UNIVERSITY OF NAIROBI

FACULTY OF ENGINEERING

Thesis Report

INVESTIGATING THE POTENTIAL USE OF TUFF AGGREGATES TO PRODUCE LIGHTWEIGHT CONCRETE

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F56/82169/2015

A thesis report submitted in partial fulfillment for the award of the Degree of Master of Science in Civil Engineering (Structural Engineering) of the University of Nairobi

November 2021

DECLARATION

I, **GEOFFREY KIPROTICH SANG**, declare that this thesis is my original work and has never been presented for any academic award in this university or any other university.

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This thesis has been submitted for examination with our approval as university supervisors.

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DEDICATION

This thesis is dedicated to my parents Mrs. Grace Monori and Mr. Kipsang Monori. Through their constant prayers, I was able to endure the difficult times until I completed this research. Secondly, I wish to dedicate this research to my dear wife Mrs. Gloria Chirchir, and my good children; Ivy C,heruto, Elsie Chemutai, and Leon Kimutai for their support, prayers, and encouragement throughout the entire period of my study at the University of Nairobi without whom I could not have completed this research work. Last but not least is to dedicate this research work to my good friend Mr. Geoffrey Koech (Jeff aka Tractor) for the support he has been giving me throughout while pursuing this research.

ACKNOWLEDGMENT

This research could not have been completed without God's grace and guidance throughout the years of my study.

I am very thankful to my first supervisor Prof. Eng. Silvester Abuodha for his invaluable comments and inputs throughout the stages of this research. Secondly, I would like to express my sincere gratitude to Dr. Eng. John Mwero, my second supervisor, for his guidance and recommendations throughout the length of this research work.

Further appreciation goes to the technicians of the civil engineering laboratory for their cooperation during the preparation and testing of materials in the laboratory.

I also thank my friends who contributed to the success of this research in one way or another.

I thank my family members for their never-ending encouragement, love, prayers and motivation throughout, my period of study.

TABLE OF CONTENTS

LIST OF ABBREVIATIONS AND ACRONYMS

AASHTO…………………...American Association of State Highway and Transportation Official

ACI …………………………American Concrete Institute

ASTM ……………………... American Society for Testing and Materials

BRE………………………... Building Research Establishment

BS…………………………...British Standards

CA…………………………. Coarse Aggregate

ESCS………………………. Expanded Shale, Clay or Slate

FA…………………………. Fine Aggregat,e

HSLDC…………………… High Strength Low-Density Concrete

LDA………………………... Low-Density Aggregate

LECA……………………… Lightweight Expanded Clay Aggregate

LWC………………………...Lightweight Concrete

NDA………………………. Normal-Density Aggregate

N/mm^2………………………Newton per square millimeter

NWC ………………………Normal-Weight concrete

Mpa…………………………Mega Pascal

OD…………………………. Oven-dried

OPS…………………………Oil Palm Shell

PCC…………………………Portland cement concrete

SDC…………………………Specified Density Concrete

SLWC………………………Structural Lightweight Concrete

SP…………………………...Superplasticizer

SSD………………………… Saturated Surface Dried

W/C………………………... Water Cement ratio

LIST OF TABLES

LIST OF FIGURES

LIST OF PLATES

ABSTRACT

Lightweight concrete can be made by using either artificial lightweight aggregates or natural lightweight aggregates. Artificial lightweight aggregates include expanded clay, ground blast furnace slag, slate, and shale. Natural aggregates include; tuff, pumice, scoria, rhyolite, and perlite among others. The rapidly increasing cost of production of artificial lightweight aggregates has however renewed the interest in the research and use of naturally occurring lightweight aggregates. The current study, therefore, assessed the performance of lightweight concrete made from crushed tuff aggregates while comparing its properties with those of conventional concrete. The procedure outlined in ACI Code was used for concrete mixed design. Some of the results obtained include the average 28-day unit densities of 2038 kg/m^3 and 2527 kg/m^3 for tuff and conventional concretes respectively. The compressive strength tests were performed at different ages of concrete following BS 1881-part 116. Specifically, the 28-day compressive strengths of 26.0 MPa and 30.0 MPa were obtained for tuff and conventional concretes respectively when presoaked aggregates were used with a water-cement ratio of 0.4. There was an increase in the 28-day compressive strength of tuff concrete while a continuous decline in the compressive strength of conventional concrete was noted when the w/c ratio was increased. It was noted in the current study that the production tuff concrete is less costly and economical than NWC. The study further confirmed that tuff aggregates can produce concrete with a splitting tensile strength of 2.9 MPa and elastic modulus of 10.6 GPa. Tuff concrete mixed designs were done according to ACI Code. This concrete is recommended for floor-filling in high-rise buildings. The study, however, did not cover durability tests on the concrete samples but recommended the same to be investigated in future studies.

