APPLICABILITY OF THE MATURITY METHOD AS A MEANS OF ESTIMATING CONCRETE STRENGTH

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Applicability of the Maturity Method as a Means of Estimating Concrete Strength

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DECLARATION

This thesis is my original work and has not been presented for a degree in any other university.

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DEDICATION

This thesis is dedicated to my parents and my brother, to whom I am greatly indebted.

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I wish to express my gratitude to my advisors for their insight and encouragement. I am also grateful to my family for their patience and support during the months of writing, and to God for being my guide.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF PLATES

APPENDICES

LIST OF ACRONYMS

ACI – American Concrete Institute

ASTM – American Society for Testing and Materials

BRE – Building Research Establishment

BS – British Standard

FHWA – Federal Highway Administration

NBS – National Bureau of Standards

OSHA – Occupational Safety and Health Administration

ABSTRACT

In Kenya, quality control of concrete involves taking samples of poured concrete, preparing specimens under controlled conditions, and subjecting the specimens to strength testing to ensure that the design strength is achieved in 28 days. However, depending on prevailing in-situ temperature conditions, concrete strength prediction based on the 28-day rule may be inaccurate. Research has shown that this conventional concrete quality control protocol, which is easily circumvented, is not effective on ensuring structural reliability of new or existing buildings, as evidenced by the collapse of seventeen buildings between 2006 and 2014 in Kenya. The maturity method of estimating concrete strength, which was developed by the US National Bureau of Standards, has been successfully used to predict the strength of concrete prepared according to American standards and without admixtures, resulting in tighter quality control of concrete. This research set out to demonstrate that the method is equally applicable to concrete prepared according to the locally used British standards, and concrete containing a plasticizer - a chemical admixture which is gaining widespread use in the production of concrete. To this end, three mixes were prepared, one according to American standards (mix A) and two according to British standards (mixes B and C). Modified lignosulphonate (Sika Plastiment BV-40), a locally available plasticizer, was used to enhance the workability of mix C. Concrete mix proportioning according to British standards resulted in a denser mix (2400 kg/m^3) than American standards (2342 kg/m^3); this was the main difference between the two standards. Cylindrical specimens (each measuring 150 mm in diameter by 300 mm deep) and beam specimens (each measuring 150 mm wide by 530 mm long by 150 mm deep) were made and cured at 23 ºC. The compressive and splitting tensile strengths of the cylinders and the flexural strength of the beams were almost identical for the three mixes. Also, the internal temperature (and hence the calculated maturity) of concrete was the same for all mixes. These findings indicate that: (1) the choice of standards used to prepare concrete

(American or British) has no effect on the strength and maturity of the resulting concrete mix; and (2) the maturity method may be applied to concrete containing a plasticizer.

CHAPTER ONE
INTRODUCTION

1.1 Background Information

1.1.1 Overview

The use of accelerated schedules in the construction of concrete structures has been necessitated by a desire to achieve economic benefits (Naik, 1992). Accurate prediction of in-situ concrete strength development can be used to shorten construction schedules and, as a result, reduce overall construction costs by determining the appropriate time to start critical construction activities such as removal of formwork and opening a pavement to traffic.

The maturity method is a useful, easily implemented, accurate means of predicting in-situ concrete strength (Crawford, 1997). It is based on the knowledge that concrete gains strength quickly when exposed to high temperatures, and slowly when exposed to low temperatures. This dependence of concrete strength on temperature presents a problem when the in-situ strength of concrete is determined using conventional methods.

Conventional non-destructive testing of in-situ concrete involves sampling the concrete before it is placed in a structure, putting the samples under controlled conditions in a laboratory (typically at room temperature), and testing the samples at regular time intervals so as to determine the rate of concrete strength development. This rate of strength gain is used to predict the strength of the concrete placed in the structure. However, the temperature of the concrete within the structure is rarely the same as that of the samples (Anderson *et al.*, 2009).

If the concrete in the structure is exposed to a higher temperature than that at which the samples have been tested in the laboratory, it will gain strength at a higher rate than the samples and achieve the desired strength more quickly than predicted. As a result, the removal of formwork or the opening of a pavement to traffic may be delayed unnecessarily, resulting in the loss of valuable construction time (Anderson *et al.*, 2009).

In contrast, if the concrete within the structure is exposed to a lower temperature than the laboratory temperature, the concrete will gain strength at a lower rate than predicted. Therefore, there is a possibility that formwork could be removed, or a pavement could be opened to traffic, before adequate strength is attained, resulting in the collapse of the structure (Anderson *et al.*, 2009).

Knowing the actual strength of in-situ concrete is important in projects where the removal of formwork from structures or the opening of pavements to traffic is a critical factor in maintaining accelerated construction schedules (Anderson *et al.*, 2009). Conventional methods of predicting in-situ concrete strength result in a conservative prediction during periods of hot weather when the temperature of the in-situ concrete may be higher than that at which samples of the concrete have been tested in a laboratory. These methods also result in an un-conservative prediction during cold weather periods when in-situ concrete temperature may be lower than the laboratory temperature. The maturity method recognizes the effect of temperature on the strength development of concrete. It provides a basis for estimating the in-situ strength of concrete by monitoring the temperature of the concrete over time.

1.1.2 Maturity Concept

Concrete gains strength through the hydration reaction between cement and water (Kosmatka, 2008). To maintain this increase in strength with age, concrete must be properly cured. This means that a satisfactory moisture content and temperature must be maintained in concrete for a period of time to allow the hydration of cement to occur. Temperature has a significant effect on concrete strength development (Garcia *et al.*, 2008). An increase in curing temperature speeds up the hydration process, leading to an increase in strength development.

The maturity method uses the curing time and temperature of concrete to compute a single parameter which is indicative of the strength of the concrete. This parameter is called “maturity” (Malhotra & Carino, 2004). The maturity of concrete is a function of the product of curing time and temperature of the concrete. The maturity rule states that a unique relationship exists between the maturity and strength of a particular concrete mixture (Malhotra & Carino, 2004). This means that if two samples of a given concrete mixture have the same maturity, they will have the same strength even though each may have been exposed to different curing times and temperatures.

Figure 1.1 illustrates the maturity concept. A concrete mixture exposed to a low temperature takes more time to reach maturity M1, whereas a concrete mixture exposed to a high temperature takes less time to reach maturity M2. If M1=M2 (i.e., area of rectangle M1 = area of rectangle M2), these two mixtures will have equal strengths even though the individual curing times and temperatures are different (Nelson, 2003).

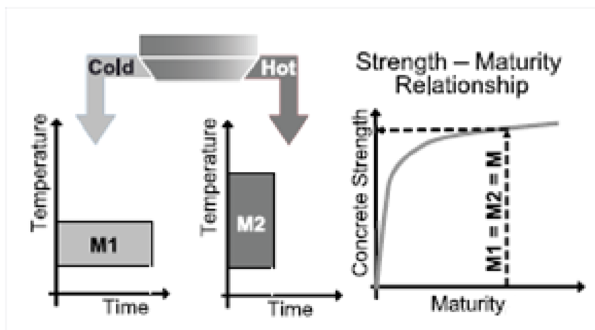

Figure 1.1: Maturity concept

1.1.3 Maturity Testing Procedure

The maturity testing procedure involves two steps (Crawford, 1997):

1. Laboratory calibration – A concrete mix which is representative of the concrete to be used for a construction project is prepared. Test specimens are prepared from the mix and a temperature sensor is inserted into at least two specimens for the purpose of recording concrete temperature for calculation of maturity values at specified ages (i.e., after 1, 3, 7, 14 and 28 days). Plate 1.1 shows a temperature sensor embedded in a concrete test cylinder.

Plate 1.1: Recording of internal concrete temperature

Strength tests are performed on the remaining specimens at the specified ages, and a strength-maturity relationship curve (also known as a calibration curve) is established.

2. Field measurement of the maturity of the concrete placed in a structure – As soon as is practicable after concrete placement, a temperature sensor is embedded into the fresh in-situ concrete. The temperature of the in-situ concrete is recorded and used to calculate the maturity of the concrete. This maturity is used together with the previously established calibration curve to estimate the in-situ concrete strength.

1.1.4 Standard Practice for Maturity Testing

A tragic display of the temperature-dependence of concrete strength gain occurred in 1973 in Fairfax County, Virginia, USA, when a multi-story building collapsed during construction, killing fourteen workers and injuring thirty-four. The U.S. National Bureau of Standards (NBS) investigated the accident at the request of the Occupational Safety and Health Administration (OSHA). The NBS report concluded that the most probable cause of the failure was the premature removal of formwork from a four-day-old floor slab which had been subjected to an average ambient temperature of only 7 ºC (Carino & Lew, 2001).

The NBS encountered difficulty in using concrete strength development data obtained under laboratory conditions to obtain a reliable estimate of the in-situ concrete strength at the time of the failure. This triggered an interest in a relatively new approach known as the maturity method for estimating in-situ concrete strength development (Carino &Lew, 2001).

In a study at the NBS, the applicability of the maturity method under simulated field conditions was investigated. This research revealed that concrete cured in the field experienced different temperatures than concrete cured in a laboratory. Further research confirmed that the maturity method could be used to estimate the in-situ strength of concrete subjected to different curing temperatures (Carino & Lew, 2001).

In 1978, there was a major construction failure of a cooling tower being constructed in Willow Island, West Virginia, USA. The incident resulted in the death of fifty-one workers. The NBS was again requested by OSHA to determine the cause of the failure. NBS concluded that the most likely cause of the collapse was insufficient concrete strength to support the applied construction loads (Carino and Lew, 2001). At the time of the failure, the concrete was only one day old and had been exposed to an estimated average ambient temperature less than 10 °C.

The accident in 1978 convinced NBS researchers that there was an urgent need for standards on estimating in-situ concrete strength during construction (Carino & Lew, 2001). The NBS began an in-depth study of the maturity method. This research led to the establishment of the standard for estimating in-situ concrete strength using the maturity method in 1987 (ASTM C 1074).

ASTM C 1074 – 04 outlines the following applications of the maturity method:

1. This method can be used to estimate the in-situ strength of concrete so as to determine the appropriate time to start critical construction activities such as: (a) removal of formwork; (b) termination of special concreting practices such as using insulation during cold weather; and (c) opening a pavement to construction or public traffic.
2. This method can be used for laboratory work involving different-sized test specimens – Test specimens with low surface-to-volume ratios experience higher early-age temperature rises than specimens with higher surface-to-volume ratios. The use of the

maturity method ensures that different-sized specimens are tested at the same maturity.

1.1.5 Quality Control of Concrete in Kenya

In Kenya, concrete is often mixed manually or with small mixers on construction sites (Fernandez, 2014). The resulting concrete mixture is then hauled in wheelbarrows and poured into formwork. Technicians from testing laboratories collect samples of the poured concrete and prepare specimens which are then stored under controlled conditions in laboratories. The specimens are subjected to compressive strength testing at pre-determined ages (7, 14, and 28 days) to ensure that the design strength is achieved in 28 days.

Engineers and inspectors determine if structures are safe based on the findings of their inspection visits to construction sites, and on the values of the compressive strength of concrete reported by materials testing laboratories. Real estate developers assume that the quality of the concrete used in construction is verified following the sampling and testing processes outlined in British codes, which are used to design structural concrete in Kenya (Fernandez, 2014).

Between 2006 and 2014, seventeen buildings collapsed in Kenya, causing eighty-four (84) deaths and two hundred and ninety (290) injuries (Fernandez, 2014). In 2009, Kenyan officials estimated that 65% of Kenya's buildings fail to meet code standards. This means that the quality control mechanisms for structural concrete currently used in Kenya are not as effective as they should be.

In 2014, a study conducted by Fernandez examined the state of the construction industry's compliance with standards for concrete used in Kenya. This was done in two ways: (1) a comparison of in-situ concrete strength test data, collected at twenty-four construction sites, with test results reported by established laboratories in Nairobi from a sample of new construction projects – In-situ concrete strength data was collected using rebound hammer tests; and (2) through a survey of fifty-one existing buildings in the metropolitan area of Nairobi. The sampled buildings included industrial, residential, commercial and religious structures. The construction sites were sufficiently diverse with regard to location, construction company size, building type, and design. They were considered a representative cross-section of the industry. The findings suggested that concrete is frequently weaker than claimed by laboratory test reports, and that current quality control practices are not effective in ensuring structural reliability of new or existing buildings (Fernandez, 2014).

The collapse of buildings in Kenya has triggered regulatory review. This has focused on zoning, building permits, and licensing, because it is commonly understood that defective designs and inadequate standards are to blame for the collapse of buildings (Fernandez, 2014). In 2011, the government of Kenya enacted two laws to improve the quality and safety of buildings: (1) the Engineers Act, which authorizes the Engineers Board of Kenya to access and inspect construction sites at will; and (2) the National Construction Authority Act, which created a National Construction Authority (NCA) with a mandate to regulate and improve the construction industry.

The NCA Act expressly states that one of its objectives is to “promote quality assurance in the construction industry”. However, despite these efforts, fundamental industry practices, including quality control protocols, have remained the same. This is partly because, until now, most of the work has focused on tighter regulation and certification of building contractors, while quality control methods remain as they have been for decades (Fernandez, 2014).

Unless better control systems are implemented, thousands of dangerously weak buildings will be built, and millions of people will likely be exposed to unnecessary risks for generations. Therefore, priority should be given to the improvement of construction quality control processes and regulation. Policymakers in government, non-governmental organizations, and professional organizations must catalyze institutional change in the construction industry as a matter of urgency. Their efforts will be most effective if attention is given to the promotion and enforcement of prudent quality control protocols that encourage engineers and inspectors to assume less and verify more (Fernandez, 2014).

1.2 Statement of the Problem

In Kenya, quality control of concrete involves taking samples of poured concrete, preparing specimens under controlled conditions, and subjecting the specimens to strength testing to ensure that the design strength is achieved in 28 days. However, depending on prevailing in-situ temperature conditions, concrete strength prediction based on the 28-day rule may be inaccurate. Research has shown that this conventional concrete quality control protocol, which is easily circumvented, is not effective in ensuring structural reliability of new or existing buildings. In 2014, a study conducted by Fernandez found that concrete used in Kenya is frequently weaker than claimed by laboratory test reports. This was evidenced by the collapse of seventeen buildings between 2006 and 2014 in Kenya.

Building failure may result in fatalities, where building occupants are either killed instantly by the collapse, or succumb to the effects of severe injuries afterwards. The economic value of the lives that are lost is often significant. In addition, a wide range of injuries are associated with building collapse: muscuskeletal injuries, head injuries, auditory damage, ocular injuries, burns, internal organ injuries and respiratory problems. The costs related to these injuries may include emergency services, physician and surgeon services, rehabilitation costs and lost income. The psychological harm experienced by injured victims (and their loved ones) due to loss or damage of property and/or bodily harm may be manifested in the form fear, helplessness, distress, depression and suicides. This inevitably leads to decreased productivity and a lower quality of life.

