ABSTRACT

Title of Thesis: REHABILITATION AND MAINTENANCE OF ROAD PAVEMENTS USING HIGH EARLY STRENGTH CONCRETE.

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The vast amount of civil infrastructure in the United States includes an extensive stretch of road networks. From an economic point of view it is more cost effective to maintain the already existing pavements rather than building new ones.

A large proportion of the traffic delays on these road networks are caused by road closures and closures of individual lanes for pavement maintenance purposes. The application of early strength concrete in pavement maintenance measures will lead to a substantial reduction in the user costs involved with the road closures caused by such maintenance. These costs involve both the actual costs of the delays in terms of time and fuel consumption, but also, more importantly, the social and economic costs associated with the safety hazards resulting from these closures.

This research is aimed at selecting two four-hour mix designs out of a total of five mix designs selected in a report made by Construction Technology Laboratories Inc. (CTL), and the Maryland State Highway Administration (MSHA), based on concrete compressive strength and freezing-thawing durability. The targeted concrete strength aimed at is a minimum of 2,000 psi (14 MPa) at four hours.

The report contains a literature review, background information and detailed description of test procedures, results analysis and selection criteria. Recommendations are made for concrete mix selection for road patching and rehabilitation.

REHABILITATION AND MAINTENANCE OF ROAD PAVEMENT

USING HIGH EARLY STRENGTH CONCRETE MIXES.

By

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DEDICATION

To the almighty for his mercy and to my parents for the sacrifice they made in bringing me this far.

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I am most grateful to God, for giving me the strength and knowledge to complete this thesis. I thank my parents for all the encouragement and support they gave me throughout the 23 years of my education. I thank my siblings for the emotional support and encouragement given me through the years of graduate school. I also thank Miss Lady Awuku-Lokko for all the help and support she gave me during the hectic times of batching and casting the concrete specimens.

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CHAPTER 1 – INTRODUCTION

1.0.0 General Overview

All civil infrastructures have a definite life span. In other words, all structures are designed to fail at some point, and this includes the vast network of road pavements in the United States. Approximately 2% of lands in the U.S are paved [Pocket guide to transportation, 2003]; this consists of flexible, rigid and composite pavements. In order to ensure that pavements achieve the purpose for which they were designed they ought to be maintained regularly and at very little cost to the road user.

The United States spends about $200M/year on highway construction; delays caused by traffic cost road users approximately $78B/year [TRB SR 260,1999]. Road maintenance and rehabilitation form the largest percentage of this figure. It is therefore necessary to curtail the high cost of maintenance to road users by developing measures to decrease traffic delays during maintenance and rehabilitation.

There is a wide perception that concrete pavements "cost too much," "take too long," or "are too difficult to repair." However, to the contrary, although the initial cost of concrete may be higher than for asphalt pavement, however concrete costs less during the pavement's life cycle. Roads can be opened faster than ever and can be repaired easily with the proper equipment, materials, processes and or procedures. Also concrete pavement restoration can return a pavement to a near-new condition at a lesser cost to the road user if measurers to decrease delay time are put in place.

1.1.0 Background Information

Deteriorating asphalt and concrete pavement infrastructure worldwide demands innovative and economical rehabilitation solutions. When desired, a properly designed and constructed bonded overlay can add considerable life to an existing pavement, by taking advantage of the remaining structural capacity of the original pavement. For patchwork and total rehabilitation, two types of thin concrete pavement overlays rely on a bond between the overlay and the existing pavement for performance. Concrete overlays bonded to existing concrete pavements are called Bonded Concrete Overlays (BCO). Concrete overlays bonded to existing asphalt pavements are called Ultra-thin White-topping (UTW). Research has shown that concrete overlays over asphalt often bond to the asphalt, and that some reduction of concrete flexural stresses may be expected from this effect. These overlays have been used to address rutting of asphalt pavements.

Bond strength and resistance to cracking are important for overlay performance. In many cases these overlays are constructed on heavily traveled pavements, making early opening to traffic important. Therefore, early strength development without compromising durability is necessary. Satisfactory performance will only occur if the overlay is of sufficient thickness and is well bonded to the original pavement. The design assumption is that if the overlay bonds perfectly with the original pavement, it produces a monolithic structure. Without bond, there is very little structural benefit from an overlay, and the overlay may break apart rapidly under heavy traffic.

The use of concrete overlays for pavement and bridge deck maintenance and rehabilitation has been in existence for several decades, both un-bonded and bonded overlays have been used in rehabilitation and maintenance of deteriorating road pavements. For both BCO and UTW overlays, characteristics of the overlay concrete have important implications for early age behavior and long-term performance.

1.2.0 High Performance Concrete (HPC)

High performance concrete is defined as "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"[Forster et al. 1994].

The design of high performance concrete mixes started in the 1980's in the private sector to protect parking structures and reinforced concrete high-rise buildings from chlorides, sulfates, alkali-silica reactivity and to curtail concrete shrinkage and creep.

HPC for pavements originated in the Strategic Highway Research Program under contract C205 [Zia et al.1991], where the mechanical properties of HPC were described and studied under actual use conditions. SHRP developed a definition of HPC (Table 1.1) and funding for limited field trials, which were to be followed by a substantial implementation period.

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%
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 (35 MPa) in 24 hours	0.35	80%
Very high strength (With Type I Cement)	10,000 psi (70 MPa) in 28 hours	0.35	80%

Table 1.1: Definition of HPC according to SHRP C-205 (Zia, et al. 1993)

In 1993, the Federal Highway Authority (FHWA) initiated a national program to encourage the use of HPC in bridges. The program included the construction of demonstration bridges in each of the FHWA regions and dissemination of the technology and results at showcase workshops. A widely publicized, mile-long concrete test section on the Chrysler Expressway in Detroit (1993) was the first High Performance Concrete pavement application. Techniques such as Belgian surface texturing, a modified German cross-section, and an Austrian exposed aggregate surface treatment were used. HPC pavements got a great boost in 1999, with the launching of a $30-million research initiative by the FHWA; this amount was increased to higher amounts with private sector participation. The Transportation Equity Act for the 21st Century included $5 million per year for applied research in rigid Portland Cement Concrete (PCC) paving. This resulted in $30 million over six years to utilize and improve concrete pavement design and construction practices. With its HPC initiative, the FHWA articulated its goal of providing the public with safe, smooth, quiet, long lasting, environmentally sound, and cost-effective concrete pavements. Performance goals for HPC pavements included an increase in pavement system service life, a decrease in construction time (including fast-track concrete paving techniques), longer life cycles such as a 30 - 50-year life, and lower maintenance costs.

The FHWA defined high performance concrete according to its properties by awarding grades to each property. This is illustrated in Table 1.2.

Performance Criteria	Standard method	FHWA HPC Performance grade		
		1	2	3
Freeze -thaw durability X=Relative Dynamic Modulus after 300 cycles	AASHTO T161 ASTM C666	60% < X <80%	80% < X	-
Scaling resistance X=Visual rating of the Surface after 50 cycles	ASTM C672	X = 4,5	X = 2,3	X = 0,1
Abrasion resistance X=avg. depth of wear in mm	ASTM C944	2.0 > X >1.0	1.0 > X >0.5	0.5 > X
Strength X=compressive strength	AASHTO T2 ASTM C39	41 < X < 55 MPa (6 < X <8 Ksi)	55 < X < 69 MPa (8 < X < 10 Ksi)	69 < X < 97 MPa (10 < X < 14 Ksi)
Elasticity X=modulus	ASTM C469	28 < X < 40 GPa (4 < X <6x10^6 psi)	40 < X < 50 GPa (6 < X < 7x10^6 < X psi)	50 GPa<=X (7.5x10^6 < X psi<= X)

Table 1.2: Definition of HPC according to Federal Highway Administration (Goodspeed, et al. 1996)

Lower maintenance costs and a decrease in construction time are a concern for this research and are the prime basis for design and research into fast track or early strength concrete mixes.

1.2.1 Early Strength / Fast Track Concrete mixes

Early strength concrete mixes are concrete mixes that, through the use of high-early-strength cement or admixtures, are capable of attaining specified strengths at an earlier age than normal concrete. This property is very useful in road pavement maintenance and rehabilitation by reducing delay costs to the road user.

Concrete or composite pavement repair is prime for maintaining existing roads. Before the advent of early strength concrete, there was no comparism of the costs of flexible pavements to rigid pavements in both initial and operating costs. This was because the initial material costs of rigid pavements and the cost of delays due to the longer closing time during maintenance and rehabilitation were far more than when asphalt was used. Since its inception, a lot of research and development has been done on early strength concrete. Early Strength can be broken down into two categories, Very Early Strength (VES) and High Early Strength (HES) concrete. VES is an Early Strength Concrete mix with two options, A and B, as follows. For VES (A) a minimum compressive strength of 2,000 psi (14 MPa) is required 6 hours after water is added to the concrete mixture using Portland cement with a maximum W/C of 0.40. For VES (B) concrete, a minimum compressive strength of 2,500 psi (17.5 MPa) is required 4 hours after water is added to the concrete mixture using Pyrament PBC-XT cement, with a maximum W/C of 0.29.

High early strength concrete is specified to have minimum compressive strength of 2,000 psi (14 MPa) but for a longer duration of 12 hours. In the context of our research, however, the word "Early" is considered to be relative; the concrete mixes to be researched will be termed "Early strength," without taking into consideration the time and place of strength gain.

These criteria were adopted after considering several factors pertinent to the construction and design of highway pavements and structures. The use of a time constraint of 4 to 6 hours for “Very Early Strength, (VES)” concrete is intended for projects with very tight construction schedules involving full-depth pavement replacements in urban or heavily traveled areas. The strength requirement of 2,000 to 2,500 psi (14 to 17.5 MPa) is selected to provide a class of concrete that would meet the need for rapid replacement and construction of pavements. Since “Very Early Strength, (VES)” concrete is intended for pavement applications where exposure to frost must be expected, it is essential that the concrete be frost resistant. Thus, it is appropriate to select a maximum W/C of 0.40, which is relatively low in comparison with conventional concrete. With a low W/C ratio, concrete durability is improved in all exposure conditions. Since VES concrete is expected to be in service in no more than 6 hours, the W/C selected might provide a discontinuous capillary pore system at about that age, as suggested by Powers et al (1959).

1.2.2 High Early Strength Concrete Vs Conventional Concrete Mixtures

Rather than using conventional concrete mixtures, High Early strength concrete mixtures are being used to decrease the delay time due to road closures. Unlike the conventional concrete mixtures, High Early strength concrete achieves its specified strength of 2,500 -3,000 psi (17.5 to 21 MPa) in 24 hours or less at an earlier age, from a few hours to several days. High strength at an early age is desirable in winter construction to reduce the length of time temporary protection is required, for high speed cast in-place construction, rapid form re-use, fast track paving and many other uses. The additional cost of high-early-strength concrete is often offset by earlier reuse of forms and removal of shores, savings in the shorter duration of temporary heating, and earlier use of the structure. In road pavement maintenance and rehabilitation, strength at an early age is beneficial when early opening of the pavement is necessary.

1.2.3 Techniques Used In Attaining Early Strength

High early strength concrete can be achieved by using one or a combination of the following techniques.

1. Use of Type III High Early Strength cement.
2. High conventional cement content.
3. Low water - cement ratio using Type I cement (0.3-0.45 by mass).
4. Higher temperatures for freshly mixed concrete
5. Chemical admixtures.
6. Silica fumes.
7. Higher curing temperatures.
8. Insulation to retain heat of hydration.
9. Special rapid hardening cements.
10. Steam or autoclave curing.

The above listed techniques can be used interchangeably or combined to achieve the desired strength. High early strength gain is not limited to the use of special proprietary cements such as Type III cement. It is now possible to achieve early strength by using locally available Portland cements, aggregates, and selected admixtures. This research uses a combination of Type III High Early Strength cement and chemical admixtures on one hand and a Low water-cement ratio and/or high conventional cement content on the other hand to attain early strength. This research will compare the combination of these techniques and of the individual techniques used.

1.3.0 Literature Review

In the past, ordinary Portland cement-based mixtures were not able to achieve early strength requirement without sacrificing necessary working, placement, and finishing times. Portland cement-based concrete mixtures usually require a minimum of 24 hours and, frequently, five to fourteen days to gain sufficient strength and allow the concrete to return to service. With the advent of various techniques and materials it is now possible to use readily available local materials to achieve early strength.

In 2001, research conducted by the University of Alabama at Birmingham, titled "Design and Quality Control of Concrete Overlays," developed and tested a range of plain and fiber reinforced concrete mixes that allowed reliable economic and durable overlay construction as well as early opening to traffic. The use of a lower water-cement ratio and a high percentage of normal cement was used in attaining early strength. It was concluded in this research that high strength concrete was appropriate for opening overlay to traffic in 24 hours or less, but normal strength may be used if traffic loading can be delayed for 48 or 72 hours.

Under the sponsorship of the New Jersey Department of Transportation a unique concrete mix was developed. This concrete mix attained a significant strength of 3,000 psi – 3,500 psi (21 to 24.5 MPa) in a period of six to nine hours for use on pavement repair in high-traffic areas [FHWA NJ 2001-015]. The use of normal Portland cement and the reliance on chemical admixtures and insulated coverings was used to attain very high temperature levels in order to attain early strength.

Research into the performance and strength of fast track concrete was done under the Strategic Highway Research Program (SHRP). This research included “Very Early Strength” (VES), and “High Early Strength” (HES) mixes developed under the SHRP project C-205 “Mechanical Behavior of High Performance Concrete.” [Zia et al.,1993]. A literature review was conducted by the Construction Technology Laboratories Inc. based on 11 Fast track mixes developed under SHRP Contract C-206 documented in a report titled “Optimization of Highway Concrete Technology,” SHRP Report C-373 (2003). In their review report they recommended 4 mixes for further research into early strength gain. Currently there are a couple of early strength design mixes available for pavement rehabilitation, notably among them are 4 X 4 mix from Master Builders.

The Maryland State Highway Administration (MDSHA) currently requires use of a 12-hour concrete mix for patching in heavily trafficked roadways in urban areas. This mix is required in order to achieve 2,500 psi (17.5MPa) compressive strength in 12 hours. However, the MDSHA now wants to reduce the concrete set time to allow the patch to be opened to traffic about 4 hours after placing the concrete in the patch. The objective of the project is to test proper concrete material mixes both designed in the lab and in the field, for composite pavements that will allow the repaired sections to be opened to traffic after four hours of concrete placement in the patch. A shorter patch repair time would minimize the disruption caused to traffic and ultimately provide longer lasting composite pavements.

The report by the Construction Technology Laboratories (CTL) was submitted to the Maryland State Highway Administration in April 2003. Based on this report, a proposal was to be made to the Maryland State Highway Administration to test the four concrete mix designs selected in the report made by CTL.

From an earlier literature review study of eleven mixes, eight mixes were considered suitable for further study, two used at a Georgia site and six used at a Ohio site. Based on the performances of these mixes during the initial trials and, considering modifications for local materials, the VES mix, the GADOT mix in Georgia, and the VES mix and the ODOT mix in Ohio were selected as the four trial mixes to be evaluated further as part of a laboratory study. Also included as one of the trial mix designs, was a 12- hour concrete mix design currently used in Maryland for fast- track paving, and designated as the control Mix.

1.4.0 Research Scope

The four concrete mixes adopted from the CTL report to the Maryland State Highway Administration (MDSHA) and the 12 hour concrete mix design currently used in Maryland are to be prepared in the Laboratory and tested for compressive strength and resistance to freezing and thawing.

A total of sixty (60) specimens are to be cast and tested for four (4) hours, twenty four (24) hours and seven (7) days' compressive strength. Twelve (12) specimens each are to be cast for each unique mix and are to comprise of four specimens for the four (4) hour compressive strength, four specimens for the twenty four (24) hours compressive strength and another four specimens for the seven (7) day test. Twenty (20) more specimens are to be cast and exposed to a minimum of three hundred (300) cycles of freeze and thaw. The resistances of the specimens to the cycles at a range of intervals are to be observed for scaling, deterioration and failure. The results are to be compared and the performance of each mix assessed accordingly.

1.5.0 Research Objective

The objective of this research is to select two (2) concrete mixes out of the five selected that will yield a compressive strength of at least 2,000 psi (14MPa) after four (4) hours of casting. The selected specimen should be able to withstand at least 300 cycles of freezing and thawing. The 2 selected mixes shall have passed both criteria. Based on the findings and recommendations of this report, another phase of this project is to be started to investigate the characteristics of the recommended mixes to field conditions. This will comprise the second phase of this project.

CHAPTER 2 – CONCRETE AND ITS CONSTITUENTS

2.0.0 Introduction

Concrete is a construction material; it has been used for a variety of structures such as highways, bridges, buildings, dams, and tunnels over the years. Its widespread use compared to other options like steel and timber is due to its versatility, durability and economy.

The external appearance of concrete looks very simple, but it has a very complex internal structure. It is basically a simple homogeneous mixture of two components, aggregates (gravel or crushed stone) and paste (cement, water and entrapped or purposely entrained air). Cement paste normally constitutes about 25%-40% and aggregates 60%-75% of the total volume of concrete. When the paste is mixed with the aggregates, the chemical reaction of the constituents of the paste binds the aggregates into a rocklike mass as it hardens. This mass is referred to as concrete.

The quality of concrete greatly depends upon the quality of the paste and the quality of hardened concrete is determined by the amount of water used in relation to the amount of cement. Thus, the less water used, the better the quality of concrete, so far as it can be consolidated properly. Although smaller amounts of water result in stiffer mixes, these mixes are more economical and can still be used with efficient vibration during placing.

The physical and chemical properties of concrete, however, can be altered by the addition of admixtures in order to attain desirable mixes for specific purposes.

2.1.0 Basic Components of Concrete

Concrete is made up of various components, primarily; concrete is made up of paste, coarse aggregates and admixtures. The basic components of concrete are the following;

2.1.1 Portland cement

There are various types of cement, however the most widely used cement in the U.S and in most parts of the world is Portland cement; it was developed by Joseph Aspdin, a British stone mason, in 1824. This product is a mixture of finely ground limestone and clay ground into powder, to create hydraulic cement that hardens with the addition of water. Portland cement is a type of cement comprised of a chemical combination of calcium, aluminum, silicon, iron and small amounts of other compounds, to which gypsum is added in order to regulate the setting time of concrete. Four major compounds make up about 90% or more of the weight of Portland cement; these are tricalcium silicate (C_3S), dicalcium silicate (C_2S), tricalcium aluminate (C_3A), and tetracalcium aluminoferrite (C_4AF). Other compounds in addition to these play important roles in the hydration process.

