 inch = 25.4 mm

As illustrated in Figure 2.2, the distribution of SA/V ratios of different types of sections and the standard quality control specimens is quite broad. From the figure it may be noted that the SA/V ratio of current quality control specimen, either 4 x 8-in. (100 x 200 mm) cylinder or 6 x 12-in. (150 x 300 mm) cylinder, has as much as 1000% or average of 435% for 4 x 8-in. (100 x 200 mm) cylinder and 291% for 6 x 12-in. (150 x 300 mm) cylinder difference from the actual member. These large variations in SA/V ratios may not result in large differences in mechanical properties for low content of cement mixes (or high w/cm mixes). However, the effects of SA/V ratio on the structural member and quality control specimens cannot be ignored when HSC is selected.

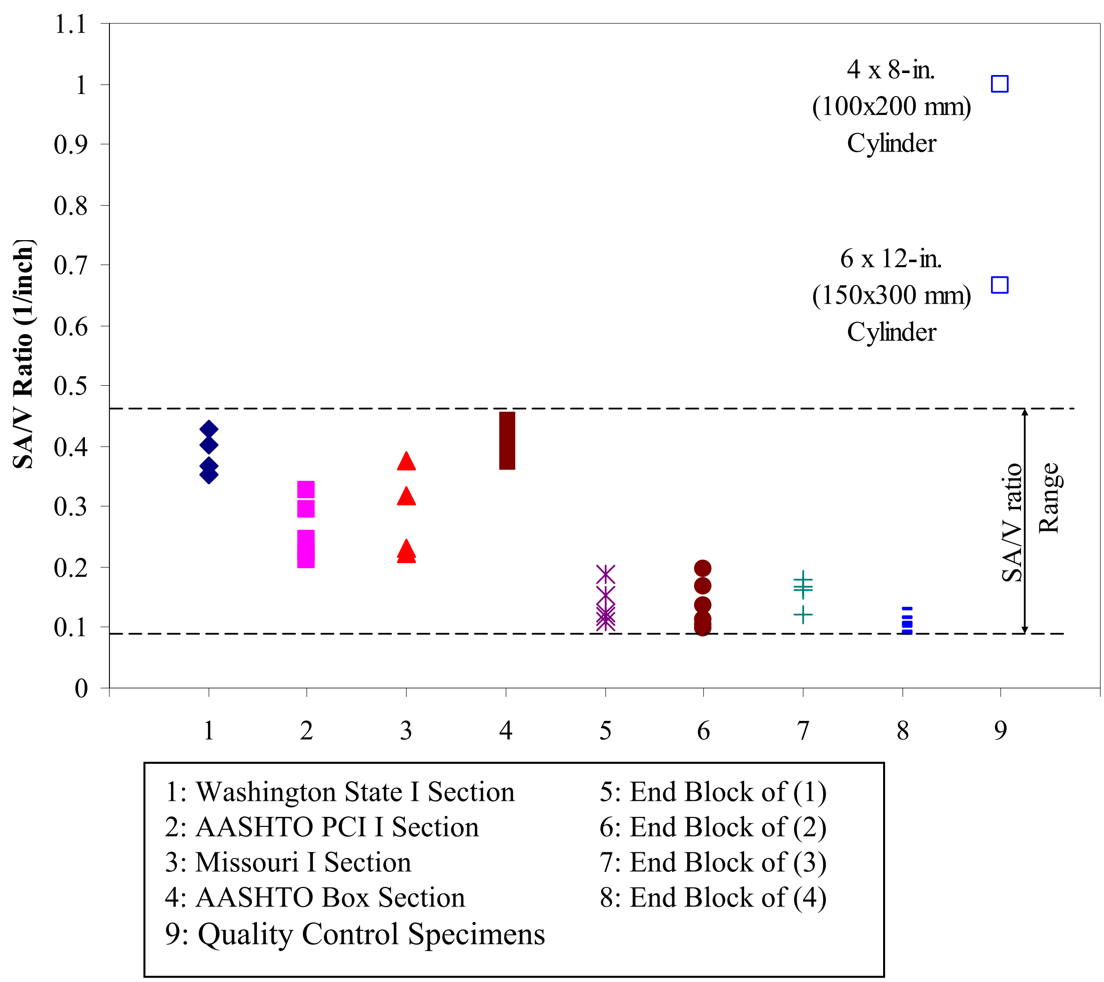


Figure 2.2. Distribution of SA/V ratios of Standard Sections

Table 2.2. SA/V Ratios of End Blocks of Standard Section Beams

AASHTO-PCI Box Beam		AASHTO-PCI I Beam		Missouri I Beam		Washington I Beam	
Type	SA/V (1/in)	Type	SA/V (1/in)	Type	SA/V (1/in)	Type	SA/V (1/in)
BI-36	0.13	Type I	0.20	Type 2	0.18	40	0.19
BI-48	0.12	Type II	0.17	Type 3	0.17	60	0.15
BII-36	0.12	Type III	0.14	Type 4	0.16	80	0.12
BII-48	0.10	Type IV	0.11	Type 6	0.12	100	0.12
BIII-36	0.11	Type V	0.10			120	0.11
BIII-48	0.09	Type VI	0.10				
BIV-36	0.10						
BIV-48	0.09						

Conversion: 1 inch = 25.4 mm

2.3. HIGH PERFORMANCE CONCRETE AND HIGH STRENGTH CONCRETE

2.3.1. Definition. The definition of High Performance Concrete (HPC) and HSC has changed significantly over the years. Prior to the introduction in 1977 of High Range Water Reducer (HRWR), also known as superplasticizers, concrete with a compressive strength of 6000 psi (40 MPa) at twenty-eight days was considered high strength. Current technology can produce concretes with compressive strengths of 8000 psi (55 MPa) in only 24 hours, which reach 28 day strengths in excess of 14,000 psi (100 MPa) [Calton (1995)].

However, strength is one criteria to HPC. Any concrete that satisfies certain criteria proposed to overcome limitations of conventional concretes may be called HPC. It may include concrete that provides either substantially improved resistance to environmental influences (durability in service) or substantially increased structural capacity while maintaining adequate durability. It may also include concrete that significantly reduces construction time to permit rapid opening or reopening of roads to traffic, without compromising long-term serviceability. Therefore, it is not possible to provide a unique definition of HPC without considering the performance requirements of the intended use of the concrete.

Forster (1994) defined HPC as "a concrete made with appropriate materials combined according to a selected mix design and properly mixed, transported, placed, consolidated, and cured so that the resulting concrete will give excellent performance in the structure in which it will be exposed, and with the loads to which it will be subjected for its design life." In discussing the meaning of HPC, Aitcin and Neville (1993) stated that "in practical application of this type of concrete, the emphasis has in many cases gradually shifted from the compressive strength to other properties of the material, such as a high modulus of elasticity, high density, low permeability, and resistance to some forms of attack."

A more broad definition of HPC was adopted by the American Concrete Institute (ACI). HPC was defined as concrete which meets special performance and uniformity requirements that cannot always be practices. The requirements may involve enhancements of (characteristics such as) placement and compaction without segregation, long-term mechanical properties, early-age strength, volume stability, service life in severe environments. Concretes possessing many of these characteristics often achieve higher strength. Therefore HPC is often of high strength, but high strength concrete may not necessarily be of High-Performance.

For the purpose of the SHRP C-205 project [Zia et al., (1993)], HPC was defined in terms of certain target strength and durability criteria as illustrated in Table 2.3. In this definition, the target minimum strength should be achieved in the specified time after water is added to the concrete mixture. The compressive strength is determined from 4 x 8”. (100 x 200 mm) cylinders tested with neoprene caps. The water-cement ratio (w/c) is based on all cementitious materials. The minimum durability factor should be achieved after 300 cycles of freezing and thawing according to ASTM C-666 (AASHTO T-161), procedure A.

Table 2.3. Definition of HPC according to SHRP C-205 [Zia, et al., (1993)]

Category of HPC	Minimum Compressive Strength	Maximum Water/ Cement Ratio	Minimum Frost Durability Factor
Very early strength (VES) Option A (with Type III cement)	2,000 psi (14 MPa) in 6 hours	0.4	80%
Very early strength (VES) Option B (with PBC-XT cement)	2,500 psi (17.5 MPa) in 4 hours	0.29	80%
High early strength (HES) (with Type III cement)	5,000 psi (17.5 MPa) in 24 hours	0.35	80%
Very high strength (VHS) (with Type I cement)	10,000 psi (70 MPa) in 28 hours	0.35	80%

Table 2.4. Definition of HPC according to FHwA [Goodspeed, et al., (1996)]

Performance Characteristics	Standard Test Method	FHwA HPC performance grade			
		1	2	3	4
Freeze-thaw durability	AASHTO T 161, ASTM C666, Procedure A	60%≤X1<80%	80%≤X1		
Scaling resistance	ASTM C672	X2=4, 5	X2=2, 3	X2=0, 1	
Abrasion resistance	ASTM C 944	2.0>X3≥1.0	1.0>X3≥0.5	0.5>X3	
Chloride penetration	AASHTO T277, ASTM C1202	3000≥X4> 2000	2000≥X4>800	800≥X4	
Strength	AASHTO T2, ASTM C39	41≤X5<55	55≤X5<69	69≤X5< 97	97 ≤X5
Elasticity	ASTM C469	28≤X6<40	40≤X6<50	50 ≤X6	
Shrinkage	ASTM C157	800>X7≥600	600>X7≥ 400	400>X7	
Specific creep	ASTM C512	75≥X8>60	60≥X8>45	45≥X8> 30	30 ≥X8

Note:

X1: relative dynamic modulus of elasticity after 300 cycles
X2: visual rating of the surface after 50 cycles
X3: avg. depth of wear in mm (1mm = 0.03937 inch)
X4: coulombs
X5: compressive strength in MPa (1 MPa = 145.0377 psi)
X6: modulus in GPa (1 GPa = 145.0377 ksi)
X7: microstrain
X8: microstrain per MPa (1 MPa = 145.0377 psi)

Based on the results of SHRP C-103 and SHRP C-205 research, the Federal Highway Administration (FHwA) has proposed criteria for four different performance grades of HPC [Goodspeed et al., (1996)]. The criteria are expressed in terms of eight performance characteristics including strength, elasticity, freezing/thawing durability, chloride permeability, abrasion resistance, scaling resistance, shrinkage, and creep as illustrated in Table 2.4. Depending on a specific application, a given HPC may require different grade of performance for each performance characteristic. For example, a bridge located in an urban area with moderate climate may require Grade 3 performance for strength, elasticity, shrinkage, creep, and abrasion resistance, but only Grade 1 performance for freezing/thawing durability, scaling resistance, and chloride permeability.

Such a HPC concrete can be either normal-strength or high-strength. At this time, Normal-Strength Concrete (NSC) defined by ACI is a concrete that has a cylinder compressive strength not exceeding 6000 psi (42 MPa). All other concrete are considered high-strength concretes.

2.3.2. Portland Cement. Portland cement is a hydraulic cement that hardens by interaction with water forming a water-resisting compound upon final set. Compared with non-hydraulic cements such as gypsum and lime, which absorb water after hardening, Portland cement is highly durable and produces high compressive strengths in mortars and concretes. Portland cement is made of finely powdered crystalline minerals composed primarily of calcium and aluminum silicates. The addition of water to these minerals produces a paste which, when hardened, becomes of stone-like strength. Its specific gravity ranges between 3.12 and 3.16 and it weighs 94 lb/ft^3 (1506.8 kg/m^3), which is the unit weight of a commercial sack or bag of cement. Its fineness or particle size can range between 10 and 50 μm. Portland cement is made to meet the specification requirements of ASTM C 150-02 (2002) for type I, IA, II, IIA, III, IIIA, IV, and V, and ASTM C 1157 (2000) for types GU, HE, MH, LH, MS, and HS. The average Portland cement particle size is 15 μm. The principal raw materials of which cement is made are:

(1) Lime (CaO) from limestone

(2) Silica (SiO_2) from clay

(3) Alumina (Al_2O_3) from clay

Cement also contains very small percentages of magnesia (MgO) and at times, some alkalis. Iron oxide is occasionally added to the mixture to aid in controlling its composition [Nawy (2001)].

Table 2.5. Types of Portland Cement Classified by ASTM 150-02 (2002)

	Description
Type I	For use when the special properties specified for any other type are not required.
Type IA	Air-entraining cement for the same uses as Type I, where air-entrainment is desired.
Type II	For general use, more especially when moderate sulfate resistance or moderate heat of hydration is desired.
Type IIA	Air-entraining cement for the same uses as Type II, where air-entrainment is desired.
Type III	For use when high early strength is desired.
Type IIIA	Air-entraining cement for the same use as Type III, where air-entrainment is desired.
Type IV	For use when a low heat of hydration is desired.
Type V	For use when high sulfate resistance is desired.

The strength of cement paste is the result of a process of hydration. This chemical process results in recrystallization in the form of interlocking crystals producing the cement gel, which has high compressive strength when it hardens [Nawy (2001)].