The economic consequences of structural failure are often dependent on: replacement/repair of the structure, which requires a significant amount of resources; temporary relocation which disrupts the work-life dynamics of displaced residents; loss of structural functionality and cleanup costs; rescue costs, which is associated with providing emergency services; regional economic effects, which depend on the cost of business interruption, as well as the cost related to job and wage losses; cost of investigation/compensation; and loss of reputation, which may be taken as the long-term effect of a structural collapse on business activities.

Following the failure of an engineering structure, reconstruction efforts lead to increased cement consumption. Cement manufacturing is responsible for the emission of vast amounts of CO_2 – a major greenhouse gas contributing to global warming. Environmental studies, as well as consequent reparative measures, will often require the expertise of trained professionals, as well as payment of certification/compliance costs to relevant authorities. In cases where the release of toxic pollutants (such as oil or industrial chemicals) may be a serious consequence of a building collapse, extra care should be taken to secure such pollutants. Collection, handling and transportation of these pollutants may require significant financial input. In addition, the cost of harming the surrounding flora and fauna is likely to be large, particularly for industrial buildings.

Unless better concrete quality control systems are implemented, thousands of weak buildings will be built, and millions of people will likely be exposed to the aforementioned consequences. The maturity method of estimating concrete strength, which was developed by the US National Bureau of Standards, has been used to monitor strength gain in many construction projects in the United States, providing tighter quality control of concrete. This study demonstrated that the maturity method is equally applicable in Kenya where: (1) structural concrete is designed according to British standards; and (2) modified lignosulphonate (Sika Plastiment BV-40), a locally available plasticizer, is used to enhance the workability of concrete. The findings of this study will promote the adoption of the maturity method in Kenya and, as a result, alleviate the problems caused by catastrophic collapse of concrete structures.

1.3 Justification for the Study

The maturity method has been successfully used to predict the strength of concrete prepared according to American standards, and without admixtures. In order to promote the adoption of this method in Kenya, where current quality control practices are not effective in ensuring structural reliability of new or existing buildings, guidance on the on the application of the method to concrete prepared according to the locally used British standards is required. In addition, this research investigated whether or not the maturity method could be applied to concrete containing a plasticizer – a chemical admixture which is gaining widespread use in the production of concrete. In a time when public agencies and contractors are concerned with escalating costs and shrinking budgets, this method provides a viable means of reducing costs without compromising safety.

1.4 Objectives

1.4.1 Main Objective

The main objective of this research was to assess the applicability of the maturity method as a means of estimating concrete strength.

1.4.2 Specific Objectives

The specific objectives of this research were:

1. To investigate the applicability of the maturity method to concrete prepared according to British standards.
2. To assess the applicability of the maturity method to concrete containing a plasticizer.

1.5 Scope and Limitations

1.5.1 Scope

The concrete used in this research was prepared according to American standards, and the locally used British standards only. The curing temperature range of the concrete was 23 ± 2º C. Hence a datum temperature of 0º C was used to compute the maturity of concrete according to equation (2.1).

1.5.2 Limitations

The concrete used in this research was proportioned according to the American Concrete Institute (ACI) and UK Building Research Establishment (BRE) mix proportioning procedures. Ordinary Portland cement, and river sand and ballast conforming to the grading requirements of both American and British standards were used to prepare the concrete. Modified lignosulphonate (Sika Plastiment BV-40) was used to increase workability. The findings of this research will likely only be applicable to the above-mentioned conditions.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

The dependence of concrete strength on temperature presents a problem when an estimate of the strength of in-situ concrete placed in a structure is based on strength development which has been derived from data obtained under standard laboratory conditions using conventional strength-estimation methods (Malhotra & Carino, 2004). The maturity method recognizes the effect of temperature on concrete strength development and provides a basis for estimating the in-situ strength gain of concrete placed in a structure by monitoring its temperature over time.

The maturity method can be used to apply better timing to construction activities which are dependent on the concrete having achieved a certain minimum strength value (such as removal of formwork). Given the high cost of delays in construction, this improved timing can result in substantial financial savings without sacrificing safety or quality. In a time when public agencies and contractors are concerned with escalating costs and shrinking budgets, this method provides a viable means of reducing costs (Crawford, 1997).

Although the maturity method continues to evolve, it still has limitations. The purpose of this chapter is to provide a review of the theoretical development, application, benefits and limitations of the maturity method.

2.2 Theoretical Background

ASTM C 1074 – 04 defines the maturity method as a technique for estimating concrete strength that is based on the knowledge that samples of a given concrete mixture attain equal strengths if they attain equal values of maturity. This method is viewed as a useful and simple means for accounting for the complex effects of time and temperature on concrete strength development (Malhotra & Carino, 2004).

Around 1950, an approach was proposed to account for the combined effects of time and temperature on strength development of concrete (Malhotra & Carino, 2004). It was proposed that the temperature history of concrete during the curing period could be used to compute a single parameter that would be indicative of concrete strength. In 1951, Saul called this parameter “maturity”. He defined the term as follows: “The maturity of concrete may be defined as its age multiplied by the average temperature above freezing that it has maintained.” Thus maturity is calculated based on temperature history using the following equation:

$$M = \sum_{0}^{t} (T_a - T_0)\Delta t \tag{2.1}$$

where:

M = maturity (time-temperature factor) at age t (ºC-hour or ºC-day)

T_a = average concrete temperature during time interval Δt (ºC)

T_0 = datum temperature (ºC)

Δt = time interval (hour or day)

Equation (2.1) is known as the Nurse-Saul equation. The datum temperature (T_0) is the temperature below which no strength gain of concrete takes place (Ghosh, 2008). In using this equation, only those time intervals in which concrete temperature is greater than T_0 are considered as contributing to strength gain.

The datum temperature depends on the type of cement used to prepare concrete, and the range of curing temperature that the concrete will be subjected to. For concrete prepared using general purpose Portland cement, and exposed to a curing temperature range from 0 to 40 °C, ASTM recommends a datum temperature of 0 °C (Carino & Lew, 2001).

If concrete temperature is plotted versus age, equation (2.1) is simply equal to the area between the datum temperature and the temperature curve (Malhotra & Carino, 2004).

This concept is demonstrated in Figure 2.1. The shaded area represents the maturity (time-temperature factor) at age t*.

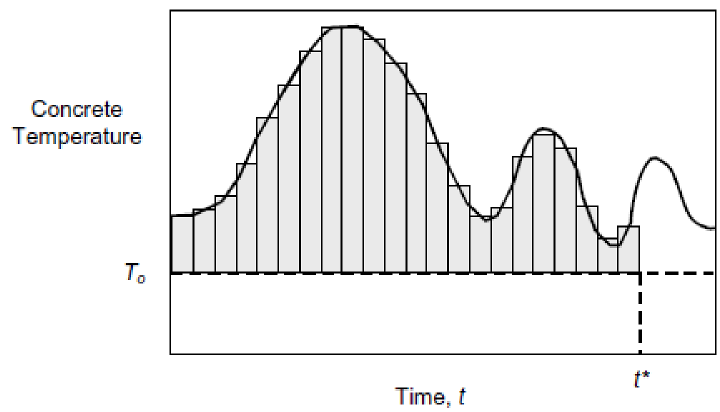

Figure 2.1: Schematic of maturity computed according to equation (1)

Saul then presented the following principle, which is now known as the maturity rule (Malhotra & Carino, 2004): “Concrete of the same mix at the same maturity has approximately the same strength whatever combination of time and temperature goes to make up that maturity.” This means that samples from a given concrete mixture will have equal strength at equal maturity regardless of their temperature history.

Saul’s introduction of the maturity rule was the result of studies dealing with accelerated curing. Bergstrom (1953), as cited by Malhotra and Carino (2004), used the maturity method to analyze previously published data on the effects of temperature on concrete strength development. He demonstrated that the maturity method was equally applicable for curing at normal temperatures

Equation (2.1) is based on the assumption that the rate of strength gain of concrete is a linear function of temperature (Malhotra & Carino, 2004). This equation can be used to convert a curing history to an equivalent age of curing at a reference temperature as follows:

$$t_e = \frac{\sum (T_a - T_0)}{(T_r - T_0)} \Delta t \tag{2.2}$$

where:

t_e = equivalent age at the reference temperature (hour or day)

T_a = average concrete temperature during time interval Δt (ºC)

T_0 = datum temperature (ºC)

T_r = reference temperature (ºC)

Δt = time interval (hour or day)

The equivalent age in equation (2.2) represents the duration of the curing period at the reference temperature that would result in the same maturity as the curing period at other temperatures (Malhotra & Carino, 2004). Rastrup, as cited by Malhotra and Carino (2004), introduced the equivalent age concept, which is a convenient method for using other functions besides equation (2.1) to account for the combined effects of time and temperature on concrete strength development.

The Nurse-Saul equation was the only accepted maturity equation until the late 1970s. Freiesleben and Pedersen (1977), as cited by Carino and Lew (2001), proposed a new equation to compute maturity from the recorded temperature history of concrete. This equation allowed the computation of maturity in terms of an equivalent age of concrete as follows:

$$t_e = \sum_0^t e^{\frac{-E}{R}\left[\frac{1}{273+T_a} - \frac{1}{273+T_r}\right]} \Delta t \tag{2.3}$$

where

t_e = maturity in terms of equivalent age at the reference temperature (hour or day)

T_a = average concrete temperature during time interval Δt (ºC)

T_r = reference temperature (ºC)

Δt = time interval (hour or day)

E = activation energy (J/mol)

R = universal gas constant 8.3144 J/ (mol K)

The key parameter in equation (2.3) is the activation energy (E). It defines the temperature sensitivity of a concrete mixture (Brooks *et al.*, 2007). The reference temperature is generally assumed to be the standard laboratory curing temperature, which is usually either 20 or 23°C.

ASTM C 1074 permits the user to express the maturity of concrete using either the time-temperature factor based on equation (2.1) or equivalent age based on equation (2.3). However, further investigation is required to determine the factors that affect the accuracy of equation (2.3) in the field (Jung & Cho, 2009).

The exponential function in equation (2.3) is the age conversion factor and is expressed in terms of the absolute temperature. Freiesleben Hansen and Pedersen proposed the following values for E (Malhotra & Carino, 2004):

E = 33 500 J/mol for $T_a \geq 20$ºC

E = 33 500 + 1470 (20 - T_a) J/mol for $T_a < 20$ºC

ASTM C 1074 recommends that a constant E in the range of 40 000 – 45 000 J/mol be used in equation (2.3) when ordinary Portland cement with no admixtures is used. However, no values are recommended for concrete prepared with admixtures (Brooks *et al.*, 2007).

2.3 Case Studies of the Application of the Maturity Method

The maturity method has been implemented in numerous construction projects to monitor strength gain. This method is gaining acceptance due to its simplicity in combining the effects of varying concrete temperatures and curing times on concrete strength development (Naik, 1992).

The maturity method was used to predict the in-situ strength of concrete slabs during construction of buildings at the University of Waterloo in 1971 and 1972 (Naik, 1992). The strengths of the in-situ concrete slabs were predicted so as to determine formwork removal times. Samples of the in-situ concrete were taken to determine the adequacy of the predictions. It was concluded that the maturity method provided satisfactory results.

Between 1973 and 1976, the maturity concept was used to determine the appropriate time for formwork removal during construction of the Canadian National Tower in Toronto, the world's tallest free-standing structure at the time (Naik, 1992). Maturity predictions were first compared with test results for cores drilled from the structure. The predictions were found to have a good correlation with core test results. The maturity method was then used to monitor the strength gain of the entire structure.

Between 1983 and 1984, a test program involving both field and laboratory tests was used to determine the accuracy of the maturity concept (Naik, 1992). The maturity method was employed to determine safe formwork stripping times for a 10-km long, 5.8-m inside diameter tunnel arch lining. The use of the maturity concept reduced winter curing time, resulting in approximately 30% saving in heat compared to the use of conventional cold weather curing. In addition, economic benefits resulted from reduction in labour costs, inspection and supervision costs, and shortened schedules.

In 1988, a study involving fast-track concrete paving in Iowa, USA incorporated the use of the maturity method. The Federal Highway Administration (FHWA) used this project to demonstrate the benefits of non-destructive testing of concrete. The study found that the maturity method was a reliable test for field estimation of concrete strength (Newbolds & Olek, 2001).

In 1995, the maturity method was used in a series of field trials covering fourteen concrete paving projects in Iowa, USA (Newbolds & Olek, 2001). The studies utilized different concrete mixtures and involved various types of pavements. It was found that regardless of the number of specimens used to develop the maturity curve, the relationship between the strength and the maturity of the concrete remained valid.

In 1996, six concrete paving projects in Iowa, USA were selected to utilize the maturity method to determine opening-to-traffic strengths (Newbolds & Olek, 2001). The results were positive and the pavements were opened to traffic eighteen hours after placement of concrete.

In a study conducted from January to April 2005, the maturity concept was applied to an industrial construction project in Edmonton, Canada so as to assess the reliability and potential benefits of using the maturity method in cold weather (Bagheri-Zadeh *et al.*, 2007). The findings indicated that the maturity method produced a reliable and accurate prediction of in-situ concrete strength on a continuous (real-time) basis during curing. This timely and accurate estimation of field concrete strength led to significant time and cost savings when the maturity method was used instead of conventional strength-estimation methods.

The maturity method has been used to monitor strength gain in many construction projects with considerable success (Naik, 1992). The use of this method can provide improvement in construction productivity, resulting in substantial time and cost savings.

2.4 Benefits and Limitations of the Maturity Method

The use of the maturity method provides a few advantages when compared to conventional methods for determining in-situ concrete strength (Myers, 2000):

1. It provides a better representation of strength gain than laboratory-cured samples of the in-situ concrete – The FHWA determined that even samples of the in-situ concrete which are cured in the field (next to a concrete structure) do not accurately reflect the true rate of hydration of the concrete placed in the structure (FHWA, 1988). The American Concrete Institute (ACI) acknowledges that even samples

drilled directly from a concrete structure do not accurately represent the strength of the concrete in the structure (ACI, 2011).