The raw materials used in the manufacture of Portland cement includes limestone, shells, and chalk or marl, combined with shale, clay, slate or blast furnace slag, silica sand, and iron ore. Manufacture requires some 80 separate and continuous operations, the use of a great deal of heavy machinery and equipment, and large amounts of heat and energy. Each step is checked by frequent chemical and physical tests in the laboratory.

The American Society for Testing and Materials (ASTM) Designation C 150 provides for eight types of Portland cement depending on the different physical and chemical requirements and the purpose the cement is needed for. These types are described in the next section.

2.1.2 Types of Portland cement

Type I

Type I is a general purpose Portland cement suitable for all uses where the special properties of other types are not required. It is used where cement or concrete is not subject to specific exposures, such as sulfate attack from soil or water, or to an objectionable temperature rise due to heat generated by hydration. Its uses include pavements and sidewalks, reinforced concrete buildings, bridges, railway structures, tanks, reservoirs, culverts, sewers, water pipes and masonry units.

Type II

Type II Portland cement is used where precaution against moderate sulfate attack is important, as in drainage structures where sulfate concentrations in groundwater are higher than normal but not unusually severe. Type II cement will usually generate less heat at a slower rate than Type I. With this moderate heat of hydration, Type II cement can be used in structures of considerable mass, such as large piers, heavy abutments, and heavy retaining walls. Its use will reduce temperature rise -- especially important when the concrete is laid in warm weather.

Type III

Type III is a high-early strength Portland cement that provides high strengths at an early period, usually a week or less. It is used when forms are to be removed as soon as possible, or when the structure must be put into service quickly. In cold weather, its use permits a reduction in the controlled curing period. Although richer mixtures of Type I cement can be used to gain high early strength, Type III may provide it more satisfactorily and more economically.

Type IA, IIA, IIIA

Specifications for three types of air-entraining Portland cement (Types IA, IIA, and IIIA) are given in ASTM C 150. They correspond in composition to ASTM Types I, II, and III, respectively, except that small quantities of air-entraining materials are inter-ground with the clinker during manufacture to produce minute, well-distributed, and completely separated air bubbles. These cements produce concrete with improved resistance to freeze-thaw action.

Type IV

Type IV is a low heat of hydration cement for use where the rate and amount of heat generated must be minimized due to the volume of concrete being poured and for ease of placement during construction. It develops strength at a slower rate than Type I cement. Type IV Portland cement is intended for use in massive concrete structures, such as large gravity dams, where the temperature rise resulting from heat generated during curing and hardening is a critical factor.

Type V

Type V is used in place of Type II when the concrete is exposed to severe sulfate action. It is principally used where soils or groundwater have a high sulfate content. Low Tricalcium Aluminate (C_3A) content, generally 5% or less, is required when high sulfate resistance is needed.

2.2.0 Aggregates

Aggregates play a major role in the properties of concrete, using the right kind of aggregate greatly influence concrete's freshly mixed and hardened properties, mixture proportions, and economy.

Aggregates can be distinguished into two distinct types based on their particle sizes. Fine aggregate consists of natural sand or crushed stone with most particles smaller than 1/5 inch (5mm). Coarse aggregates consist of one or a combination of gravels and crushed aggregate with particles predominantly larger than 1/5 inch (5mm) and generally between 3/8 and 1-1/2 inches (9.5 and 37.5mm). Natural aggregates are obtained by either dredging or digging from a pit, river, lake or sea-bed. Crushed aggregates are produced by the crushing of quarry rock, boulders, cobbles, or large size gravels.

Aggregates must be set to some standards in order to be most useful in engineering structures. They must be clean, hard, strong, durable particles free of absorbed chemicals, coating of clay and other fine materials in amounts that could affect hydration and the bond of the cement paste. Aggregates with low resistance to weathering should be avoided in concrete mixes.

2.3.0 Aggregate Characteristics

Resistance to Abrasion and Degradation

An aggregate's resistance to abrasion and degradation is a good indication of its quality. In road paving construction, this characteristic is essential since the aggregate is subjected to abrasion. The most common test for abrasion resistance is the Los Angeles abrasion test performed in accordance with ASTM C131 or C535.

Aggregate Grades and Grading Limits

The particle size and distribution of an aggregate is termed grading. It is determine by a sieve analysis in accordance to ASTM C136. The seven standard ASTM C33 sieves for fine aggregates have openings ranging from 150µm to 3/8in (9.5mm). There are thirteen standard sieves for coarse aggregates that range from 0.046 inches to 4 inches (101.6mm). Grading and grading limits are usually expressed as percentages of materials passing through each sieve.

It is important to specify grading limits and maximum aggregate size because it affects the relative aggregate proportions as well as cement and water requirements, workability, pump-ability, economy, porosity, shrinkage and durability of concrete. It is thus important to acquire aggregates comprised of a collection of sizes so as to reduce the total volume of voids between aggregates during mixing.

Shape and Texture

The shape of aggregates influences the properties of concrete mixes. Angular, elongated particles and rough-textured aggregate produce more workable concrete than smooth, rounded, compact aggregates. Flat and elongated particles should be avoided or at least limited to 15% by weight of the total aggregate.

Resistance to Freezing and Thawing

This characteristic is related to the aggregate porosity, absorption, permeability, and pore structure. For instance, when an aggregate particle is fully saturated and cannot accommodate the expansion and hydraulic pressure that occurs during freezing of water in the aggregate, it expands and finally disintegrates.

Strength and Shrinkage

An aggregate’s tensile strength ranges from 30 psi to 2,300 psi (0.21 to 16.1MPa) and its compressive strength from 10,000 psi to 40,000 psi (70 to 280 MPa). This is important in high strength concrete.

Aggregates with high absorption properties may have high shrinkage on drying. Other characteristics include unit weight and voids, specific gravity, absorption, surface moisture, strength and shrinkage.

Handling and Storage of Aggregates

To minimize segregation, degradation and contamination by deleterious substances, aggregates should be handled and stored in an appropriate fashion by stockpiling them in thin layers of uniform thickness. The most appropriate and economical method of stockpiling is the truck dump method; however, when aggregates are not delivered by truck, an acceptable and less expensive way is to form the stockpile in layers using a clamshell bucket.

Washed aggregates should be stockpiled in sufficient time so that they can drain to have uniform moisture content before use.

2.4.0 Admixture

Admixtures are additives other than water, aggregates, hydraulic cement, and fibers that are added to the concrete batch immediately before or during mixing to improve specific characteristics of the concrete. There are two types of admixtures, chemical and mineral admixtures. These when properly used, offer certain beneficial effects to concrete, including improved quality of concrete during the stages of mixing, transporting, placing and curing in adverse weather, reduction in the cost of concrete construction, avoidance of certain emergencies during concrete mix operations, and achievement of certain properties.

A survey by the National Ready Mix Concrete Association reported that 39% of all ready-mixed concrete producers use fly ash, and at least 70% of produced concrete contains a water-reducer admixture. The chemical composition of admixtures vary and, since many perform more than one function, it is necessary that all admixtures to be used in any concrete mix should meet specifications and tests should be made to evaluate the effect of the admixtures on the properties of the concrete mix.

The beneficial effects admixtures have on concrete are due to the following properties they posses;

- Protection against Freeze- Thaw Cycles –Improve Durability
- Water Reduction in the Mix
- Increase in Concrete Strength
- Corrosion Protection
- Set Acceleration
- Strength Enhancement

- Set Retardation
- Crack Control (shrinkage reduction)
- Flowability
- Finish Enhancement

2.4.1 Mineral Admixtures

Mineral admixtures are usually added to concrete in large amounts to enhance its workability; improve its resistance to thermal cracking, alkali-aggregate expansion and sulfate attack; reduce permeability; increase strength; and enable a reduction in the cement content, thus improving the concrete mix properties.

Mineral admixtures affect the nature of the hardened concrete through hydraulic or pozzolanic activity. Pozzolans are cementitious materials and include natural pozzolans (such as the volcanic ash used in Roman concrete), fly ash and silica fume.

Fly Ash

Fly ashes are finely divided residues resulting from combustion of ground or powdered coal. They are generally finer than cement and consist mainly of glassy-spherical particles as well as residues of hematite and magnetite, char, and some crystalline phases formed during cooling.

Fly ash improves the workability in concrete, reduces segregation, bleeding, heat evolution and permeability, inhibits alkali-aggregate reaction, and enhances sulfate resistance. Because Portland cement concrete pavement is largely dependent on high volumes of cement, the use of fly ash as an admixture is important where economy is important factor.

Silica Fume

Silica fume, also known as micro-silica, is a byproduct of the reduction of high-purity quartz with coal in electric furnaces in the production of silicon and ferrosilicon alloys. Silica Fume is also collected as a byproduct in the production of other silicon alloys such as ferrochromium, ferromanganese, ferromagnesium, and calcium silicon. Silica Fume consists of very fine vitreous particles, which makes it a highly effective pozzolanic material. It has been found that Silica Fume improves compressive strength, bond strength, and abrasion resistance; reduces permeability; and, therefore, helps in protecting reinforcing steel from corrosion.

Granulated Blast Furnace Slag

Intergrinding the granulated slag with Portland cement clinker makes Portland blast furnace slag cement. Its use as a mineral admixture did not start until the late 1970s. Ground granulated blast-furnace slag is the granular material formed when molten iron blast furnace slag is rapidly chilled by immersion in water.

2.4.2 Chemical Admixtures

Chemical admixtures are added to concrete to modify its properties. They ensure the quality of concrete during mixing/transporting/placing/curing. They are added mainly for the entrainment of air, reduction of water or cement content, plasticization of fresh concrete mixtures, or control of setting time. They are added in smaller amounts as compared to mineral admixtures. They fall into the following categories: Air entrainers, Water reducers, Set retarders, Set accelerators, and Superplasticizers.

Air Entrainers

Air entrainment is the process whereby many small air bubbles are incorporated into concrete and become part of the matrix that binds the aggregate together in the hardened concrete. The formation of small air bubbles dispersed uniformly through the cement paste increases the freeze-thaw durability of concrete and improves concrete's resistance to surface scaling caused by chemical deicers.

Besides the increase in freeze-thaw and scaling resistances, air-entrained concrete is more workable than non-entrained concrete. The use of air-entraining agents also reduces bleeding and segregation of fresh concrete.

Water Reducers

Water-reducing admixtures are groups of products that are added to concrete to achieve certain workability (slump) at a lower w/c than that of control concrete. In other words they are used to reduce the quantity of mixing water required to produce concrete of a certain slump, to reduce water-cement ratio, or to increase slump. Water-reducing admixtures are used to improve the quality of concrete and to obtain specified strength at lower cement content. They also improve the properties of concrete containing marginal- or low-quality aggregates and they help in placing concrete under difficult conditions. When these are used, the water content in concrete is reduced by approximately 5% - 30% depending on whether the reducer is high range or not. Despite reduction in water content, water reducers can cause significant increases in drying shrinkage.

The basic role of water reducers is to deflocculate the cement particles bounded together and release the water tied up in these units, producing more fluid paste at lower water contents. Its effectiveness in concrete is a function of its chemical composition, concrete temperature, cement composition and fineness.

Water reducers have been used primarily in bridge decks, low-slump concrete overlays, and patching concrete.

Set Retarders

Retarders delay hydration of cement without affecting its long-term mechanical properties. They are used to offset the effect of high temperatures in concrete, which decrease setting times, or to avoid complications when unavoidable delays between mixing and placing occur. Their use enables farther hauling, thus eliminating the cost of relocating central mixing plants; they allow more time for texturing or plastic grooving of concrete pavements, allow more time for hand finishing around the headers at the start and end of the production day, help to eliminate cold joints in two-course paving and in the event of equipment breakdown, delay the set for special finishing processes such as an exposed aggregate surface.

Retarders can also be used to resist cracking due to form deflection that can occur when horizontal slabs are placed in sections. Generally, there is some reduction in strength at early ages when retarders are used.

Set Accelerators

To increase the rate of early strength development or to shorten the time of setting, or both, accelerating admixtures are added to concrete. Chemical compositions of accelerators include some inorganic compounds such as soluble chlorides, carbonates, silicates, fluosilicates, and some organic compounds such as triethanolamine.

Calcium chloride ($CaCL_2$) is the most common accelerator used in concrete among the above listed. Because calcium chloride in reinforced concrete can promote corrosion activity of steel reinforcement, especially in moist environments, a growing interest in using "chloride-free" accelerators as replacement for calcium chloride has been observed.

Superplastisizers

These are a class of water reducers originally developed in Japan and Germany in the early 1960's and introduced in the United States in the mid-1970's. They are linear polymers containing sulfonic acid groups attached to polymer backbone at regular intervals.

All Superplastisizers belong to one of four commercial formulations. These are;

Sulfonate melamine-formaldehyde condensate (SMF)

Sulfonated naphthalene-formaldehyde condesates (SNF)

Modified lingnosulfonates (MLS)

Polycarboxylate derivatives

The sulfuric acid groups are responsible for neutralizing surface charges on the cement particles and causing dispersions. The water tied up in the cement particle agglomerations is reduced in this process, thus reducing the viscosity of the paste and concrete. Superplastisizers are mainly used in concrete to produce flowing concrete with very high slump in the range of 7-9 inches (177.8-228.6mm) to be used in very heavily reinforced structures and in placements where adequate consolidation by vibration cannot be readily achieved. It is also essential in the production of high strength concrete at water-cement ratios ranging from 0.3 to 0.4. The type, dosage, and time of addition of superplasticizers, water –cement ratio and the nature or amount of cement determine the effectiveness of the superplasticizer to increase the slump in concrete mixes. In most types of cement, however, superplasticizers have been found to increase the workability of concrete.

The ability of superplasticizers to reduce water requirements by 12-25% without affecting the workability have been found to produce high strength concrete and lower permeability. They can produce coarser than normal air void systems in air entrained concrete. Research by Siebel (1987) indicated that high workability concrete containing superplasticizers can be made with high freeze-thaw resistance, but air content must be increased relative to concrete without superplasticizers. The research also showed that the type of superplasticizer has nearly no influence on the air-void system.

A problem associated with high range water reducers is slump loss. In another research, by Whiting and Dziedzic (1989), it was found that slump loss with time was very rapid although it was claimed that the second –generation high range water reducers did not suffer as much from the slump loss phenomenon as the first-generation conventional water reducers did. Slump loss of flowing concrete was however found to be less severe for co-polymeric based admixtures.

The problem of slump loss can be overcome by adding the admixture to concrete just before it is placed. However, the problems that might arise out of this may be inadequate dosage control.

Specialty Admixture

These include corrosion inhibitors, shrinkage control, alkali-silica reactivity inhibitors, and coloring. They can be used with Portland cement, or blended cement either individually or in combinations.

2.5.0 Water for Mixing Concrete

All natural and processed water that is drinkable and has no pronounced taste or odor can be used as mixing water for making concrete if as it has no chemicals that will react with the concrete constituents to change its required properties or standards. An example of this is the use of saline water, which can cause dampness of the concrete, efflorescence (white deposits of precipitated salts on the surface of the concrete), increased risk of corrosion (rust) damage to embedded reinforcement, and damage to paint systems. It is therefore advisable not to use such water for durable concrete work, and its use is generally avoided. However, some water, which may not be suitable for drinking, may still be safe for mixing concrete.

Pipe borne drinking water supplies are generally safe for making concrete; however, if in doubt of the quality of water being used, a simple test to verify its usability is to simply make two sets of cubes or cylinders of the same mix, one with the doubtful water, and the other set with distilled water, purified water, tap water, or other drinkable water of good quality. By using the second mix as reference, if the suspected water produces concrete of twenty eight (28)-day compressive strengths for at least 90% of the strength of the reference set, then it can be considered suitable for mixing concrete. If however it falls below this percentage, its use will depend on how far below it falls, and the standards and use for which the concrete is to be used, ASTM C94

ASTM C94/ 92 specifies limits of chemicals allowed in mixing water for concrete and provides a useful guide as to allowances that have worked in practice.

It is acknowledged that the quality of the constituents of a concrete mix plays an important role in the quality of the concrete; however, the best materials will fail if incorporated into a concrete mixture in an improper manner or if the concrete is subsequently incorrectly mixed or transported. It is therefore important to ensure that the batching process and sequence during loading of the concrete mixer is as important as the quality of materials that make up the concrete mix.

2.6.0 Properties of Concrete

The desired properties required in any concrete mix are the following;

Workability

This is the ease at which concrete is placed, consolidated and finished. Concrete mixes should be workable but not segregated or bleeding excessively. Entrained air improves workability and reduces the chances of segregation.

Proper consolidation of concrete makes the use of stiffer mixes possible. Stiffer mixes tend to be more economical and are achieved by reducing the water to cement ratio or using larger proportions of coarse aggregates and a smaller proportion of fine aggregates, resulting in improved quality and economy.

Resistance to Freezing and Thawing and Deicing Chemicals

A desired design requirement in concrete structures and pavements is to achieve a long life span with as little maintenance cost as possible. As such the concrete must be able to resist the harsh natural conditions it is exposed to. The most destructive weathering factor that concrete is exposed to is freezing and thawing while the concrete is wet, especially in the presence of deicing chemicals. The freezing of the water in the paste, the aggregates or both, mainly causes deterioration.

As the water in moist concrete freezes, it produces osmotic and hydraulic pressures in the capillaries and pores of the cement paste and aggregate. Hydraulic pressures are caused by the 9% expansion of water upon freezing, in which growing ice crystals displace unfrozen water. If a capillary is above critical saturation (91.7% filled with water), hydraulic pressures result as freezing progresses. At lower water contents, no hydraulic pressure should exist.

If the pressure exceeds the tensile strength of the paste or aggregate, the cavity will dilate and rupture. The accumulative effect of successive freeze-thaw cycles and disruption of paste and aggregate eventually cause significant expansion and deterioration of the concrete. Deterioration is visible in the form of cracking, scaling, and crumbling. Air entrainment is helpful in this respect and makes concrete highly resistant to deterioration due to this factor.