CHAPTER ONE

INTRODUCTION

1.1 Background to the Study

Euro code 2 Part 1-1 (2008) defines lightweight concrete as concrete with a density of not more than 2200 kg/m^3 and compressive strength of at least 17 MPa consisting of artificial or natural lightweight aggregates having a particle density of less than 2000 kg/m^3. Neville (2000) stated that the average 28-day density of conventional concrete ranges from 2200 kg/m^3 to 2600 kg/m^3. Faizul (2016) noted that due to high self-weight, conventional concrete becomes uneconomical material to use in some construction works especially those located in earthquake-prone areas. Therefore, many attempts have been made not only to reduce the self-weight of the concrete but also to increase its efficiency as a structural material.

The three types of lightweight concrete include aerated concrete, no-fines concrete, and lightweight aggregate concrete (Mulgund & Kulkarni, 2018).). Aerated concrete is produced by introducing an air-entraining agent such as aluminium powder, hydrogen peroxide, hydrolyzed protein, or resin during mixing. This concrete is also referred to as cellular or gas concrete. No-fines concrete is composed of only cement and low-density coarse aggregate. It is produced by omitting fine aggregate components in the mix. Lightweight aggregate concrete is produced by using lightweight aggregates with a specific gravity lower than 2.6 in the mix.

The current study explored the possibility of utilizing natural tuff aggregates mined from existing rock deposits available in Kenya which are commonly carved into masonry blocks and used in the construction of walls in buildings. The research, therefore, focused on the production of lightweight aggregate concrete from the combination of tuff coarse aggregate, natural sand, cement, and water while conventional concrete was prepared for control experiment using granite coarse aggregate in place of tuff aggregate. The production of artificial lightweight aggregates is a costly venture which also requires a lot of energy. It is against this backdrop therefore that investigation on the use of natural lightweight aggregates to make structural lightweight concrete becomes relevant. Tuff is a type of extrusive igneous rock that is formed from the products of an explosive volcanic eruption. This rock is made up of ejected fragments of bedrocks, tephra, volcanic ash, magma, and other materials. The ejected material is propelled through the air, which then falls back to the Earth's surface and consolidates forming tuff rock during and after the volcanic eruption (Hobart King, 2019). Tuff rocks are found in many parts of Kenya including Bahati and Kedowa areas in Rift Valley Province where the volcanic eruption took place many years ago. The material is commonly used in the production of carved masonry blocks for use in the construction of buildings and walls since it is soft and light in weight. The products of tuff rock are also used for artifacts, monuments, sculptures, and small figurines. Incorporating wholly or partially of tuff aggregates in concrete production could broaden the range of natural materials available in our country that are potentially suitable for making lightweight concrete. The use of the material will certainly enhance its diversification and demand as a concrete-making material in the construction industry and subsequently improve its economic value.

The demand for structural lightweight concrete in Kenya is likely to rise owing to an increase in the need for Earthquake resistant high-rise buildings, tilt-up and precast materials especially in urban areas. Buildings constructed with heavy materials are more vulnerable to seismic forces than those constructed with lightweight materials (Vandanapu et al., 2018). Therefore, reducing the mass of concrete structures is of utmost importance in reducing their risks of failure due to seismic forces. This type of concrete also has advantages of higher strength to weight ratio, better tensile strain capacity, lower coefficient of thermal expansion, and superior heat and sound insulation characteristics due to air voids in the lightweight aggregates.