Table 2.6. Types of Hydraulic Cement Classified by ASTM C1157-00a (2000)

	Description
Type GU	Hydraulic cement for general construction. Use when one or more of the special types are not required.
Type HE	High Early-Strength
Type MS	Moderate Sulfate Resistance.
Type HS	High Sulfate Resistance
Type MH	Moderate Heat of Hydration
Type LH	Low Heat of Hydration

2.3.3. Coarse Aggregates. Coarse Aggregate is classified as such if the smallest size of the article is greater than 0.25 inch (6 mm). Properties of the coarse aggregate affect the final strength of the hardened concrete and its resistance to disintegration, weathering, and other destructive effects. The mineral coarse aggregate must be clean of organic impurities and must bond well with the cement gel. The common types of coarse aggregate are (1) Natural crushed stone, (2) Natural gravel, (3) Artificial coarse aggregates, (4) Heavyweight (extra high-density) and nuclear-shielding aggregates [Nawy (2001)].

The important parameters of coarse aggregate are its shape, texture and the maximum size. Since the aggregate is often several times stronger than the paste, its strength is not a major factor for normal strength concrete, or for HES and VES concretes. However, the aggregate strength becomes important in the case of HSC or lightweight aggregate concrete. Surface texture and mineralogy affect the bond between the aggregates and the paste as well as the stress level at which micro-cracking begins. The surface texture, therefore, may also affect the modulus of elasticity, the shape of the stress-strain curve and, to a lesser degree, the compressive strength, these effects will be more pronounced in HES and VES concretes. Tensile strengths may be very sensitive to differences in aggregate surface texture and surface area per unit volume [Zia et al., (1997)].

The effect of the coarse aggregate size on concrete strength was investigated by Cook (1989) who used limestone of two different sizes: 3/8 inch (10 mm) and 1 inch (25 mm). A superplasticizers was used in all the mixes. In general, for a given w/c ratio, the smallest size of the coarse aggregate produced the highest strength; however, it was feasible to produce compressive strengths in excess of 10 ksi (69 MPa) using a 1 inch (25 mm) maximum size aggregate when the mixture was properly proportioned with a HRWR admixture. In the similar study by de Larrard and Belloc (1992) using crushed limestone aggregate, Portland cement, silica fume, and superplasticizers for eight different concrete mixtures, it was shown that better performances and economy could be achieved with 3/4 to 1 inch (20 to 25 mm) maximum size aggregates even though previous researchers had suggested that 3/8 to 0.5 inch (10 to 12 mm) is the maximum size of aggregates preferable for making HSC.

Studies [Neville (1990); Aitcin et al., (1990)] also indicated that the aggregate type plays a role in the bond characteristic. In the case of limestone, dolomite, and siliceous aggregates, a chemical reaction between the rocks and cement matrix can enhance the bond characteristics for HSC.

2.3.4. Fine Aggregates. Fine aggregate is a smaller filler made of sand. It ranges in size from No. 4 [0.187-in (4.75 mm)] to No. 100 (150 μm) U.S. standard sieve sizes. A good fine aggregate should always be free of organic impurities, clay, or any deleterious material or excessive filler of size smaller than No. 100 (150 μm) sieve. It should preferably have a well-graded combination conforming to the American Society of

Testing and Materials (ASTM) sieve analysis standards, particularly that sand has a significant effect on the consistency of fresh concrete. For radiation-shielding concrete, fine steel shot and crushed iron ore are used as fine aggregate.

2.3.5. Effects of Mineral Admixtures. Mineral Admixtures mainly consist of fly ash, slag and silica fume. All of these materials react with calcium in a pozzolanic manner and are thereby referred to as pozzolans. A pozzolan contains amorphous silica which is sufficiently reactive to combine with calcium hydroxide to form C-S-H in the presence of water. When a pozzolan is combined with cement, it will react with the calcium hydroxide formed during hydration. The pozzolanic reaction does not begin until CH is produced by the cementitious reaction. The reaction with the pozzolan converts the CH crystals to the highly cementitious C-S-H. The strength of the paste is thereby increased while the permeability is decreased. However, since the pozzonlanic reaction does not occur until CH becomes available for reaction, it does not contribute to the concrete strength until later-ages. Since the cementitious and pozzolanic reactions occur at different times, the hydration temperature at any given time is generally lower than that of a concrete containing the same volume of Portland cement. The control of temperature can be beneficial in most situations as previously discussed. The effect on the properties of fresh and hardened concrete of each of these mineral admixtures depends on the chemical and physical properties of each. While each mineral admixture will vary depending on its source, there are characteristics which generally apply to each type.

Concrete producers typically choose mineral admixtures that are locally available. When used as a replacement material by producers, the mineral admixture generally reduces the cost of the cementitious material. Along the northern rust belt, slag, fly ash, and silica fume are more typically used while fly ashes are used in the south and southwest. Therefore, it is mainly an economic criteria which drives the usage of a particular mineral admixture in a region [Myers (1998)].

Fly ash Fly ash is a finely divided residue that is a by-product of the combustion of ground or powered coal exhaust fumes of coal-fired power stations. It is composed of siliceous and aluminous ingredients and possesses limited cementitious property unless so finely divided that its fineness is at least equal to or less than the fineness of the cement with which it is mixed. In the presence of the water in the mixture, it reacts with the calcium hydroxide for cement hydration at ordinary temperatures, thereby forming compounds that have high cementitious qualities.

Based on ASTM definition, fly ash is classified into two groups: ASTM Class F and ASTM Class C fly ash. ASTM Class F fly ash normally has a low calcium oxide content (less than 10%), but a high proportion of high silica content silicate glass, quartz, mullites, and magnetites of low reactivity. High-calcium fly ashes gain more early strength than do low-calcium fly ashes. The particle size ranges from 1μm to 1 mm or greater, with a specific gravity of solid fly ash particles normally ranging between 2.2 and 2.8. In ASTM Class C the sum of SiO_2, Al_2O_3, and Fe_2O_3 is equal to or greater than 50% but equal to or less than 70%. It has a high calcium oxide content (more than 10% and often 15-30%). It also has a high proportion of particles finer than 10 μm.

The effects of fly ash on concrete can be described as 2 aspects: the effects on fresh concrete and hardened concrete. ACI 232.2R-96 has pointed out fly ash will improve the workability and pumpability, reduce bleeding, extend the time of setting of the fresh concrete, while has lower compressive strength at early ages but gain higher strength at the later ages, acquire a little bit higher Modulus of Elasticity (MOE) and exhibit better performance in the long-term behavior of hardened concrete [Nawy (2001); Myers (1998)].

Slag There are three main types of slag which include two blast furnace slags (granulated and palletized) and one steel slag. Non-ferrous slag is not being used as an admixture in concrete [Mehta (1983)]. Blast furnace slag is a by-product in the production of pig iron. The type of blast slag is determined by the method used for cooling the molten slag. If the liquid slag is quenched using water, granulated slag results. If the liquid slag is quenched with air and water, pelletized slag results. Both materials must be ground to a fineness of 1953 to 2930 ft^2/lb (400 to 600 m^2/kg) to exhibit satisfactory cementitious and pozzolanic characteristics [Mehta (1983)]. Water demand for slags is generally higher than for an equal volume of cement due to its coarse texture and higher fineness. However, these slags can exhibit similar strengths equal to reference concrete at 7 days [Hogan et al., (1981)].

Silica fume Silica fume is a relatively more recent pozzolanic material that has received considerable attention in both research and application. It is a by-product resulting from the use of high-purity quartz with coal in an electric ore furnace in the production of silicon and ferrosilicon alloys. These are smooth and spherical particles which are nearly pure silicon dioxide and one hundred times finer than cement and fly ash particles. Silica fume is highly pozzolanic in nature and has an extremely high water demand and requires the use of a water reducer. To achieve adequate workability, the slump of mixes containing silica fume should be increased to offset the stickiness or cohesiveness due to the abundance of fine particles. Mixes with silica fume also exhibit reduced bleeding characteristics that require special consideration in curing such as proper fogging to avoid plastic shrinkage cracks [Myers (1998)].

2.3.6. Effects of Chemical Admixtures. The use of chemical admixtures in the production of HSC can significantly improve the quality of the concrete. In order to select the best type and dosage rate of a chemical admixture for a HSC, the admixture should be optimized and tested under the expected field conditions. Chemical admixtures may be separated into several general groups including water reducers and retarders, HRWRs, air entraining admixtures and accelerating admixtures.

<u>Retarders</u> The high cementitious material content and generally low water to cementitious material (w/cm) ratio have made the use of water reducers and retarders a necessity for the production of HSC. Water reducers are an effective means of decreasing a high mixing water demand for proper workability. Water reducers lower a given w/cm ratio while maintaining a given slump resulting in improved concrete strength, impermeability, and durability. The use of retarding admixtures also reduces the mixing water requirement, but more importantly prolongs the plastic state of the concrete mix and aids in reducing the temperature rise of the fresh concrete due to the heat of hydration. Most HSC incorporate very high contents of cement where high heat of hydration temperatures are encountered. These high temperatures can impact mechanical and material property development. By reducing the temperature rise during initial hydration, the later-age properties of the concrete including strength and elastic modulus can be improved. Longer set times, particularly during hot-weather concreting, can benefit the overall quality of the concrete.

High range water reducer (HRWR) The use of HRWR, often referred to as a superplasticizers, is considered to be very important in the production of very high strength concretes with compression strengths in excess of 10,000 psi (68.9 MPa). The use of a HRWR in a HSC mix allows for a significant reduction in the w/cm ratio below a value of 0.40. HRWR's are high molecular weight anionic surfactants derived from sulfonated formaldehyde of naphthalene or melamine. HRWR deflocculates the clumps of cement grains and fluidizes the mixture. The higher dosage rates of HRWR over mid-range or conventional water reducers allows for an increased and more uniform dispersion of hydration products and thereby a more effective use or optimization of the cementitious material.

Accelerating admixtures Set-accelerating admixtures are used to increase the rate of hydration of C_3S. Applications may include emergency concrete repair where very rapid development of rigidity is required or for cold weather concreting where they can offset delay caused by cold temperatures. The use of set-accelerating admixtures with HSC is generally not recommended since it increases the heat of hydration and detrimentally affects the long-term properties of the concrete. Furthermore, in many cold weather concreting applications, temperature related placement concerns are reduced due to the high cement content of most HSC mix designs [Myers (1998)].

2.3.7. Effects of Air Entraining Admixtures. Air-entraining admixtures entrain air in the concrete to provide an air void system for the movement of freezing water within the cement paste. An adequate well distributed air void system provides a relief space for expanding water which minimizes the amount of micro-cracking damage due to moist freeze-thaw cycles. The use of air entraining admixtures for HSC mix designs is only desirable for components subjected to freeze-thaw cycles in excess of the critical saturation threshold (91.7%). Air entrainment is not recommended in other cases since entrained air can decrease a concrete's strength significantly at high dosages. The rule of thumb is that a five percent loss in strength may be anticipated for every one percent of air entrainment. Research at the University of Texas at Austin [Ernzen et al., (1992)] and elsewhere has shown that a minimum of three percent air entrainment is required for HSC. This is slightly lower than conventional mix designs due to the pore structure of HSC with low w/cm ratios. HSC mix designs are naturally more durable due to their higher strength, lower permeability, and decreased porosity. While research studies to date have not specifically investigated the levels of porosity and time-to-saturation for HSC mix designs with low w/cm ratios, it is safe to assume that the reduced porosity of these mixes require a longer period of time to exceed the critical saturation threshold for a given moisture exposure. Air entrainment should only be used in HSC where specifically required for exposure conditions and should always be tested under field conditions due to the high variability of air entrainments with other chemical and mineral admixtures [Myers (1998)].

2.4. STRENGTH OF THE CONCRETE

2.4.1. General Description. Concrete properties such as elastic of modulus, tensile or flexural strength, shear strength, stress-strain relationships and bond strength are usually expressed in terms of uniaxial compressive strength of concrete cylinders that are moist cured to 28 days for NSC or 56 days / 90 days for HSC. Compressive strength is the common basis for design for most structures, other than pavements, and even then is the common method of routine quality control testing. The terms “strength” and “compressive strength” are used virtually interchangeably.

Unlike other engineering materials such as steel, concrete develops its mechanical properties under highly variable conditions in the job site. The inherent variability of constituents combined with different methods of placement and curing can significantly affect the properties of hardened concrete in the field. On the other hand, concrete is a non-homogeneous combination of large aggregate pieces surrounded by mortar and paste which can fail in a number of ways. The non-homogeneous nature of concrete means that merely getting a strength test result, the essential first step in any analysis, is a complicated process.

However, regardless of the difficulties involved in achieving the exact strength of the concrete, people have to obtain the reliable information of the load-capacity of the concrete member. Consequently, standard procedures have been established to provide some consistent points of reference.

2.4.2. Traditional Quality Control Strength. ASTM has regulated the standards of storing, making and curing quality control specimens, such as ASTM C 192-00 (2000) and ASTM C31-00 (2000). The key point for those standards is using the compressive strength of the cylinder cured in standard moist environment for acceptance for specific strength of the target concrete. The reasons for this as stated by Calton (1995) are:

Simplicity: Immersing specimens in a tank of lime saturated water, the easiest way to provide moist curing, is far simpler than trying to account for the complicated effects of temperature and moisture variations in the field. Only recently has the technology become available to monitor certain curing condition inside a concrete structure.