2. The maturity method enables contractors/engineers to measure strength within a concrete structure at any time and as many times as necessary until the desired strength is achieved (Trost * Fox, 2004) – Conventional strength-estimation methods require making, curing, and testing samples prepared from the in-situ concrete. If all the samples are tested too early (when the measured in-situ concrete strength is too low), there will be no samples to test at a later time. If the samples are tested too late (when the measured strength of in-situ concrete is higher than required), valuable construction time will have been lost. The maturity method can be used to provide in-situ concrete strength measurements at just the right time.
3. The maturity method provides better timing for strength-dependent construction activities (Trost & Fox, 2004) – Because the maturity method can be used to accurately measure in-situ concrete strength at any time, better timing can be applied to construction activities which are dependent on the in-situ concrete having achieved certain minimum strength values (such as removal of formwork). This improved timing can reduce construction delays, resulting in financial savings.
4. The maturity method enables in-situ concrete strength measurement at "lowest strength" locations (Trost & Fox, 2004) – Concrete within a structure will gain strength at different rates in different locations depending on temperature conditions within the structure. Thinner concrete sections tend to generate and retain less heat than sections with more mass and/or less surface area. The maturity method can be used to take measurements at locations where strength gain is likely to be slowest. This will provide assurance that subsequent construction work does not begin until adequate strength has been gained within the entire structure.
5. The "pinpoint" capability of measuring strength in a concrete structure using the maturity method can also be used to target strength measurements in locations where critical stresses are expected for certain loading conditions (Trost & Fox, 2004).

Although the maturity method continues to evolve, it still has limitations (Kim & Rens, 2008). These limitations as outlined in ASTM C 1074 – 04 are:

1. Concrete must be maintained in a condition that permits cement hydration – This means that concrete must be cured properly so as to maintain increase in strength with age. If this is not the case, then strength estimates based on the maturity method are meaningless (Crawford, 1997).
2. The maturity method does not take into account the effects of early age concrete temperature on long-term ultimate strength – This means that the maturity method is only useful in estimating the strength gain of concrete at early ages, generally less than 28 days old.
3. The maturity method needs to be supplemented by other indicators of the potential strength of concrete – ASTM C 1074 – 04 requires verification of the strength of in-situ concrete before performing critical construction operations, such as formwork removal (Carino &Lew, 2001). This is because there is no assurance that the in-situ concrete has the same mixture proportions as the concrete used to develop a relationship between strength and maturity in the laboratory. Methods of verification of concrete strength include other in-situ tests that measure the strength of concrete, such as the rebound hammer test. However, this limitation can easily be overcome by adopting strict batching procedures for in-situ concrete.

2.5 Summary of Literature Review

Concrete quality control mechanisms currently used in Kenya are not as effective as they should be (Fernandez, 2014). Although the collapse of buildings has triggered regulatory review, the focus has been on tighter regulation and certification of building contractors while quality control methods remain as they have been for decades. No effort has been made to explore the adoption of better quality control protocols. This research sought to determine the applicability of the maturity method, which has been successfully used to predict concrete strength in the United States, to the local construction industry where structural concrete is designed according to British standards.

In addition, prior to this study, the maturity method had only been applied to concrete prepared without admixtures (Brooks *et al.*, 2007). As chemical admixtures become an increasingly indispensable component of concrete, it is prudent that the applicability of the maturity method to concrete containing admixtures be addressed. This research sought to determine the applicability of the maturity method to modified lignosulphonate (Sika Plastiment BV-40), a locally available plasticizer which has recently been introduced to the local construction industry.

CHAPTER THREE: MATERIALS AND METHODS

3.1 Introduction

Three concrete mixtures, one prepared according to American standards (mix A) and two prepared according to British standards (mixes B and C), were used in this research. For all mixtures, class 42.5 ordinary Portland cement was used as a binder, and locally available natural river sand and ballast were used as fine and coarse aggregate respectively. Potable water was used to mix and cure the concrete. Each mix was designed to have an average 28-day compressive strength of 25 N/mm^2 and a maximum aggregate size of 20 mm. The water-cement ratio of mix A was obtained from Table A2. For mixes B and C, the water-cement ratio was restricted to 0.5. Modified lignosulphonate (Sika Plastiment BV-40) was added to mix C so as to increase workability without adjusting the water-cement ratio. The following data was first determined for mix design purposes: (1) sieve analyses of fine and coarse aggregates; (2) unit weights of fine and coarse aggregates; and (3) specific gravity and water absorption of fine and coarse aggregates.

3.2 Investigating the Applicability of the Maturity Method to Concrete Prepared According to British Standards

3.2.1 Experimental Setup

3.2.1.1 Properties of Aggregates

Sieve analyses of fine and coarse aggregates were done in accordance with ASTM C 136 – 96a for mix A, and BS 812 – Part 103.1:1985 for mix B. The fineness modulus of aggregate, which is an indicator of the fineness of an aggregate, was calculated by adding the cumulative percentages retained on each of the following sieves, and dividing the sum by 100: 150 µm, 300 µm, 600 µm, 1.18 mm, 2.36 mm, 4.75 mm, 9.5 mm, 19.0 mm, 37.5 mm, and larger.

The unit weight of aggregate, which is the mass of a unit volume of the aggregate, was determined in accordance with ASTM C 29 – 03 for mix A, and BS 812 – 2: 1995 for mix B. The specific gravity and absorption of fine aggregate was determined in accordance with ASTM C 128 – 97 for mix A, and BS 812 – 2: 1995 for mix B. The specific gravity, which is the ratio of the mass of a unit volume of a material to the mass of the same volume of water, was calculated on the basis of saturated surface-dry fine aggregate. Water absorption was calculated as a percentage of dry mass. The specific gravity and water absorption of coarse aggregate was determined in accordance with ASTM C 127 – 93 for mix A, and BS 812 – 2: 1995 for mix B. The specific gravity was calculated on the basis of saturated surface-dry coarse aggregate, and water absorption was calculated as a percentage of dry mass.

3.2.1.2 Concrete Mix Proportioning

Concrete mix proportioning was done in accordance with the American Concrete Institute (ACI) mix design procedure (ACI 211.1 - 91) for mix A; the slump of this mix, which was designed to be 25-50 mm, was determined in accordance with ASTM C 143 – 05a. The approximate amount of mixing water required to produce a 25-50 mm (1-2 in.) slump in non-air-entrained concrete with a maximum aggregate size of 20 mm (4/5 in.) was found from Table A1 to be 312 lb/yd^3 (184 kg/m^3). From Table A2, the water-cement ratio needed to produce a strength of 25 N/mm^2 (3623 psi) in non-air-entrained concrete was found (by interpolation) to be about 0.6. Therefore, the amount of cement required per cubic yard of concrete was 312/0.6 = 520 lb/yd^3 (307 kg/m^3).

The fineness modulus of sand (determined from sieve analysis of fine aggregate) and the maximum size of coarse aggregate were used to estimate the volume of coarse aggregate required per unit volume of concrete from Table A3. The unit weight of coarse aggregate (determined experimentally) was used to calculate the weight of coarse aggregate required per unit volume of concrete as shown in equation (3.1):

$$W = V \times U = 1035 \ \ kg/m^3 \tag{3.1}$$

where:

W = weight of coarse aggregate required per unit volume of concrete, kg;

V = volume of coarse aggregate required per unit volume of concrete, (= 0.646); and

U = unit weight of coarse aggregate (1602 kg/m^3).

With the quantities of water, cement, and coarse aggregate established, the remaining material comprising the unit volume of concrete consisted of fine aggregate. The estimated weight of a unit volume of non-air-entrained concrete made with aggregate having a maximum size of 20 mm (4/5 in.) was found from Table A4 to be 3970 lb/yd^3 (2342 kg/m^3). The weight of fine aggregate required per unit volume of concrete was calculated using equation (3.2)

$$FA = 2342 - (W + C + CA) = 816 \ kg/m^3 \tag{3.2}$$

where:

FA = weight of fine aggregate required per unit volume of concrete

W = weight of water required per unit volume of concrete (184 kg/m^3)

C = weight of cement required per unit volume of concrete (307 kg/m^3)

CA = weight of coarse aggregate required per unit volume of concrete (1035 kg/m^3)

Table 3.1: Mix proportions for 1 m^3 of grade 25 concrete

Mix	Water (kg)	Cement (kg)	Fine Aggregate (kg)	Coarse Aggregate (kg)	Density (kg/m^3)
A	184	307	816	1035	2342

Concrete mix proportioning was done in accordance with the United Kingdom Building Research Establishment (BRE) mix design procedure for mix B. The slump of this mix, which was designed to be 30-60 mm, was determined in accordance with BS 1881 – 102:1983.

As a result of the variability of concrete in production, the mix was designed to have a mean strength greater than the specified characteristic strength by an amount termed the margin. Thus the target mean strength was calculated using equation (3.3):

$$f_m = f_c + ks \tag{3.3}$$

where:

f_m = the target mean strength (in N/mm^2)

f_c = the specified characteristic strength (in N/mm^2)

ks = the margin (in N/mm^2), which is the product of:

s = the standard deviation (in N/mm^2), and

k = a constant (= 1.96)

A standard deviation (s) of 8 N/mm^2 obtained from line A in Figure A1 was used because previous information concerning the variability of strength tests comprised fewer than 20 results.

The margin was then derived from equation (3.4):

$$ks = 1.96 \times 8 = 16 \tag{3.4}$$

For a specified characteristic strength of 25 N/mm^2, the target mean strength was found to be 41 N/mm^2 using equation (3.5).

$$f_m = 25 + 16 = 41\ N/mm^2 \tag{3.5}$$

Class 42.5 Portland cement was used to prepare the concrete mixture. The coarse aggregate was crushed. Table A5 showed that for the materials used, the estimated 28-day compressive strength for a free-water/cement ratio of 0.5 was 49 N/mm^2.

By applying an estimated 28-day strength of 49 N/mm^2 to Figure A2 and plotting a strength versus free-water cement ratio curve by interpolation between the curves adjacent to the entered value, it was found that a free-water/cement ratio of about 0.55 was required for a target mean strength of 41 N/mm^2. This free-water/cement ratio was higher than the specified maximum value of 0.5. Hence a free-water/cement ratio of 0.5 was used in the rest of the mix design.

The free-water content of the concrete mixture was determined using equation (3.6). For crushed aggregate with a maximum size of 20 mm, and a specified slump of 30-60 mm, the required free-water content was 210 kg/m^3.

A cement content of 420 kg/m^3 was obtained using equation (3.6).

$$Cement\ content = \frac{free - water\ content}{free - water / cement\ ratio} = \frac{210}{0.5} = 420\ kg/m^3 \tag{3.6}$$

Using a relative density of 2.7 and a free-water content of 210 kg/m^3, a wet density of concrete of about 2400 kg/m^3 was obtained from Figure 3.3.

Equation (3.7) gave a total aggregate content of 1770 kg/m^3.

$$Total\ aggregate\ content\ (saturated\ and\ surface - dry) = D - C - W \tag{3.7}$$

where:

D = the wet density of concrete (= 2400 kg/m^3)

C = the cement content (= 420 kg/m^3)

W = the free-water content (= 210 kg/m^3)

A fine aggregate proportion of 38% was obtained from Figure A3 using the percentage of fine aggregate passing a 600 μm sieve (found to be 40%), the maximum aggregate size (20 mm), required slump (30-60 mm) and derived free-water/cement ratio (0.5).

Finally, the fine and coarse aggregate contents were obtained using equations (3.8) and (3.9).

$$Fine\ aggregate\ content = total\ aggregate\ content \times proportion\ of\ \ fines \tag{3.8}$$

$$Coarse\ aggregate\ content = total\ aggregate\ content - \ fine\ aggregate\ content \tag{3.9}$$

Table 3.2: Mix proportions for 1 m^3 of grade 25 concrete

Mix	Water (kg)	Cement (kg)	Fine Aggregate (kg)	Coarse Aggregate (kg)	Density (kg/m^3)
B	210	420	673	1097	2400

3.2.2 Data Collection Procedure

Thirty-four (34) cylindrical concrete specimens and seventeen (17) concrete beam specimens were made and cured in accordance with ASTM C 192 – 02 for mix A, and thirty-four (34) cylindrical concrete specimens and seventeen (17) concrete beam specimens were made and cured in accordance with BS EN 12390 – 2:2000 for mix B (Plates 3.1 and 3.2).

Plate 3.1: Cylindrical concrete specimens for compressive and splitting tensile strength testing

Plate 3.2: Concrete beam specimens for flexural strength testing

The compressive strength of concrete, which is the measured maximum resistance of a concrete specimen to axial loading, was determined in accordance with ASTM C 39 – 14 for mix A, and BS EN 12390 – 3: 2002 for mix B (Plate 3.3). Three cylindrical specimens were tested at each test age and the average compressive strength was computed.

Plate 3.3: Compressive strength testing

The splitting tensile strength of concrete, which is a measure of the resistance of concrete to longitudinal stress, was determined in accordance with ASTM C 496 – 04 for mix A, and BS EN 12390 – 6: 2009 for mix B (Plate 3.4). Three cylindrical specimens were tested at each test age and the average splitting tensile strength was determined.

Plate 3.4: Splitting tensile strength testing

The flexural strength of concrete, which is a measure of the ability of concrete to resist deformation under load, was determined using a simple beam with third-point loading in accordance with ASTM C 78 – 02 for mix A, and BS EN 12390 – 5: for mix B (Plate 3.5). Three beam specimens were tested at each test age and the average flexural strength was computed.

Plate 3.5: Flexural strength testing

Maturity testing (Plate 3.6) was done in accordance with ASTM C 1074 – 04 for mix A, and mix B. For each concrete mixture, temperature sensors were embedded in the centres of three cylindrical specimens and three beam specimens as soon as practicable after the specimens were made. The temperature sensors were immediately connected to data loggers which recorded the temperature of the concrete specimens at intervals of 0.5h.

Plate 3.6: Maturity testing

3.2.3 Data Analysis

The 1-day, 3-day, 7-day, and 14-day strengths of each mix were computed as percentages of the 28-day design strength so as to ascertain consistency with previous research on concrete strength development. For all instrumented specimens the internal temperature of concrete was recorded at intervals of 0.5h. The recorded values were used to calculate maturity, which was then presented in tabular form (Table 4.3).

By plotting the observed strength of mix A (the control mix) against the corresponding maturity values, a function which produced a best-fit curve was developed. A relationship between strength (y) and maturity (x) was then expressed in the form of an equation. This equation was used to compute compressive strength, against which the compressive strength of mix B was compared. The maximum deviation of the observed compressive strength of mix B from the estimated compressive strength was then determined.