In the rush to meet the Millennium Development Goals (MDGs) and the Vision 2030, the Kenyan government has initiated many flagship projects which could see massive consumption of concrete (Ndung'u, Thugge, & Otieno, 2011). The use of more economical, eco-sustainable, and environmentally friendly materials in construction cannot be overemphasized given that Kenya is a developing country. It's envisaged that the introduction of natural tuff aggregates to replace conventional aggregates in the production of concrete will reduce the global cost of construction of structures and can contribute to a reduction in carbon emissions into the atmosphere. Many researchers recognize the advantages offered by lightweight concrete as manifested by the existence of lightweight structures throughout the world.

1.2 Problem Statement

Overreliance on conventional coarse aggregates for the production of concrete has been the common practice in concrete production in Kenya for a long time. The main sources of conventional construction materials for making concrete known to many builders in the construction industry are mainly granite, gravel, limestone and, gneiss rocks. This research study sought to explore an alternative material other than the common artificially manufactured materials used to produce lightweight concrete. If tuff aggregate material is adopted for use in the construction industry, overreliance on conventional materials will reduce and subsequently lead to a reduction in the overall cost of construction. Tuff aggregate concrete was expected to address the problems of the high cost of handling, transportation, and placing of wet normal-weight concrete in the construction industry. Vandanapu et al. (2018) confirmed that tall buildings constructed with normal-weight concrete are more susceptible to disastrous effects of seismic forces than those done with lightweight concretes. Effects of seismic forces particularly on high-rise buildings made of concrete can be reduced by using lightweight construction materials. The possibility of using tuff aggregates to make concrete from the list of the few natural materials available in Kenya can solve the problems of higher dead load due to normal-weight concrete in structures particularly high-rise buildings. Furthermore, the problem of high water absorption associated with tuff aggregate concrete can be addressed by either presoaking the aggregates or incorporating a superplasticizer while mixing. By incorporating a polycarboxylic Ether-based superplasticizer, the problem of water absorption and compaction of tuff aggregate concrete was addressed. Subsequently, the compressive strength which was lower than that of normal-weight concrete due to the lower rigidity of tuff aggregates was slightly improved by adding a superplasticizer.

1.3 Objectives of the Research

1.3.1 General Objective

The objective of this research is to investigate the suitability of tuff rock aggregates for the production of low-density concrete while comparing its properties with those of conventional concrete.

1.3.2 Specific Objectives

i. To determine a mixture proportion of materials that can produce lightweight concrete with a 28-day compressive strength of at least 25 MPa.
ii. To establish the effect of water-cement ratio and superplasticizer on the fresh and hardened properties of tuff and conventional concretes.
iii. To determine the mechanical properties of structural tuff concrete such as compressive strength, tensile strength, and elastic modulus.

1.4 Research Questions

i. Were the values of unit-weight and compressive strength sufficient to qualify tuff concrete as a structural lightweight concrete?
ii. How can the problem of water absorption during the mixing of tuff aggregate concrete be addressed?
iii. What are some of the economic benefits of using tuff concrete over normal-weight concrete?

1.5 Justification of the Study

Rapid increase in construction activities in Kenya has led to acute shortage of conventional materials in the construction industry owing to our country's effort to achieve the Vision 2030 and other Millennium Development Goals (MDGs). Secondly, the ordinary concrete commonly used is quite heavy with a density ranging from 2200 kg/m^3 to 2600 kg/m^3 (Neville, 2000) contributing significantly to the dead weight of the structures. By using lightweight aggregates made from tuff, the density of concrete can be reduced considerably. Lightweight concrete also provides a better insulation effect against heat and sound to structures. It is also suitable for earthquake-proof structures. LWC also finds use generally in decks of long-span bridges, fire, and corrosion protection, covering for architectural purposes, heat insulation on roofs, insulation of water pipes, filling for roofs, construction of walls for partitioning, and panel walls in framed structures. The use of tuff aggregate minimizes the demand for the extraction of conventional aggregates such as granite and gravel. The use of artificial lightweight aggregates contributes to high carbon emission and high consumption of energy during the manufacturing process which makes them costly and environmentally risky compared to natural tuff aggregates.