Consistency: Test results from moist-cured specimens are far more consistent than most other means of strength determination. The strength variations of moist-cured specimens are far smaller than those of field cured specimens. Variations in moisture and temperature and the variations in strength they cause are minimized by the moist curing process. Smaller strength variations do indicate a higher level of precision for moist-cured specimens, but not necessarily a higher level of accuracy.

Politics: Using moist cured cylinders gives an indication of the highest potential for strength development in a given sample of concrete. The significance of this value depends on the point of view of the user. An engineer would obviously prefer to know whether the concrete in a structure is capable of carrying certain loads. A concrete producer is primarily concerned with his product's potential strength, unaffected by the placement and curing procedures of a contractor. Thus far, the interests of concrete producers have prevailed.

Although ASTM Moist-Cured cylinders provide the standard for the engineers to evaluate the quality of the concrete, such method eliminate all other factors like placement, member shape and size, curing condition which greatly affects the actual strength in the concrete member. Therefore, member-cured (job-cured) cylinders are introduced to the actual practice. The member-cured cylinders are placed beside or near the actual member in the job site and cured in the similar environment including the ambient temperature and moisture. Curing concrete test specimens at the construction site and under job conditions is considered more representative of the curing applied to the structure. Tests of member-cured specimens is highly desirable especially in HSC application and are necessary when determining the time of form removal which is very important in the Precast / Prestressed Plant. However, the member-cured cylinders shouldn't be used for quality control testing for the acceptance for the specific compressive strength. This is because the in-situ concrete strength can be affected by many factors at the job site, such as size and shape of the section at different locations of the structural member, ambient temperature, type of formwork, accelerated curing temperature and so on, it needs a standard curing condition is needed to evaluate the quality of concrete including materials and batching. Compressive strength specimens of concrete made or cured under other than standard conditions provide additional information but are often analyzed and reported separately.

2.4.3. Ways of Approaching Higher Strength. To produce higher strength concretes, several parameters have to be optimized in addition to mixture design, although the design of the concrete mixture is a major factor in achieving the desired strength. Several methods can be applied to achieve high strength. In general, high strength concrete contains higher quality aggregates, a higher Portland cement content, and a low w/cm ratio. The addition of water-reducing admixtures, superplasticizers, polymers, slag or silica fume is common today. Following are the major factors that have to be taken into account [Nawy (2001), Myers (1998)]:

(1) *Water to Cementitious Ratio (w/cm)*: The production of high strength concrete requires the use of low w/cm ratio. In order to produce concrete with 28 day compressive strengths ranging from 6,000 psi (41.4 MPa) to 14,000 psi (96.5 MPa), a w/cm ratio in the range of 0.25 to 0.52 is required by weight. In addition, a HRWR is required when the very low w/cm ratio are used to produce HSC.

(2) *Cement Characteristics and Content*: The cement used for HSC should have good strength producing properties. The fact that the cements from different manufacturers which meet the same ASTM specification does not mean that concretes produced with each of the different brand cements will perform similarly. It is difficult to predict the interaction of particular cement with other constituents within a mix design. For this reason, trial mixes varying locally available cements is suggested to investigate which cement produces optimum strength and thereby the most cost effective mix design.

(3) *Mineral Admixtures*: The replacement rate of mineral admixtures is different for each type. ACI Committee 211 has established some guidelines for the replacement of Portland cement with other cementitious materials as illustrated in Table 2.6 [ACI 211.4R-93 (1993)]. Silica fume at additional rates in the rage of 7% ~ 15% by weight of cement has been used in the production of high strength concrete. No specific guidelines or recommendations are given in terms of an optimum percentage of replacement materials to use. This may be determined through trial mixes until specification requirements are satisfied.

Table 2.6. ACI 211 Guidelines for Portland Cement Replacement

Type	Portland Cement Replacement Percent by Weight
Fly Ash	
Class F	15% ~ 25%
Class C	20% ~ 35%
Blast Furnace Slag	30% ~ 50%

(4) *Aggregate Quality and Interaction with Cement Paste*: HSC contains a larger volume of coarse aggregate than conventional normal strength mixes. The higher aggregate content in HSC mixes helps to reduce the water demand of the concrete and increase the elastic modulus of the concrete. Since many HSC elements are thinner and have longer span lengths, the elastic modulus of the concrete becomes more critical in addressing serviceability requirements. For fine aggregate, since HSC mix designs have a high content of cementitious materials, the gradation of the fine aggregate is not so important as in conventional normal strength concrete. Recommendations for fine aggregates are those which minimize the mixing water and provide good workability. Generally sands meeting ASTM C 33 requirements and having a fineness modulus between 2.6 and 3.2 perform well in HSC mixes.

(5) *Chemical admixture*: In the production of HSC, it is generally advantageous to use certain chemical admixtures such retarders, water reducers, and HRWRs. Dosage rate recommendations are provided in most cases by the manufacturer. However, the optimum dosage rate of any chemical admixture must be determined through field trial batches conducted under the expected job conditions. In general, the production of HSC will result in the use of higher than recommended dosages of chemical admixtures.

2.4.4. Early Strength of HSC. Traditionally, interest in the strength and other properties of concrete has been focused on those at 28 days and beyond. In the past several years, there has been an increasing interest in the strength and other properties of concrete at ages less than 28 days. There are at least three factors which have contributed to this increased interest in early strength: (1) fast-paced construction schedules that expose concrete to significant structural loads at early ages, (2) development of specialty cements or admixtures that enable the achievement of higher strength at early ages, and (3) recognition that long-term performance of concrete is greatly affected by its early-age history.

At the present time, there is no generally accepted definition of early strength. Any strength measured at ages less than the standard 28 days is regarded as early strength. Since the properties of concrete depend closely on the degree of cement hydration, one definition of early strength could be the strength at the age corresponding to 50% hydration of cement. For concrete made with ordinary Portland cement (ASTM Type I) and cured at a standard temperature of 70^{o}F (21^{o}C), approximately 50% of the cement will hydrate within 3 days. In an Engineering Foundation Conference on Properties of Concrete at Early Ages [Carino et al., (1989)], it was recommended that the early age could possibly be defined as the period during which the properties of concrete undergo rapid change [Zia et al., (1997)].

In this study, the early strength of the concrete is defined as the concrete strength of 24 hours after placement.

2.5. CURING TEMPERATURE OF THE CONCRETE

2.5.1. Curing and Curing Temperature. The strength development with time is a function of the curing techniques as well as constituent materials. ACI Committee 363 defined curing as "*the process of maintaining a satisfactory moisture content and a favorable temperature in concrete during the hydration period of the cementitious materials so that desired properties of the concrete can developed*" [ACI C 363R-92

(1997)]. The temperature mentioned in the above definition maintained for some period after the concrete has set, is called the Curing Temperature.

2.5.2. Effects of Temperature on the Concrete Strength. Hydration of cement is an exothermic reaction and will cause the concrete to reach a peak temperature after some period of time. Dating back to 1958, Klieger (1958) pointed out the influence of the initial and curing temperature on the concrete strength: increasing the mixing and curing temperatures resulted in increased strengths at early ages and considerably lower strengths at later ages. Temperature affected the flexural strength in much the same way (Figure 2.3).

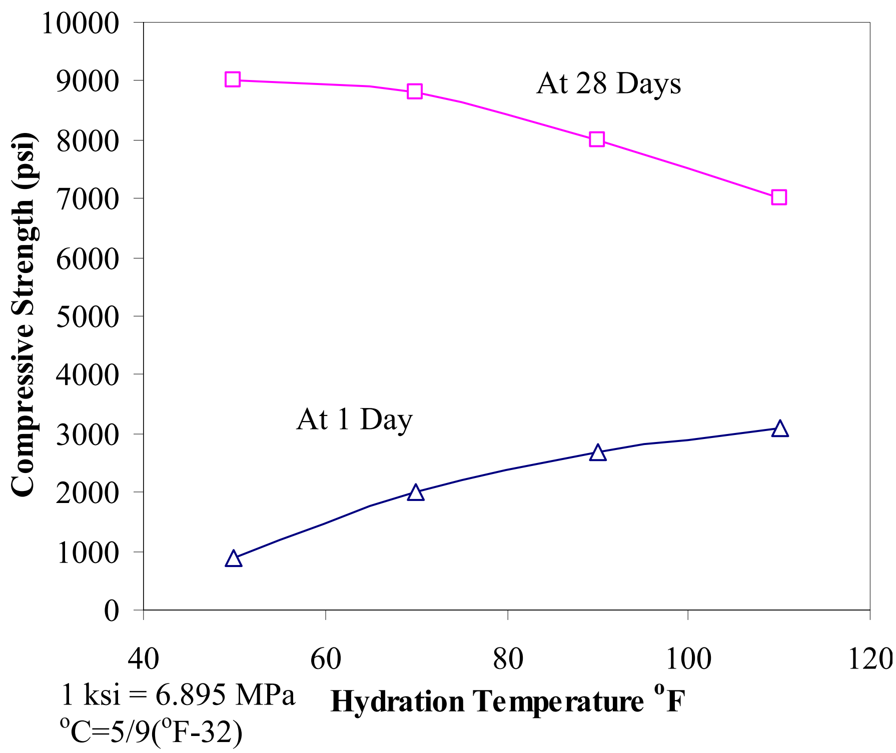


Figure 2.3. Temperature Effect on the Strength of Early and Later-age

Burg (1996) reported that early age compressive strength of concrete cast and cured at high temperature was greater than concrete cast and cured at 73°F (23°C).

However, after 7 days, compressive strength of concrete cast and cured at high temperature was lower than concrete cast and cured at 73^{o}F (23^{o}C). Concrete cast and cured at low temperature had initial strength lower than concrete cast and cured at high temperature. Lura et al., (2001) observed the same situation and concluded that "although higher temperatures improve the initial strength development, the value at a later stage, 14 or 28 days, seems to be penalized, as already observed by other authors."

Myers (1998) reported after monitoring the Louetta Road Overpass U-Beams and North Concho River Overpass AASHTO Type IV Beams, there was a second order polynomial trend line between the maximum hydration temperature and compressive strength at release of prestressing strands. That trend line indicated an improvement in compressive strength up to a hydration temperature of approximately 180^{o}F (82.2^{o}C) and when the temperature was above 160 ~ 180 ^{o}F (71 ~ 82 ^{o}C), micro-cracking was observed and resulted in a strength decrease. He also concluded that the trend line of 56 days compressive strength and maximum hydration temperature indicated a linear decrease in compressive strength with an increase in maximum hydration temperature at later-age. Kehl (1998) reported in his thesis that, at release stage, for temperatures above 160 ~ 180^{o}F (71 ~ 82^{o}C), the situation gets more complicated. Extremely high curing temperatures may affect the crystalline structure development of the concrete. Since cement hydration is accelerated by heat, extremely high curing temperatures may drive the cement to hydrate too quickly. This could create concrete that is unable to develop the same crystalline structure it would at lower curing temperatures. Furthermore, the concrete was observed being unable to fully develop to its potential strength and elastic modulus capabilities. Although there was no in-depth research found in this study discussing the reason for this observation, it may be explained that, it is the poorly developed crystalline structure that prevent the concrete from gaining further strength even when the concrete is fully hardened. For the effect of 24-hour curing temperatures on the 56-day design compressive strength of HSC, the reasons for the continual decrease in compressive strength are probably the same factors discussed in the stage of release. As curing temperatures increases, hydration of the cement accelerates. This could result in concrete that hardens too quickly to develop a strong, dense crystalline matrix it is capable of developing.

The high cement content, normal to HSC, causes elevated temperatures to develop during hydration, despite the fact that the heat release per weight of cementitious materials decreases at low w/cm ratio due to the lack of available water for hydration [FIB/CEB (1990)]. Cook et al., (1992) observed that the temperature rise during hydration increases as the cement content and concrete strength increase. This explains the fact that high strength concrete shows a higher rate of strength development than conventional concrete at early-ages.

The effect of curing temperature on the strength development of fly ash concretes was also investigated. Barrow and Carrasquillo (1988) observed that an increase in the curing temperature resulted in a greater strength increase in the fly ash concrete than in the control concrete. They concluded that fly ash concrete is more temperature sensitive. Owens (1985) reported that the deleterious effect of elevated temperatures due to hydration at early ages is more pronounced for a control concrete than for fly ash concrete. Leshchinsky and Velichko (1991) noted that the use of fly ash in concrete subjected to heat treatment results in an increase in strength.

2.5.3. Factors Affecting Hydration Temperature. According to Harrison (1983), there are 5 factors that affect the hydration temperature. They are 1) types and quantities of cementitious materials; 2) size and shape of the section; 3) insulating effectiveness of the formwork; 4) concrete placing temperature and 5) ambient condition. The actual hydration temperature within the member is some combination of those five factors. Furthermore, the effect of the concrete temperature appears to be most critical in the first 24 to 48 hours of curing [Plum (1989)].