3.3 Assessing the Applicability of the Maturity Method to Concrete Containing a Plasticizer

3.3.1 Experimental Setup

3.3.1.1 Properties of Aggregates

Sieve analyses of fine and coarse aggregates were done in accordance with ASTM C 136 – 96a for mix A, and BS 812 – Part 103.1:1985 for mix C. The unit weight of aggregate was determined in accordance with ASTM C 29 – 03 for mix A, and BS 812 – 2: 1995 for mix C. The specific gravity and absorption of fine aggregate was determined in accordance with ASTM C 128 – 97 for mix A, and BS 812 – 2: 1995 for mix C. The specific gravity was calculated on the basis of saturated surface-dry fine aggregate. Water absorption was calculated as a percentage of dry mass. The specific gravity and water absorption of coarse aggregate was determined in accordance with ASTM C 127 – 93 for mix A, and BS 812 – 2: 1995 for mix C. The specific gravity was calculated on the basis of saturated surface-dry coarse aggregate, and water absorption was calculated as a percentage of dry mass.

3.3.1.2 Concrete Mix Proportioning

Concrete mix proportioning was done in accordance with the American Concrete Institute (ACI) mix design procedure (ACI 211.1 - 91) for mix A, and the United Kingdom Building Research Establishment (BRE) mix design procedure for mix C. Modified lignosulphonate (Sika Plastiment BV-40), a locally available plasticizer, was added to mix C at a dosage of 0.2% by weight of cement as recommended by the manufacturer. The plasticizer was dispersed in the mixing water before addition. The slump value of mix C was determined in accordance with BS 1881 – 102:1983.

Table 3.3: Mix proportions for 1 m^3 of grade 25 concrete

Mix	Water (kg)	Cement (kg)	Fine Aggregate (kg)	Coarse Aggregate (kg)	Plasticizer (kg)	Density (kg/m^3)
A	184	307	816	1035	-	2342
C	210	420	673	1097	0.84	2400.84

3.3.2 Data Collection Procedure

Thirty-four (34) cylindrical concrete specimens and seventeen (17) concrete beam specimens were made and cured in accordance with ASTM C 192 – 02 for mix A, and thirty-four (34) cylindrical concrete specimens and seventeen (17) concrete beam specimens were made and cured in accordance with BS EN 12390 – 2:2000 for mix C (Plates 3.1 and 3.2).

The compressive strength of concrete, which is the measured maximum resistance of a concrete specimen to axial loading, was determined in accordance with ASTM C 39 – 14 for mix A, and BS EN 12390 – 3: 2002 for mix C (Plate 3.3). Three cylindrical specimens were tested at each test age and the average compressive strength was computed.

The splitting tensile strength of concrete, which is a measure of the resistance of concrete to longitudinal stress, was determined in accordance with ASTM C 496 – 04 for mix A, and BS EN 12390 – 6: 2009 for mix C (Plate 3.4). Three cylindrical specimens were tested at each test age and the average splitting tensile strength was determined.

The flexural strength of concrete, which is a measure of the ability of concrete to resist deformation under load, was determined using a simple beam with third-point loading in accordance with ASTM C 78 – 02 for mix A, and BS EN 12390 – 5: for mix C (Plate 3.5). Three beam specimens were tested at each test age and the average flexural strength was computed.

Maturity testing (Plate 3.6) was done in accordance with ASTM C 1074 – 04 for mix A, and mix C. For each concrete mixture, temperature sensors were embedded in the centres of three cylindrical specimens and three beam specimens as soon as practicable after the specimens were made. The temperature sensors were immediately connected to data loggers which recorded the temperature of the concrete specimens at intervals of 0.5h.

3.3.3 Data Analysis

The 1-day, 3-day, 7-day, and 14-day strengths of each mix were computed as percentages of the 28-day design strength so as to ascertain consistency with previous research on concrete strength development. For all instrumented specimens the internal temperature of concrete was recorded at intervals of 0.5h. The recorded values were used to calculate maturity, which was then presented in tabular form (Table 4.3).

By plotting the observed strength of mix A against the corresponding maturity values, a function which produced a best-fit curve was developed. A relationship between strength (y) and maturity (x) was then expressed in the form of an equation. This equation was used to compute compressive strength, against which the compressive strength of mix C was compared. The maximum deviation of the observed compressive strength of mix C from the estimated compressive strength was then determined.

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1 Properties of Aggregates

For both American standards (ASTM C 33 – 03) and British standards (BS 882: 1992), locally available river sand and coarse aggregate conformed to the grading requirements for suitability of use as aggregates (Figure 4.1 and Figure 4.2). The grading of aggregates affects the relative aggregate proportions as well as cement and water requirements. Aggregates that have a uniform distribution of particle sizes produce a workable concrete mixture (Kosmatka, 2008).

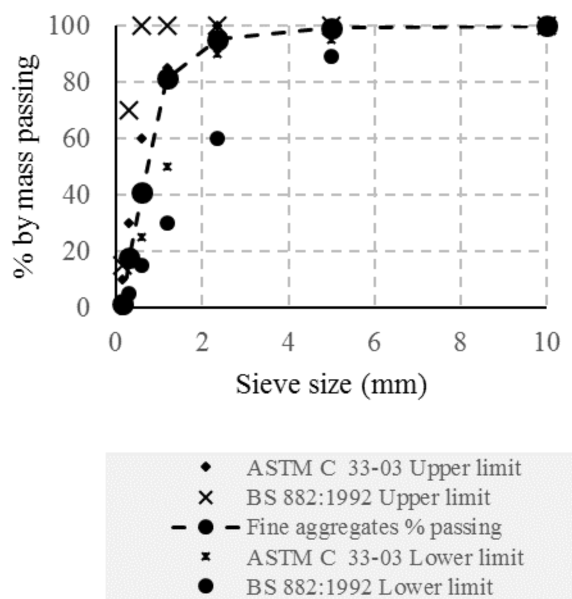

Figure 4.1: Fine aggregate particle size distribution

In addition, the fineness modulus of the fine aggregate used in this research was found to be 2.6 (Table 4.1). This value was used to determine the volume of coarse aggregate to be used per unit volume of concrete prepared according to American standards. Fine aggregates with a fineness modulus of 2.5 and under will produce concrete with low compressive strength (Kosmatka, 2008). Hence the selected river sand was suitable for use as fine aggregate.

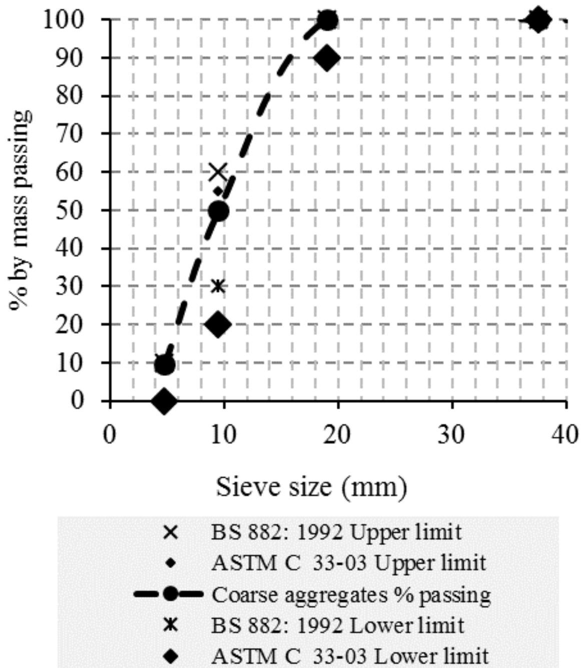

Figure 4.2: Coarse aggregate particle size distribution

Table 4.1: Properties of aggregates

Type of aggregate	Fineness modulus	Unit weight (kg/m^3)	Specific gravity	Water absorption (%)
Fine aggregate	2.6	1500	2.7	3.4
Coarse aggregate	5.3	1600	2.7	3.2

Unit weight, specific gravity and water absorption of fine and coarse aggregates were found to be approximately the same regardless of the standards used (Table 4.1). The unit weight of aggregate was used to determine the weight of aggregate to be used per unit volume of concrete. The approximate unit weight of aggregates commonly used in concrete ranges from about 1120 to 1760 kg/m^3 (Kosmatka, 2008).The unit weight of the aggregates used in this research was within this range (Table 4.1). The specific gravity of aggregate was used to calculate the volume that the aggregate would occupy in the concrete mixture. Most natural aggregates have specific gravities of between 2.4 and 2.9 (Kosmatka, 2008). Therefore, the selected river sand and ballast were suitable for use as fine and coarse aggregate respectively.

The specific gravity of the aggregates used in this research was found to be 2.7 (Table 4.1). The water absorption of aggregate was used to calculate the change in the mass of the aggregate due to water absorbed in the pore spaces within the constituent particles. The amount of water used in the concrete mixture was adjusted for the moisture conditions of the aggregates to meet the designated water requirement (Kosmatka, 2008). The aggregates used in this study met all the acceptance criteria of both American and British standards.

A slump value of 35 mm was obtained for mixes A and B. Mix C, to which a plasticizer was added, achieved a slump value of 45 mm. Addition of a plasticizer enhances the workability of concrete. In mix C, the focus was not so much on the effect of the plasticizer on workability as it was on the presence of the plasticizer and how it affected the maturity of the resulting mix.

4.2 Investigating the Applicability of the Maturity Method to Concrete Prepared According to British Standards

The compressive strengths of mixes A, and B were found to be approximately equal at each test age (Table 4.2).

Table 4.2: Compressive strength (N/mm^2) of grade 25 concrete

Mix	Age of concrete (days)				
	1	3	7	14	28
A	3.8	9.1	16.5	21.7	24.7
B	4.0	9.5	16.8	22.3	24.9

Comparable concretes generally provide the same strengths with identical water-cement ratios regardless of the concrete composition (Popovics and Ujhelyi, 2008). The 28-day compressive strength of mix A was about 99% of the design compressive strength (25 N/mm^2) (Table 4.2).

The 1-day, 3-day, 7-day, and 14-day compressive strengths of mix A were found to be about 15, 37, 67, and 88% of the 28-day design strength respectively (Table 4.2). For mix B, the 28-day compressive strength was found to be approximately 100% of the design compressive strength (25 N/mm^2) (Table 4.2). The 1-day, 3-day, 7-day, and 14-day compressive strengths of mix B were found to be about 16, 38, 67, and 90% of the 28-day design strength respectively (Table 4.2). The compressive strength development of each mix was consistent with previous research on concrete strength development (Kosmatka *et al.*, 2003). Failure to achieve the exact 28-day compressive strength was likely due to errors in batching processes.

For all instrumented specimens, the maturity of concrete was computed using equation (2.1) and found to be the same at each test age (Table 4.3). The internal temperature of concrete was not affected by the composition of the mix. The maturity values for each mix may be found in the appendix (Tables A16 – A21). Because the surface area-to-volume ratio of the cylindrical and beam specimens used in this study was the same (= 0.03), the measured internal temperature of all specimens was the same (ASTM C 1074 – 04).

Table 4.3: Maturity of cylindrical concrete specimens prepared from control mix (A)

Age (h)	Temp. (ºC)	Average Temp. (ºC)	Maturity Increment (ºC-h)	Cumulative Maturity (ºC-h)
0	22.5	-	-	0
0.5	22.5	22.5	11.25	11.25
1	23.32	22.92	11.46	22.71
...	...	...	...	...
24	21.4	22.94	11.47	548.75
...	...	...	...	...
72	21.28	22.92	11.46	1643.36
...	...	...	...	...
168	24.76	23	11.5	3833.61
...	...	...	...	...
336	21.12	23.02	11.51	7670.96
...	...	...	...	...
672	21.72	22.52	11.26	15364.61

By plotting the observed compressive strength of mix A (the control mix) against the corresponding maturity values, a function which produced a best-fit curve was developed (Figure 4.3).

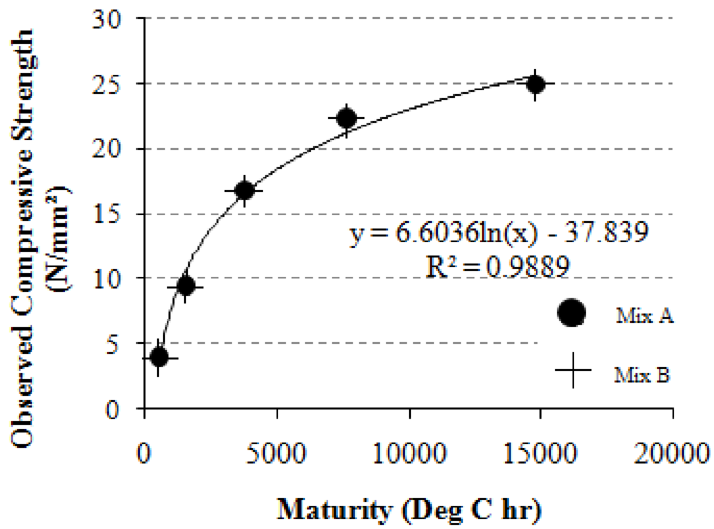

Figure 4.3: Calculated compressive strength-maturity relationship curve

The compressive strength of each mix increased with increasing maturity. The relationship between compressive strength (y) and maturity (x) was expressed in the form of an equation (equation (4.1)).

$$y = 6.6036 \ln (x) - 37.839 \tag{4.1}$$

Equation (4.1) was used to compute compressive strength, against which the compressive strength of mix B (Figure 4.4) was compared.

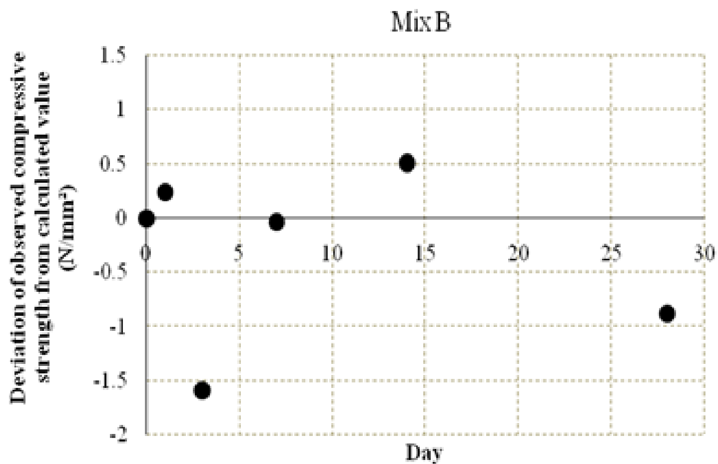

Figure 4.4: Deviation of observed compressive strength of mix B from calculated compressive strength

The maximum deviation of the observed compressive strength of mix B from the estimated compressive strength was found to be only 1.6 N/mm^2 (about 17% of the estimated value) (Figure 4.4). Therefore, the maturity method gives an accurate estimate of the compressive strength of concrete irrespective of the choice of concrete mix design standard.

The 1-day, 3-day, 7-day, 14-day, and 28-day splitting tensile strengths of mix A were found to be about 11, 11, 10, 14, and 13% of the corresponding compressive strengths respectively (Tables 4.2 and 4.4). For mix B, the 1-day, 3-day, 7-day, 14-day, and 28-day splitting tensile strengths were found to be about 13, 12, 11, 14, and 13% of the corresponding compressive strengths respectively (Tables 4.2 and 4.4).