Concrete's resistant to freezing and thawing, rests on the quality of the hardened paste [ERDC/CRREL TR-02-5]. Hence, the development of the pore structure inside the cement paste is fundamental to understanding the freeze–thaw resistance of concrete

An approach to increasing concrete's resistance to freeze–thaw damage is to modify its microstructure, because concrete readily absorbs water, when it is in a wet environment and then cooled to below 0°C, any water that freezes inside the concrete will expand and, depending on the nature of the internal pore structure, could lead to internal micro-cracks. There are several mechanisms responsible for this damage, so preventing it is complex.

There are several methods used to decrease the impact caused by freezing water, these include

1) Incorporating entrained air into the concrete to relieve pressures caused by freezing water.

2) Using low water-to-cement ratios to minimize the type of voids in which water typically freezes.

3) Using silica fume to refine the pore system so that water may not be able to freeze at normal ambient temperatures.

Freeze-Thaw durability is determined by a laboratory test procedure ASTM C666, "Standard Test Method for resistance of Concrete to Rapid Freezing and Thawing."

Permeability and Water-tightness

Permeability is the ability of concrete to resist water penetration or other substances. Pavements as well as other structures depending on their use require very little or no penetration of water. Water-tightness is the ability of the concrete to retain water without visible leakage; this property is desirable in water retaining or confined structures.

Permeability and water tightness is a function of the permeability of the paste and aggregates, the gradation of the aggregates and the relative proportion of paste to aggregate.

These are related to water-cement ratio and the degree of cement hydration or length of moist curing.

Strength

This is defined as the maximum resistance of a concrete specimen to axial loading. The most common measure of concrete strength is the compressive strength. It is primarily a physical property, which is used in design calculations of structural members. General use concrete has a compressive strength of 3,000 psi – 5,000 psi (21.0 – 35.0 MPa) at an age of twenty-eight (28) days whilst high strength concrete has a compressive strength of at least 6,000 psi (42.0 MPa).

In pavement design, the flexural strength of concrete is used; the compressive strength can be used, however, as an index of flexural strength, once the empirical relationship between them has been established.

The flexural strength is approximated as 7.5 to 10 times the square root of the compressive strength whilst the tensile strength is approximated as 5 to 7.5 times the square root of the compressive strength. The major factors, which determine the strength of a mix, are: The free water-cement ratio, the coarse aggregate type (Harder coarse aggregates result in stronger concrete.), and the cement properties.

Wear resistance

Pavements are subjected to abrasion; thus, in this type of application concrete must have a high abrasion resistance. Abrasion resistance is closely related to the compressive strength of the concrete.

Economy

Since the quality of concrete depends mainly on the water to cement ratio, to reduce the cost of concrete due to the volume of cement in the mix, the water requirement should be minimized to reduce the cement requirement. Adopting any of the following methods or a combination of any two or all three as follows can minimize the cost of concrete;

- Use the stiffest mix possible.
- Use the largest size aggregate practical for the job.
- Use the optimum ratio of fine to coarse aggregate.

CHAPTER 3 – SAMPLE PREPARATION, MATERIALS AND TEST METHODS

3.0.0 Introduction

The previous two chapters gave a brief overview of past research, into concrete as a construction material, and the essence of early strength concrete in pavement maintenance and rehabilitation. This chapter details the procedures, materials used and specifications adopted in the preparation of the concrete specimens. The various test methods and test procedures are also detailed and explained.

To attain early strength, the mix designs adopted from the SHRP-C-373 report by the Construction Technology Laboratory (CTL) made use of the following techniques:

- Use of Type III High Early Strength cement.
- Low water - cement ratio (0.3-0.45 by mass) using Type I cement.
- Use of chemical admixtures to enhance workability and durability.

The water to cement ratios varied from 0.3 to 0.45 depending on the specimen in question. The use of normal Portland cement (Type I), and High Early Strength Portland cement (Type III) was employed with various dosages of different kinds of admixtures depending on the concrete quality and specifications required in an attempt to attain the specified strength and durability requirements. The coarse aggregate-fine aggregate, and the cement-fine aggregate ratio were also varied in each mix.

3.1.0 Research Procedure

This research was divided into two phases. Phase I included preparation, casting, curing and testing of the various concrete specimens for compressive strength in accordance with ASTM C 39/C 39M -01. Phase II of this research comprised the preparing, casting, curing and testing of the resistance of the concrete specimens to rapid freezing and thawing conditions in accordance with ASTM C 666-97.

The concrete was mixed and cured in accordance with ASTM C192/ 192M-02. A total of 4 designed mixes adopted from the literature review by the Construction Technology Laboratory and a mix obtained from the Maryland State Highway Authority (MSHA) used as a control mix were batched and tested.

3.2.0 Materials

The aggregates used in this research were obtained from Aggregates Industries. All admixtures were obtained from WR-Grace and the cement from Greenwald Industry. Products Co. Clean pipe-borne water was used.

The materials used in this research and their sources are summarized in Table 3.1.

Material	Type /Manufacturer	Vendor	MSHA Approval
Cement	Lehigh Type I	Greenwald Ind. Products Co.	Approved
	Lehigh Type III	Greenwald Ind. Products Co.	Approved
Fine Aggregate	Mortar sand	Aggregate Industries	Approved
Coarse Aggregate	¾" Quarry Gravel	Aggregates Industries	Approved
Admixtures	Accelerator (Polar set)	Grace Construction Products	Approved
	HRWA (ADVA Flow)	Grace Construction Products	Approved
	AEA (Darex II)	Grace Construction Products	Approved

Table 3.1: Source of Materials

3.2.1 Material Preparations

The aggregates were passed through a sieve to determine the gradation (the distribution of aggregate particles, by size, within a given sample) in order to determine compliance with mix design specifications. This was done using a tray shaker. Both the coarse and fine aggregates were oven dried to establish a standard uniform weight measurement throughout the test. The dry weights of the aggregates were used in this research. The amount of water was adjusted to reflect the free water necessary for the aggregate to be used in their dry state.

Figure 3.1: Fine and Coarse aggregates being dried in oven

3.3.0 Concrete Mix

3.3.1 Mix Characteristics and Specifications

The mix specifications obtained from the CTL report were adjusted to match the bulk saturated surface dry specific gravity and Absorption of the aggregates to be used. The coarse and fine aggregates obtained from Aggregate Industries were found to have a Bulk SSD of 2.72 and 2.59, respectively, and absorption of 0.36% and 1.36%, respectively. All aggregates were oven dried before use. Tables 3.2 and 3.3 show the proposed mix specifications at SSD and adjusted weights (dry weights) based on the absorption properties of the coarse and fine aggregates found by laboratory methods in accordance with ASTM C127-01and C128-01, respectively.

	MIX DESIGN Materials at SSD (Cubic yard basis)				
MIX	**1**	**2**	**3**	**4**	**CONTROL**
Cement Type	III	I	III	I	I
Cement, Ib	870	752	915	900	800
Coarse Aggregate, Ib	1732	1787	1124	1596	1772
Fine Aggregate, Ib	831	1015	1218	1125	1205
Water, Ib	339	286	412	270	242
Accelerator, (PolarSet), gal.	6	3.5	6	6	16
HRWR (ADVA Flow), oz.	43.5	37.6	45.8	45	40
Darex II AEA, oz.	43.5	15	73.2	45	16
W/C Ratio	0.39	0.38	0.45	0.30	0.30

Table 3.2: Proposed mix specifications at SSD:

3.3.2 Actual mix specifications (Dry weights):

To ensure that the mix proportions were exact according to specifications for laboratory testing, the dry weights of the aggregates were calculated and the water-cement ratio adjusted. The mix design obtained from the report by CTL was based on the saturated surface dry density (SSD) of the aggregates. Because aggregates vary in SSD, the absorption of the aggregates used in this research was calculated in accordance to ASTM C-127 and C-128 for coarse and fine aggregates respectively.

To find the SSD and absorption of the aggregates, the aggregates were oven dried to a condition where there was no change in mass. The dry weights of the aggregates were measured and recorded. The aggregates were then immersed in water to a state where they were fully saturated. The weights of the fully saturated aggregates were measured and the absorption computed as follows;

Weight at SSD = X g

Absorption (ABS) = Y%

Dry Weight =? g

Water at SSD =? g

Dry Weight + Water at SSD = weight at SSD

ABS + Dry weight = weight at SSD

$((100\%+Y\ \%)\ /100)_{\text{of dry weight}} = X$ g

Dry Weight = X g / ((100+Y)/100)

Weight of water = Weight at SSD – Dry weight.

Knowing the quantity of water that the aggregate will absorb when fully saturated, the dry weights of the aggregate was computed as shown above and the amount of absorbed water at SSD was added to the amount of free water to get the total weight of water required for the mix. Allowance was also allowed for the use of Polarset since each liter of Polarset added to a concrete mix will contribute 0.78 kg (6.5 lbs/gal) of water to that mix.

Table 3.3 shows the actual mix specifications for all 5 mixes.

	MIX DESIGN Materials Dry Weight (Cubic yard basis)				
MIX	**1**	**2**	**3**	**4**	**CONTROL**
Cement Type	**III**	**I**	**III**	**I**	**I**
Cement, Ib	870	752	915	900	800
Coarse Aggregate, Ib	1726	1781	1124	1590	1766
Fine Aggregate, Ib	820	1001	1218	1110	1189
Water, Ib	356.3	306.1	412	290.8	264.5
Accelerator, (PolarSet), gal.	6	3.5	6	6	1
HRWR (ADVA Flow), oz.	43.5	37.6	45.8	45	40
Darex II AEA, oz.	43.5	15	73.2	45	16
W/C Ratio	0.45	0.44	0.51	0.37	0.34

Table 3.3: Actual mix specifications

3.4.0 Phase I - Compressive Strength Test

This phase consists of applying a compressive axial load to a molded cylinder until failure occurs in accordance with ASTM C39/C 39M-01.

The material for each mix design was batched based on the actual mix specifications in Table 3.3 above. The concrete was mixed and cured in accordance with ASTM C192/ 192M-02, “Standard Practice for Making and Curing Test Specimens in the Laboratory,” making sure the inner surface of the mixer was wetted to compensate for the loss of free water due to absorption by the surface of the mixer.

The concrete components were mixed in an electrically driven mixer. A shovel was used to scoop the mixed concrete into a large wheelbarrow and a "slump test" was used to test the water content of the concrete. The cone was 1’-0” high, with a top opening of 4” diameter and a bottom opening of 8” diameter. The mixed concrete was placed into the cone through the top, a bar was used to compact the concrete, and remove air voids, within the cone. The cone was then lifted clear. By laying a bar on top of the cone, it was possible to measure how far the concrete "slumped."

6”x12” cylindrical plastic molds were filled and compacted using an external table vibrator to remove air voids. A total of 60 cylindrical specimens were cast, four (4) for each of the 3-test conditions (4 hours, 24 hours, and 7 days) for a total of 5 different mixes. The 20 specimens were then de-molded, weighed and tested after 4 hours to obtain the compressive strength. The same procedure was repeated after 24 hours and seven (7) days to obtain the compressive strength after that period of placing. The seven (7) day-old specimen was placed in a curing tank after twenty –four (24) hrs.

Fig.3.2: Cast cylindrical specimen

Fig.3.3: De-molding the cylindrical specimens

Fig.3.4: De-molded specimen for 4 hr compressive strength test

Fig.3.5: Specimen in the compression machine

Fig. 3.6: Specimen under compression

3.5.0 Phase II –Resistance to freeze and thaw

The same mix design specification in Table 3.1 was used in the preparation of the specimen in this phase. Procedure A, “Rapid Freezing in water and Thawing in water” was adopted for this test in accordance with ASTM C 666-97.

Prism-shaped steel molds with internal dimensions of 3” x 4” cross-sectional area and 16” lengths were used in this phase. After casting, the exposed parts of each specimen were covered with aluminum foil to prevent drying and shrinkage. All 20 specimens were de-molded after 24 hours. The de-molded specimens were cured in a plastic curing tank for 14 days. After 14 days of curing, each specimen was placed in a freeze and thaw chamber for the freeze and thaw cycle to begin.

Each specimen was placed in a container filled with water at the beginning of the freezing phase of the cycle. The temperature of the chamber was lowered from 40° F to 0° F and raised from 0° F to 40° F within 2 to 5 hours. At intervals ranging from 10–25 cycles of exposure to freeze and thaw, each specimen was removed from the chamber, weighed and made to undergo transverse vibration. This was to enable the weight of the specimen, and the transverse frequency to be measured and documented.

Fig. 3.7: Prism specimen covered with foil to prevent drying and shrinkage

Fig. 3.8: Freeze and thaw chamber

Fig.3.9: Specimen being removed from chamber for testing at thaw machine breakdown

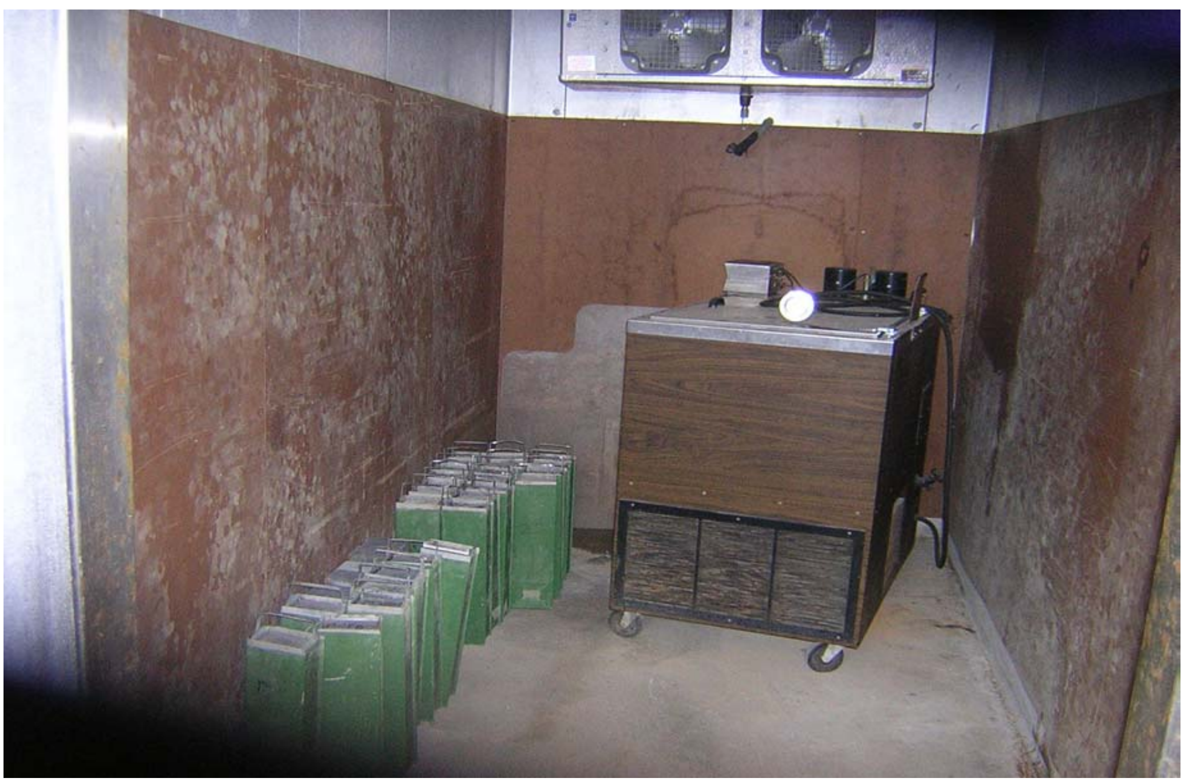

Fig.3.10: Storage Freezer used as storage facility during freeze and thaw failure

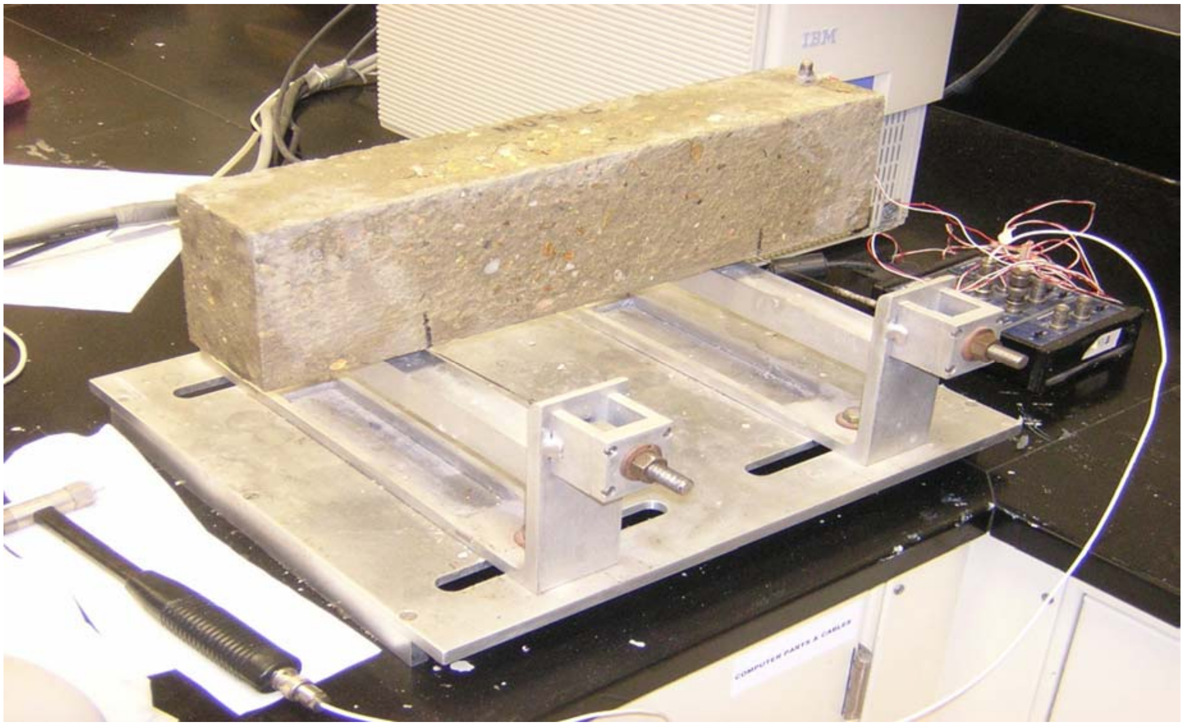

Fig.3-11: Specimen undergoing transverse vibration

Fig.3-12: Results of transverse vibration of specimen shown on the monitor screen

3.6.0 Identification of specimen

Each specimen was identified based on the nomenclature assigned to it. For the cylindrical specimen tested for compressive strength, a nomenclature of MC1A depicted Mix 1, specimen A. For a specimen used in the freeze and thaw test, a nomenclature of MU1A depicted Mix 1, specimen A.