1) *Types and Quantities of Cementitious Materials*: Types and quantities of cementitious materials play the most important role in the hydration temperature. It is accepted that the concrete temperature increases about 10 ~ 12 ^{o}F / 100 lb cementitious material / yd^3 (9.38 ~ 11.3 ^{o}C / 100 kg cementitious material / m^3). It is the major source of the heat within the concrete. Carlton (1995) verified the above relationship for both Type I and Type III cement. The cementitious materials most commonly consist of cement and fly ash, and both materials influence the amount of heat generated during hydration. However, it has been shown that the partial replacement of cement with fly ash can reduce the total temperature rise of the concrete and delay the time at which the maximum temperature occurs [Mani et al., (1990)].

2) *Section Shape and Size*: Surface Area-to-Volume (SA/V) ratio can be used as the indicator of the section shape and size of the member. SA/V ratio has a large effect on the temperature rise. It has been documented that larger concrete sections can produce larger internal temperature rises [Cannon (1987)]. As the internal heat of hydration cannot dissipate as quickly as possible in massive sections as it does in thin or small section. This is because the SA/V ratio is very low in the massive member. Plum also

found the internal temperature profile of a thick section follows a temperature profile similar to the adiabatic temperature profile of the concrete [Plum (1989)].

3) *Formwork, Placing Temperature and Ambient Condition*: The insulating effectiveness of the formwork can cause heat to be retained or dissipated in the concrete. The placing temperature of the concrete sets the level at which heat generation begins. And the ambient conditions affect how much heat is contributed from the environment to the concrete [Kehl (1998)].

The factors of cementitious material and SA/V ratio of the member are internal characteristics of the member that can be designated off-jobsite. The factors of formwork, placing temperature and ambient condition are external conditions that can only be controlled in the field. All those factors are important and should be considered when predicting the internal temperature rise of the concrete.

2.6. CURRENT STRENGTH VERIFICATION METHODOLOGY

The primary means of determining the quality of concrete is the molded, standard cured cylinders. However, even under the fairly restrictive guidelines, there are still many factors that make the test results variable. Therefore, several standard test procedures have been established by ASTM to provide more reliable strength data.

2.6.1. ASTM Moist Cured Cylinders. ASTM Moist Cured Cylinders are made for control specimens. They are molded and kept in a moist room where the temperature is $73 \pm 3\ ^{o}F$ ($23 \pm 2\ ^{o}C$) until the moment of test. ASTM Moist Cured Cylinders are used in the field for 1) Acceptance testing for specified strength; 2) Checking adequacy of mixture proportions for strength; and 3) Quality control [ASTM C31–00 (2000)]. In the laboratory, they are able to be used for 1) Mixture proportioning for project concrete; 2) Evaluation of different mixtures and materials; 3) Correlation with nondestructive tests; and 4) Providing specimens for research purposes [ASTM C192–00 (2000)].

2.6.2. Member Cured Cylinders. Member Cured Cylinders are stored in or on the structure, near to the point of deposit of the concrete represented as possible. They are provided with the same temperature and moisture environment as the structural work until the time of release of the strands or removal of the formwork. Member Cured Cylinders are made for 1) Determination of the time the structure is permitted to be put in service, 2) Comparison with test results of standard cured specimens or with test results from various in-place test methods, 3) Adequacy of curing and protection of concrete in the structure, and 4) Form or shoring removal time requirements [ASTM C31-00 (2000)].

2.6.3. Accelerated Cured Cylinders. The Accelerated Cured Cylinders are exposed to accelerated curing condition that permit the specimens to develop a significant portion of their ultimate strength within a time period ranging from 5 to 49 hours, depending upon the procedure that is used. There are four standard procedures: A) Warm Water, B) Boiling Water, C) Autogenous, and D) High Temperature and Pressure. The significance for this type of cylinders is: 1) to provide and indication of the potential strength of a specific concrete mixture at the earliest practical time, 2) to provide information on the variability of the production process for use in quality control, 3) to evaluate concrete strength by using the accelerated early strength obtained from any of the four procedures in the same way conventional 28-day strengths have been used in the past, with suitable changes in the expected strength values [ASTM C684–99 (1999)].

2.6.4. Effects of Testing Variables. Even when the quality control cylinders are made identical, there are still many testing variables that affect the final test results. The variables summarized by Zia et al., (1997) in their State-of-the-Art report included mold type, specimen size, end conditions, and rate of loading. Each variable will be addressed in the following sections.

Mold type The effect of mold type on strength was reported by Carrasquillo et al., (1988a) that use of 6 x 12-inch (150 x 300 mm) plastic molds gave strengths slightly lower than steel molds and use of 4 x 8-inch (100 x 200 mm) plastic molds gave negligible difference with steel molds. A report by French and Mokhtarzadeh (1993) also indicated that the compressive strength of a cylinder cast in 6 x 12-inch (150 x 300 mm) heavy-gauge reusable steel molds was 2.5% higher than that of cylinders cast in flexible single-use plastic molds. It appeared that as long as the manual rodding method was used to consolidate the concrete, the effect of mold type on the compressive strength of concrete was insignificant. Carrasquillo et al., (1988a) recommended that steel molds should be used for concrete with compressive strengths up to 15 ksi (103 MPa). For higher strength concrete, it seems logical that steel molds should also be used.

Specimen size Many studies [Tanigawa et al., (1990); Baalbaki et al., (1992); French et al., (1993); Aitcin et al., (1994); Carino et al., (1994)] have been conducted to investigate the specimen size effect on the compressive strength. Comparisons were usually made between the compressive strength 4 x 8-inch (100 x 200 mm) cylinders and that of 6 x 12-in. (150 x 300 mm) cylinders. Generally, 4 x 8-inch (100 x 200 mm) cylinders exhibit higher strengths than 6 x 12-in. (150 x 300 mm) cylinders due to the least flaws effect. The difference may vary from 2% to 10% with a common value being

5%, and the difference is lower for higher strength concrete. Burg and Ost (1992) reported, however, that their test data showed that the strength of 4 x 8-inch (100 x 200 mm) cylinders was within 1% of the strength of 6 x 12-in. (150 x 300 mm) cylinders. A contradiction to this trend is the study reported by Carrasquillo et al., (1988a) which showed that the compressive strength of 4 x 8-inch (100 x 200 mm) cylinders were approximately 7% lower than 6 x 12-in. (150 x 300 mm) cylinders. However, it is generally accepted that the strengths of 4 x 8-inch (100 x 200 mm) cylinders are 5% higher than that of the 6 x 12-in. (150 x 300 mm) cylinders.

End conditions The preparation of the end conditions (cappings) of the concrete cylinder can significantly affect the measured compressive strength. Generally speaking, the standard sulfur mortar capping is suitable for concrete strength up to about 7.5 ksi (52 MPa). For higher strength concrete, different procedures are used to prepare the end conditions of cylinders for compressive testing. One procedure is the parallel grinding of the ends of the cylinders, thereby eliminating the need for end caps. While grinding is regarded as the best procedure, it entails expensive equipment and longer preparation time so that it is not practical for field application. Another procedure is the use of an unbonded cap consisting of a restraining cap and an elastomeric pad as insert. The unbonded cap system is far more cost-effective and can be easily equipped by any laboratory and used in the field. A previous study [Carasquillo et al., (1988b)] showed that for concrete strengths between 4 and 10 ksi (28 and 69 MPa), the use of polyurethane inserts with aluminum restraining caps produced average test results within 5% of those obtained using sulfur mortar caps. For concrete strengths below 11 ksi (76 MPa), the use of neoprene inserts with steel restraining caps yielded average test result within 3% of those obtained using sulfur mortar caps. For higher strength concrete, the use of either unbonded capping system became questionable. A new technique for the unbonded cap system has been developed recently in France [Boulay el al. (1992); Boulay and de Larrard (1993)] in which the neoprene insert is replaced by dry sand (the sand box). The strength of the confined sand seems to have no limit and when used as the capping system, the results it produces are comparable to those obtained with grinding for concrete strength between 7.25 to 11.6 ksi (50 to 80 MPa).

Rate of loading It is generally understood that the measured compressive strength of concrete increases with increasing rate of loading. Many studies of the subject have been conducted over the past seven decades, covering a wide range of strength of concrete of 2.5 ~ 8.8 ksi (17.4 to 60.4 MPa), strain rate (10-6/s to 10/s), specimen size and shape (cylinder, cube, and prism), curing and testing conditions (wet and dry), and loading mechanism (electrohydraulic servosystem, pressure-activated piston system, ballistic pendulum and drop hammer apparatus).

2.6.5. Allowance Criteria for Testing. To date, there is no definition for allowance criteria to recommend the use of match curing technology to accurately predict the mechanical properties of the HSC in the members.

In this research study, ACI 318-99 section 5.6.3.3 is used as the with-in test base of allowance criteria for cylinder acceptance. ACI 318-99 Section 5.6.3.3 says: "*Strength level of an individual class of concrete shall be considered satisfactory if both of the following requirements are met: (a) Every arithmetic average of any three consecutive strength tests equals or exceeds* f'_c; *(b) No individual strength test (average of two cylinders) falls below* f'_c *by more than 500 psi.*" In the "State-of-the-Art Report on High-

Strength Concrete" reported by ACI Committee 363 [ACI 363R-92 (1997)], section 4.8.2 summarized that the standard deviation for HSC became uniform in the range of 500 ~ 700 psi (3.5 ~ 4.8 MPa). Therefore, 500 psi (3.5 MPa) is used as the cylinders acceptance deviation value. If a required design strength of HSC is 10,000 psi (70.0 MPa) for example, using this criteria the allowance deviation percentage can be determined as 500 psi / 10,000 psi = 5%.

In the discussion of this research study, a deviation of 5% is adopted as the allowance criteria. This value is somewhat arbitrary but was selected as a conservative level to recommend match curing technology.

2.6.6. Current Researches on the Quality Control Testing of HSC in the Precast / Prestressed Plant. Many researches on the quality control testing of HSC in the precast / prestressed plant have been conducted recently. It has been found [Carlton (1995); Kehl (1998); Myers (1998)] that HSC precast beams experience significant heat development during curing, which can result in large temperature gradients and reduced strength at later ages. At the same time, the difference of temperatures within the member and quality control cylinders can be as much as 77^{o}F (43^{o}C) [Carlton (1995)]. Myers et al., (1997) reported that member cured cylinders used to determine release of the prestressing strands underestimated the compressive strength by as much as 27.9% or average of 19.1% for the mix design investigated based on the results of the match cured cylinders at release while ASTM moist cured cylinders used to verify the design strength at 56 days overestimated the compressive strength by as much as 15.9% or 9.8% based on the results of the match cured cylinders at 56 days. Kehl (1998) concluded that current procedures for curing quality control specimens in the precast concrete industry produce specimens which are subjected to curing temperatures that can be very different than the actual concrete in the member. He further recommended that match curing technology should be used for curing quality control specimens in the precast concrete industry without regard to SA/V ratio or cement content.

2.7. MATCH CURING TECHNOLOGY

Match curing technology, which is also referred as temperature-matched curing, can be tracked back to 1920's. During that time, laboratory research in England and the United States started to investigate the effect of cement types, cement contents, and concrete placing temperatures on the temperature rise and strength development of the concrete. This was done by monitoring the temperature conditions within Mass Concrete and match curing specimens at the same temperature profile. The specimens were then compared with standard specimens cured at normal room temperatures [Cannon (1987)]. In the 1970's, the John Laing Research and Development company in England developed a temperature-matched curing system that was available for hire and use in the field. Several companies made use of the equipment for field applications that included the determination of the curing time required before removal of formwork. In the 1980s, the uses for match curing technology expanded to include determining stress transfer times and uses on slip form operation projects [Cannon (1987)]. In 1989, Hirst described the use of thermal matched cubes to speed up fast track construction on a major building in Blackwall Yard, London [Hirst (1989)].

Recently the match curing technology has been used in research and field environments where the actual strength of the concrete needs to be evaluated. The primary use in the field is to determine the time of the formwork removal and prestress strand release and in some states such as Texas, it is also used for strength verification.

2.7.1. Maturity Method. Maturity Method is defined by ASTM C1074 as a technique for estimating concrete strength that is based on the assumption that samples of a given concrete mixture attain equal strengths when subjected to an equal value of the maturity index. The theory is based on the fact that temperature and time are two critical factors that affect the strength of the concrete, especially in the first 24 hours after placement. The maturity of the concrete is determined by multiplying an interval of time by the temperature of the concrete during that interval. This product is summed over time, and the maturity of the concrete is equal to the sum of these time-temperature products.

The maturity rule [Saul (1951)] stated that, provided moisture was available for hydration, samples of the same concrete would have equal strengths when maturity indices were equal irrespective of their actual temperature histories. As seen in Figure 2.4, regardless how the temperature developed, if the maturities of each profile are equal (M1=M2), the compressive strengths are equal. Therefore maturity method becomes one of the techniques for estimating the in-place strength gain of the conventional concrete. The field procedure is also simple involving measurement of the temperature profile within the member to deduce to maturity index, which is then used to estimate the in-situ strength based on the previously established relationship between strength and the maturity index.