1.6 Scope of the Study

Within the scope of this research, an extensive testing program was conducted on concrete materials which include grading and determination of the physical properties of aggregates. Bulk density, as well as specific gravity tests, were measured. This was followed by carrying out concrete mixed designs according to the America Concrete Institute (1998). Testing of concrete specimens to determine the fresh concrete properties such as slumps and compaction factors were conducted. Mechanical properties such as compressive strength, splitting tensile strength, and modulus of elasticity were determined and compared. During this study, the conventional concrete made from coarse granite aggregate components was used for a control experiment.

1.7 Limitations of the Study

Samples of tuff aggregate from only one source were investigated in this research. The experimental results, therefore, were only limited and specific to tuff aggregate concrete made from tuff aggregate obtained from Bahati area site in Nakuru County and may not be replicated for general applications when other sources are considered. It was not possible to sample tuff aggregates from other sources because of time limitations. The crushing of aggregate materials was done manually by hand which was a slow and tedious process. Crushing by hand negatively affects properties of aggregates as such as flakiness indices, coefficients of uniformity, and coefficients of curvature. These properties have a bearing on the strength of concrete. The mix design of concrete was carried out according to America Concrete Institute (1998). The water-cement ratio in this code is just an approximated value. The actual values were determined experimentally in the lab to get the practical and workable water-cement ratios that were incorporated during the current study. Concrete mix design codes applicable to normal-weight concrete mixes are difficult to apply for lightweight aggregate concretes. This was due to the lack of accurate water absorption and free moisture content of lightweight aggregates making the application of accurate water/cement ratio difficult when proportioning the components. Investigations on the chemical properties of concretes were not covered in this study.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

This chapter outlines some of the research studies conducted on lightweight concrete used in the past. Some literature reviewed includes the occurrence of tuff rocks, the physical properties of lightweight concrete made from materials other than tuff, and the various applications of lightweight concrete in the past. The study on the use of tuff aggregate as a concrete making material seeks to place it as an alternative construction material to provide economical and sustainable solutions to alleviate some of the engineering problems associated with the use of normal-weight concrete in the construction industry.

2.2 The Occurrence of Tuff Rocks

Hobart King (2019) defined tuff (Plate 2.1) as an extrusive igneous rock that forms from the products of an explosive volcanic eruption. The ejected material in a volcanic eruption is classified into three types which include volcanic gases having a mixture of steam, carbon dioxide, and sulfur compounds. The other material is lava, which is magma expelled out from the volcano to the earth's surface. The third material is tephra which is composed of solid materials of varying shapes and sizes propelled through the air. Tephra is produced when magma in the volcano is blown out by the forceful expansion of hot gases thrown out of the volcano. The forceful explosions produce solid materials that are discharged from the vent of the volcano. Therefore, tuff is a lightweight consolidated rock that contains compacted and cemented fragments of bedrocks, tephra, volcanic ash, magma, and other materials. When the ejected material is compacted and cemented, it results in the formation of tuff rock through a process called consolidation. Tuff rock contains minerals such as calcite and chlorite. There are different types of tuff rocks which include welded tuff, rhyolite tuff, trachyte tuff, andesitic tuff, basaltic tuff, and ultramafic tuff depending on among others their texture. The Romans used it often for construction purposes as it is a relatively soft rock. The Rapa Nui people used it to make most of the Moai statues on Easter Island. The quantity of the volcanic tuff rocks available in Kenya cannot be estimated with any accuracy but may be of the order of several 100,000 km^3.