Table 4.4: Splitting tensile strength (N/mm^2) of grade 25 concrete

Mix	Age of concrete (days)				
	1	3	7	14	28
A	0.4	1	1.6	3	3.1
B	0.5	1.1	1.8	3.2	3.3

The splitting tensile strengths of mixes A and B were almost identical. This may be attributed to the fact that the two mixes had approximately the same water-cement ratios. The effect of using different standards was insignificant. In addition, values of splitting tensile strength were within the prescribed range of 8 to 14% of corresponding values of compressive strength (Kosmatka *et al.*, 2003).

The maturity of mixes A and B, which was computed using equation (2.1), was the same at each test age. The maturity values for each of the three mixes used in this study may be found in the appendix (Tables A16 – A21). Mix composition had no effect on the internal temperature of concrete. For all mixes, splitting tensile strength increased with increasing maturity. The splitting tensile strength of mix A was plotted against corresponding maturity values and a best-fit curve was drawn through the data (Figure 4.5).

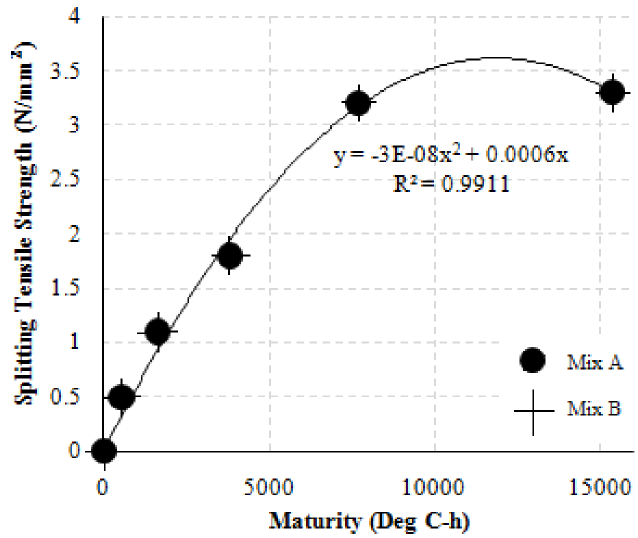

Figure 4.5: Calculated splitting tensile strength-maturity relationship curve

Splitting tensile strength (y) was related to maturity (x) according to equation (4.2).

$$y = -3E - 08x^2 + 0.0006x \qquad (4.2)$$

Equation (4.2) was used to calculate values of splitting tensile strength, against which the splitting tensile strength of mix B (Figure 4.6) was compared. The maximum deviation of mix B from the estimated values was found to be only 0.9 N/mm^2 (about 9% of the estimated value). Hence the maturity method provides a satisfactory estimate of the splitting tensile strength of concrete regardless of the mix design standards used.

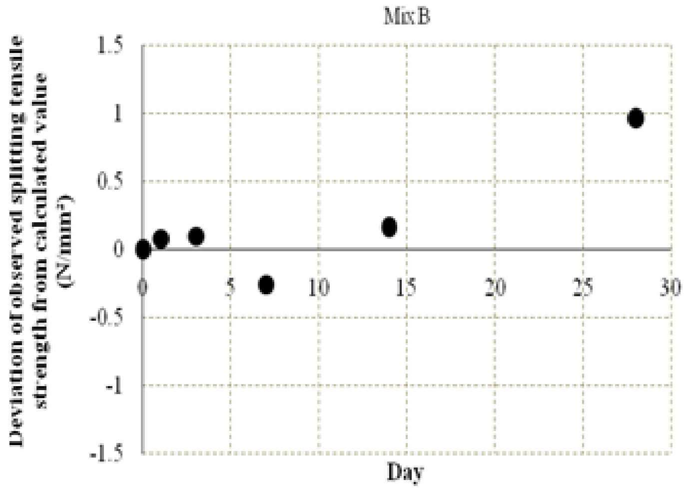

Figure 4.6: Deviation of observed splitting tensile strength of mix B from calculated splitting tensile strength

The 1-day, 3-day, 7-day, 14-day, and 28-day flexural strengths of mix A were found to be about 16, 11, 13, 14, and 13% of the corresponding compressive strengths respectively (Tables 4.2 and 4.5). For mix B, the 1-day, 3-day, 7-day, 14-day, and 28-day flexural strengths were found to be about 18, 13, 14, 15, and 14% of the corresponding compressive strengths respectively (Tables 4.2 and 4.5). Because mixes A and B had approximately the same water-cement ratios, the flexural strengths of the three mixes were not significantly different. The flexural strength of concrete was not affected by the use of different standards, or the addition of a plasticizer.

Table 4.5: Flexural strength (N/mm^2) of grade 25 concrete

Mix	Age of concrete (days)				
	1	3	7	14	28
A	0.6	1	2.1	3.1	3.2
B	0.7	1.2	2.4	3.3	3.5

Mixes A and B had the same maturity at each test age. The maturity values for each of the three mixes used in this study may be found in the appendix. The effect of concrete mix composition on the internal temperature of the concrete was not significant. The flexural strength and maturity of mix A were used to obtain the curve shown in Figure 4.8. Flexural strength (y) was related to maturity (x) according to equation (4.3).

$$y = -3E - 08x^2 + 0.0007x \tag{4.3}$$

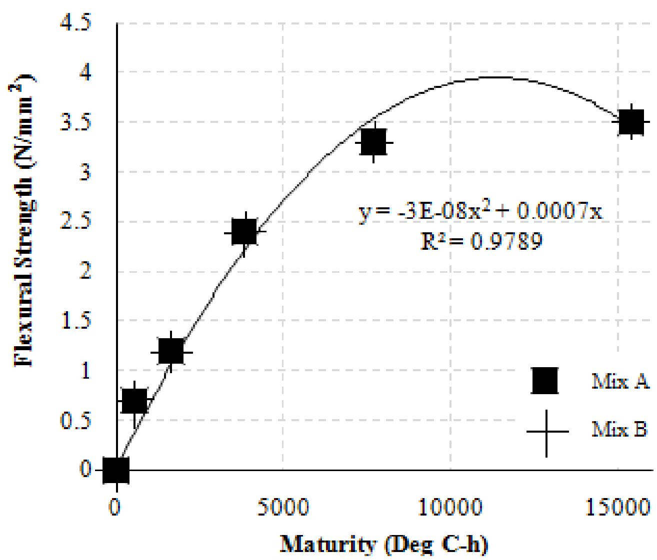

Figure 4.7: Calculated flexural strength-maturity relationship curve

Equation (4.3) was used to calculate values of flexural strength, against which the flexural strength of mix B (Figure 4.7) was compared. The maximum deviation of mix B was found to be only 0.5 N/mm^2 (about 15% of the estimated value) (Figure 4.8).

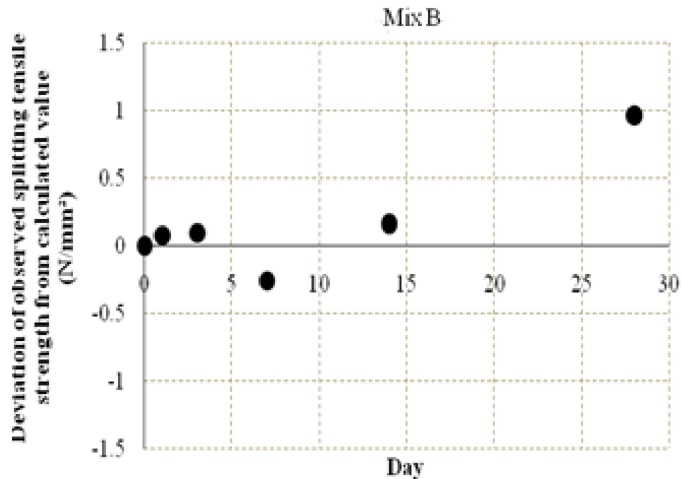

Figure 4.8: Deviation of observed flexural strength of mix B from calculated flexural strength

Clearly, the maturity method may be used to estimate the flexural strength of concrete and the results are independent of the mix design standards used.

4.3 Assessing the Applicability of the Maturity Method to Concrete Containing a Plasticizer

The compressive strengths of mixes A and C were found to be approximately equal at each test age (Table 4.6).

Table 4.6: Compressive strength (N/mm^2) of grade 25 concrete

Mix	Age of concrete (days)				
	1	3	7	14	28
A	3.8	9.1	16.5	21.7	24.7
C	3.9	9.7	17	22.1	24.8

Comparable concretes generally provide the same strengths with identical water-cement ratios regardless of the concrete composition (Popovics & Ujhelyi, 2008). The 1-day, 3-day, 7-day, and 14-day compressive strengths of mix A were found to be about 15, 37, 67, and 88% of the 28-day design strength respectively (Table 4.2). For mix C, the 28-

day compressive strength was found to be about 99% of the design compressive strength (25 N/mm^2) (Table 4.2). The 1-day, 3-day, 7-day, and 14-day compressive strengths of mix C were found to be about 16, 39, 69, and 89% of the 28-day design strength respectively (Table 4.2). The compressive strength development of each mix was consistent with previous research on concrete strength development (Kosmatka *et al.*, 2003). Failure to achieve the exact 28-day compressive strength was likely due to errors in batching processes.

For all instrumented specimens, at each test age, the maturity of concrete was the same. This is because the internal temperature of concrete was not affected by the composition of the mix. The maturity values for each mix may be found in the appendix (Tables A16 – A21).

By plotting the observed compressive strength of mix A against the corresponding maturity values, a function which produced a best-fit curve was developed (Figure 4.9).

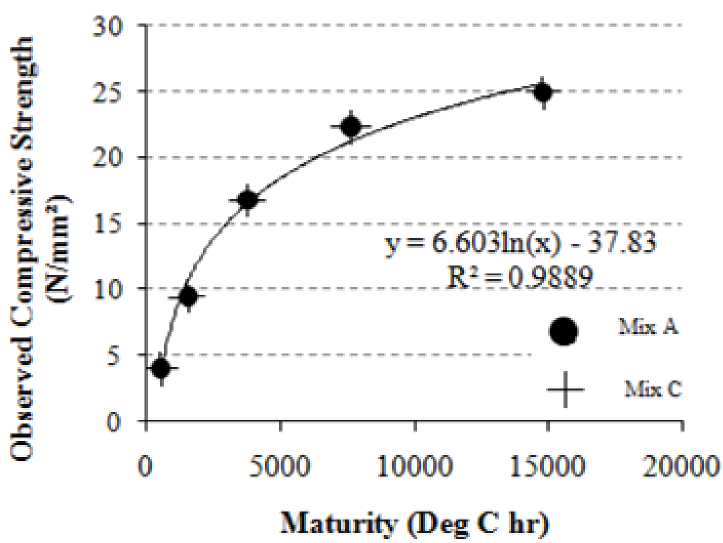

Figure 4.9: Calculated compressive strength-maturity relationship curve

The compressive strength of each mix increased with increasing maturity. The relationship between compressive strength (y) and maturity (x) was expressed in the form of an equation (equation (4.1)).

$$y = 6.603 \ln (x) - 37.83 \tag{4.1}$$

Equation (4.1) was used to compute compressive strength, against which the compressive strength of mix C (Figure 4.10) was compared.

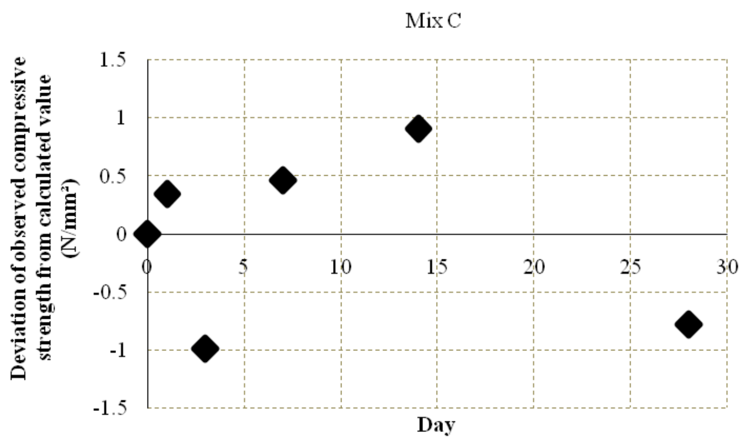

Figure 4.10: Deviation of observed compressive strength of mix C from calculated compressive strength

The maximum deviation of the observed compressive strength of mix C from the estimated compressive strength was found to be only 1.0 N/mm^2 (about 11% of the estimated value) (Figure 4.10). Therefore, the maturity method gives an accurate estimate of the compressive strength of concrete irrespective of the presence of modified lignosulphonate (Sika Plastiment BV-40), a locally available plasticizer.

The 1-day, 3-day, 7-day, 14-day, and 28-day splitting tensile strengths of mix A were found to be about 11, 11, 10, 14, and 13% of the corresponding compressive strengths respectively (Tables 4.2 and 4.7). For mix C, the 1-day, 3-day, 7-day, and 14-day splitting tensile strengths were found to be about 15, 10, 11, 14, and 13% of the corresponding compressive strengths respectively (Tables 4.6 and 4.7).

Table 4.7: Splitting tensile strength (N/mm^2) of grade 25 concrete

Mix	Age of concrete (days)				
	1	3	7	14	28
A	0.4	1	1.6	3	3.1
C	0.6	1	1.8	3.1	3.2

The splitting tensile strengths of mixes A and C were not significantly different. This may be attributed to the fact that the two mixes had approximately the same water-cement ratios. The effect of adding a plasticizer was insignificant. In addition, values of splitting tensile strength were within the prescribed range of 8 to 14% of corresponding values of compressive strength (Kosmatka *et al.*, 2003).

The maturity of mixes A and C, which was computed using equation (2.1), was the same at each test age. The maturity values for each of the three mixes used in this study may be found in the appendix. Mix composition had no effect on the internal temperature of concrete. For all mixes, splitting tensile strength increased with increasing maturity. The splitting tensile strength of mix A was plotted against corresponding maturity values and a best-fit curve was drawn through the data (Figure 4.11).