In the Maturity Method, selecting the proper Maturity Function and Maturity Index are the important issues. Currently there are two maturity functions being used widespread:

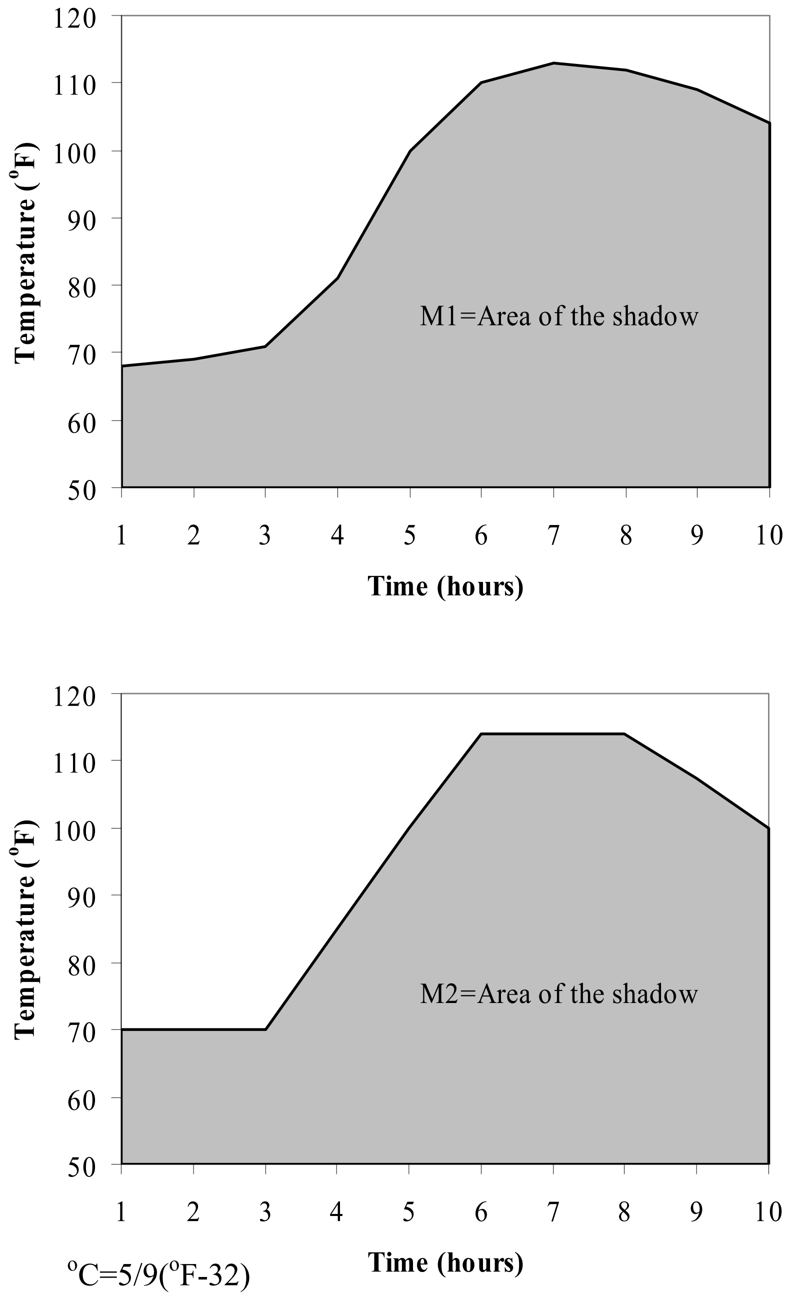


Figure 2.4. Maturity of the Fresh Concrete

1) *Temperature-Time Factor*: Temperature-Time Factor, also known as Nurse-Saul Function, was one of the earliest and still widely–used functions for computing a maturity index and is defined as follows:

$$M = \sum_{0}^{t}(T_a - T_0)\Delta t \qquad (1)$$

Where:

M = Maturity Index at age t, ^{o}C-hours or ^{o}C-days

T_a = Average concrete temperature during time interval Δt, ^{o}C

T_0 = Datum temperature, ^{o}C

Δt = Time interval, hours or days

Because of its simplicity, Eq. (1) has won widespread acceptance. However, It has some inherent limitations. Malhotra (1975) presented some constraints for the maturity method to be valid as follows: (1) Initial concrete temperature is between 60 and 80 ^{o}F (15.6 and 26.7 ^{o}C), (2) Concrete is maintained in a environment that permits further hydration, and (3) Maturity is represented by curing at normal temperatures between the ages of 3 and 28 days.

2) *Equivalent age*: To overcome the shortages of the Temperature-Time Factor, Freiesleben Hansen and Pedersen presented an improved function as an alternative to the temperature-time factor. This function converts the measured temperature history of the concrete to an equivalent age of curing at the reference temperature. The Equivalent Age t_e is defined in Eq. (2):

$$t_e = \sum_{0}^{t} e^{-\frac{E}{R}\left(\frac{1}{T_a}-\frac{1}{T_r}\right)} \Delta t \qquad (2)$$

Where:

t_e = The equivalent age at the reference temperature, hours and days

E = Activation Energy, J/mole

R = Universal gas constant, 8.314 J/oK-mole

T_a = Average absolute temperature of the concrete during Δt, ^{o}K

T_r = Absolute reference temperature, ^{o}K

Δt = Time interval, hours or days

The investigation of the applicability of the maturity method to High Performance of Concrete was conducted by Carino et al., (1992). A revised maturity function Eq. (2) of Equivalent Age is defined in Eq. (3):

$$t_e = \sum_0^t e^{-B(T-T_r)} \Delta t \qquad (3)$$

Where:

t_e = The equivalent age at the reference temperature, hours and days

B = Temperature sensitivity factor, $^{o}C^{-1}$

T = Concrete Temperature, ^{o}C

T_r = Reference temperature, ^{o}C

Δt = Time interval, hours or days

Eq. (3) provides simpler equation and more physical significance compared with the apparent activation energy. Carino et al., (1992) concluded that (1) the maturity method is applicable for describing the effects of time and temperature on the strength development of the low w/cm ratio mixtures that were investigated, (2) the linearhyperbolic strength-age model is suitable for the mixtures that were investigated, (3) the rate constant for strength development can be related to the curing temperature using an exponential function in which the temperature sensitivity factor B, is used to describe the temperature dependence, and (4) the “limiting” strengths computed by fitting the strength-age models did not decrease in a consistent manner as the curing temperature increased.

Further studies were conducted by Cetin (1995). Cetin illustrated how the maturity method underestimated the early strength and overestimated the later strength for accelerated heat cured specimens. The high cementitious content of the HSC mix and the mass sections of concrete created a “burn-out” effect (high initial strength gain followed by limited long-term strength gain) as described by Cetin. He concluded that the traditional maturity-strength relationship based on standard cured cylinders does not appear to be representative of strengths of concretes subjected to accelerated hear curing.

The limitations of applying maturity method for predicting the strength development of the concrete are pointed out by Mindness et al., (1981): (1) the maturity functions do not consider the humidity conditions during the curing that is very important for concrete hydration, (2) the maturity functions cannot be applied to massive concrete, for the heat loss from such member is far less than the normal specimens. Only ambient temperatures are considered while the contributions from the heat of hydration are ignored, (3) they are not suitable for low maturities where significant strength gain has not yet begun, (4) they are not applicable when large temperature variations in the concrete, (5) strength is also affected by both the chemical composition and the fineness of the cement since these both affect the rate of hydration. The w/cm ratio may also influence the results, and (6) although there seems to be an initial correlation between maturity and strength for accelerated strength tests, the relationship breaks down when the accelerated cure specimens are subsequently cooled and moist cured.

2.7.2. Match Curing System. Based on the Maturity Method, the Match Curing System tries to cure the specimens under the same temperature development profile within the member. The actual temperature profile that the Match Curing System follows can be from the concrete member measured by thermocouple wires in the real time, or from pre-programmed data in the controller. The system controls the heater / cooler to let the match-curing specimens keep the same temperature as desired. Figure 2.5 shows how the match curing system works.

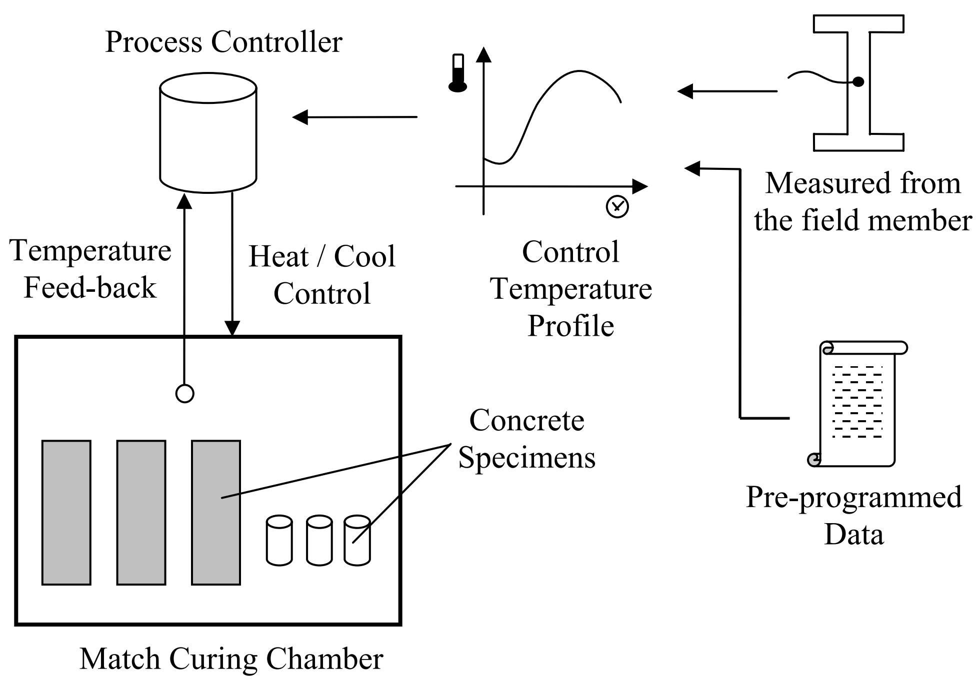


Figure 2.5. Flow Chart of Match Curing System

In practice, based on the requirements and needs, the Match Curing System can be as large as an Environmental Chamber, or as small as just a Heater Cylinder Mold. Some of the Match Curing Chambers only have the heaters to rise the temperature within the container, which required the ambient temperature shall lower than the expected lowest curing temperature.

2.7.3. Benefits for Match Curing Technology. Match Curing Technology provides the method for evaluating the actual concrete strength within any point of the structural member, as well as simulating any hydration temperature development profile for researches. It can remove the unreliable estimation only based on experiences from concrete casting operations with accurate prediction of time for formwork stripping, prestressing or lifting operations. This could be very critical for in-situ construction of buildings, roads, bridges and tunnels, as well as for controlling the curing rate in the manufacture of concrete products such as reinforced and prestressed concrete elements, railway sleepers and pipes.

Direct benefit for the match curing technology, as stated by Myers (1998), was allowing the precaster to strip forms and release pretensioned strands approximately 2 hours sooner for the section shape and mix design investigated compared to conventional quality control specimens.

3. EXPERIMENTAL PROGRAM

3.1. INTRODUCTION

The experimental program for the investigation of the influence of the surface area-to-volume (SA/V) ratio on the quality control of high performance concrete is described in the this chapter.

The program was broken down into two studies with a total of four phases. The first part is the Laboratory Study, which includes Phases I, II, and III. The second part is the Field Study, which includes Phase IV. Phase I measured the temperature development profiles of eight high strength concrete cubes that represent eight different SA/V ratios ranging from 0.25 1/inch to 1.0 1/inch. These cubes were subjected to 3 ambient temperatures of 70^{o}F, 130^{o}F, and 160^{o}F (21.1^{o}C, 54.4^{o}C, and 71.1^{o}C), 2 water to cement ratios (w/cm=0.25, w/cm=0.30) and 2 fly ash replacement levels (with / without 35% replacement of cement with fly ash by weight). Phase II established the equations for the relationship between peak hydration temperature and SA/V ratio and modeled the temperature development profiles of each condition from the data obtained in phase I. Four values of SA/V ratio: 0.1, 0.2, 0.4, 0.6 1/inch, were selected as the standard key points for match curing in the next phase. Phase III match-cured groups of 4 x 8-in. (100 x 200 mm) cylinders following the programmed temperature profiles developed in Phase II. Release (24 hours) and 56-days compressive strength and modulus of elasticity of cylinders were measured respectively during this phase. Phase IV measured the temperature development profiles and in-situ strengths at different locations in a Missouri Type 2 girder fabricated at Egyptian Concrete Co. in Bonne Terre, Missouri. Eight cubes similar to Phase I were fabricated using the same concrete as the girders. These were produced to monitor their temperature profiles and in-situ strengths as a supplement to the results in the laboratory (as illustrated in Figure 3.1).