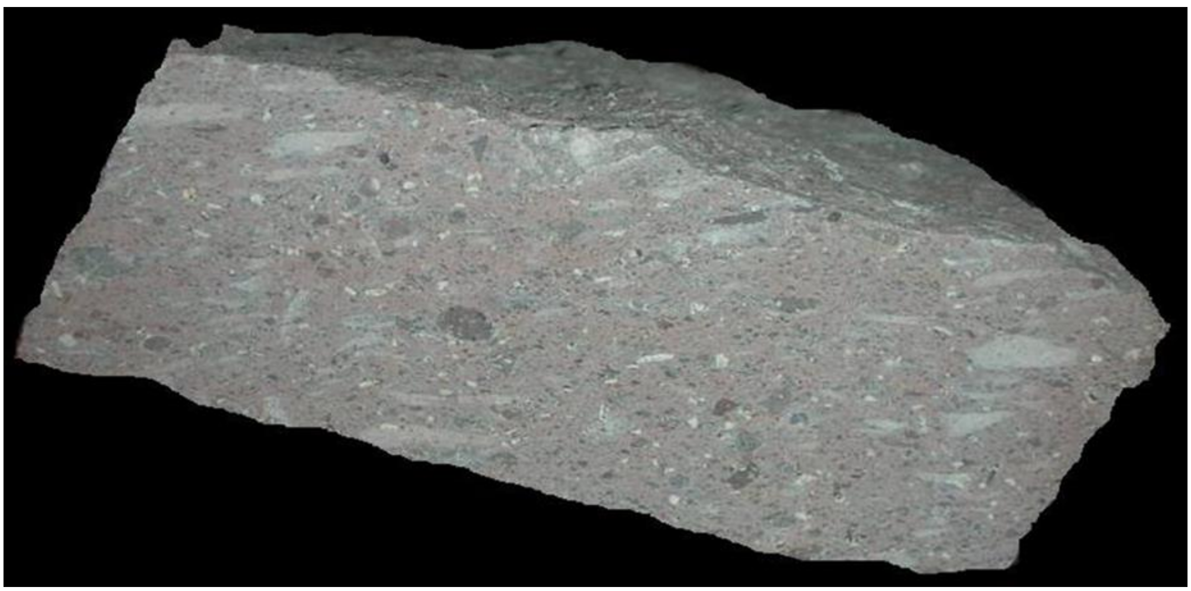

Plate 2.1: Sampled tuff rock from Bahati quarry in Nakuru County. (Source: Author)

2.3 Ancient Applications of Structural Lightweight Concrete

The application of lightweight concrete dates back to the times of the Roman Empire approximately more than two thousand years ago. Some of the most significant structures constructed with lightweight concrete include; the Pantheon Dome, the Port of Cosa as well as the Coliseum (ACI Committee 213, 2003). It was noted that the builders chose to use natural lightweight pumice and scoria aggregates from the volcanic materials instead of using the locally available beach sand and gravel aggregates to construct the structures. The construction of the Pantheon dome with a diameter of 43.3 m was completed in 27 B.C. During its construction, the builders used lightweight aggregate concrete of varying densities with the higher density concrete used near the base where the stresses were higher and lower density concrete used near the top of the dome where the stresses were lower (Holm & Bremner, 2000). In the case of the Pantheon Dome, the aggregate used for the upper dome region consisted of alternating layers of pumice, giving the concrete a low density of 1,350 kg/m^3. The foundation of the structure was constructed using travertine as an aggregate, which resulted in the concrete of density 2,200 kg/m^3. Holm and Bremner (2000) found out in their study that the Pantheon structure was still in a good condition despite being subjected to forces of nature for a long time, and it is still being used for spiritual purposes to date. The Coliseum is an ancient amphitheater of massive size with a fifty thousand seating capacity. It was constructed in 75 to 80 A.D. The foundation of the Coliseum was made of lightweight concrete utilizing crushed volcanic lava as aggregate. Its walls were constructed with porous lightweight crushed bricks. The spaces and the vaults between the walls were made of porous-tufa cut stone as documented in American Concrete Institute Committee 213 (2003).