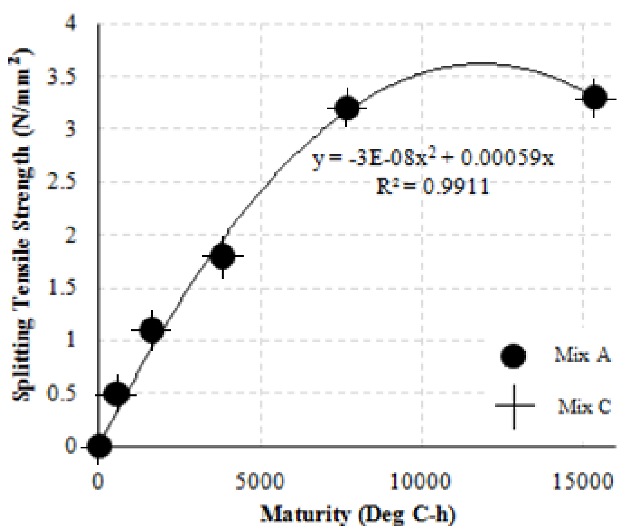

Figure 4.11: Calculated splitting tensile strength-maturity relationship curve

Splitting tensile strength (y) was related to maturity (x) according to equation (4.2).

$$y = -3E - 08x^2 + 0.00059x \qquad (4.2)$$

Equation (4.2) was used to calculate values of splitting tensile strength, against which the splitting tensile strength of mix C (Figure 4.12) was compared. The maximum deviation of mix C from the estimated values was found to be only 0.25 N/mm^2 (about 23% of the estimated value) respectively. Hence the maturity method provides a satisfactory estimate of the splitting tensile strength of concrete regardless of the addition of modified lignosulphonate (Sika Plastiment BV-40).

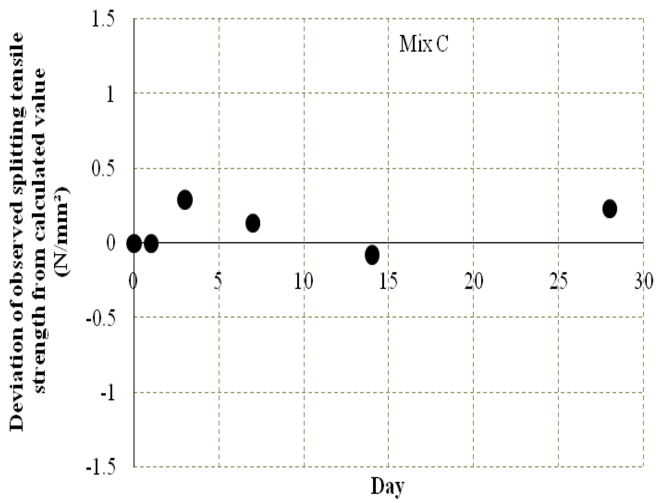

Figure 4.12: Deviation of observed splitting tensile strength of mix C from calculated splitting tensile strength

The 1-day, 3-day, 7-day, 14-day, and 28-day flexural strengths of mix A were found to be about 16, 11, 13, 14, and 13% of the corresponding compressive strengths respectively (Tables 4.2 and 4.8). For mix C, the 1-day, 3-day, 7-day, and 14-day flexural strengths were found to be about 15, 12, 14, 15, and 14% of the corresponding compressive strengths respectively (Tables 4.6 and 4.8). Because mixes A and C had approximately the same water-cement ratios, the flexural strengths of the three mixes were not significantly different. The flexural strength of concrete was not affected by the addition of a plasticizer.

Table 4.8: Flexural strength (N/mm^2) of grade 25 concrete

Mix	Age of concrete (days)				
	1	3	7	14	28
A	0.6	1	2.1	3.1	3.2
C	0.6	1.2	2.3	3.4	3.5

Mixes A and C had the same maturity at each test age. The maturity values for each mix may be found in the appendix. The effect of concrete mix composition on the internal temperature of the concrete was not significant. The flexural strength and maturity of mix B were used to obtain the curve shown in Figure 4.14. Flexural strength (y) was related to maturity (x) according to equation (4.3).

$$y = -3E - 08x^2 + 0.0007x \qquad (4.3)$$

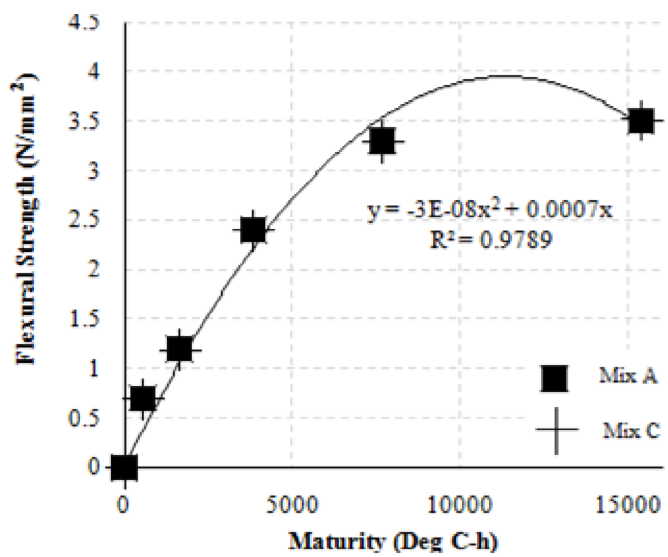

Figure 4.13: Calculated flexural strength-maturity relationship curve

Equation (4.3) was used to calculate values of flexural strength, against which the flexural strength of mix C (Figure 4.14) was compared. The maximum deviation of mix C was found to be only 0.25 N/mm^2 (about 36% of the estimated value) (Figure 4.14).

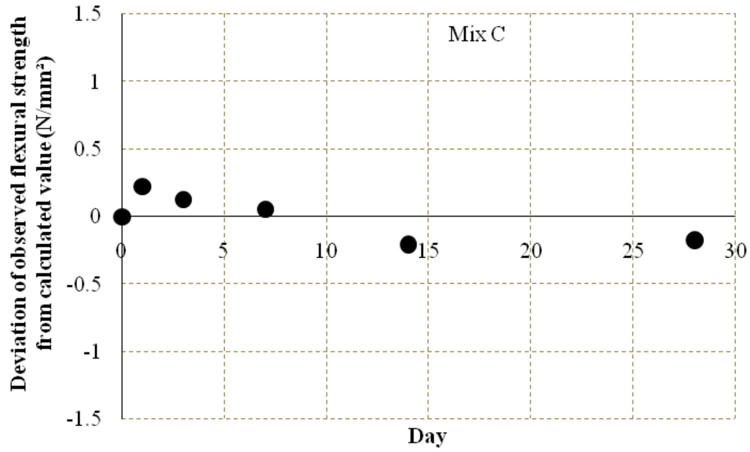

Figure 4.14: Deviation of observed flexural strength of mix C from calculated flexural strength

Clearly, the maturity method may be used to estimate the flexural strength of concrete and the results are independent of the presence of modified lignosulphonate (Sika Plastiment BV-40).

CHAPTER FIVE: CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The aim of this research was to assess the applicability of the maturity method as a means of estimating concrete strength. The following conclusions were drawn:

1. The maturity method is applicable to concrete prepared according to British standards.
2. The maturity method may be applied to concrete containing modified lignosulphonate (Sika Plastiment BV-40), a locally available plasticizer.

5.1.1 Recommendations

In order to prevent catastrophic collapse of concrete structures during/after construction in Kenya, it is recommended that the maturity method be used to determine concrete strength, particularly before strength-dependent activities such as removal of formwork are performed.

5.1.2 Further Work

It is recommended that further research be conducted on the applicability of the maturity method to concrete prepared using different types of cement and/or concrete containing different admixtures.

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APPENDIX

Table A1: Approximate mixing water requirements for different slumps and maximum sizes of aggregates

(Source: ACI 211.1 – 91)

	Water (lb/yd^3 of Concrete for Indicated Nominal Maximum Sizes of Aggregate)							
Slump (in.)	3/8 in.	1/2 in.	3/4 in.	1 in.	1-1/2 in.	2 in.	3 in.	6 in.
Non-air-entrained concrete								
1 to 2	350	335	315	300	275	260	220	190
3 to 4	385	365	340	325	300	285	245	210
6 to 7	410	385	360	340	315	300	270	—
Approximate amount of entrapped air in non-air-entrained concrete (%)	3	2.5	2	1.5	1	0.5	0.3	0.2
Air-entrained concrete								
1 to 2	305	295	280	270	250	240	205	180
3 to 4	340	325	305	295	275	265	225	200
6 to 7	365	345	325	310	290	280	260	—
Recommended average total air content (percent for level of exposure)								
Mild exposure	4.5	4.0	3.5	3.0	2.5	2.0	1.5	1.0
Moderate exposure	6.0	5.5	5.0	4.5	4.5	4.0	3.5	3.0
Extreme exposure	7.5	7.0	6.0	6.0	5.5	5.0	4.5	4.0

Table A2: Relationship between water-cement ratio and compressive strength of concrete

(Source: ACI 211.1 – 91)

Compressive Strength at 28 Days (psi)	Water/Cement Ratio (by weight)	
	Non-Air-Entrained Concrete	Air-Entrained Concrete
6000	0.41	—
5000	0.48	0.40
4000	0.57	0.48
3000	0.68	0.59
2000	0.82	0.74

Table A3: Volume of coarse aggregate per unit volume of concrete

(Source: ACI 211.1 – 91)

Maximum Size of Aggregate (in.)	Volume of Dry-Rodded Coarse Aggregate per Unit Volume of Concrete for Different Fineness Moduli of Sand			
	2.40	2.60	2.80	3.00
3/8	0.50	0.48	0.46	0.44
1/2	0.59	0.57	0.55	0.53
3/4	0.66	0.64	0.62	0.60
1	0.71	0.69	0.67	0.65
1 1/2	0.75	0.73	0.71	0.69
2	0.78	0.76	0.74	0.72
3	0.82	0.80	0.78	0.76
6	0.87	0.85	0.83	0.81

Table A4: First estimate of weight of fresh concrete

(Source: ACI 211.1 – 91)

Maximum Size of Aggregate (in.)	First Estimate of Concrete Weight (lb/yd^3)	
	Non-Air-Entrained Concrete	Air-Entrained Concrete
3/8	3840	3690
1/2	3890	3760
3/4	3960	3840
1	4010	3900
1-1/2	4070	3960
2	4120	4000
3	4160	4040
6	4230	4120

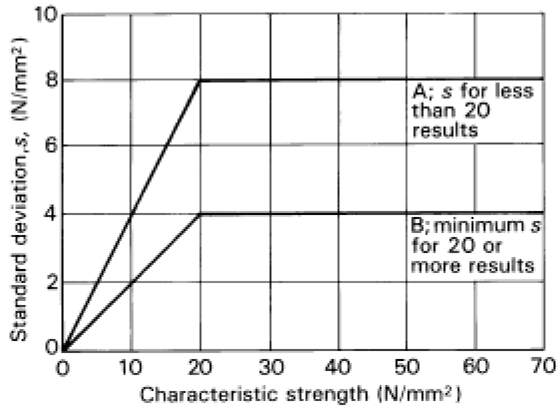

Figure A1: Relationship between standard deviation and characteristic strength

(Source: BRE, 1997)

Table A5: Approximate compressive strengths of concrete mixes made with a free-water/cement ratio of 0.5

(Source: BRE, 1997)

Cement strength class	Type of coarse aggregate	Compressive strengths (N/mm^2) Age (days) 3	7	28	91
42.5	Uncrushed	22	30	42	49
	Crushed	27	36	49	56
52.5	Uncrushed	29	37	48	54
	Crushed	34	43	55	61

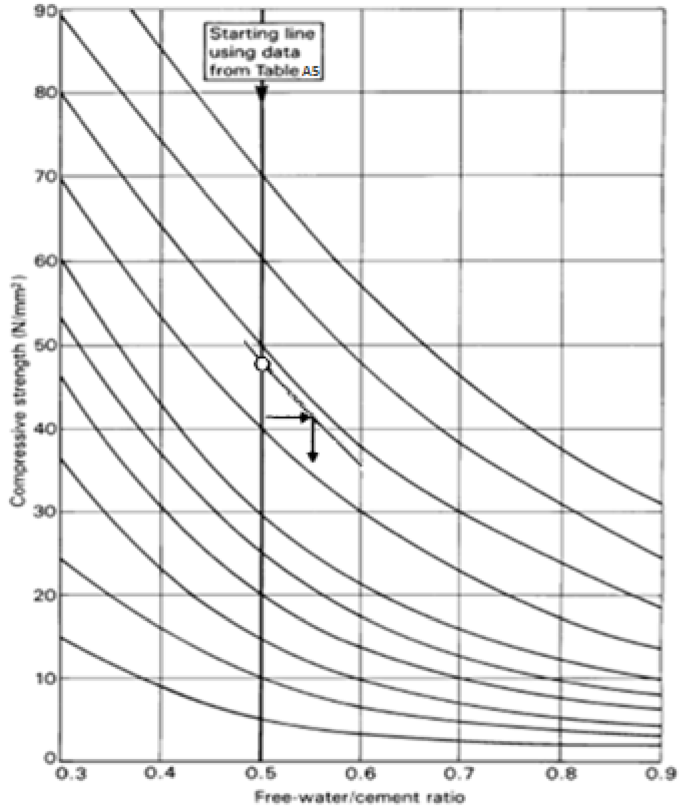

Figure A2: Relationship between compressive strength and free-water/cement ratio (Source: BRE, 1997)

Table A6: Approximate free-water contents (kg/m^3) required to give various levels of workability

(Source: BRE, 1997)

Slump (mm)		0–10	10–30	30–60	60–180
Vebe time (s)		>12	6–12	3–6	0–3
Maximum size of aggregate (mm)	Type of aggregate				
10	Uncrushed	150	180	205	225
	Crushed	180	205	230	250
20	Uncrushed	135	160	180	195
	Crushed	170	190	210	225
40	Uncrushed	115	140	160	175
	Crushed	155	175	190	205

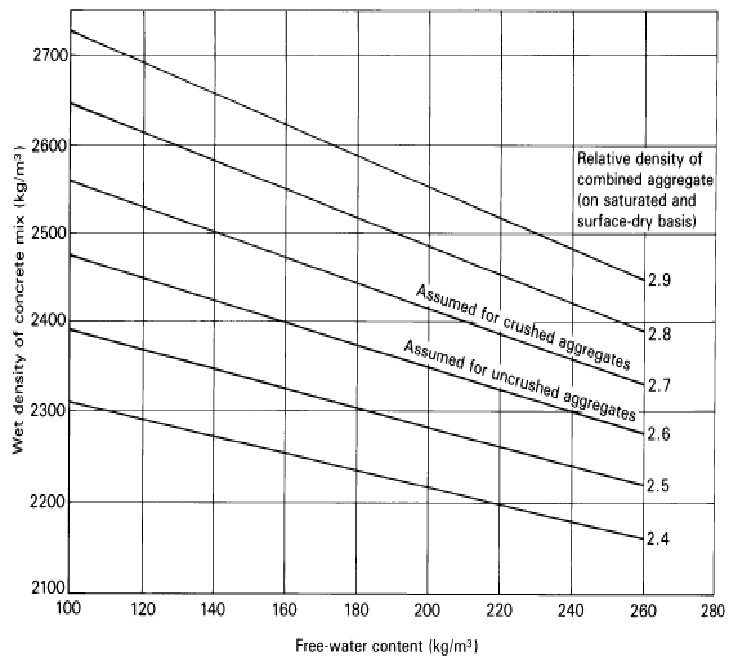

Figure A3: Estimated wet density of fully compacted concrete

(Source: BRE, 1997)