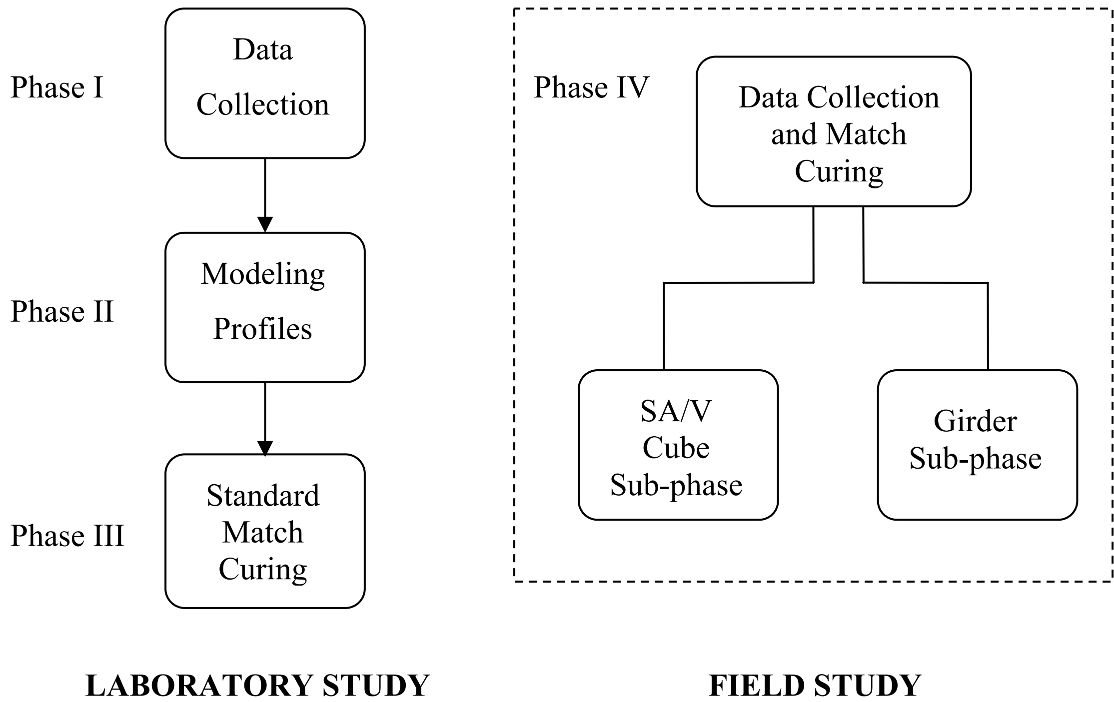


Figure 3.1. Flow Chart of the Research

3.2. GENERAL DESCRIPTION OF THE EXPERIEMENT

3.2.1. Materials. This experimental study involved the use of Portland cement, coarse aggregate, fine aggregate, fly ash, mix water and High Range Water Reducer (HRWR). Locally available materials including coarse/fine aggregate and mix water were used in both laboratory and field studies. Table 3.1 lists the type and source of materials used in this research study.

Table 3.1. Material Selections in the Research

Material	Type	
	Laboratory Study	Field Study
Portland Cement	HOLNAM, TYPE GU, I (see Appendix E)	RIVER, TYPE GU, I
Coarse Aggregate	9/16" (14.3 mm) Gasconade Formation (see Appendix G)	Iron MT Trap Rock
Fine Aggregate	Little Piney Creek at Rolla, MO (see Appendix G)	Mississippi river Sand CL-A
Fly Ash	ISG Resources, Class C (see Appendix F)	n/a
Mix Water	Rolla, MO	City of Bonne Terre
HRWR	RHEOBUILD3000FC, Type F (see Appendix H)	Adva Cast
Silica Fume	n/a	Force 10000~10004
Retarder	n/a	Daratard 17 ~ 21

3.2.2. Concrete Mix Designs. Water to Cementitious (w/cm) ratio and fly ash content are two major mix design factors affecting HSC. In this study of the laboratory study, two w/cm ratios: w/cm=0.25 & 0.30, were selected and the effects of 35% fly ash replacement by weight were also studied. The reason for selecting w/c=0.25 and 0.30 in this study was that they are mostly utilized in the precast / prestressed plant in practice and 35% replacement of Fly Ash Class C is the maximum limit specified by ACI 211 (as seen in Table 2.6). Therefore, three mix designs were investigated in the laboratory study (summarized in Table 3.2).

In concrete mix design used in the field section is followed by the same mix design as used in the precast / prestressed HSC girder at Egyptian Concrete Company in Bonne Terre, Missouri. The typical mix deign is shown in Table 3.3. The purpose of using the same mix design in the plant is to verify the applicability of experiments completed in the lab.

Table 3.2. Three Concrete Mix Designs in Laboratory Study

	Mix Designation			Unit	Description
	L25	L30	L25FA		
w/cm	0.25	0.3	0.25		
Coarse Aggregate	1918	1965	1918	lb/yd^3	9/16" Gascornade Formation
Fine Aggregate	1029	928	1029	lb/yd^3	Little Piney Creek at Rolla, MO
Cement	988	850	642	lb/yd^3	Type I
Fly Ash	0	0	346	lb/yd^3	ASTM Class C
Mix Water	247	255	247	lb/yd^3	Rolla, MO
HRWR	200	127	200	oz/yd^3	ASTM Type F

Conversion: 1 lb/yd^3 = 0.5933 kg/m^3

Table 3.3. Concrete Mix Design (F25) at Egyptian Concrete in Bonne Terre, MO

Material	Quantity	Unit	Description
Coarse Aggregate	1987	lb/yd^3	Iron MT Trap Rock
Fine Aggregate	927	lb/yd^3	Mississippi River sand CL-A
Cement	850	lb/yd^3	Type I
Mix Water	225	lb/yd^3	City of Bonne Terre
HRWR	90	oz/yd^3	ASTM C-494 Type F
Silica Fume	50	lb/yd^3	Force 10000~10004
Retarder	3~4	lb/yd^3	ASTM C-494 Type D
Air Entrainment	4.5~6.5%	$fl.oz/yd^3$	Daravair 1400 ~ 1404
w/cm	0.25		
Slump	8	inch	

Conversion: 1 lb/yd^3 = 0.5933 kg/m^3; 1 inch = 25.4 mm

3.2.3. SA/V Cube Selections. Selecting the dimension and size of the cubes was based on section SA/V ratios and size constraints in the lab. After careful review of section SA/V, as much as eight cube molds were selected to produce 8 concrete cubes with SA/V ratios ranging from 0.25 ~ 1.0 1/inch (see in Figure 3.2). These ratios represent the most standard cross-sections in fields and quality control cylinders as tabulated in Table 3.4.

Table 3.4. SA/V Cube Selection

No.	SA/V (1/in.)	Dimension (LxWxH in.)	Volume (ft^3)	Description
1	1.00	6 x 6 x 6	0.125	Represents 4x8-in. cylinder
2	0.67	9 x 9 x 9	0.422	Represents 6x12-in. cylinder
3	0.50	12 x 12 x 12	1.000	Represent Standard Cross-Sections
4	0.45	13.5 x 13.5 x 13	1.371	
5	0.40	15 x 15 x 15	1.953	
6	0.35	17 x 17 x 17.5	2.927	
7	0.30	20 x 20 x 20	4.630	
8	0.25	24 x 24 x 24	8.000	

Conversion: 1 inch = 25.4 mm

The SA/V cube molds were designed to be used repeatly. Each mold can be assembled and disassembled very easily. The thickness of the plywood plate is 0.75-in. (19.05 mm). That can let the mold keep its stiffness and shape during concrete placement and curing.

Figure 3.2. Representative SA/V Cube Plywood Molds

3.2.4. Curing Temperature. Curing temperature is one of the most important factors that affects the hydration temperature within the concrete member. The principal of selecting the accelerated curing temperature was based on practice and current DOT restrictions. A survey of DOT's codes for ambient and accelerated curing temperature is tabulated in Table 3.5.

Table 3.5. Representative DOT's Specifications for Ambient and Curing Temperatures

	Preset Time (Hour)	Maintain Ambient Temperature (^oF)	Steam Curing Rate (^oF)	Maximum Steam Curing Temperature (^oF)
MoDOT	4	50 ~ 70	<40	<160
TxDOT	3	n/a	<40	<180, 165 ~ 180 <1hr
IlDOT	n/a	n/a	<40	<160
MnDOT	3	50 ~ 59	<27	<158
RiDOT	4	50 ~ 59	<40	<150

Conversion: $^oC = 5/9(^oF - 32)$

As shown in Table 3.5, the DOT specifications referenced have similar requirements for maximum steam curing temperatures and maintaining ambient temperatures. The final design of curing temperature for this research is included as Table 3.6. Room temperature was varied from 60 to 75 ^{o}F (15.6 to 23.9 ^{o}C).

Table 3.6. Curing Temperature in this Research

Curing Temperature (^{o}F)	Preset Time (Hour)	Steam Curing Rate (^{o}F)
Room	n/a	n/a
130	4	<40
160	4	<40

Conversion: $^{o}C = 5/9(^{o}F - 32)$

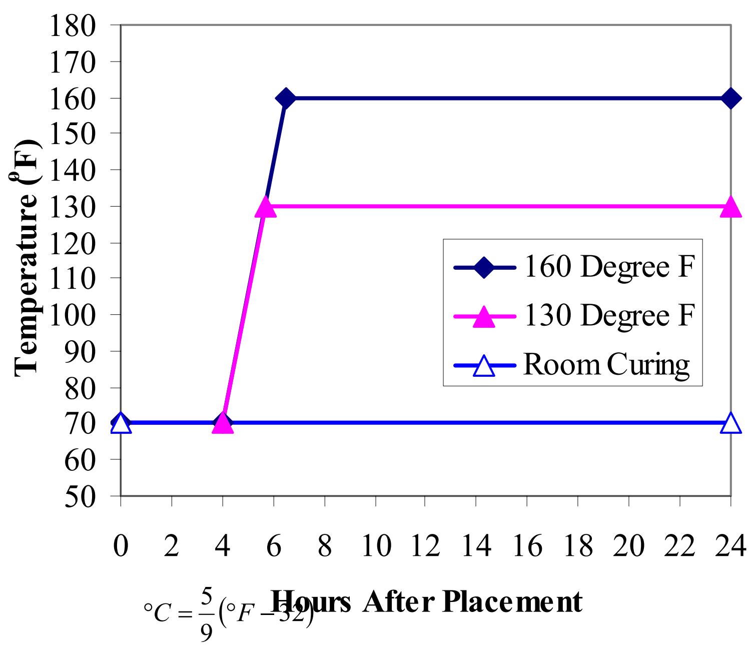


Figure 3.3. Accelerated Curing Temperature Profiles

In order to reflect the behavior of HSC and simulate the actual practice in the field, accelerated curing temperature profiles for 130°F (54.4°C) and 160°F (71.1°C) were developed as shown in Figure 3.3. Those two profiles were pre-programmed before curing the specimens in an environmental chamber that can automatically control the temperature within the chamber while were implemented by manually controlling the temperature during the time of curing specimens in the oven.

3.2.5. Curing Conditions & Specimens. Curing conditions play an important role in material properties development. The quality control specimens can have different test results when cured in the different curing conditions. For this research, three curing conditions were considered and studied: ASTM Moist Cured Cylinder, Member Cured Cylinder and Match Cured Cylinder. All of the specimens were 4 x 8-inch (100 x 200 mm) cylinders and were made according to the procedures of ASTM C192-00 (2000) in the laboratory study and ASTM C 31-00 (2000) in the field study. For ASTM Moist Cured and Member Cured Cylinders, the molds were made of plastic. For match cured cylinders, the molds were made of steel.

<u>ASTM moist cured cylinder</u> This type of cylinder represents the standard quality control cylinder used for testing the design strength of concrete. In the laboratory study, cylinders were kept in a moist room at the temperature of 73 ± 3°F (23 ± 2°C) with a relative humidity of not less than 95%. The cylinders in the field study were kept in a 70°F (21.1°C) room for approximately 24 hours after casting. They were then demolded and cured at 73 ± 3°F (23 ± 2°C) in a moist-curing room until the moment of test [as seen in Figure 3.4 (a)]. This group of cylinders was used as the control specimens.

Member cured cylinder In this curing condition, the cylinders were placed directly next to the member (SA/V cubes in laboratory or girders in field) until the time of demold (24 hours after casting in laboratory) or release (approximately 2 days after casting in the field). Cylinders were all moved into the moist room at the temperature of 73 ± 3°F (23 ± 2°C) until testing [as seen in Figure 3.4(b)]. This type of curing condition created a situation where the cylinder is exposed to some of the heat generated by the member during the initial hydration and somewhat more representative for the actual strength within the member.