Holm and Bremner (2000) observed that the Roman Port of Cosa which was constructed about 273 B.C incorporating both pumice and scoria aggregates is still in existence and serves as a shining example of how durable the concrete structures have been. At the time of its operation, some of the structures found in the port include a harbor supported by concrete piers, a fishery, factories for making amphoras, and a lighthouse. For several years, the structures in the port have resisted the forces of nature which include corrosion, abrasion, and siltation that have taken place over a long period (Gazda & McCann, 1987). As of today, the aforementioned Roman structures stand as a shred of evidence to the length of time lightweight concrete structures have been in existence. This is a clear indication that the builders by 273 B.C knew that lightweight aggregates were more suitable for marine structures than natural gravel and beach sand.

2.4 Recent Applications of Structural Lightweight Concrete

2.4.1 Use of lightweight concrete in marine structures

The use of lightweight aggregate after the Romans was limited. This trend changed when manufactured lightweight aggregates became commercially available in the 20th century as documented in American Concrete Institute Committee 213 reports (2003). Many marine structures including ships and offshore oil exploration platforms have been built using lightweight concrete. The first modern application of high-performance lightweight concrete was utilized in building the American Fleet Corporation ship in 1920. The concrete used was produced from expanded shale, clay, and slate (ESCS) with a unit weight of 1760 kg/m^3 and compressive strength of 35 MPa (American Concrete Institute, 2014).

2.4.2 Use of lightweight concrete in high-rise buildings

Many residential, industrial, and commercial buildings have also been constructed around the world using lightweight concrete. Tunç et al., (2018) found that the floor system of the Bank of America Corporate Center (Plate 2.2) was constructed using lightweight concrete to produce floor slabs of 117 mm thickness supported by deep beams placed at 3 m centre to centre. The strength of the concrete used ranged from 43 Mpa to 51 MPa while the density was 1890 kg/m^3. It is one of the tallest buildings in Charlotte in North Carolina with a height of 265 m and sixty storeys.

Plate 2.2: Bank of America Corporate Center.

(Source: Wikipedia.org/wiki/File: Bank of America Corporate Center)

A study done by Zareef (2010) found that the frame and the floor system of the 28-storey Park Plaza Hotel in St. Louis were done using structural infra lightweight concrete. The same researcher also found that fourteen additional stories were added to the existing 14-storey building of the South Western Bell Telephone Office in Kansas City by using lightweight concrete. The builders confirmed that the foundations and underpinnings would support additional eight floors of normal-weight concrete. However, the same foundations could support additional fourteen floors doubling the storeys of the building to twenty-eight floors. Zareef (2010) found that the floor system of 30-storey Standard Bank building in Johannesburg, 139 m high, was done using lightweight concrete. The floor slabs in the building were constructed using precast double-T units which were steam-cured and lifted one day after casting to achieve a rapid erection time. The unit slabs were 10 m long, 3.16 m wide, and 75 mm thick between ribs at 1.58 m centres. The expanded clay was used as coarse (20/10 mm) aggregate with natural sand, giving a concrete mix with a dry density of 1950 kg/m^3 at an age of 28 days. Other buildings constructed with lightweight concrete include 42-storey Prudential Life Building in Chicago, the U.S.A which has lightweight concrete floors, and the 18-storey Statler Hilton Hotel in Dallas which has lightweight concrete frames and flat plate floors as documented in American Concrete Institute Committee 213 (2003).