3.7.0 Apparatus

General Apparatus

1. Concrete mixer
2. Tamping rod 5/8" diameter and approximately 24in. long.
3. Mallet
4. External Vibrator (table vibrator)
5. Small tools (shovel, trowel, wood float, straight edge, ruler, scoop, slump apparatus)
6. Sampling and mixing pan
7. Air content apparatus
8. Scale (large and small scales)
9. Curing tank

Phase I

1. 6" x12" cylindrical molds
2. Compression testing machine

Phase II

1. 3"x 4"x16" prism molds
2. Freeze and thaw chamber
3. Freezing chamber
4. Temperature measuring equipment
5. Dynamic testing apparatus conforming to the requirements of Test Method C215
6. Tempering tank

3.8.0 Materials

The following materials were used for this research; Type I and III cements, ¾" coarse aggregates (gravel), fine aggregate (mortar sand), admixtures (PolarSet, ADVA Flow and Darex II from Grace construction products)

CHAPTER 4 – TEST RESULTS AND DISCUSSIONS

4.0.0 Introduction

This chapter reports the results obtained from the laboratory tests of the various test specimens. It attempts to analyze the results obtained and report them in a graphical and tabular format. It deals with the compression test results as an isolated criterion and then the freeze and thaw test results as another. It finally attempts to analyze the various mixes combining both criteria.

The mixes employed in this research were designed to attain a compressive strength of at least 2,500 psi (17.5 MPa) in 4 hours or less, it was also expected that the mixes would go through at least 300 cycles of freeze and thaw without failing or excessive scaling.

A summary of the test results is discussed in the sections that follow.

4.1.0 Properties of the concrete mixes.

The property of a concrete mix depicts its strength, durability and performance under loading. Properties affecting concrete characteristics measured in this research include the following;

- Air content
- Consistency

When in its fresh state, concrete should be plastic or semi-fluid and generally capable of being molded by hand. This does not include a very wet concrete which can be cast in a mold, but which is not pliable and capable of being molded or shaped like a lump of modeling clay nor a dry mix, which crumbles when molded into a slump cone.

Tables 4.1 and 4.2 illustrate a summary of the properties of the concrete mixes used in this research.

It is assumed that conditions remained constant throughout the preparation and testing of the various samples.

Mix constituents per total weight of constituents

		Mix 1	Mix 2	Mix 3	Mix 4	Mix 5
Proportion of constituents	**Cement Type**	III	I	III	I	I
	Cement	0.227	0.194	0.246	0.228	0.1930
	Fine Aggregates	0.214	0.259	0.323	0.281	0.2860
	Coarse Aggregates	0.451	0.46	0.301	0.403	0.4250
	Air entrainment	0.0007	0.0002	0.0012	0.0007	0.0002
	HRWR	0.0007	0.0006	0.0008	0.0007	0.0006
	Water	0.093	0.079	0.116	0.074	0.064

Table 4.1: The various ratios of mix constituents to the total weight of the mix

Concrete properties

Properties	Mix 1	Mix 2	Mix 3	Mix 4	Mix 5
Unit weight/Ib/Ft^3	137.89	136.30	133.39	135.95	122.19
Air content	7%	7.50%	4.50%	5.40%	17%
Slump	1/8"	1/8"	2"	None	None
Consistency	Medium	Medium	High	None	None

Table 4.2: Concrete properties

The slump test is the most generally accepted method used to measure the consistency of concrete. The slump results in Table 4.2 show that “Mix 3” had the best consistency and “Mix 4” and “Mix 5” had the worst consistencies. This result was expected due to the proportions of water and water reducers in the different mixes. Mix 3 containing 11.6% and 0.08% of water and High range water reducer respectively by weight of the total constituents was expected to be most workable. The opposite was expected for “Mix 4” and “Mix 5” as shown in Table 4.1.

Due to poor consistency of “Mix 4” and “Mix 5”, no slump was recorded for those mixes, the formed cone either collapse totally or did not show any slump when the slump cone was removed.

4.2.0 Compressive test results

One of the most important strength related parameters used to define the “Early strength” of a concrete mix is its compressive strength. The average results are as shown in Tables 4.3a – 4.3c below. Early strength concrete is widely accepted to be concrete that can gain a compressive strength in the range of 2,500 psi and 3,500 psi (17.5 and 24.5MPa) within 24 hours or less.

4 Hour Test Results				
Specimen No	**Specimen Age**	**Average Weight lb (kg)**	**Average Load Ib (kg)**	**Comp. Strength psi (MPa)**
Mix 1	4 hrs	28.0 (12.7)	64,625 (29,313)	2,285 (15.8)
Mix 2	4 hrs	28.5 (12.9)	24,000 (10,886)	849 (5.9)
Mix 3	4 hrs	27.5 (12.4)	77,625 (35,210)	2,745 (18.9)
Mix 4	4 hrs	27.0 (12.2)	23,667 (10,735)	837 (5.8)
Mix 4	4 hrs	27.0 (12.2)	23,625 (10,716)	835 (5.8)

Table 4.3a: 4 Hours Compressive Average Strength

24 Hour Test Result				
Specimen No	**Specimen Age**	**Average Weight lb (kg)**	**Average Load Ib (kg)**	**Comp. Strength psi (MPa)**
Mix 1	24 hrs	28.0 (12.7)	135,500 (72,745)	4,792 (39.1)
Mix 2	24 hrs	27.6 (12.5)	98,875 (45,983)	3,497 (24.7)
Mix 3	24 hrs	27.6 (12.5)	140,250 (78,641)	4,960 (42.3)
Mix 4	24 hrs	27.2 (12.3)	52,375 (41,163)	1,852 (22.1)
Mix 5	24 hrs	27.1 (12.3)	53,000 (42,694)	1,874 (23.0)

Table 4.3b: 24 Hours Average Compressive Strength

7 Day Test Result				
Specimen No	**Specimen Age**	**Average Weight lb (kg)**	**Average Load Ib (kg)**	**Comp. Strength psi (MPa)**
Mix 1	7days	28.3 (12.8)	160,375 (72,745)	5,671 (39.1)
Mix 2	7days	27.7 (12.6)	101,375 (45,983)	3,585 (24.7)
Mix 3	7days	27.7 (12.6)	173,375 (78,641)	6,131 (42.3)
Mix 4	7days	27.1 (12.3)	90,750 (41,163)	3,209 (22.1)
Mix 5	4 hrs	27.2 (12.3)	94,125 (42,694)	3,329 (23.0)

Table 4.3c: 7 days Average Compressive Strength

For the raw data obtained from the laboratory, refer to Appendix B.

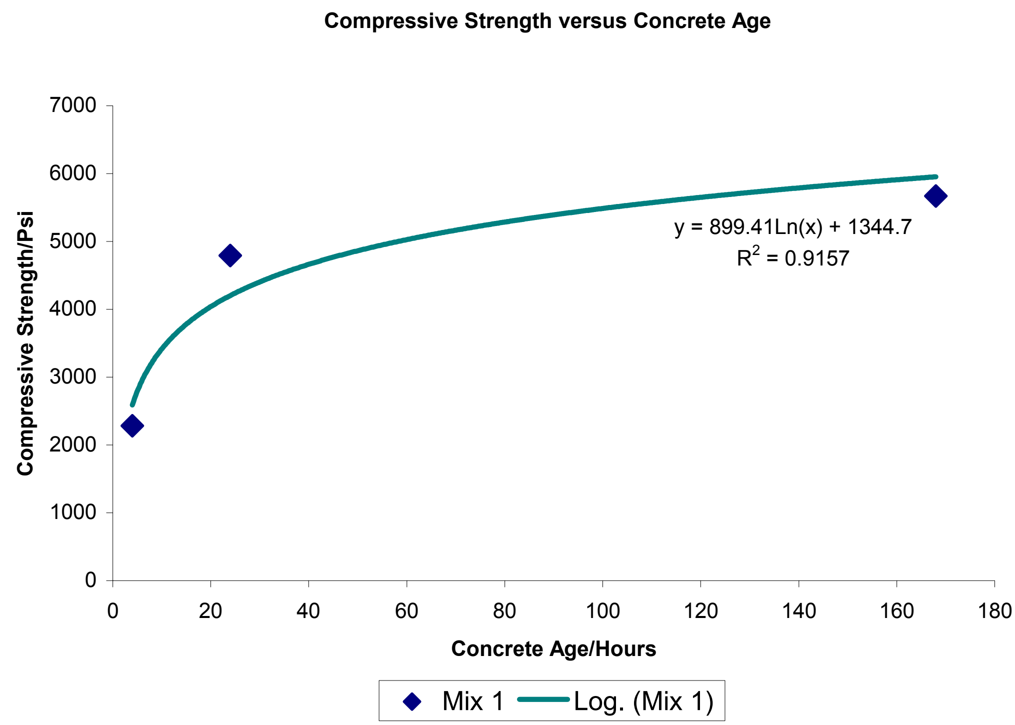

Figure 4.1a: Variation of Compressive strength of "Mix 1" with Age

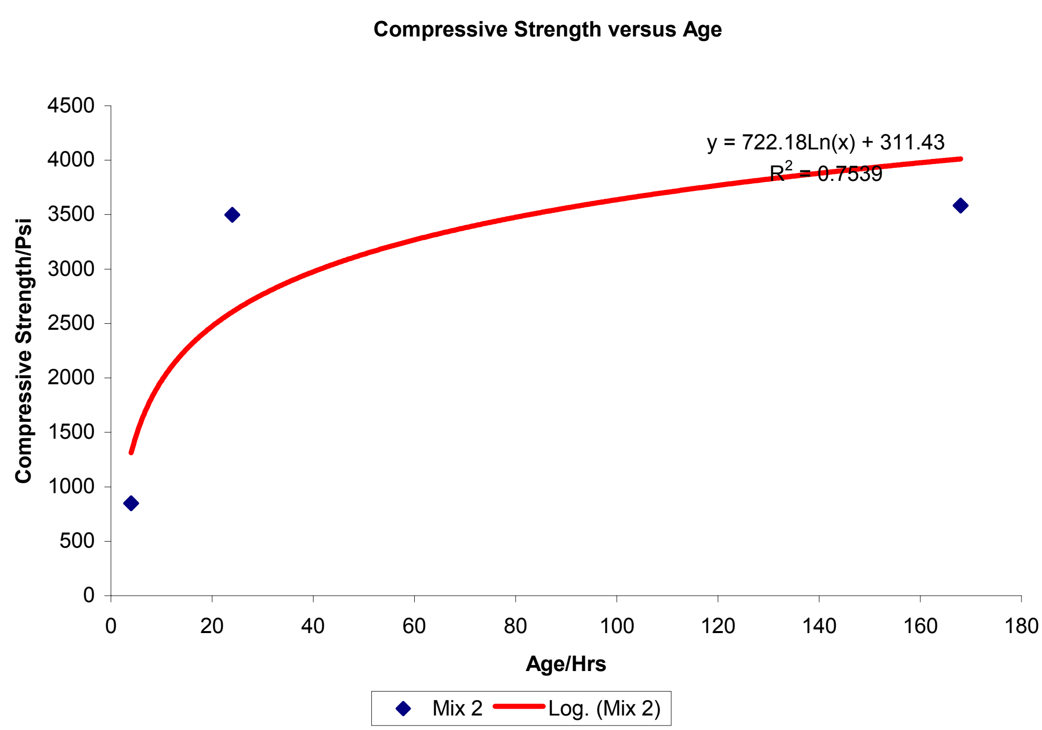

Figure 4.1b: Variation of Compressive strength of "Mix 2" with Age.

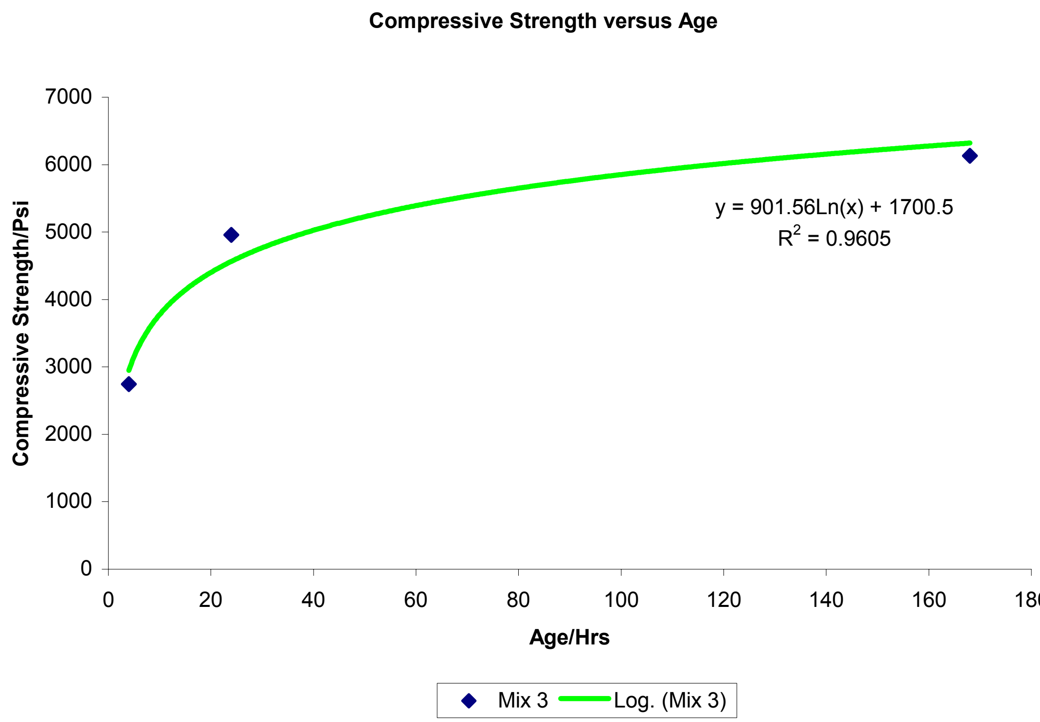

Figure 4.1c: Variation of Compressive strength of "Mix 3" with Age.

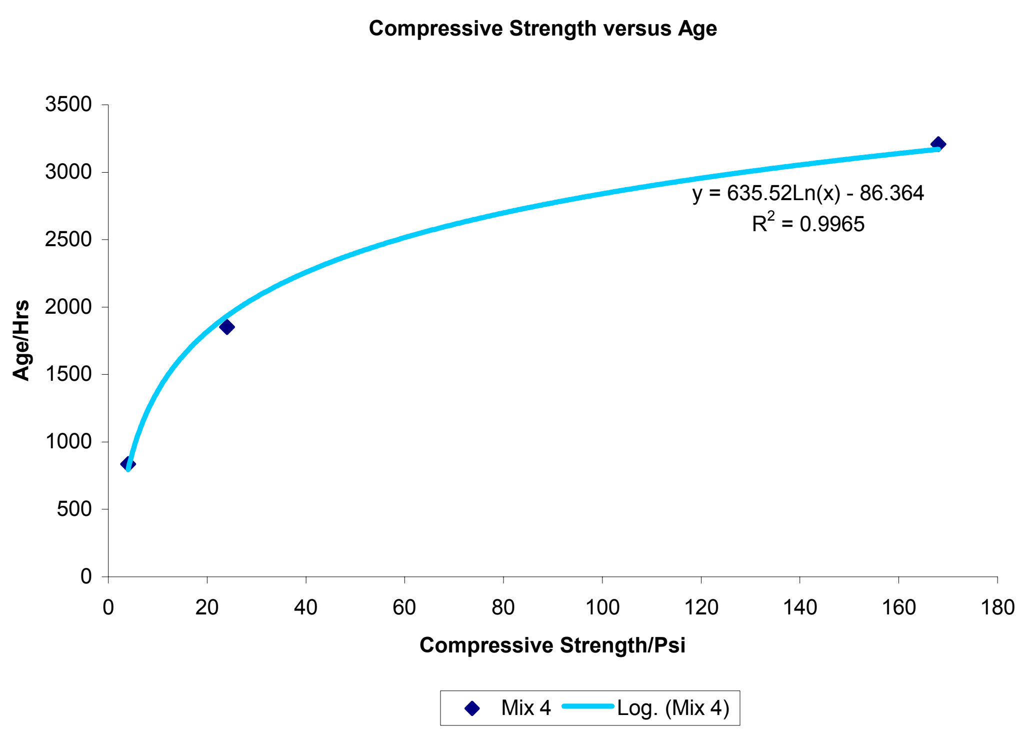

Figure 4.1d: Variation of Compressive strength of "Mix 4" with Age.

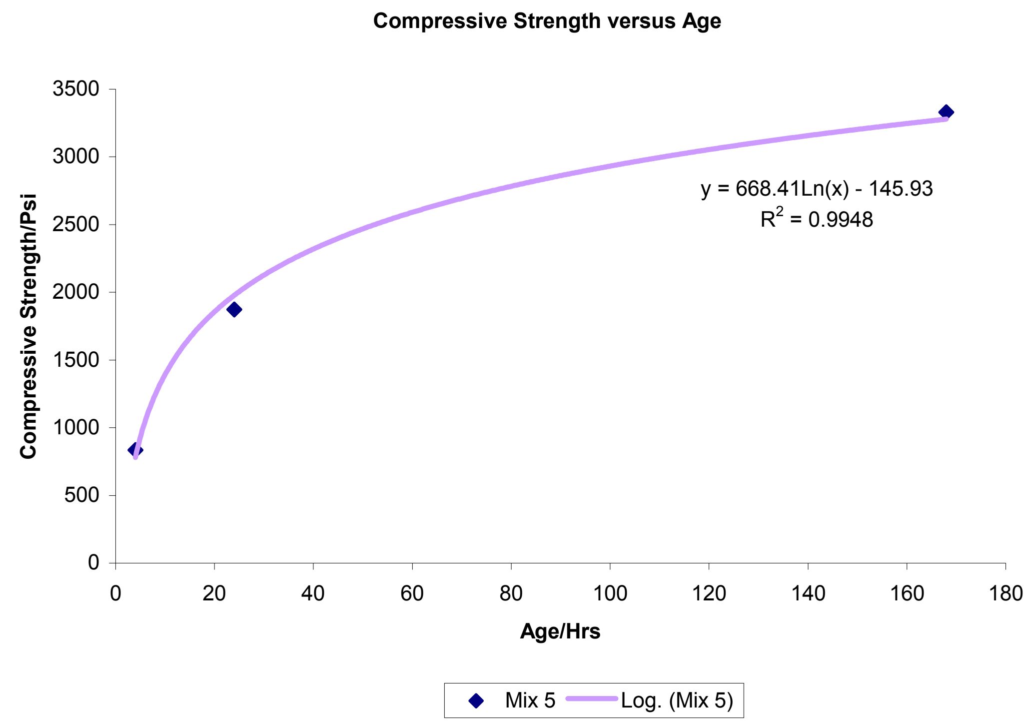

Figure 4.1e: Variation of Compressive strength of "Mix 5" with Age.