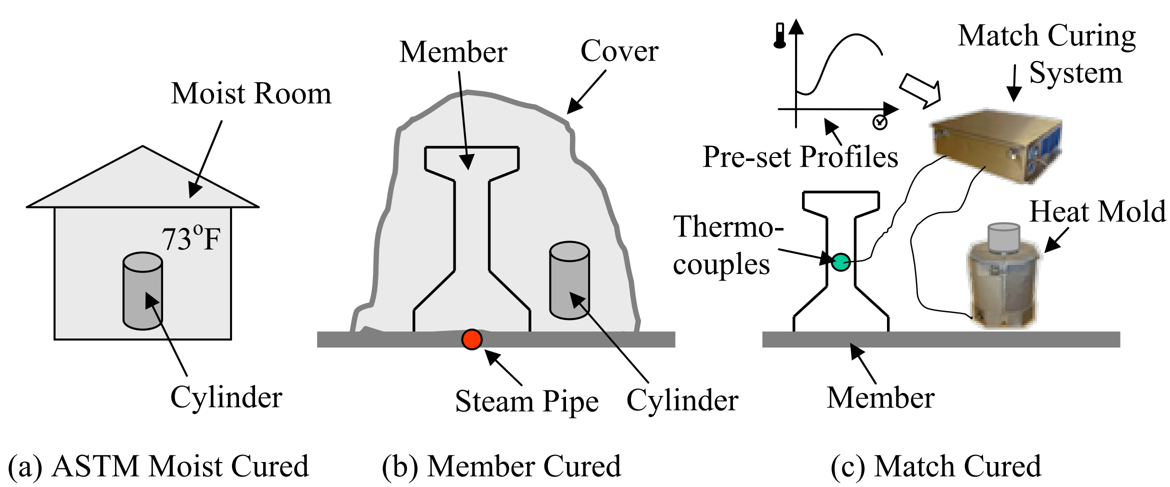


Figure 3.4. Three Curing Conditions

Match cured cylinder By using the SURE CURE match curing system, a commercially available system by Products Engineering, Inc., match-cured cylinders were cured under the exact temperature development profile or pre-set specified temperature profile to represent the actual curing condition in the field. This is done through the use of special cylinder molds that are equipped to heat the concrete in the mold using an internal temperature probe to specify and monitor temperature. Match-cured condition incorporates factors such as cementitious contents of the concrete and SA/V ratios of the member, in addition to the factor of ambient temperature. Match cured cylinders were used to provide actual mechanical properties for comparison with the results of conventional quality control cylinders. In this study, a 24 hours time period was selected since this is the typical time of the higher temperature development in the hydration profile and the maximum time frame at which accelerated curing may occur. The cylinders were match cured for 24 hours after casting the concrete and were stored in the same moist room (relative humidity not less than 95%) as ASTM moist cured cylinder and member cured cylinders after demold until the moment of test [as seen in Figure 3.4.(c)].

A 24 hours time period was selected since this is the typical time of the higher temperature development in the hydration profile and the maximum time frame that accelerated curing may occur.

3.2.6. Apparatus. The apparatus used in this research study included concrete mixer, match curing system, oven, environmental chamber and hydraulic operated compression test machine.

<u>Concrete mixer</u> All laboratory mixings were batched in a 9 ft^3 (6.88 m^3) capacity rotary drum, shown in Figure 3.5, using 3.0 ~ 5.0 ft^3 per batch. All mixings procedures were followed by ASTM standard practice for making and curing concrete test specimens in the laboratory [ASTM C 192-00 (2000)] and ACI state-of-the-art report on High-Strength Concrete [ACI 363R-92 (1992)]. The amount of mixing water was adjusted by the moisture condition of the coarse and fine aggregate before each batch.

Figure 3.5. Whiteman Rotary Mixer Used for Laboratory Produced Concrete

Match Curing System The SURE CURE Match Curing System was utilized in this research. The system is composed of three major parts: I/O cabinet, Personal Computer and Heat Molds (shown in Figure 3.6).

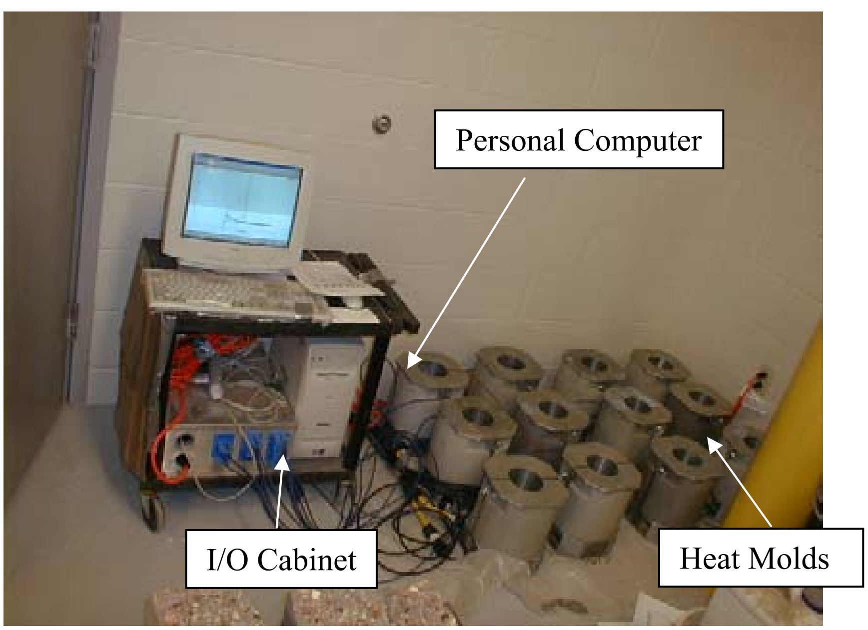


Figure 3.6. SURE CURE Match Curing System

The I/O Cabinet is a device that monitors and digitalizes the temperature signals as well as activates its output relays under the control of the personal computer. It could accept fourteen thermocouple inputs and eight of which have output control capability. The I/O Cabinet takes up to 20 seconds for the controller to sample the temperature of a channel and determine where to activate or deactivate its heat output control. Every 6 minutes the controller writes each channel's temperature onto the hard disk in the computer. The accuracy of measuring the temperature is ± 0.1 ^{o}F (0.056 ^{o}C).

The personal computer is an IBM-compatible PC that is used to control the I/O Cabinet by running its specific software. It acts as the interface between the user and the control unit, as well as storing measured data from the channels or pre-set temperature profiles. Therefore the computer controls the cylinder temperatures to follow a reference temperature profile being monitored simultaneously or a temperature profile programmed by the user.

Heat mold is a special 4 x 8-in. (100 x 200 mm) steel mold that embedded internal electronic heat unit and thermocouples. It can heat the fresh concrete within the molds when the I/O Cabinet activates the power to its channel. The thermocouples within the mold can transfer the temperature of the fresh concrete to the computer for its control to match the specific temperature profile, whether from in-situ member or pre-programmed profile.

In the laboratory, SURE CURE system was used not only to monitor the temperature development profile within the SA/V cubes and conventional quality control cylinders, but also to match curing the cylinder followed the modeled temperature profiles. In the field, the system was mainly used to monitor the member temperatures at various locations in the member and match cured the cylinder followed the actual temperature profiles within the member at the same time.

Temperature-controllable unit To obtain the temperature development profile within the member under different curing temperature, a temperature-control unit was required. Two temperature-control units had been used throughout the research. The first one was the environmental chamber at Engineering Research Laboratory (ERL), the University of Missouri-Rolla (UMR) (shown in Figure 3.7). This walk-in chamber can accommodate large concrete specimens and the curing temperature profile can be programmed prior to curing. Another feature of this chamber is that it has both a heating and a cooling unit. Therefore, the chamber can have the quick response to the desired temperature.

Figure 3.7. Environmental Chamber at ERL, UMR

The second temperature-controllable unit was an oven in the material lab of Butler-Carlton Civil Engineering Building at the University of Missouri – Rolla (shown in Figure 3.8). This oven can only heat relatively small concrete specimens and manually control the curing temperature in real time.

Selecting the oven or environmental chamber to cure the concrete specimens was based on the size and weight of the specimens. If all the specimens in a batch could be cured in the oven, it would be utilized. On the other hand, when the specimens were too large or heavy for the oven to accommodate, the environmental chamber would be selected..

Figure 3.8. Oven at Butler-Carlton Materials Lab, UMR

Hydraulic operated compression test machine A hydraulic operated compression test machine was the major test equipment to access the compressive strength and modulus of elasticity (MOE) of the test cylinders. Figure 3.9 shows two compression test machines used in this study. All testing of laboratory-produced cylinders and later-age field-produced cylinders were tested at the University of Missouri – Rolla. Early-age testing of field-produced cylinders were tested at the Egyptian Concrete Company in Bonne Terre, Missouri.

in Bonne Terre, MO

Figure 3.9. Hydraulic Operated Compression Test Machine

3.2.7. Standard Test Procedures. The mechanical properties of the concrete investigated in this research were the compressive strength and modulus of elasticity of the concrete. Corresponding ASTM specifications were strictly followed.

Compressive strength ASTM C39-01 The compressive strength tests were performed in accordance with ASTM C39-01 "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens". All the cylinders were 4-in. (100 mm) in diameter and 8-in. (200 mm) in height. Neoprene pads were inserted in the retaining steel caps with appropriate durometer hardness in lieu of sulfur mortar caps (shown in Figure 3.10). While testing, the loading rate was controlled at 35 ± 15 psi per second (241 ± 103 kPa per second), which means in this study 440 ± 188 lb per second (200 ± 85 kg / second) of loading, until failure of the specimens.

Figure 3.10. Compressive Strength Test

Modulus of Elasticity ASTM C469-94 Modulus of Elasticity tests were performed in accordance with ASTM C469-94 "Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression". The specimens being tested were the same cylinders used for compressive tests that were conducted after the Modulus of Elasticity test as shown in Figure 3.11. The loading rate was controlled at 35 ± 5 psi per second (241 ± 34 kPa per second), or 440 ± 63 lb per second (200 ± 28.6 kg per second) until failure.

Figure 3.11. Modulus of Elasticity Test

Before the modulus of elasticity test was conducted, the compressometer had been calibrated in accordance with the ASTM specification [ASTM C469-94 (1994)]. The equation of calculating the Modulus of Elasticity is as follows:

$$E = \frac{(g_2 - g_1)(2 \times 5.375)}{(\varepsilon_2 - 0.000050)} \tag{4}$$

Where:

E = Chord modulus of elasticity, psi

g_2 = Gage reading corresponding to 40% of ultimate load

g_1 = Gage reading corresponding to a longitudinal strain ε_1 = 50 millionths psi

ε_2 = Longitudinal strain produced by gage reading g_2

The Compressometer was used both in the laboratory and in the field. Therefore Equ. (4) could be applied to all the calculations of the Modulus of Elasticity in this research.

3.3. PHASE I: MEASURING TEMPERATURE DEVELOPMENT

Phase I of this research was the data collection step in the laboratory study. It involved batch designs, concrete batching & placing and temperature monitoring & recording. The temperature data measured in this phase was the essential information for the analysis and modeling in Phase II.

3.3.1. Objectives. The objectives of this phase were:

Designate the mix designs, curing temperatures and SA/V cubes re suitable for research.

Batch and pour the SA/V cubes and measure the temperature development profiles for the first 24 hours after placement of each cube or cylinder.

3.3.2. Test Matrix. The materials (as shown in Table 3.1), mix designs (as shown in Table 3.2.), SA/V cube selections (as shown in Table 3.4) and accelerated curing temperatures (as shown in Table 3.6 and Figure 3.3) had been designated before work started in the laboratory. Nine different groups combined with different factors were conducted in this phase. Table 3.7 shows the SA/V cubes and cylinders that were used to measure temperature profiles in the nine groups. The decision of whether specific SA/V cubes were used for measured temperature profiles was made based on the requirements for modeling, lab-work availability, minimizing the number of mixes per batch and accommodation of the environmental chamber / oven.

Table 3.7. Summary of SA/V Cubes and Cylinders Measured in Phase I

No	w/cm Ratio	Fly Ash	Curing Tempe-rature	Cylinder	SA/V Ratio of Cubes							
				4"x8"	1	0.67	0.5	0.45	0.4	0.35	0.3	0.25
1	0.25	NO	Room	√	√	√	√	√	√	√	√	
2			130^{o}F	√	√	√				√	√	
3			160^{o}F	√	√	√	√	√	√	√	√	
4		YES	Room	√	√	√	√	√	√	√		
5			130^{o}F	√	√	√		√				
6			160^{o}F	√	√	√	√	√				
7	0.3	NO	Room	√	√	√		√	√			
8			130^{o}F	√	√	√	√					
9			160^{o}F	√	√	√	√					

Conversion: ^{o}C = 5/9 (^{o}F – 32) ; 1 inch = 25.4 mm

3.3.3. Test Setup. Due to the different accelerated curing temperatures, the test setups had three curing cases: room (Figure 3.13), oven (Figure 3.14) and environmental chamber (Figure 3.15), based on the same flow chart as shown in Figure 3.12.