2.4.3 Use of lightweight concrete in bridges and marine structures

To date, the application of structural lightweight concrete has been extended to the construction of world-famous bridges with nearly 500 bridges incorporating the concrete into decks, beams, girders, or piers (Holm & Bremner, 2000). It has been in use in the United States and Canada for more than 50 years and with nearly 200 concrete bridges containing lightweight aggregates built. In USSR, over 100 bridges have been constructed with lightweight aggregate concrete in the last 20 years. Lightweight concrete bridges which have been successfully built range from simple reinforced concrete footbridges to long-span post-tensioned segmental box girder bridges. Holm and Bremner (2000) observed that weight savings from 20 to 35 percent on the superstructure have been achieved when lightweight concrete was used instead of normal-weight concrete, with consequent savings being made on pre-stressing and reinforcement steel on superstructures and substructures. Overall cost savings from 20 percent or more have been possible after allowance has been made for the higher initial cost of lightweight aggregates in cases where artificial ones are used. The researchers have also observed that concrete bridges built with lightweight concrete performed satisfactorily well in service and there is increasing evidence that the durability property of such concrete is as good as in normal-weight concrete. The few cases of unsatisfactory performance are attributable to inadequate detail design or poor quality control during the construction process. Lightweight concrete has been particularly advantageous for precast and pre-stressed components in reducing handling costs where access to the site is limited or where the ground conditions are difficult. Thus it has found potential application in the upgrading of existing bridges where disruption to traffic flows must be kept to a minimum.

Other applications of lightweight concrete were found in the construction of Stolmen Bridge and Heidrun Tension Leg Platform (Holm & Bremner, 2000). Stolmen Bridge (Plate 2.3) is a cantilevered prestressed box girder bridge constructed in 1998 using lightweight concrete. To reduce its weight, the center of the main span was constructed with high-performance low-density concrete with a density of 1,940 kg/m^3. The short end spans were ballasted with gravel concrete. It has a main span length of 301m and a total length of 46 m and was the world's longest cantilever box-girder bridge before it was surpassed in 2006 by the Shibanpo Second Yangtze River Bridge in China. The 184m portion in the middle of the main span of the bridge was constructed with high-strength lightweight concrete. Approximately a total of 1600 m^3 of lightweight concrete was used during the construction of Stolmen Bridge. The 28-day mean compressive cube strength of concrete was 70.4 MPa and the mean concrete density of 28-day water cured concrete was 1940 kg/m^3. Plate 2.4 below shows the Heidrun Tension Leg

Platform built in 1995 in the Heidrun field of the Norwegian North Sea which floats over 345 m deep sea. It is the largest floating concrete structure carrying the largest deck load. In the construction of the Heidrun Tension Leg Platform, 65,700 m^3 of lightweight concrete was used. High-performance low-density concrete with a compressive strength of 70 MPa and a maximum density of 1950 kg/m^3 cast-in-place concrete was also used. The concrete density used for the slip-forming sections of the structure was 2000 kg/m^3. A study by Mishutn et al., (2017) has shown that concrete with porous aggregates has excellent durability characteristics in the harsh operating conditions in waters with sulfates and chlorides. This was in respect to the caisson Tarsiut Island floating structure made of porous lightweight concrete in the Beaufort Sea (Canada). The structure was built for the extraction of sand in 1982 and is still in operation to date.

Plate 2.3: Stolmen Bridge in Norway.
(Source: https://en.wikipedia.org/wiki/Stolma_Bridge)

Plate 2.4: Heidrun Tension Leg Platform, Norwegian North Sea.

(Source: <u>Https://Heidrun Tension Leg Platform, North Sea (Escsi, 2010))</u>

2.4.4 Use of lightweight concrete in stadia structures

Apart from buildings and bridges, some world stadia have been constructed with lightweight concrete. A significant example is Calgary Saddledome Stadium which was built in 1983 for the 1988 Winter Olympics in Canada (ACI Committee 213, 2003). It is a 20,000-seat capacity coliseum comprising all its precast structural members made of lightweight concrete. Lightweight concrete is increasingly being used in the prefabricated construction industry because it is easier to tow and less expensive to handle. The main aim of using lightweight concrete is to reduce a dead load of a concrete structure, which then allows the structural designer to reduce the size of columns, footings, and other load-bearing elements in the structure.

2.5 Specified Density Concrete

Specified density concrete (SDC) can be defined as concrete with a higher unit weight than that of the lowest lightweight concrete possible when all low-density aggregates are used but less than that of normal-weight concrete (Holm & Ries, 2000). For example, between the generally accepted maximum densities ranging from 1850 kg/m^3 to 2200 kg/m^3 of lightweight concrete and the typically assumed normal-weight concrete of density 2400 kg/m^3, any density may be specified and developed. The current study focused on average concrete densities ranging from 1850 kg/m^3 and 2200 kg/m^3 for tuff aggregate concretes. The application of specified density concrete is increasingly being adopted in Europe, US, and Canada. The use of SDC is driven by builders' decisions to optimize the density of concrete to improve structural efficiency and reduce the cost of transportation of concrete and its products.