Figures 4.1a-4.1e show increasing strength of the samples of concrete as a function of curing time. It can be noticed that strength gain is quite rapid at first for all samples. The results obtained from the laboratory tests shown in Tables 4.3a-4.3e show that "Mix 1" and "Mix 3" with compressive strength of 2,285 psi and 2,745 psi (16.0 and 19.0 MPa) in 4 hours and 4,792 psi and 4,959 psi (33.5 and 34.7 MPa) in 24 hours fall within the criteria for the definition of early strength concrete. Although "Mix 2" did not achieve the compressive strength desired in four hours, its compressive strength increased drastically within 24 hours and 7days. "Mix 4" and "Mix 5" did not show any strength characteristics to be considered as an "Early Strength" mix within 4 hours to 24 hours. Although tests were not done for 14 days and 28 days, the shape of the curve makes it quite clear that strength continues to increase well beyond a month, research has shown that under favorable conditions, concrete is still "maturing" after 18 months.

4.3.0 Summary of Compressive strength Results

A logarithmic regression line was the best trend line fit for the data acquired from the laboratory test results. The regression equations for the various mixes are tabulated in Table 4.4 below and Table 4.5 gives the compressive strength results based on this.

Mix	Logarithmic Regression Equation	R^2 Value
1	y = 899.41Ln(x) + 1344.7	R2 = 0.9157
2	y = 722.18Ln(x) + 31.43	R2 = 0.7539
3	y = 901.56Ln(x) + 1700.5	R2 = 0.9605
4	y = 635.52Ln(x) - 86.364	R2 = 0.9965
5	y = 668.41Ln(x) - 145.93	R2 = 0.9948

Table 4.4: Logarithmic Regression equations for Laboratory test results

Mix	Compressive Strength/Ksi (Mpa)		
	4hrs	**24hrs**	**7days**
1	2.592 (17.87)	4.203 (28.98)	5.953 (41.04)
2	1.033 (7.122)	2.327 (16.04)	3.732 (25.73)
3	2.950 (20.34)	4.566 (31.48)	6.320 (43.57)
4	0.795 (5.48)	1.933 (13.33)	3.170 (21.86)
5	0.781 (5.38)	1.978 (13.64)	3.279 (22.61)

Table 4.5: Compressive Strengths of various mixes

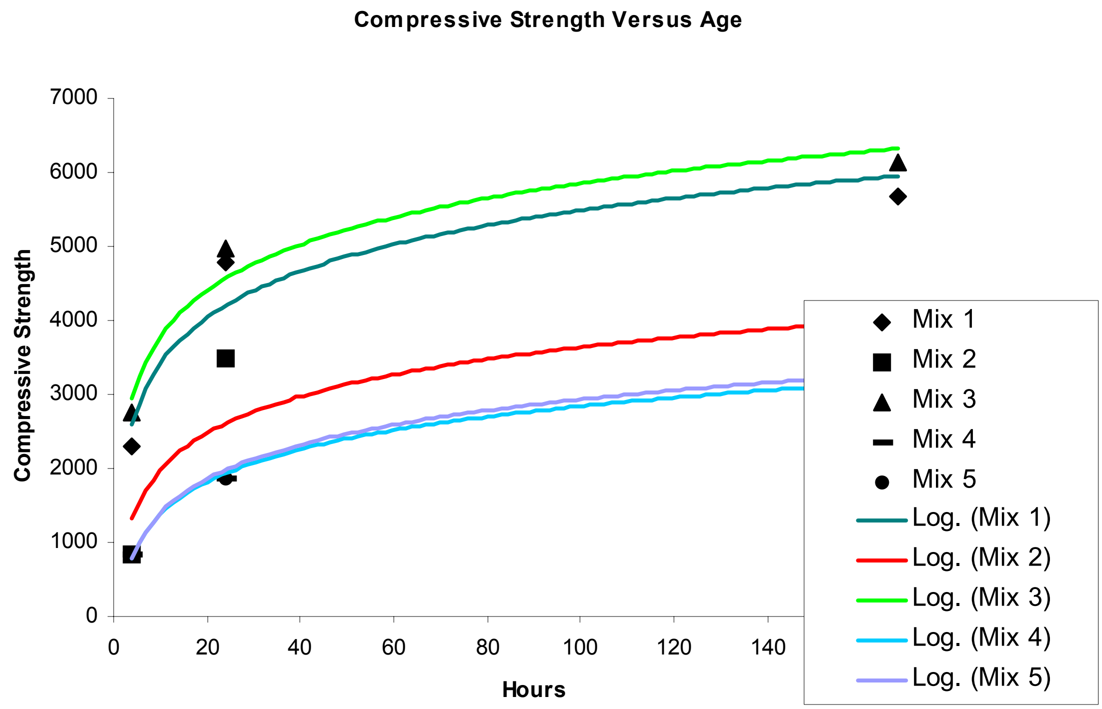

Figure 4.2: Compressive strength of the various mixes with Age

4.4.0 Freeze and Thaw test results

Tables 4.6a-4.6e show the laboratory results obtained from the freeze and thaw tests. During the tests, there were machine breakdowns on three occasions and the samples were stored in a freezer in accordance to specifications. Although the results obtained are with an assumption that testing conditions remain the same during subsequent tests, practically that is never the case. The laboratory room conditions varied slightly in between cycles.

The "Relative Dynamic Modulus of Elasticity (RDM)" was calculated based on the Resonance Transverse Frequency obtained from tests carried out in the Laboratory. The "Durability Factor" was also calculated based on the RDM using the following formulas in accordance with ASTM C666.

4.4.1 Relative Dynamic Modulus of Elasticity

$P_c = (n_1^2 / n^2) \times 100$

Where:

P_c = Relative dynamic modulus of elasticity, after c cycles of freezing and thawing in percentage

n = Fundamental transverse frequency at 0 cycles of freezing and thawing, and

n_1 = Fundamental transverse frequency after c cycles of freezing and thawing

4.4.2 Durability Factor

DF = PN/M

P = Relative dynamic modulus of elasticity, at N cycles in percentage

N = Number of cycles at which P reaches the specified minimum value for discontinuing the test or the specified number of cycles at which the exposure is to be terminated, whichever is less, and

M = Specified number of cycles at which exposure is to be terminated.

To arrive at these values, the procedure used for judging the acceptability of the durability factor results obtained in the Laboratory as outlined in ASTM C666 Section 11.0 was used. This required finding the average of the Fundamental frequencies and standard deviation of the specimens. The raw data of this can be found in Appendix A

Mix 1				
Cycle	Mass(g)	Frequency	Relative Dynamic Modulus of Elasticity (P_c) (%)	Durability Factor (%) (DF)
0	7093	2149	100	100
24	7093	2079	94	94
39	7124	2093	95	95
51	7121	2071	93	93
69	7118	2035	90	90
81	7110	1996	86	86
95	7099	1956	83	83
107	7093	1967	84	84
134	7018	1947	82	82
148	7009	1912	79	79
175	7032	1875	76	76
189	7014	1852	74	74
201	6999	1764	67	67
227	6982	1819	72	72
252	6952	1769	68	68
270	6930	1752	66	66
289	6926	1843	74	74
314	6902	1800	70	70
338	6686	1708	63	63

Table 4.6a: Elastic Modulus and Durability Factors for Mix 1

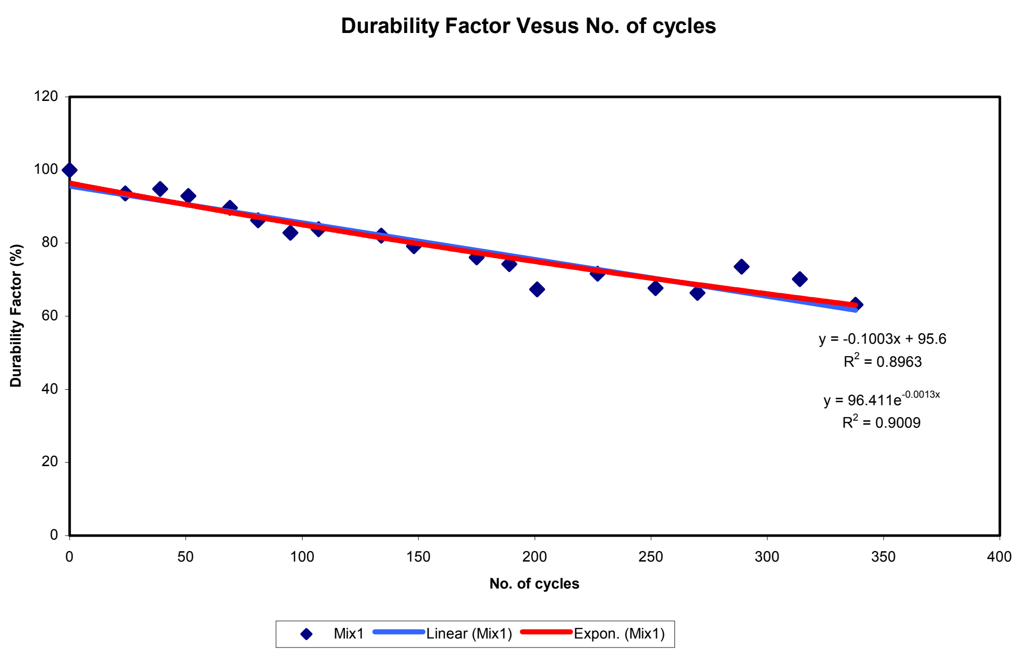

Figure 4.3a: Graph of durability vs No of cycles for “mix 1”

Mix 2				
Cycle	Mass (g)	Frequency	Relative Dynamic Modulus of Elasticity (P_c) (%)	Durability Factor (%) (DF)
0	7254	2118	100	100
24	7254	2075	96	96
39	7247	2073	96	96
51	7242	2071	96	96
69	7226	2074	96	96
81	7211	2073	96	96
95	7182	2063	95	95
107	7194	2076	96	96
134	7179	2068	95	95
148	7166	2069	95	95
175	7150	2071	96	96
189	7139	2061	95	95
201	7126	2071	96	96
227	7126	2071	96	96
252	7110	2073	96	96
270	7095	2057	94	94
289	7087	2060	95	95
314	7089	2068	95	95
338	7075	2061	95	95

Table 4.6b: Elastic Modulus and Durability Factors for Mix 2

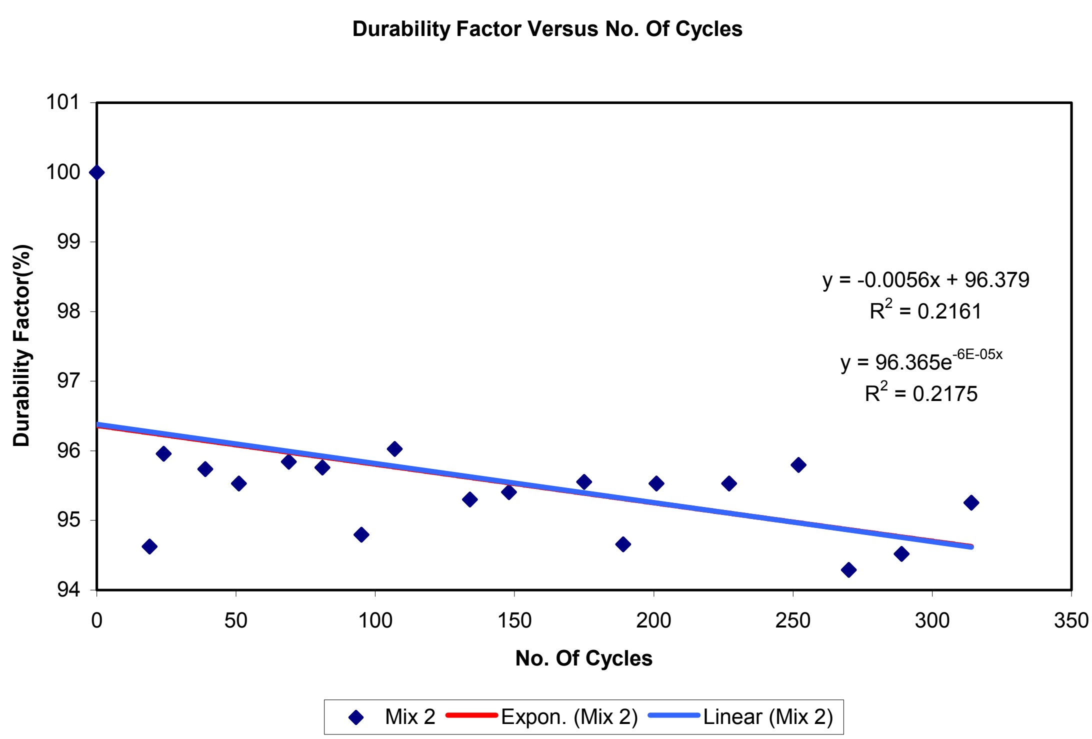

Figure 4.3b: Graph of durability vs No of cycles for “mix 2”

Mix 3				
Cycle	Mass (g)	Frequency	Relative Dynamic Modulus of Elasticity (P_c) (%)	Durability Factor (%) (DF)
0	6916	2011	100	100
24	6904	1989	98	98
39	6899	1985	97	97
51	6893	1967	96	96
69	6888	1955	95	95
81	6877	1939	93	93
95	6869	1921	91	91
107	6865	1916	91	91
134	6848	1873	87	87
148	6838	1836	83	83
175	6814	1829	83	83
189	6805	1788	79	79
201	6805	1788	79	79
227	6798	1733	74	74
252	6763	1633	66	66
270	6739	1593	63	63
289	6758	1628	66	66
314	6743	1596	63	63
338	6725	1515	57	57

Table 4.6c: Elastic Modulus and Durability Factors for Mix 3

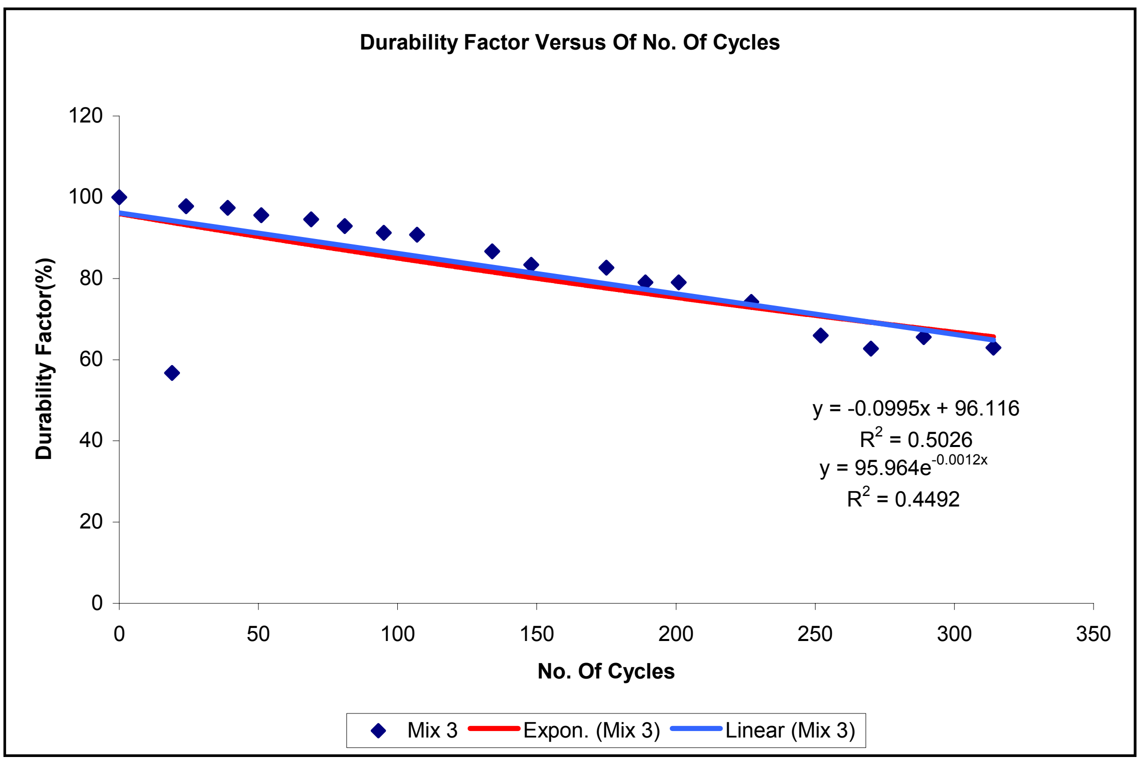

Figure 4.3c: Graph of durability vs No of cycles for “mix 3”

Mix 4				
Cycle	**Mass (g)**	**Frequency**	**Relative Dynamic Modulus of Elasticity (P_c) (%)**	**Durability Factor (%) (DF)**
0	7384	2196	100	100
24	7377	2165	97	97
39	7374	2170	98	98
51	7371	2164	97	97
69	7371	2157	97	97
81	7368	2153	96	96
95	7367	2152	96	96
107	7368	2161	97	97
134	7373	2146	95	95
148	7371	2146	96	96
175	7391	2157	96	96
189	7388	2136	95	95
201	7390	2141	95	95
227	7392	2152	96	96
252	7387	2155	96	96
270	7329	2055	88	88
289	7419	2175	98	98
314	7419	2173	98	98
338	7415	2164	97	97