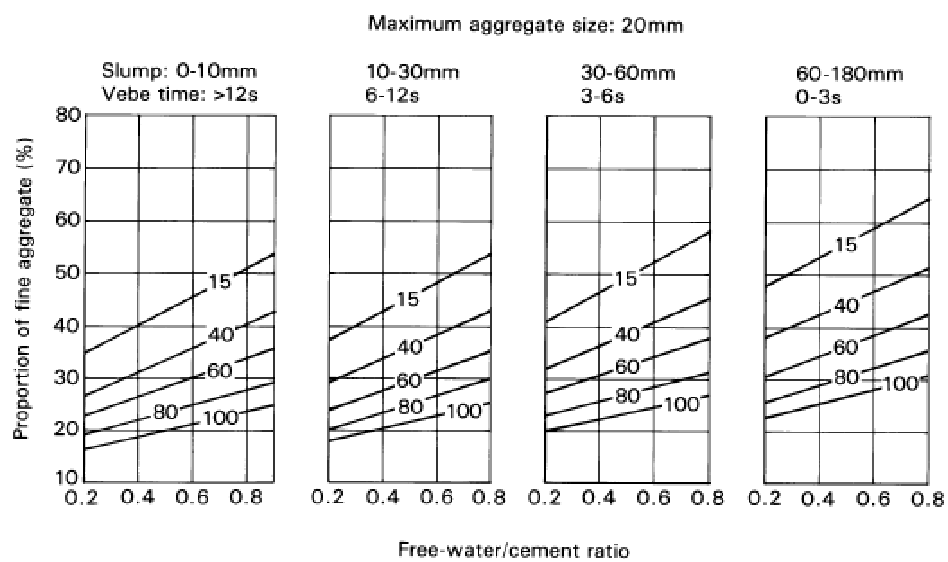

Figure A4: Recommended proportions of fine aggregate according to percentage passing a 600 µm sieve

(Source: BRE, 1997)

Table A7: Compressive strength of cylindrical concrete specimens for mix A

Specimen number	Age (days)	Cylinder weight (kg)	Dimensions (mm by mm)	Cylinder density (kg/m^3)	Cylinder area (mm^2)	Failure load (N)	Compressive strength (N/mm^2)	Average compressive strength (N/mm^2)
S1-1	1	12.410	150 x 300	2339.933	17 678.571	66 665.891	3.771	3.8
S2-1	1	12.407	150 x 300	2339.367	17 678.571	65 675.891	3.715	
S3-1	1	12.419	150 x 300	2341.630	17 678.571	69 317.677	3.921	
S1-3	3	12.421	150 x 300	2342.007	17 678.571	160 645.175	9.087	9.1
S2-3	3	12.420	150 x 300	2341.818	17 678.571	158 682.853	8.976	
S3-3	3	12.431	150 x 300	2343.892	17 678.571	161 546.782	9.138	
S1-7	7	12.398	150 x 300	2337.670	17 678.571	289 645.707	16.384	16.5
S2-7	7	12.409	150 x 300	2339.744	17 678.571	291 572.672	16.493	
S3-7	7	12.415	150 x 300	2340.875	17 678.571	292 085.350	16.522	
S1-14	14	12.390	150 x 300	2336.162	17 678.571	369 093.205	20.878	21.7
S2-14	14	12.430	150 x 300	2343.704	17 678.571	402 063.740	22.743	
S3-14	14	12.411	150 x 300	2340.121	17 678.571	381 980.884	21.607	
S1-28	28	12.403	150 x 300	2338.613	17 678.571	431 038.918	24.382	24.7
S2-28	28	12.422	150 x 300	2342.195	17 678.571	438 764.454	24.819	
S3-28	28	12.434	150 x 300	2344.458	17 678.571	440 638.382	24.925	

Table A8: Splitting tensile strength of cylindrical concrete specimens for mix A

Specimen number	Age (days)	Cylinder weight (kg)	Dimensions (mm by mm)	Cylinder density (kg/m^3)	Cylinder area (mm^2)	Failure load (N)	Tensile strength (N/mm^2)	Average tensile strength (N/mm^2)
S1-1	1	12.420	150 x 300	2341.818	70 714.286	29 912.143	0.423	
S2-1	1	12.412	150 x 300	2340.310	70 714.286	28 144.286	0.398	0.4
S3-1	1	12.404	150 x 300	2338.801	70 714.286	27 366.429	0.387	
S1-3	3	12.417	150 x 300	2341.253	70 714.286	68 522.143	0.969	
S2-3	3	12.429	150 x 300	2343.515	70 714.286	78 068.572	1.104	1.0
S3-3	3	12.421	150 x 300	2342.007	70 714.286	74 532.857	1.054	
S1-7	7	12.423	150 x 300	2342.384	70 714.286	116 678.572	1.650	
S2-7	7	12.379	150 x 300	2334.088	70 714.286	105 930.000	1.498	1.6
S3-7	7	12.411	150 x 300	2340.121	70 714.286	112 577.143	1.592	
S1-14	14	12.409	150 x 300	2339.744	70 714.286	212 213.572	3.001	
S2-14	14	12.402	150 x 300	2338.424	70 714.286	205 000.715	2.899	3.0
S3-14	14	12.432	150 x 300	2344.081	70 714.286	218 577.858	3.091	
S1-28	28	12.430	150 x 300	2343.704	70 714.286	222 962.144	3.153	
S2-28	28	12.419	150 x 300	2341.630	70 714.286	219 285.001	3.101	3.1
S3-28	28	12.398	150 x 300	2337.670	70 714.286	211 506.429	2.991	

Table A9: Flexural strength of concrete beam specimens for mix A

Specimen number	Age (days)	Beam weight (kg)	Dimensions (mm by mm by mm)	Beam density (kg/m^3)	Beam area (mm^2)	Failure load (N)	Flexural strength (N/mm^2)	Average flexural strength (N/mm^2)
S1-1	1	28.02	150 x 150 x 530	2349.686	7 500	5272.5	0.703	
S2-1	1	27.97	150 x 150 x 530	2345.493	7 500	5002.5	0.667	0.6
S3-1	1	27.91	150 x 150 x 530	2340.461	7 500	3757.5	0.501	
S1-3	3	27.93	150 x 150 x 530	2342.138	7 500	7582.5	1.011	
S2-3	3	27.89	150 x 150 x 530	2338.784	7 500	6592.5	0.879	1.0
S3-3	3	27.90	150 x 150 x 530	2339.623	7 500	7312.5	0.975	
S1-7	7	28.01	150 x 150 x 530	2348.847	7 500	16 147.5	2.153	
S2-7	7	27.95	150 x 150 x 530	2343.816	7 500	15 780.0	2.104	2.1
S3-7	7	27.92	150 x 150 x 530	2341.300	7 500	14 932.5	1.991	
S1-14	14	27.87	150 x 150 x 530	2337.107	7 500	21 577.5	2.877	
S2-14	14	27.92	150 x 150 x 530	2341.300	7 500	23 160.0	3.088	3.1
S3-14	14	28.10	150 x 150 x 530	2356.394	7 500	24 007.5	3.201	
S1-28	28	27.89	150 x 150 x 530	2338.784	7 500	23 182.5	3.091	
S2-28	28	27.93	150 x 150 x 530	2342.138	7 500	24 022.5	3.203	3.2
S3-28	28	27.91	150 x 150 x 530	2340.461	7 500	23 947.5	3.193	

Table A10: Compressive strength of cylindrical concrete specimens for mix B

Specimen number	Age (days)	Cylinder weight (kg)	Dimensions (mm by mm)	Cylinder density (kg/m^3)	Cylinder area (mm^2)	Failure load (N)	Compressive strength (N/mm^2)	Average compressive strength (N/mm^2)
S1-1	1	12.721	150 x 300	2398.572	17 678.571	70 396.070	3.982	
S2-1	1	12.699	150 x 300	2394.424	17 678.571	67 991.784	3.846	4.0
S3-1	1	12.731	150 x 300	2400.458	17 678.571	74 532.855	4.216	
S1-3	3	12.709	150 x 300	2396.310	17 678.571	162 695.889	9.203	
S2-3	3	12.720	150 x 300	2398.384	17 678.571	166 408.389	9.413	9.5
S3-3	3	12.734	150 x 300	2401.024	17 678.571	175 848.746	9.947	
S1-7	7	12.733	150 x 300	2400.835	17 678.571	297 883.921	16.850	
S2-7	7	12.691	150 x 300	2392.916	17 678.571	294 772.493	16.674	16.8
S3-7	7	12.741	150 x 300	2402.343	17 678.571	299 209.814	16.925	
S1-14	14	12.724	150 x 300	2399.138	17 678.571	397 732.490	22.498	
S2-14	14	12.733	150 x 300	2400.835	17 678.571	398 121.419	22.520	22.3
S3-14	14	12.696	150 x 300	2393.859	17 678.571	388 857.848	21.996	
S1-28	28	12.725	150 x 300	2399.327	17 678.571	435 281.775	24.622	
S2-28	28	12.731	150 x 300	2400.458	17 678.571	441 138.739	25.123	24.9
S3-28	28	12.727	150 x 300	2399.704	17 678.571	440 496.954	24.917	

Table A11: Splitting tensile strength of cylindrical concrete specimens for mix B

Specimen number	Age (days)	Cylinder weight (kg)	Dimensions (mm by mm)	Cylinder density (kg/m^3)	Cylinder area (mm^2)	Failure load (N)	Tensile strength (N/mm^2)	Average tensile strength (N/mm^2)
S1-1	1	12.730	150 x 300	2400.269	70 714.286	33 872.143	0.479	
S2-1	1	12.729	150 x 300	2400.081	70 714.286	35 427.857	0.501	0.5
S3-1	1	12.735	150 x 300	2401.212	70 714.286	38 397.857	0.543	
S1-3	3	12.700	150 x 300	2394.613	70 714.286	67 956.429	0.961	
S2-3	3	12.699	150 x 300	2394.424	70 714.286	67 532.143	0.955	1.1
S3-3	3	12.720	150 x 300	2398.384	70 714.286	95 393.572	1.349	
S1-7	7	12.723	150 x 300	2398.949	70 714.286	124 952.143	1.767	
S2-7	7	12.722	150 x 300	2398.761	70 714.286	120 992.143	1.711	1.8
S3-7	7	12.731	150 x 300	2400.458	70 714.286	128 841.429	1.822	
S1-14	14	12.736	150 x 300	2401.401	70 714.286	220 275.000	3.115	
S2-14	14	12.741	150 x 300	2402.343	70 714.286	236 185.715	3.340	3.2
S3-14	14	12.740	150 x 300	2402.155	70 714.286	226 073.572	3.197	
S1-28	28	12.698	150 x 300	2394.236	70 714.286	224 093.572	3.169	
S2-28	28	12.701	150 x 300	2394.801	70 714.286	226 356.430	3.201	3.3
S3-28	28	12.727	150 x 300	2399.704	70 714.286	241 347.858	3.413	

Table A12: Flexural strength of concrete beam specimens for mix B

Specimen number	Age (days)	Beam weight (kg)	Dimensions (mm by mm by mm)	Beam density (kg/m^3)	Beam area (mm^2)	Failure load (N)	Flexural strength (N/mm^2)	Average flexural strength (N/mm^2)
S1-1	1	28.702	150 x 150 x 530	2406.876	7 500	6720	0.896	
S2-1	1	28.615	150 x 150 x 530	2399.581	7 500	5190	0.692	0.7
S3-1	1	28.493	150 x 150 x 530	2389.350	7 500	4417.5	0.589	
S1-3	3	28.606	150 x 150 x 530	2398.826	7 500	9352.5	1.247	
S2-3	3	28.611	150 x 150 x 530	2399.245	7 500	9382.5	1.251	1.2
S3-3	3	28.590	150 x 150 x 530	2397.484	7 500	8887.5	1.185	
S1-7	7	28.700	150 x 150 x 530	2406.709	7 500	20 917.5	2.789	
S2-7	7	28.659	150 x 150 x 530	2403.270	7 500	17 760	2.368	2.4
S3-7	7	28.623	150 x 150 x 530	2400.252	7 500	16 095	2.146	
S1-14	14	28.611	150 x 150 x 530	2399.245	7 500	24 720	3.296	
S2-14	14	28.598	150 x 150 x 530	2398.155	7 500	24 022.5	3.203	3.3
S3-14	14	28.609	150 x 150 x 530	2399.078	7 500	24 555.0	3.274	
S1-28	28	28.622	150 x 150 x 530	2400.168	7 500	26 737.5	3.565	
S2-28	28	28.617	150 x 150 x 530	2399.748	7 500	26 227.5	3.497	3.5
S3-28	28	28.624	150 x 150 x 530	2400.335	7 500	26 827.5	3.577	

Table A13: Compressive strength of cylindrical concrete specimens for mix C

Specimen number	Age (days)	Cylinder weight (kg)	Dimensions (mm by mm)	Cylinder density (kg/m^3)	Cylinder area (mm^2)	Failure load (N)	Compressive strength (N/mm^2)	Average compressive strength (N/mm^2)
S1-1	1	12.719	150 x 300	2398.195	17 678.571	67 249.284	3.804	3.9
S2-1	1	12.723	150 x 300	2398.949	17 678.571	68 522.141	3.876	
S3-1	1	12.730	150 x 300	2400.269	17 678.571	69 158.570	3.912	
S1-3	3	12.720	150 x 300	2398.384	17 678.571	171 305.353	9.690	9.7
S2-3	3	12.729	150 x 300	2400.081	17 678.571	172 401.424	9.752	
S3-3	3	12.698	150 x 300	2394.236	17 678.571	168 353.032	9.523	
S1-7	7	12.727	150 x 300	2399.704	17 678.571	308 897.671	17.473	17.0
S2-7	7	12.717	150 x 300	2397.818	17 678.571	293 181.422	16.584	
S3-7	7	12.725	150 x 300	2399.327	17 678.571	301 720.171	17.067	
S1-14	14	12.731	150 x 300	2400.458	17 678.571	390 714.098	22.101	22.1
S2-14	14	12.718	150 x 300	2398.007	17 678.571	388 769.455	21.991	
S3-14	14	12.721	150 x 300	2398.572	17 678.571	391 297.491	22.134	
S1-28	28	12.689	150 x 300	2392.539	17 678.571	432 117.311	24.443	24.8
S2-28	28	12.742	150 x 300	2402.532	17 678.571	445 517.668	25.201	
S3-28	28	12.722	150 x 300	2398.761	17 678.571	437 668.382	24.757	