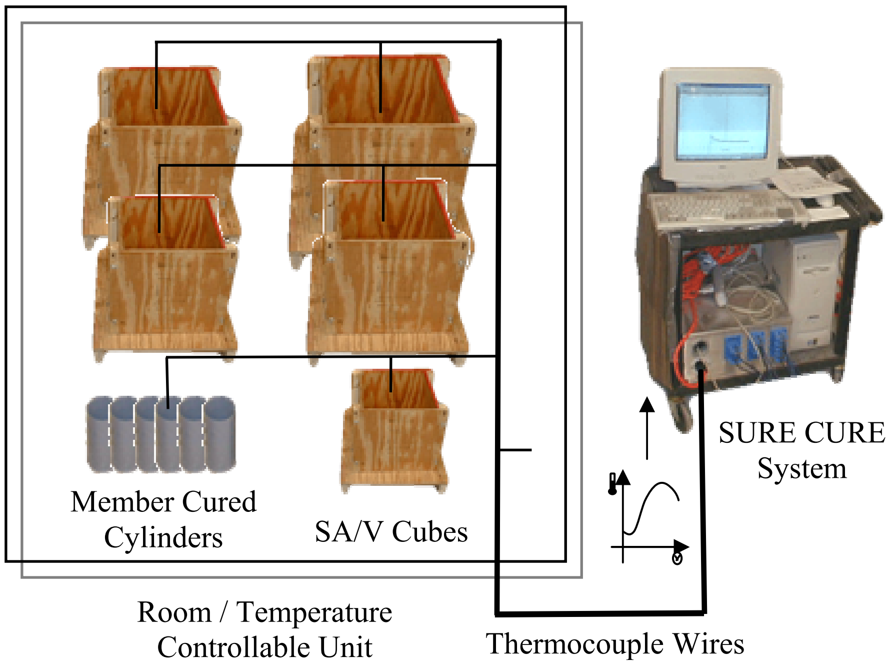


Figure 3.12. Flow Chart of Test Setup for Temperature Profile Measurement

The temperature was measured by coupled thermocouples connected to the I/O Cabinet. The ambient temperature around the specimens was also monitored during curing.

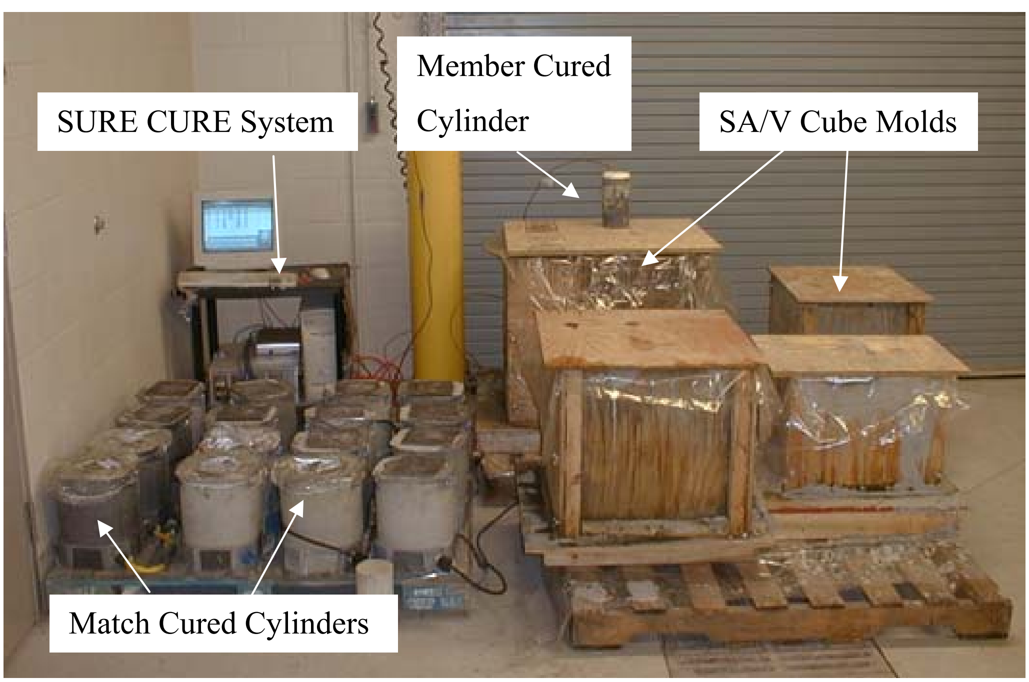


Figure 3.13. Test Setup in the Room Condition

In practice, the standard match cured cylinders in Phase III were conducted along with the later part of the groups with the same concrete mix design. For example, after having been measured the temperature profiles of Group No. 1 in Table 3.7 (w/cm=0.25 + without fly ash replacement + room condition), the standard match curing of group No. 1 was performed at the same time of measuring temperature profiles of group w/cm=0.25/ without Fly Ash/ Curing Temperature = 130^{o}F (54.4^{o}C). The purpose was to minimize the number of batches with the same concrete mix design in order to decrease the variations from batches to batches, as well as to save the project cost and time. Therefore, Phase I, II, and III were virtually executed interactively. Figure 3.13 and Figure 3.15 show the cases discussed above.

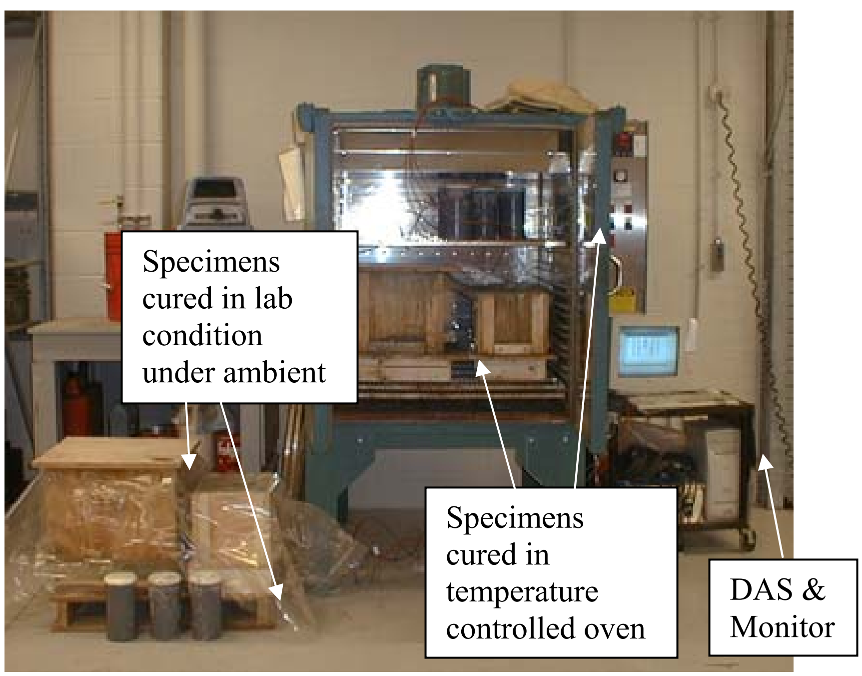


Figure 3.14. Test Setup in the Room and Oven Condition (Phase I & III)

Climate Control Room

Figure 3.15. Test Setup in the Environmental Chamber Condition (Phase I)

3.4. PHASE II: MODELING TEMPERATURE DEVELOPMENT PROFILES

After collecting the necessary temperature profiles of each group in Table 3.7, the process of data analysis and standard temperature profile modeling was performed. All of the work conducted in Phase II was executed in the lab and the standard match curing in Phase III would be based on the results in this phase.

3.4.1. Objectives. The objectives of this phase were:

Model the relationship between the peak hydration temperature and SA/V ratio for the conditions shown in Table 3.7.

Model the relationship between the maturity value and SA/V ratio for the conditions shown in Table 3.7.

Calculate the peak hydration temperatures and maturities of concrete cubes with SA/V ratio equaled to 0.1, 0.2, 0.4, 0.6 1/in., based on the equations established from Objective 1) and 2).

Construct the temperature development models for SA/V ratio equal to 0.1, 0.2, 0.4, 0.6 and 1.0 1/in. [for 4 x 8-in. (100 x 200 mm) cylinder)].

Objective 4) is the desired result required for Phase III.

3.4.2. Methodology. Recall that hydration temperature and first 24 hours' maturity of the concrete greatly affects the mechanical properties of HSC. Since it was difficult to produce the low SA/V ratio cubes such as SA/V=0.1 1/inch in the lab, the 2nd order equations of relationships between SA/V ratio and peak hydration temperature as well as maturity were established based on the limited measured data from the cubes with the SA/V ratios from 0.25 to 1.0 1/inch to predict the peak hydration temperature and maturity of extremely low SA/V ratio cube. The peak hydration temperature and maturity of desired SA/V cubes could be derived from such equations. Another benefit for establishing the equations other than using the original measured data was to standardize the information and reduce the variations of different batches. It must be pointed out that the equations established in this research were not intended to be utilized in all kinds of HSC, even those employing with the same mix design. This is because there are other factors such as type and size of aggregates, cement, or even batch procedures can affect the properties of the concrete. The equations here were only used to standardize the information and to predict the peak hydration temperature and maturity of the cubes investigated.

Once the peak hydration temperature and its maturity in the first 24 hours were obtained, the temperature development profile was constructed. Based on the Maturity Method, the new temperature profile had the same peak hydration temperature and maturity as the design values. Thus the mechanical properties of the concrete cured in such new temperature development profile were identical to the concrete cube with the same SA/V ratio. As shown in Figure 3.16, a typical temperature development profile included pre-set period, peak hydration temperature and drop point. During the modeling, the pre-set period was always selected to be 4 hours and the peak hydration temperature was equal to the value calculated from the peak hydration temperature relationship. The first point of the peak temperature and the drop point were designated to satisfy the requirement for the same value of the area below the curve in Figure 3.16 and maturity obtained from the equation.

The detailed information of Phase II will be further discussed in Section 4.

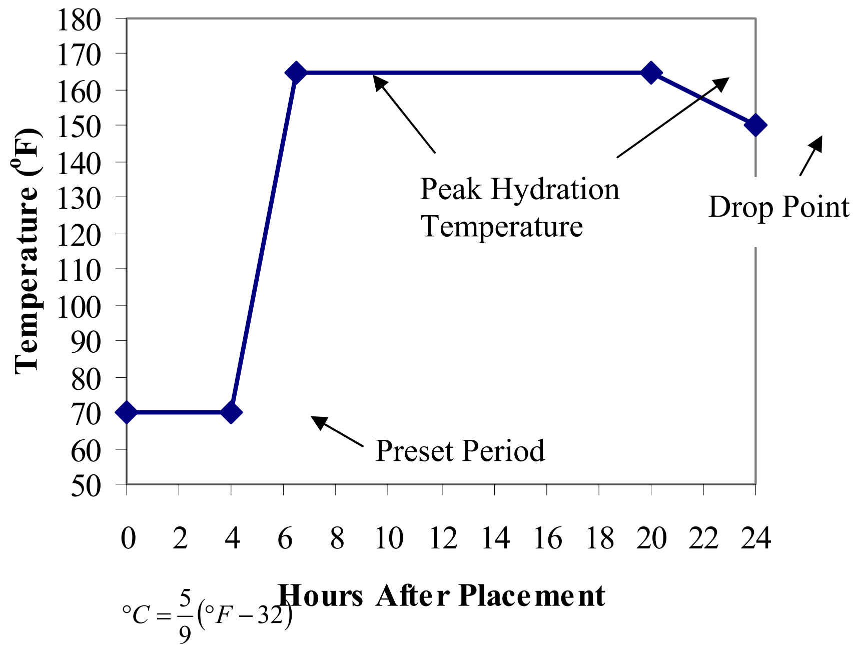


Figure 3.16. Sample of Temperature Development Profile Model

3.5. PHASE III: MATCH CURING CYLINDERS

Phase III was the critical phase throughout the research. In this phase, nine groups of SA/V ratios were used to match-cure the standard cylinders under the temperature development models developed in Phase II.

3.5.1. Objectives. The objectives of this phase were:

Match cure the standard cylinders under the temperature development models developed in Phase II.

Conduct compressive strength test and modulus of elasticity test of the match-cured cylinders in release (24 hours) and 56-day age.

Data was analyzed after the test procedures had been accomplished.

3.5.2. Test Matrix. There were nine groups of concrete investigated in this phase (as shown in Table 3.8). By using the SURE CURE match curing system, cylinders could be match cured under different temperature development profiles. Thus it could simulate the different curing conditions of the concrete.

Table 3.8. Summary of the Standard Match Curing Cylinders in Phase III

Group	Item	Description			Represents of SA/V Ratio of Cubes				
		w/cm Ratio	35% Fly Ash	Curing Tempe-rature	0.1	0.2	0.4	0.6	1*
1	L25-70	0.25	NO	70°F	√	√	√	√	√
2	L25-130			130°F	√	√	√	√	√
3	L25-160			160°F	√	√	√	√	√
4	L25FA-70	0.25	YES	70°F	√	√	√	√	√
5	L25FA-130			130°F	√	√	√	√	√
6	L25FA-160			160°F	√	√	√	√	√
7	L30-70	0.3	NO	70°F	√	√	√	√	√
8	L30-130			130°F	√	√	√	√	√
9	L30-160			160°F	√	√	√	√	√

Notes :
"*" represents of the member cured cylinder of 4 x 8-in. (100 x 200 mm) in corresponding group and served as the control cylinder.

Conversion: °C = 5/9 (°F – 32)

3.5.3. Test Setup. The flow chart of the test setup in this phase is shown in Figure 3.12. The SURE CURE system was used to control the temperature in the heated molds in order to match the pre-programmed temperature development profiles. Eight (8) channels were used to match cure the cylinders. Every four (4) molds were match cured under the same temperature profile to represent one specific SA/V ratio. Among the four cylinders, two of them were used for early-age testing while the other two were used for later-age