Table 4.6d: Elastic Modulus and Durability Factors for Mix 4

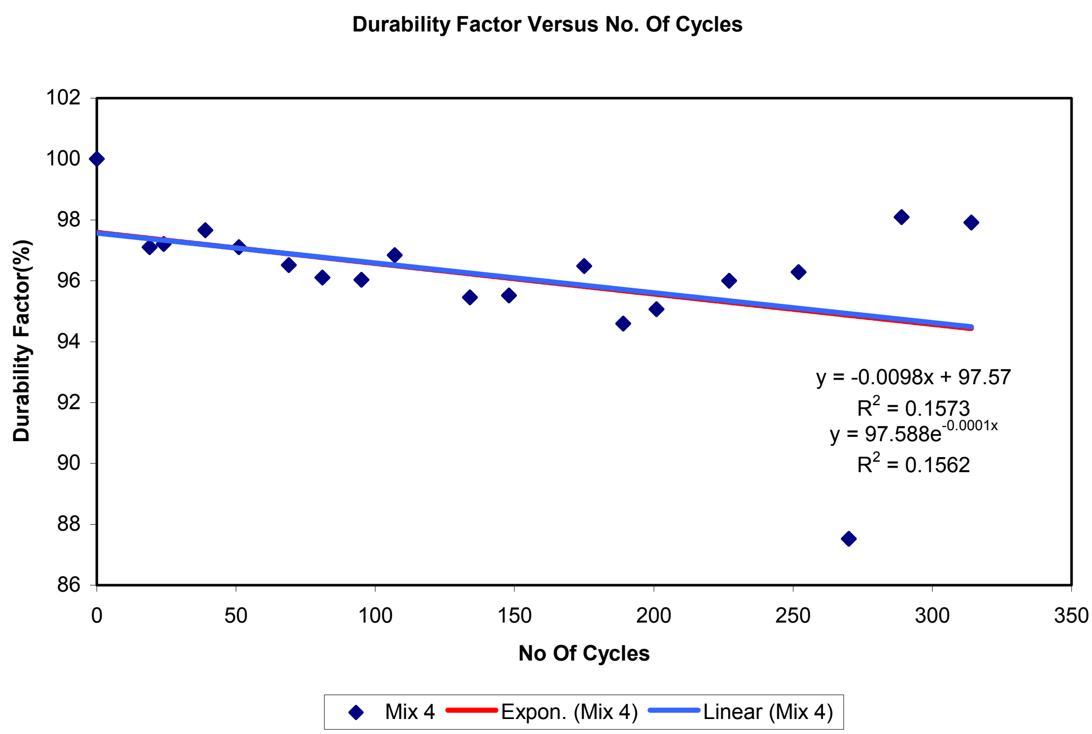

Figure 4.3d: Graph of durability vs No of cycles for “mix 4”

Mix 5				
Cycle	Mass (g)	Frequency	Relative Dynamic Modulus of Elasticity (P_c) (%)	Durability Factor (%) (DF)
0	0	7312	2198	100
24	24	7377	2181	98
39	39	7368	2172	98
51	51	7364	2169	97
69	69	7362	2168	97
81	81	7358	2168	97
95	95	7354	2165	97
107	107	7352	2165	97
134	134	7357	2165	97
148	148	7354	2165	97
175	175	7349	2176	98
189	189	7348	2167	97
201	201	7346	2170	97
227	227	7347	2175	98
252	252	7343	2197	100
270	270	7345	2191	99
289	289	7345	2189	99
314	314	7343	2185	99
338	338	7341	2178	98

Table 4.6e: Elastic Modulus and Durability Factors for Mix 5

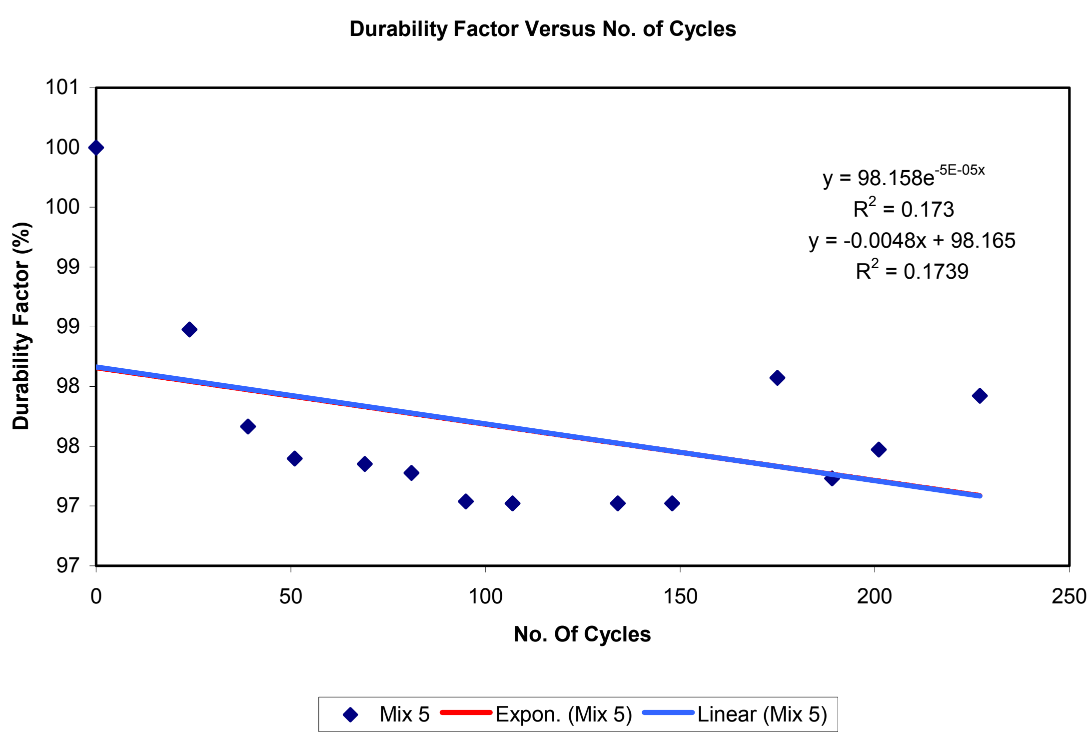

Figure 4.3e: Graph of durability vs No of cycles for “mix 5”

4.5.0 Summary of Freeze and Thaw Tests

	Linear regression		Exponential regression	
	Equation	R2 value	Equation	R2 value
Mix 1	y=-0.1003X + 95.6	0.8963	Y=96.411e-0.0013	0.9009
Mix2	y=-0.0056X + 96.379	0.2161	Y=96.365e-6E-05X	0.2175
Mix3	y=-0.0995X + 96.116	0.5026	Y=95.964e-0.0012X	0.4492
Mix4	y=-0.0098X + 97.57	0.1573	Y=97.588e-0.0001X	0.1562
Mix 5	y=-0.0048X + 98.165	0.1739	Y=98.158e-5E-05X	0.1730

Table 4.7: Linear and exponential regression equations for freeze and thaw data.

	Linear regression	
	Equation	Durability factor at 300th cycle
Mix 1	y=-0.1003X + 95.6	65.51
Mix2	y=-0.0056X + 96.379	94.699
Mix3	y=-0.0995X + 96.116	66.266
Mix4	y=-0.0098X + 97.57	94.63
Mix 5	y=-0.0048X + 98.165	96.725

Table 4.8: Predicted 300th cycle durability factors.

For simplicity, it was decided to use the linear regression equation in predicting the durability factor at the 300th cycle because both trends were almost identical. Notably from Table 4.8, none of the mixes fell below 60% durability factor. However, the 3 mixes with Type I cement and lowest water-cement ratio fared better in this durability test.

In a research by Powers et al. he concluded that entrained air voids act as empty chambers in the paste for the freezing and migrating water to enter, thus relieving the pressures described above and preventing damage to the concrete. Upon thawing, most of the water returns to the capillaries due to capillary action and pressure from air compressed in the bubbles. Thus the bubbles are ready to protect the concrete from the next cycle of freezing and thawing.

The three mixes that fared best among the lot were mixes that may have likely more air pockets in them due to inadequate consolidation during placing.

CHAPTER 5-CONCLUSIONS, OBSERVATIONS AND RECOMMENDATIONS

5.0.0 Conclusions and Observations

The primary conclusion expected from this research was to determine if all the mixes researched into, fell into the category of High Performance concrete and thus was either Very early strength (VES), High early strength (HES) or not an Early strength mix. It was finally expected to recommend which two mixes based on the strength and durability requirements of High Performance concrete were the best.

Based on the results of this investigation, the following conclusions can be drawn;

5.1.0 Strength Criterion: Compressive strength

1 High Performance concrete can be produced with a variety of mix options including the use of;

(a) Type III Portland cement and

(b) Type I or Type III Portland cement with a low water-cement ratios by using superplasticizers to achieve moderate to high consistencies.

2 Although the water-cement ratio plays an important role in attaining early strength, for concrete to be poured and consolidated, it has to workable. The consistency of an early strength mix should not be compromised in an attempt to acquire its strength. It was concluded in this research that “mix 4” and “mix 5” attained low early strengths due to inadequate consolidation.

3 In order to make use of a lower water to cement ratio in acquiring early strength, the right dosage of superplasticizers must be used. A slump of at least 2” must be obtained in order to attain good consolidation in a laboratory setting.

4 The two mixes with type III Portland cement “mix 1” and “mix 3” fell in the Very early Strength (VES) category of High Performance concrete, attaining the required strengths of a minimum of 2,000-2,500 psi (14-17.5 MPa) within four (4) hours. “Mix 2”, “mix 4” and “mix 5” can be considered as High early strength concrete (HES) accordingly, attaining a strength of approximately 2,000 psi (14.0 MPa) within twenty-four (24) hours as shown in Table 4.3.

5 “Mix 1” and “mix 3” which utilizes Type III early strength Portland cement achieved the best results for the strength criterion.

5.2.0 Durability Criterion: Freeze and thaw resistance

From earlier research discussed in the literature review of this paper, it was established that;

- Dry concrete is unaffected by repeated freeze and thaw.
- The development of pore structure inside cement paste is fundamental to freeze–thaw resistance of concrete.
- Capillary porosity of a concrete cement paste becomes a factor in concrete’s resistance to freeze and thaw at water-cement ratios above 0.36. At water cement ratios below this value, the only porosity in the paste is the gel porosity, which is very minute and has no effect on frost action.

- The durability of concrete depends mostly on its resistance to frost action (freeze and thaw) and can be enhanced by modifying the pore structure of the concrete. This modification depends on the water-cement ratio of the mix, the degree of saturation, and air bubbles (entrapped air and entrained air).

MIX	**MIX DESIGN Materials Dry Weight (Cubic yard basis)**				
	1	**2**	**3**	**4**	**5**
Cement Type	**III**	**I**	**III**	**I**	**I**
W/C Ratio	0.410	0.410	0.470	0.320	0.320
Proportion of water content by mass in Paste	0.174	0.149	0.162	0.126	0.117
Proportion of fines by mass in paste	0.826	0.851	0.838	0.874	0.883
Proportion of Air Entrainment by mass in paste	0.0013269	0.0004551	0.0018018	0.0012209	0.00044356
Frost Resistance (Durability Factor)	66	95	66	95	97

Table 5.1: Factors affecting resistance to freeze and thaw

From Table 5.1 above, the following conclusions are made on the resistance of the various mixes to Freeze and thaw;

1 The consistency/workability of the concrete mix should be taken into consideration when attempting to increase the strength and durability of a concrete mix by decreasing its water-cement ratio.

2 The durability factor of a concrete prism exposed to freeze-thaw cycles depicts its durability. The higher this factor, the less susceptible the mix is to freeze and thaw. Drier mixes have a tendency to have higher durability factors. Air entrainment is also a means to attain higher durability factors in a concrete mix.

3 Coarser cement tends to produce pastes with higher porosity than that produced by finer cement (Powers et al 1954). Type III cement is by far finer in nature than

Type I, the fact that there may have been more pore spaces for freezable water to expand in "mix 2" which uses Type I cement may have been the reason for the better durability performance.

4 Cement pore structure develops by the gradual growth of gel into the space originally occupied by the anhydrous cement and mixing water [ERDC/CRREL TR-02-5]. Taking into consideration of the water-cement ratio and the proportion by mass of water in the paste of the various mixes, the capillary porosity of the paste in "mix 2", "mix 4" and "mix 5" is less than that of "mix 1" and "mix 3". Because there is less freezable water in the drier mixes ("mix 2", "mix 4" and "mix 5"), there is little or no impact of the hydraulic pressures during freezing on the internal structure of the paste hence the better results obtained for durability.

5 The ratio by mass of air entrainment in the various mixes may have aided their resistance to frost action, but its effect on "mix 4" and "mix 5" was negligible since there was virtually no expandable freezable water to fill the air voids.

6 All the mixes had samples going through all 300 cycles of freeze and thaw, "Mix 4" and "mix 5" were more durable in this respect (resistance to freeze and thaw). They did not show any signs of deterioration after the freeze and thaw cycle had ended. The other three mixes showed some signs of scaling and some of the samples failed. Some of the failures were considered, however, as abnormalities in the mixing procedures.

7 Because of the variability of water-cement ratio and superplasticizers used, no conclusion could be made as to the optimal dosage of admixtures.

8 Adjustment of the factors that enhance either the strength or durability of the various mixes could be done for “mix 1”, “mix 2” and “mix 3” because there is room for water content adjustment to resist freeze and thaw as well as to increase strength. Since “mix 4” and “mix 5” make use of low water-cement ratio to achieve early strength, adjusting the water content will increase the strength a little but may compromise with its durability.

5.3.0 Recommendations

The results of this research are summarized in Table 5.2.

Mix	**Durability Factor (%)**	**Compressive Strength/ ksi (MPa)**		
		4hrs	**24hrs**	**7days**
1	66	2.592 (17.87)	4.203 (28.98)	5.953 (41.04)
2	95	1.033 (7.122)	2.327 (16.04)	3.732 (25.73)
3	66	2.950 (20.34)	4.566 (31.48)	6.320 (43.57)
4	95	0.795 (5.48)	1.933 (13.33)	3.170 (21.86)
5	97	0.781 (5.38)	1.978 (13.64)	3.279 (22.61)

Table 5.2: Summary of results

The following recommendations are made by taking into consideration observations of the results obtained during preparation, testing and evaluation of results obtained from the tests conducted in the course of this research;

1 Mix 1 and Mix 3 by all indication achieved early strength much quicker than the other mixes, the consistency of these mixes were also good and as such can be placed and formed with ease under all conditions. Their durability factor values exceeded the limits for the Freeze and thaw durability factor criteria (60%) for

failure in 300 cycles set for this research by a small margin, their lower durability characteristics as compared to the other mixes could be improved by adjusting the factors that dictate their resistance to freeze and thaw i.e. water-cement ratio and air entrainment.

2 Mix 2, which makes use of lower water-cement ratio, and Type I cement could also be further studied into since it shows good strength gain after 4 hours and a better freeze-thaw resistance. This mix is also an alternative option of using of Type III cement.

3 Finally, this research recommends the choice in order of the best overall strength and durability performance the use of an **adjusted/modified** "mix 1" and "mix 3" as the best 2 mixes and "mix 2" as a control mix for the Phase II of this research.

Average of Mass and Frequency for 0 cycle

Specimen#	A	B	C	Mass	Frequency				P_c	DF	Av. DF	σ	Avg. Mass	Avg. Frequency
MU1A	0	0	0	7073	2000	+	177.0	2177	100	100	100	0	7093	2149
MU1B	0	0	0	7039	2000	+	139.0	2139	100	100				
MU1C	0	0	0	7127	2000	+	141.5	2142	100	100				
MU1D	0	0	0	7133	2000	+	139.0	2139	100	100				
MU2A	0	0	0	7235	2000	+	103.0	2103	100	100	100	0	7254	2118
MU2B	0	0	0	7303	2000	+	126.5	2127	100	100				
MU2C	0	0	0	7229	2000	+	127.0	2127	100	100				
MU2D	0	0	0	7249	2000	+	117.0	2117	100	100				
MU3A	0	0	0	6966	1800	+	217.0	2017	100	100	100	0	6916	2011
MU3B	0	0	0	6867	1800	+	217.0	2017	100	100				
MU3C	0	0	0	6911	1800	+	211.0	2011	100	100				
MU3D	0	0	0	6921	1800	+	200.0	2000	100	100				
MU4A	0	0	0	7462	2000	+	198.0	2198	100	100	100	0	7384	2196
MU4B	0	0	0	7422	2000	+	185.0	2185	100	100				
MU4C	0	0	0	7336	2000	+	211.0	2211	100	100				
MU4D	0	0	0	7315	2000	+	190.0	2190	100	100				
MU5A	0	0	0	7290	2000	+	225.5	2226	100	100	100	0	7312	2198
MU5B	0	0	0	7359	2000	+	207.5	2208	100	100				
MU5C	0	0	0	7118	2000	+	153.0	2153	100	100				
MU5D	0	0	0	7481	2000	+	204.5	2205	100	100				

Table A-FT-1

Table of Average Mass and Frequency for 24th cycle

Specimen#	A	B	C	Mass	Frequency				P_c	DF	Av. DF	σ	Avg. Mass	Avg. Frequency
MU1A	24	0	24	7072	1900	+	175.0	2075	95	95	97	1	7093	2079
MU1B	24	0	24	7043	1900	+	141.0	2041	95	95				
MU1C	24	0	24	7125	1900	+	195.0	2095	98	98				
MU1D	24	0	24	7130	1900	+	204.0	2104	98	98				
MU2A	24	0	24	7239	1900	+	163.0	2063	98	98	98	0	7254	2075
MU2B	24	0	24	7302	1900	+	179.0	2079	98	98				
MU2C	24	0	24	7228	1900	+	180.5	2081	98	98				
MU2D	24	0	24	7248	1900	+	178.0	2078	98	98				
MU3A	24	0	24	6943	1800	+	192.0	1992	99	99	99	0	6904	1989
MU3B	24	0	24	6853	1800	+	197.0	1997	99	99				
MU3C	24	0	24	6904	1800	+	183.5	1984	99	99				
MU3D	24	0	24	6919	1800	+	181.5	1982	99	99				
MU4A	24	0	24	7450	2000	+	173.5	2174	99	99	99	0	7377	2165
MU4B	24	0	24	7416	2000	+	154.5	2155	99	99				
MU4C	24	0	24	7332	2000	+	179.0	2179	99	99				
MU4D	24	0	24	7309	2000	+	153.5	2154	98	98				
MU5A	24	0	24	7291	2000	+	186.5	2187	98	98	99	0	7377	2181
MU5B	24	0	24	7358	2000	+	173.0	2173	98	98				
MU5C	24	0	24	7128	2000	+	185.0	2185	101	101				
MU5D	24	0	24	7483	2000	+	183.0	2183	99	99				