Table A14: Splitting tensile strength of cylindrical concrete specimens for mix C

Specimen number	Age (days)	Cylinder weight (kg)	Dimensions (mm by mm)	Cylinder density (kg/m^3)	Cylinder area (mm^2)	Failure load (N)	Tensile strength (N/mm^2)	Average tensile strength (N/mm^2)
S1-1	1	12.731	150 x 300	2400.458	70 714.286	46 035.000	0.651	
S2-1	1	12.698	150 x 300	2394.236	70 714.286	41 509.286	0.587	0.6
S3-1	1	12.720	150 x 300	2398.384	70 714.286	42 570.000	0.602	
S1-3	3	12.728	150 x 300	2399.892	70 714.286	77 149.286	1.091	
S2-3	3	12.719	150 x 300	2398.195	70 714.286	75 310.715	1.065	1.0
S3-3	3	12.707	150 x 300	2395.933	70 714.286	68 380.715	0.967	
S1-7	7	12.688	150 x 300	2392.350	70 714.286	111 445.715	1.576	
S2-7	7	12.731	150 x 300	2400.458	70 714.286	134 286.429	1.899	1.8
S3-7	7	12.722	150 x 300	2398.761	70 714.286	132 872.143	1.879	
S1-14	14	12.729	150 x 300	2400.081	70 714.286	226 639.287	3.205	
S2-14	14	12.695	150 x 300	2393.670	70 714.286	204 859.287	2.897	3.1
S3-14	14	12.723	150 x 300	2398.949	70 714.286	219 850.715	3.109	
S1-28	28	12.699	150 x 300	2394.424	70 714.286	210 162.858	2.972	
S2-28	28	12.727	150 x 300	2399.704	70 714.286	226 710.001	3.206	3.2
S3-28	28	12.732	150 x 300	2400.646	70 714.286	233 427.858	3.301	

Table A15: Flexural strength of concrete beam specimens for mix C

Specimen number	Age (days)	Beam weight (kg)	Dimensions (mm by mm by mm)	Beam density (kg/m^3)	Beam area (mm^2)	Failure load (N)	Flexural strength (N/mm^2)	Average flexural strength (N/mm^2)
S1-1	1	28.598	150 x 150 x 530	2398.155	7 500	4252.5	0.567	
S2-1	1	28.602	150 x 150 x 530	2398.491	7 500	4417.5	0.589	0.6
S3-1	1	28.615	150 x 150 x 530	2399.581	7 500	4672.5	0.623	
S1-3	3	28.617	150 x 150 x 530	2399.748	7 500	9052.5	1.207	
S2-3	3	28.579	150 x 150 x 530	2396.562	7 500	8160.0	1.088	1.2
S3-3	3	28.611	150 x 150 x 530	2399.245	7 500	8970.0	1.196	
S1-7	7	28.621	150 x 150 x 530	2400.084	7 500	17 280.0	2.304	
S2-7	7	28.590	150 x 150 x 530	2397.484	7 500	16 627.5	2.217	2.3
S3-7	7	28.623	150 x 150 x 530	2400.252	7 500	17 385.0	2.318	
S1-14	14	28.619	150 x 150 x 530	2399.916	7 500	25 492.5	3.399	
S2-14	14	28.620	150 x 150 x 530	2400.000	7 500	25 515.0	3.402	3.4
S3-14	14	28.618	150 x 150 x 530	2399.832	7 500	25 402.5	3.387	
S1-28	28	28.621	150 x 150 x 530	2400.084	7 500	26 962.5	3.595	
S2-28	28	28.599	150 x 150 x 530	2398.239	7 500	25 687.5	3.425	3.5
S3-28	28	28.627	150 x 150 x 530	2400.587	7 500	27 007.5	3.601	

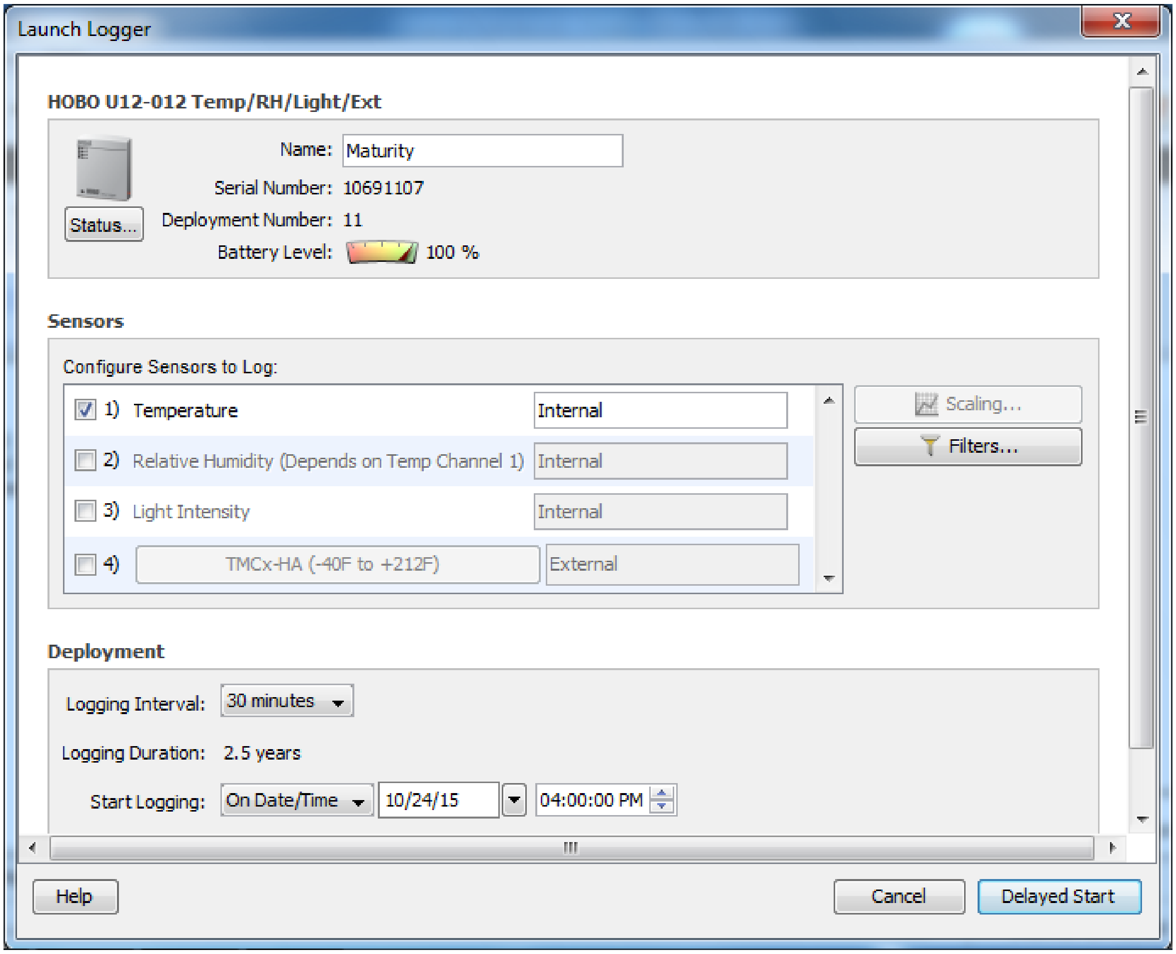

Plate A1: HOBOware Data Logger Software Configuration

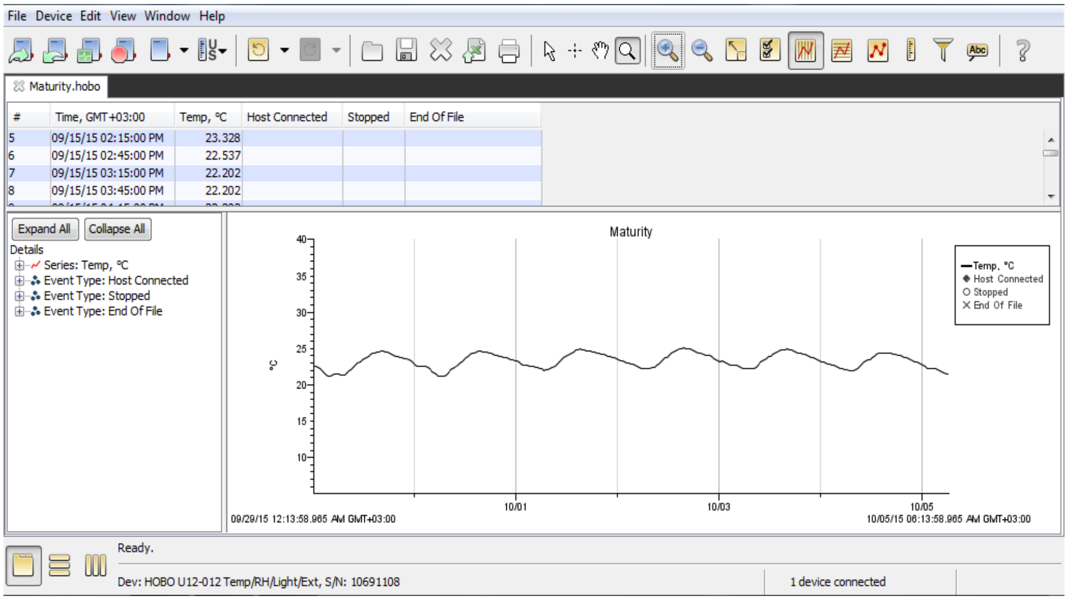

Plate A2: HOBO Data Logger Readout

Table A16: Maturity values for cylindrical concrete specimens prepared from mix A

Age (h)	Temperature (°C)	Age increment (h)	Average temperature (°C)	Temperature-time factor increment (°C-h)	Temperature-time factor cumulative (°C-h)
0	22.48				0
0.5	22.52	0.5	22.5	11.25	11.25
1.0	23.32	0.5	22.92	11.46	22.71
...	...	...	...	...	...
24.0	21.4	0.5	22.94	11.47	548.75
...	...	...	...	...	...
72.0	21.28	0.5	22.92	11.46	1643.36
...	...	...	...	...	...
168.0	24.76	0.5	23	11.5	3833.61
...	...	...	...	...	...
336.0	21.12	0.5	23.02	11.51	7670.96
...	...	...	...	...	...
672.0	21.72	0.5	22.52	11.26	15364.61

Table A17: Maturity values for concrete beam specimens prepared from mix A

Age (h)	Temperature (ºC)	Age increment (h)	Average temperature (ºC)	Temperature-time factor increment (ºC-h)	Temperature-time factor cumulative (ºC-h)
0	22.49				0
0.5	22.51	0.5	22.5	11.25	11.25
1.0	23.32	0.5	22.92	11.46	22.71
...	...	...	...	...	...
24.0	21.4	0.5	22.94	11.47	548.75
...	...	...	...	...	...
72.0	21.28	0.5	22.92	11.46	1643.36
...	...	...	...	...	...
168.0	24.76	0.5	23	11.5	3833.61
...	...	...	...	...	...
336.0	21.12	0.5	23.02	11.51	7670.96
...	...	...	...	...	...
672.0	21.72	0.5	22.52	11.26	15364.61

Table A18: Maturity values for cylindrical concrete specimens prepared from mix B

Age (h)	Temperature (ºC)	Age increment (h)	Average temperature (ºC)	Temperature-time factor increment (ºC-h)	Temperature-time factor cumulative (ºC-h)
0	22.5				0
0.5	22.5	0.5	22.5	11.25	11.25
1.0	23.32	0.5	22.91	11.46	22.71
...	...	...	...	...	...
24.0	21.4	0.5	22.94	11.47	548.75
...	...	...	...	...	...
72.0	21.28	0.5	22.92	11.46	1643.36
...	...	...	...	...	...
168.0	24.76	0.5	23	11.5	3833.61
...	...	...	...	...	...
336.0	21.12	0.5	23.02	11.51	7670.96
...	...	...	...	...	...
672.0	21.72	0.5	22.52	11.26	15364.61

Table A19: Maturity values for concrete beam specimens prepared from mix B

Age (h)	Temperature (ºC)	Age increment (h)	Average temperature (ºC)	Temperature-time factor increment (ºC-h)	Temperature-time factor cumulative (ºC-h)
0	22.5				0
0.5	22.51	0.5	22.51	11.25	11.25
1.0	23.32	0.5	22.92	11.46	22.71
...	...	...	...	...	...
24.0	21.4	0.5	22.94	11.47	548.75
...	...	...	...	...	...
72.0	21.28	0.5	22.92	11.46	1643.36
...	...	...	...	...	...
168.0	24.76	0.5	23	11.5	3833.61
...	...	...	...	...	...
336.0	21.12	0.5	23.02	11.51	7670.96
...	...	...	...	...	...
672.0	21.72	0.5	22.52	11.26	15364.61

Table A20: Maturity values for cylindrical concrete specimens prepared from mix C

Age (h)	Temperature (ºC)	Age increment (h)	Average temperature (ºC)	Temperature-time factor increment (ºC-h)	Temperature-time factor cumulative (ºC-h)
0	22.49				0
0.5	22.51	0.5	22.5	11.25	11.25
1.0	23.32	0.5	22.92	11.46	22.71
...	...	...	...	...	...
24.0	21.4	0.5	22.94	11.47	548.75
...	...	...	...	...	...
72.0	21.28	0.5	22.92	11.46	1643.36
...	...	...	...	...	...
168.0	24.76	0.5	23	11.5	3833.61
...	...	...	...	...	...
336.0	21.12	0.5	23.02	11.51	7670.96
...	...	...	...	...	...
672.0	21.72	0.5	22.52	11.26	15364.61

Table A21: Maturity values for concrete beam specimens prepared from mix C

Age (h)	Temperature (ºC)	Age increment (h)	Average temperature (ºC)	Temperature-time factor increment (ºC-h)	Temperature-time factor cumulative (ºC-h)
0	22.5				0
0.5	22.5	0.5	22.5	11.25	11.25
1.0	23.32	0.5	22.92	11.46	22.71
...	...	...	...	...	...
24.0	21.4	0.5	22.94	11.47	548.75
...	...	...	...	...	...
72.0	21.28	0.5	22.92	11.46	1643.36
...	...	...	...	...	...
168.0	24.76	0.5	23	11.5	3833.61
...	...	...	...	...	...
336.0	21.12	0.5	23.02	11.51	7670.96
...	...	...	...	...	...
672.0	21.72	0.5	22.52	11.26	15364.61