Table A-FT-2

Table of Average Mass and Frequency for 39th cycle

Specimen#	A	B	C	Mass	Frequency				P_c	DF	Av. DF	σ	Avg. Mass	Avg. Frequency
MU1A	15	24	39	7073	1800	+	253.0	2053	94	94	98	0	7124	2093
MU1B	15	24	39	7049	1800	+	141.5	1942	91	91				
MU1C	15	24	39	7123	1800	+	285.5	2086	97	97				
MU1D	15	24	39	7125	1800	+	299.5	2100	98	98				
MU2A	15	24	39	7228	1800	+	259.0	2059	98	98	98	0	7247	2073
MU2B	15	24	39	7298	1800	+	269.0	2069	97	97				
MU2C	15	24	39	7220	1800	+	288.5	2089	98	98				
MU2D	15	24	39	7243	1800	+	274.5	2075	98	98				
MU3A	15	24	39	6913	1800	+	181.5	1982	98	98	99	0	6899	1985
MU3B	15	24	39	6933	1800	+	193.5	1994	99	99				
MU3C	15	24	39	6846	1800	+	178.5	1979	98	98				
MU3D	15	24	39	6903	1800	+	185.0	1985	99	99				
MU4A	15	24	39	7450	1900	+	282.0	2182	99	99	99	0	7374	2170
MU4B	15	24	39	7413	1900	+	257.0	2157	99	99				
MU4C	15	24	39	7327	1900	+	283.0	2183	99	99				
MU4D	15	24	39	7306	1900	+	258.5	2159	99	99				
MU5A	15	24	39	7277	1900	+	293.5	2194	99	99	98	0	7368	2172
MU5B	15	24	39	7352	1900	+	270.0	2170	98	98				
MU5C	15	24	39	FAILED										
MU5D	15	24	39	7475	1900	+	252.0	2152	98	98				

Table A-FT-3

Table of Average Mass and Frequency for 51st cycle

Specimen#	A	B	C	Mass	Frequency				P_c	DF	Av. DF	σ	Avg. Mass	Avg. Frequency
MU1A	12	39	51	7074	1800	+	223.5	2024	93	93	97	0	7121	2071
MU1B	12	39	51	7050	1800	+	58.0	1858	87	87				
MU1C	12	39	51	7120	1800	+	269.0	2069	97	97				
MU1D	12	39	51	7121	1800	+	273.5	2074	97	97				
MU2A	12	39	51	7223	1800	+	250.0	2050	97	97	98	0	7242	2071
MU2B	12	39	51	7295	1800	+	271.0	2071	97	97				
MU2C	12	39	51	7215	1800	+	287.0	2087	98	98				
MU2D	12	39	51	7235	1800	+	274.0	2074	98	98				
MU3A	12	39	51	6929	1800	+	162.0	1962	97	97	98	0	6893	1967
MU3B	12	39	51	6839	1800	+	177.5	1978	98	98				
MU3C	12	39	51	6898	1800	+	161.5	1962	98	98				
MU3D	12	39	51	6907	1800	+	165.0	1965	98	98				
MU4A	12	39	51	7448	1900	+	270.5	2171	99	99	99	0	7371	2164
MU4B	12	39	51	7410	1900	+	255.0	2155	99	99				
MU4C	12	39	51	7325	1900	+	283.0	2183	99	99				
MU4D	12	39	51	7303	1900	+	247.5	2148	98	98				
MU5A	12	39	51	7276	1900	+	284.5	2185	98	98	98	0	7364	2169
MU5B	12	39	51	7352	1900	+	271.0	2171	98	98				
MU5C	12	39	51	FAILED										
MU5D	12	39	51	7466	1900	+	251.0	2151	98	98				

Table A-FT-4

Table of Average Mass and Frequency for 69th cycle

Specimen#	A	B	C	Mass	Frequency				P_c	DF	Av. DF	σ	Avg. Mass	Avg. Frequency
MU1A	18	51	69	7069	1800	+	207.0	2007	92	92	95	2	7118	2035
MU1B	18	51	69	7030	1700	+	116.5	1817	85	85				
MU1C	18	51	69	7123	1700	+	299.5	2000	93	93				
MU1D	18	51	69	7113	1700	+	369.5	2070	97	97				
MU2A	18	51	69	7196	1700	+	354.0	2054	98	98	98	0	7226	2074
MU2B	18	51	69	7291	1700	+	377.0	2077	98	98				
MU2C	18	51	69	7194	1700	+	389.0	2089	98	98				
MU2D	18	51	69	7224	1700	+	375.5	2076	98	98				
MU3A	18	51	69	6922	1700	+	259.5	1960	97	97	97	0	6888	1955
MU3B	18	51	69	6836	1700	+	270.0	1970	98	98				
MU3C	18	51	69	6894	1700	+	249.0	1949	97	97				
MU3D	18	51	69	6902	1700	+	243.0	1943	97	97				
MU4A	18	51	69	7447	1900	+	273.5	2174	99	99	98	1	7371	2157
MU4B	18	51	69	7411	1900	+	253.0	2153	99	99				
MU4C	18	51	69	7324	1900	+	270.0	2170	98	98				
MU4D	18	51	69	7302	1900	+	233.0	2133	97	97				
MU5A	18	51	69	7274	1900	+	284.0	2184	98	98	98	0	7362	2168
MU5B	18	51	69	7350	1900	+	271.0	2171	98	98				
MU5C	18	51	69	FAILED										
MU5D	18	51	69	7463	1900	+	250.0	2150	98	98				

Table A-FT-5

Table of Average Mass and Frequency for 81st cycle

Specimen#	A	B	C	Mass	Frequency				P_c	DF	Av. DF	(	Avg. Mass	Avg. Frequency
MU1A	12	69	81	7060	1700	+	288.0	1988	91	91	93	3	7110	1996
MU1B	12	69	81	7021	1600	+	175.0	1775	83	83				
MU1C	12	69	81	7117	1700	+	222.5	1923	90	90				
MU1D	12	69	81	7103	1800	+	269.5	2070	97	97				
MU2A	12	69	81	7173	1800	+	250.5	2051	98	98	98	0	7211	2073
MU2B	12	69	81	7283	1800	+	271.5	2072	97	97				
MU2C	12	69	81	7178	1800	+	294.0	2094	98	98				
MU2D	12	69	81	7211	1800	+	276.0	2076	98	98				
MU3A	12	69	81	6918	1700	+	229.5	1930	96	96	96	1	6877	1939
MU3B	12	69	81	6814	1700	+	272.0	1972	98	98				
MU3C	12	69	81	6885	1700	+	234.5	1935	96	96				
MU3D	12	69	81	6891	1700	+	219.5	1920	96	96				
MU4A	12	69	81	7443	1900	+	270.5	2171	99	99	98	1	7368	2153
MU4B	12	69	81	7407	1900	+	253.5	2154	99	99				
MU4C	12	69	81	7321	1900	+	260.5	2161	98	98				
MU4D	12	69	81	7300	1900	+	227.0	2127	97	97				
MU5A	12	69	81	7269	1900	+	283.5	2184	98	98	98	0	7358	2168
MU5B	12	69	81	7345	1900	+	270.0	2170	98	98				
MU5C	12	69	81	FAILED										
MU5D	12	69	81	7459	1900	+	249.0	2149	97	97				

Table A-FT-6

Table of Average Mass and Frequency for 95th cycle

Specimen#	A	B	C	Mass	Frequency				P_c	DF	Av. DF	σ	Avg. Mass	Avg. Frequency
MU1A	14	81	95	7048	1800	+	145.5	1946	89	89	91	4	7099	1956
MU1B	14	81	95	6996	1500	+	194.5	1695	79	79				
MU1C	14	81	95	7108	1700	+	172.5	1873	87	87				
MU1D	14	81	95	7090	1900	+	140.0	2040	95	95				
MU2A	14	81	95	7090	1900	+	140.5	2041	97	97	97	0	7182	2063
MU2B	14	81	95	7275	1800	+	262.5	2063	97	97				
MU2C	14	81	95	7163	1900	+	178.5	2079	98	98				
MU2D	14	81	95	7201	1900	+	168.5	2069	98	98				
MU3A	14	81	95	6912	1700	+	202.5	1903	94	94	96	1	6869	1921
MU3B	14	81	95	6806	1700	+	259.5	1960	97	97				
MU3C	14	81	95	6878	1700	+	211.5	1912	95	95				
MU3D	14	81	95	6881	1700	+	210.0	1910	96	96				
MU4A	14	81	95	7445	1900	+	264.5	2165	98	98	98	1	7367	2152
MU4B	14	81	95	7405	1900	+	256.5	2157	99	99				
MU4C	14	81	95	7320	1900	+	265.0	2165	98	98				
MU4D	14	81	95	7299	1900	+	222.0	2122	97	97				
MU5A	14	81	95	7266	1900	+	283.0	2183	98	98	98	1	7354	2165
MU5B	14	81	95	7343	1900	+	270.0	2170	98	98				
MU5C	14	81	95	FAILED										
MU5D	14	69	83	7453	1900	+	241.5	2142	97	97				

Table A-FT-7

Table of Average Mass and Frequency for 107th cycle

Specimen#	A	B	C	Mass	Frequency				P_c	DF	Av. DF	σ	Avg. Mass	Avg. Frequency
MU1A	12	95	107	7039	1700	+	199.0	1899	87	87	92	5	7093	1967
MU1B	12	95	107	6990	1500	+	160.5	1661	78	78				
MU1C	12	95	107	7100	1700	+	172.0	1872	87	87				
MU1D	12	95	107	7086	1900	+	162.5	2063	96	96				
MU2A	12	95	107	7157	1900	+	152.0	2052	98	98	98	0	7194	2076
MU2B	12	95	107	7270	1900	+	181.5	2082	98	98				
MU2C	12	95	107	7154	1900	+	190.0	2090	98	98				
MU2D	12	95	107	7194	1900	+	180.0	2080	98	98				
MU3A	12	95	107	6911	1700	+	198.0	1898	94	94	95	1	6865	1916
MU3B	12	95	107	6799	1700	+	270.0	1970	98	98				
MU3C	12	95	107	6873	1700	+	200.0	1900	94	94				
MU3D	12	95	107	6876	1700	+	197.5	1898	95	95				
MU4A	12	95	107	7446	1900	+	272.5	2173	99	99	98	1	7368	2161
MU4B	12	95	107	7406	1900	+	260.0	2160	99	99				
MU4C	12	95	107	7320	1900	+	276.5	2177	98	98				
MU4D	12	95	107	7300	1900	+	235.0	2135	97	97				
MU5A	12	95	107	7265	1900	+	283.0	2183	98	98	98	0	7352	2165
MU5B	12	95	107	7337	1900	+	270.0	2170	98	98				
MU5C	12	95	107	FAILED										
MU5D	12	95	107	7454	1900	+	241.0	2141	97	97				

Table A-FT-8

Table of Average Mass and Frequency for 134th cycle

Specimen#	A	B	C	Mass	Frequency				P_c	DF	Av. DF	σ	Avg. Mass	Avg. Frequency
MU1A	27	107	134	7003	1600	+	177.5	1778	82	82	91	4	7018	1947
MU1B	27	107	134	6946	1500	+	65.5	1566	73	73				
MU1C	27	107	134	7067	1600	+	262.0	1862	87	87				
MU1D	27	107	134	6969	1700	+	331.0	2031	95	95				
MU2A	27	107	134	7142	1700	+	330.0	2030	97	97	98	1	7179	2068
MU2B	27	107	134	7256	1800	+	274.0	2074	98	98				
MU2C	27	107	134	7138	1800	+	290.5	2091	98	98				
MU2D	27	107	134	7180	1800	+	277.5	2078	98	98				
MU3A	27	107	134	6899	1800	+	82.0	1882	93	93	93	1	6848	1873
MU3B	27	107	134	6782	1800	+	116.5	1917	95	95				
MU3C	27	107	134	6853	1700	+	129.0	1829	91	91				
MU3D	27	107	134	6858	1700	+	163.0	1863	93	93				
MU4A	27	107	134	7452	1800	+	367.0	2167	99	99	98	1	7373	2146
MU4B	27	107	134	7407	1800	+	356.5	2157	99	99				
MU4C	27	107	134	7325	1900	+	260.5	2161	98	98				
MU4D	27	107	134	7308	1900	+	198.0	2098	96	96				
MU5A	27	107	134	7268	1900	+	283.0	2183	98	98	98	1	7357	2165
MU5B	27	107	134	7342	1900	+	270.0	2170	98	98				
MU5C	27	107	134	FAILED										
MU5D	27	107	134	7460	1900	+	241.0	2141	97	97				

Table A-FT-9

Table of Average Mass and Frequency for 148th cycle

Specimen#	A	B	C	Mass	Frequency				P_c	DF	Av. DF	σ	Avg. Mass	Avg. Frequency
MU1A	14	134	148	6990	1600	+	142.0	1742	80	80	89	5	7009	1912
MU1B	14	134	148	6916	1400	+	79.5	1480	69	69				
MU1C	14	134	148	7050	1600	+	210.5	1811	85	85				
MU1D	14	134	148	6969	1700	+	313.0	2013	94	94				
MU2A	14	134	148	7132	1700	+	339.5	2040	97	97	98	1	7166	2069
MU2B	14	134	148	7247	1700	+	375.0	2075	98	98				
MU2C	14	134	148	7120	1800	+	279.0	2079	98	98				
MU2D	14	134	148	7165	1800	+	283.0	2083	98	98				
MU3A	14	134	148	6886	1800	+	55.0	1855	92	92	91	2	6838	1836
MU3B	14	134	148	6770	1700	+	184.5	1885	93	93				
MU3C	14	134	148	6846	1500	+	283.0	1783	89	89				
MU3D	14	134	148	6850	1700	+	122.5	1823	91	91				
MU4A	14	134	148	7448	1800	+	371.0	2171	99	99	98	1	7371	2146
MU4B	14	134	148	7404	1900	+	257.5	2158	99	99				
MU4C	14	134	148	7323	1900	+	258.5	2159	98	98				
MU4D	14	134	148	7309	1900	+	198.0	2098	96	96				
MU5A	14	134	148	7265	1900	+	283.0	2183	98	98	98	1	7354	2165
MU5B	14	134	148	7339	1900	+	270.0	2170	98	98				
MU5C	14	134	148	FAILED										
MU5D	14	134	148	7459	1900	+	241.0	2141	97	97				

Table A-FT-10

Table of Average Mass and Frequency for 175th cycle

Specimen#	A	B	C	Mass	Frequency				P_c	DF	Av. DF	σ	Avg. Mass	Avg. Frequency
MU1A	27	148	175	6966	1400	+	237.5	1638	75	75	88	5	7032	1875
MU1B	27	148	175	6867	1200	+	210.5	1411	66	66				
MU1C	27	148	175	7031	1400	+	366.5	1767	82	82				
MU1D	27	148	175	7033	1700	+	283.0	1983	93	93				
MU2A	27	148	175	7124	1700	+	332.5	2033	97	97	98	1	7150	2071
MU2B	27	148	175	7232	1800	+	278.5	2079	98	98				
MU2C	27	148	175	7106	1800	+	287.0	2087	98	98				
MU2D	27	148	175	7139	1800	+	285.0	2085	98	98				
MU3A	27	148	175	6865	1700	+	135.5	1836	91	91	91	2	6814	1829
MU3B	27	148	175	6751	1700	+	182.0	1882	93	93				
MU3C	27	148	175	6813	1700	+	83.0	1783	89	89				
MU3D	27	148	175	6828	1700	+	114.0	1814	91	91				
MU4A	27	148	175	7447	1800	+	365.5	2166	99	99	97	2	7391	2157
MU4B	27	148	175	7406	1800	+	363.0	2163	99	99				
MU4C	27	148	175	7321	1800	+	342.5	2143	97	97				
MU4D	27	148	175	7284	1800	+	259.0	2059	94	94				
MU5A	27	148	175	7261	1900	+	300.0	2200	99	99	98	0	7349	2176
MU5B	27	148	175	7335	1900	+	269.0	2169	98	98				
MU5C	27	148	175	FAILED										
MU5D	27	148	175	7450	1900	+	260.0	2160	98	98				

Table A-FT-11

Table of Average Mass and Frequency for 189th cycle

Specimen#	A	B	C	Mass	Frequency				P_c	DF	Av. DF	σ	Avg. Mass	Avg. Frequency
MU1A	14	175	189	6952	1500	+	54.5	1555	71	71	87	4	7014	1852
MU1B	14	175	189	FAILED										
MU1C	14	175	189	7014	1500	+	260.0	1760	82	82				
MU1D	14	175	189	7015	1600	+	344.0	1944	91	91				
MU2A	14	175	189	7114	1700	+	322.0	2022	96	96	97	1	7139	2061
MU2B	14	175	189	7218	1700	+	360.5	2061	97	97				
MU2C	14	175	189	7099	1700	+	379.0	2079	98	98				
MU2D	14	175	189	7126	1700	+	382.5	2083	98	98				
MU3A	14	175	189	6858	1700	+	113.5	1814	90	90	89	2	6805	1788
MU3B	14	175	189	6739	1700	+	132.0	1832	91	91				
MU3C	14	175	189	6802	1700	+	38.0	1738	86	86				
MU3D	14	175	189	6821	1700	+	69.0	1769	88	88				
MU4A	14	175	189	7442	1800	+	341.5	2142	97	97	97	1	7388	2136
MU4B	14	175	189	7403	1800	+	342.5	2143	98	98				
MU4C	14	175	189	7320	1800	+	323.5	2124	96	96				
MU4D	14	175	189	7279	1800	+	236.5	2037	93	93				
MU5A	14	175	189	7258	1900	+	290.0	2190	98	98	98	1	7348	2167
MU5B	14	175	189	7335	1900	+	271.0	2171	98	98				
MU5C	14	175	189	FAILED										
MU5D	14	175	189	7449	1900	+	240.0	2140	97	97				

Table A-FT-12

Table of Average Mass and Frequency for 201st cycle

Specimen#	A	B	C	Mass	Frequency				P_c	DF	Av. DF	σ	Avg. Mass	Avg. Frequency
MU1A	12	189	201	6944	1400	+	107.0	1507	69	69	82	3	6999	1764
MU1B	12	189	201	FAILED										
MU1C	12	189	201	6998	1500	+	199.5	1700	79	79				
MU1D	12	189	201	7000	1600	+	229.0	1829	86	86				
MU2A	12	189	201	7106	1800	+	208.0	2008	95	95	97	1	7134	2052
MU2B	12	189	201	7216	1800	+	263.0	2063	97	97				
MU2C	12	189	201	7096	1800	+	256.5	2057	97	97				
MU2D	12	189	201	7117	1800	+	279.5	2080	98	98				
MU3A	12	189	201	6851	1500	+	276.0	1776	88	88	86	1	6798	1733
MU3B	12	189	201	6730	1500	+	253.0	1753	87	87				
MU3C	12	189	201	6793	1500	+	201.5	1702	85	85				
MU3D	12	189	201	6817	1500	+	200.5	1701	85	85				
MU4A	12	189	201	7443	1900	+	254.5	2155	98	98	97	1	7390	2141
MU4B	12	189	201	7403	1900	+	254.5	2155	99	99				
MU4C	12	189	201	7324	1900	+	214.5	2115	96	96				
MU4D	12	189	201	7277	1900	+	122.0	2022	92	92				
MU5A	12	189	201	7256	1900	+	295.0	2195	99	99	98	1	7346	2170
MU5B	12	189	201	7330	1900	+	273.5	2174	98	98				
MU5C	12	189	201	FAILED										

Table A-FT-13

Table of Average Mass and Frequency for 227th cycle

Specimen#	A	B	C	Mass	Frequency				P_c	DF	Av. DF	σ	Avg. Mass	Avg. Frequency
MU1A	26	201	227	6939	1400	+	46.0	1446	66	66	85	5	6982	1819
MU1B	26	201	227	FAILED										
MU1C	26	201	227	6983	1400	+	322.5	1723	80	80				
MU1D	26	201	227	6981	1600	+	314.5	1915	90	90				
MU2A	26	201	227	7096	1700	+	335.0	2035	97	97	98	1	7126	2071
MU2B	26	201	227	7204	1700	+	382.5	2083	98	98				
MU2C	26	201	227	7089	1700	+	370.5	2071	97	97				
MU2D	26	201	227	7117	1700	+	394.0	2094	99	99				
MU3A	26	201	227	6842	1600	+	164.0	1764	87	87	84	2	6791	1689
MU3B	26	201	227	6723	1600	+	39.5	1640	81	81				
MU3C	26	201	227	6785	1600	+	92.5	1693	84	84				
MU3D	26	201	227	6816	1600	+	58.5	1659	83	83				
MU4A	26	201	227	7443	1800	+	369.0	2169	99	99	98	1	7392	2152
MU4B	26	201	227	7403	1800	+	359.0	2159	99	99				
MU4C	26	201	227	7331	1800	+	327.0	2127	96	96				
MU4D	26	201	227	7281	1800	+	223.5	2024	92	92				
MU5A	26	201	227	7259	1900	+	303.5	2204	99	99	98	1	7347	2175
MU5B	26	201	227	7331	1900	+	270.5	2171	98	98				
MU5C	26	201	227	FAILED										
MU5D	26	201	227	7450	1900	+	250.0	2150	98	98				

Table A-FT-13

Table of Average Mass and Frequency for 252nd cycle

Specimen#	A	B	C	Mass	Frequency				P_c	DF	Av. DF	σ	Avg. Mass	Avg. Frequency
MU1A	25	227	252	6900	1200	+	201.0	1401	64	64	83	2	6952	1769
MU1B	25	227	252	FAILED										
MU1C	25	227	252	6951	1400	+	325.5	1726	81	81				
MU1D	25	227	252	6954	1500	+	312.0	1812	85	85				
MU2A	25	227	252	7073	1700	+	330.5	2031	97	97	98	1	7110	2073
MU2B	25	227	252	7191	1800	+	293.5	2094	98	98				
MU2C	25	227	252	7075	1800	+	284.5	2085	98	98				
MU2D	25	227	252	7102	1800	+	285.0	2085	98	98				
MU3A	25	227	252	6817	1600	+	140.5	1741	86	86	81	4	6763	1633
MU3B	25	227	252	6696	1500	+	87.0	1587	79	79				
MU3C	25	227	252	6753	1500	+	168.0	1668	83	83				
MU3D	25	227	252	6785	1400	+	138.0	1538	77	77				
MU4A	25	227	252	7437	2000	+	181.0	2181	99	99	98	2	7387	2155
MU4B	25	227	252	7399	2000	+	176.0	2176	100	100				
MU4C	25	227	252	7326	2000	+	107.5	2108	95	95				
MU4D	25	227	252	7275	1900	+	137.5	2038	93	93				
MU5A	25	227	252	7256	1900	+	315.5	2216	100	100	99	0	7343	2197
MU5B	25	227	252	7328	1900	+	299.0	2199	100	100				
MU5C	25	227	252	FAILED										

Table A-FT-14

Table of Average Mass and Frequency for 270th cycle

Specimen#	A	B	C	Mass	Frequency				P_c	DF	Av. DF	σ	Avg. Mass	Avg. Frequency
MU1A	18	252	270	FAILED										
MU1B	18	252	270	FAILED										
MU1C	18	252	270	6921	1500	+	131.0	1631	76	76	82	6	6930	1752
MU1D	18	252	270	6940	1600	+	272.0	1872	88	88				
MU2A	18	252	270	7050	1800	+	200.0	2000	95	95	97	1	7095	2057
MU2B	18	252	270	7178	1800	+	267.5	2068	97	97				
MU2C	18	252	270	7064	1900	+	180.5	2081	98	98				
MU2D	18	252	270	7090	1900	+	180.0	2080	98	98				
MU3A	18	252	270	6800	1400	+	259.5	1660	82	82	79	3	6739	1593
MU3B	18	252	270	6677	1300	+	196.5	1497	74	74				
MU3C	18	252	270	6740	1400	+	223.0	1623	81	81				
MU3D	18	252	270	FAILED										
MU4A	18	252	270	7437	1900	+	271.0	2171	99	99	93	0	7329	2055
MU4B	18	252	270	7399	1900	+	264.5	2165	99	99				
MU4C	18	252	270	7329	1900	+	154.5	2055	93	93				
MU4D	18	252	270	7274	1900	+	122.5	2023	92	92				
MU5A	18	252	270	7259	1900	+	310.5	2211	99	99	99	1	7345	2191
MU5B	18	252	270	7329	1900	+	296.5	2197	100	100				
MU5C	18	252	270	FAILED										
MU5D	18	252	270	7448	1900	+	264.5	2165	98	98				

Table A-FT-15

Table of Average Mass and Frequency for 289th cycle

Specimen#	A	B	C	Mass	Frequency				P_c	DF	Av. DF	σ	Avg. Mass	Avg. Frequency
MU1A	19	270	289	FAILED										
MU1B	19	270	289	FAILED										
MU1C	19	270	289	FAILED										
MU1D	19	270	289	6926	1600	+	243.0	1843	86	86	86	0	6926	1843
MU2A	19	270	289	7039	1800	+	213.5	2014	96	96	97	1	7087	2060
MU2B	19	270	289	7169	1800	+	274.0	2074	98	98				
MU2C	19	270	289	7055	1800	+	271.0	2071	97	97				
MU2D	19	270	289	7083	1800	+	279.5	2080	98	98				
MU3A	19	270	289	6790	1400	+	233.5	1634	81	81	81	0	6758	1628
MU3B	19	270	289	FAILED										
MU3C	19	270	289	6726	1400	+	223.0	1623	81	81				
MU3D	19	270	289	FAILED										
MU4A	19	270	289	7438	1900	+	280.5	2181	99	99	99	0	7419	2175
MU4B	19	270	289	7399	1900	+	269.5	2170	99	99				
MU4C	19	270	289	7330	1900	+	125.0	2025	92	92				
MU4D	19	270	289	7274	1900	+	105.5	2006	92	92				
MU5A	19	270	289	7261	1900	+	305.0	2205	99	99	96	1	7345	2189
MU5B	19	270	289	7329	1900	+	295.5	2196	99	99				
MU5C	19	270	289	FAILED										
MU5D	19	270	289	7444	1900	+	266.0	2166	98	98				

Table A-FT-16

Table of Average Mass and Frequency for 314th cycle

Specimen#	A	B	C	Mass	Frequency				P_c	DF	Av. DF	σ	Avg. Mass	Avg. Frequency
MU1A	25	289	314	FAILED										
MU1B	25	289	314	FAILED										
MU1C	25	289	314	FAILED										
MU1D	25	289	314	6902	1500	+	299.5	1800	84	84	84	0	6902	1800
MU2A	25	289	314	7018	1800	+	201.5	2002	95	95	97	1	7089	2068
MU2B	25	289	314	7156	1800	+	264.0	2064	97	97				
MU2C	25	289	314	7040	1800	+	272.5	2073	97	97				
MU2D	25	289	314	7069	1800	+	266.0	2066	98	98				
MU3A	25	289	314	6774	1400	+	190.0	1590	79	79	79	0	6743	1596
MU3B	25	289	314	FAILED										
MU3C	25	289	314	6711	1400	+	201.5	1602	80	80				
MU3D	25	289	314	FAILED										
MU4A	25	289	314	7439	1900	+	279.0	2179	99	99	99	0	7419	2173
MU4B	25	289	314	7398	1900	+	267.0	2167	99	99				
MU4C	25	289	314	7325	1800	+	169.5	1970	89	89				
MU4D	25	289	314	7275	1800	+	191.0	1991	91	91				
MU5A	25	289	314	7259	1900	+	302.5	2203	99	99	99	1	7343	2185
MU5B	25	289	314	7327	1900	+	292.5	2193	99	99				
MU5C	25	289	314	FAILED										
MU5D	25	289	314	7441	1900	+	260.5	2161	98	98				

Table A-FT-17

Table of Average Mass and Frequency for 338th cycle

Specimen#	A	B	C	Mass	Frequency				P_c	DF	Av. DF	σ	Avg. Mass	Avg. Frequency
MU1A	24	314	338	FAILED										
MU1B	24	314	338	FAILED										
MU1C	24	314	338	FAILED										
MU1D	24	314	338	6686	1400	+	307.5	1708	80	80	80	0	6686	1708
MU2A	24	314	338	7005	1700	+	277.5	1978	94	94	97	0	7075	2061
MU2B	24	314	338	7146	1900	+	150.5	2051	96	96				
MU2C	24	314	338	7030	1900	+	174.0	2074	98	98				
MU2D	24	314	338	7050	1900	+	157.5	2058	97	97				
MU3A	24	314	338	6756	1300	+	210.0	1510	75	75	75	0	6725	1515
MU3B	24	314	338	FAILED										
MU3C	24	314	338	6695	1300	+	219.0	1519	76	76				
MU3D	24	314	338	FAILED										
MU4A	24	314	338	7435	1900	+	265.5	2166	99	99	99	0	7415	2164
MU4B	24	314	338	7395	1900	+	262.5	2163	99	99				
MU4C	24	314	338	7321	1700	+	266.5	1967	89	89				
MU4D	24	314	338	7266	1700	+	266.0	1966	90	90				
MU5A	24	314	338	7258	1900	+	287.5	2188	98	98	98	0	7341	2178
MU5B	24	314	338	7324	1900	+	287.0	2187	99	99				
MU5C	24	314	338	FAILED										

Table A-FT-18

Compressive strength for 4hrs, 24hrs and 7days for mix 1

Mix 1				
Cement, Ib	870.000			
Coarse Aggregate, Ib	1726.000			
Fine Aggregate, Ib	820.000			
Water, Ib	356.300			
Accelerator, (PolarSet), gal.	6.000			
HRWR (ADVA Flow), oz.	43.500			
Darex II AEA, oz.	43.500			
W/C Ratio	0.41			
4 hourTest Results				
Specimen	**Test Time**	**Weight**	**Load / lb**	**Comp.Strength / psi**
MC1A	4 hrs	28.3	65500	2316.3
MC1B	4 hrs	28.3	64000	2263.2
MC1C	4 hrs	28.1	64000	2263.2
MC1D	4 hrs	28	65000	2298.6
24 hour Test Result				
Specimen	**Test Time**	**Weight**	**Load / lb**	**Comp.Strength / psi**
MC1E	24 hrs	28.2	142000	5021.6
MC1F	24 hrs	28	130000	4597.2
MC1G	24 hrs	28.1	142000	5021.6
MC1H	24 hrs	28.2	128000	4526.5
7 Day Test Result				
Specimen	**Test Time**	**Weight**	**Load / lb**	**Comp.Strength / psi**
MC1K	7days	28.4	160000	5658.1
MC1L	7days	28.4	159500	5640.4
MC1M	7days	28.1	158000	5587.4
MC1N	7days	28.2	164000	5799.6

Table C-S1

Compressive strength for 4hrs, 24hrs and 7days for mix 2

Mix 2				
Cement, Ib	752.000			
Coarse Aggregate, Ib	1781.000			
Fine Aggregate, Ib	1001.000			
Water, Ib	306.100			
Accelerator, (PolarSet), gal.	3.500			
HRWR (ADVA Flow), oz.	37.600			
Darex II AEA, oz.	15.000			
W/C Ratio	0.41			
4 hourTest Results				
Specimen	**Test Time**	**Weight**	**Load / lb**	**Comp.Strength / psi**
MC2A	4 hrs	28.5	24000	848.7163166
MC2B	4 hrs	28.4	25000	884.0794964
MC2C	4 hrs	28.4	24000	848.7163166
MC2D	4 hrs	28.5	23000	813.3531367
24 hour Test Result				
Specimen	**Test Time**	**Weight**	**Load / lb**	**Comp.Strength / psi**
MC2E	24 hrs	28.2	98000	3465.591626
MC2F	24 hrs	28	99000	3500.954806
MC2G	24 hrs	28	100500	3553.999576
MC2H	24 hrs	28.2	98000	3465.591626
7 Days Test Result				
Specimen	**Test Time**	**Weight**	**Load / lb**	**Comp.Strength / psi**
MC2K	7 days	28.2	102000	3607.044345
MC2L	7 days	28	103000	3642.407525
MC2M	7 days	28	102000	3607.044345
MC2N	7 days	28.2	98500	3483.273216

Table C-S2

Compressive strength for 4hrs, 24hrs and 7days for mix 3

Mix 3				
Cement, Ib	915.000			
Coarse Aggregate, Ib	1124.000			
Fine Aggregate, Ib	1218.000			
Water, Ib	412.000			
Accelerator, (PolarSet), gal.	6.000			
HRWR (ADVA Flow), oz.	45.800			
Darex II AEA, oz.	73.200			
W/C Ratio	0.45			
4 hourTest Results				
Specimen	**Test Time**	**Weight**	**Load / lb**	**Comp.Strength / psi**
MC3A	4 hrs	27.5	78000	2758.3
MC3B	4 hrs	27.6	79000	2793.7
MC3C	4 hrs	27.5	76000	2687.6
MC3D	4 hrs	27.5	77500	2740.6
24 hour Test Result				
Specimen	**Test Time**	**Weight**	**Load / lb**	**Comp.Strength / psi**
MC3E	24 hrs	27.8	140000	4950.8
MC3F	24 hrs	27.6	140000	4950.8
MC3G	24 hrs	27.5	139000	4915.5
MC3H	24 hrs	27.5	142000	5021.6
7 Day Test Result				
Specimen	**Test Time**	**Weight**	**Load / lb**	**Comp.Strength / psi**
MC3K	7days	27.8	168000	5941.0
MC3L	7days	27.7	175000	6188.6
MC3M	7days	27.6	174000	6153.2

Table C-S3

Compressive strength for 4hrs, 24hrs and 7days for mix 4

Mix 4				
Cement, Ib	900.000			
Coarse Aggregate, Ib	1590.000			
Fine Aggregate, Ib	1110.000			
Water, Ib	290.800			
Accelerator, (PolarSet), gal.	6.000			
HRWR (ADVA Flow), oz.	45.000			
Darex II AEA, oz.	45.000			
W/C Ratio	0.32			
4 hourTest Results				
Specimen	**Test Time**	**Weight**	**Load / lb**	**Comp.Strength / psi**
MC1A	4 hrs	27.5	25500	901.8
MC1B	4 hrs	27.6	22500	795.7
MC1C	4 hrs	27.5	FAILED	FAILED
MC1D	4 hrs	27.5	23000	813.4
24 hour Test Result				
Specimen	**Test Time**	**Weight**	**Load / lb**	**Comp.Strength / psi**
MC1E	24 hrs	27.8	49000	1732.8
MC1F	24 hrs	27.6	58000	2051.1
MC1G	24 hrs	27.5	50500	1785.8
MC1H	24 hrs	27.5	52000	1838.9
7 Day Test Result				
Specimen	**Test Time**	**Weight**	**Load / lb**	**Comp.Strength / psi**
MC1K	7days	27.8	90000	3182.7
MC1L	7days	27.7	88000	3112.0
MC1M	7days	27.6	95000	3359.5
MC1N	7days	27.6	90000	3182.7

Table C-S4

Compressive strength for 4hrs, 24hrs and 7days for mix 5

Mix 5				
Cement, Ib	800.000			
Coarse Aggregate, Ib	1766.000			
Fine Aggregate, Ib	1189.000			
Water, Ib	264.500			
Accelerator, (PolarSet), gal.	16.000			
HRWR (ADVA Flow), oz.	40.000			
Darex II AEA, oz.	16.000			
W/C Ratio	0.33			
4 hourTest Results				
Specimen	**Test Time**	**Weight**	**Load / lb**	**Comp.Strength / psi**
M501	4 hrs	27.5	23000	813.4
M502	4 hrs	27.6	24000	848.7
M503	4 hrs	27.5	23000	813.4
M504	4 hrs	27.5	24500	866.4
24 hour Test Result				
Specimen	**Test Time**	**Weight**	**Load / lb**	**Comp.Strength / psi**
M505	24 hrs	27.8	54000	1909.6
M506	24 hrs	27.6	52500	1856.6
M507	24 hrs	27.5	53500	1891.9
M508	24 hrs	27.5	52000	1838.9
7 Day Test Result				
Specimen	**Test Time**	**Weight**	**Load / lb**	**Comp.Strength / psi**
M509	7days	27.8	95000	3359.5
M510	7days	27.7	94500	3341.8
M511	7days	27.6	93000	3288.8

Table C-S5

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