Some of the structures where specified density concrete has been used include bridges, marine structures, precast elements, and consumer products with compressive strengths ranging from 20-70 Mpa and densities ranging from 1200 kg/m^3 to 2200 kg/m^3. This type of concrete is achieved by customizing the mixture proportions by replacing part or all of the normal density aggregates in the mixture with either coarse or fine aggregates having both the density and specific gravity less than 1600 kg/m^3 and 1.60 respectively.

The first structure constructed in 1981 with SDC is the Tarsiut Caisson Retained Island in Vancouver, Canada. The concrete used was a specified density concrete of with a density of 2,240 kg/m^3 (Holm & Ries, 2000). Another structure constructed in 1996 with SDC is Heidron Floating Oil Production Platform in Norway. The platform was constructed with approximately 70,000 m^3 of high-strength low-density concrete with a density of 2000 kg/m^3 to improve the buoyancy of the floating platform. The Hibernia Oil Platform built in 1998 in the North Atlantic Ocean is another significant application of SDC. To improve the buoyancy of the structure, the normal density aggregate was replaced with approximately 50% of the low-density aggregate (LDA) to make lightweight concrete with a unit weight of 2170 kg/m^3. Bridges, where specified density concrete has been utilized, include; the Shelby Creek bridge in Kentucky constructed in 1982 and the Norwegian prestressed box-girder Raftsundet bridge built in 1998. The Shelby Creek bridge was constructed with concrete with a density of 2080 kg/m^3. The Raftsundet bridge is a box girder where HSLDC was utilized for the 220-metre main span. At the time of its construction; the bridge was the longest box bridge in the world.

American Concrete Institute Standard Building Code, ACI 318 (2003) provides engineers with adequate guidance on how to design structural low-density concrete with compressive strength ranging from 20-35 Mpa. The code defines the difference in engineering properties of normal-weight concrete and low-density concrete.

2.6 The Compressive Strength of Lightweight Concrete

American Concrete Institute Committee 213 (2003) defines a structural lightweight concrete as a concrete having an air-dry equilibrium density of between 1120 kg/m^3 and 1920 kg/m^3 and a 28-day compressive strength higher than 17 MPa. In case its 28-day compressive strength is 40 MPa and above, it is referred to as high-strength lightweight concrete. On the other hand, Euro code 2 Part 1-1 of section 11 (2008) specifies the density range of structural lightweight concrete to be between 1050 kg/m^3 and 2050 kg/m^3 and average 28-day compressive strength of at least 17 MPa.

The three types of tests that are normally used to determine the compressive strength of concrete include cube, cylinder, and prism tests. Neville (2000) adopted a concrete cube test method for compressive tests on concrete for quality control purposes during his research studies. The current research adopted Neville's method of testing compressive strength. Shafigh et al., (2018) confirmed that Oil palm boiler clinker (OPBC) waste material could be used to replace fine and coarse aggregate in a concrete mixture to produce cheap and environmental structural low-density concrete. They obtained high-strength concrete with a 28-day compressive strength of between 60–70 MPa and a dry density of between 1990 kg/m^3 to 2250 kg/m^3. This was by far a better concrete than most commonly produced normal-weight concretes in Kenya today. Kosmatka et al., (2002) established that the compressive strength of concrete at an age of 7 days is about 75% of the 28-day strength for normal-weight concrete. Therefore, if the compressive strength of concrete is known at a particular age, then the formwork can be removed and reused in other works thus maximizing its use and resulting in a reduction of the project cost. Rahman et al., (2018) investigated the properties lightweight concrete produced from red clay, rice husk ash, and glass powder as a replacement for lightweight aggregate. In their research, they found similar compressive strengths with those of normal-weight concretes when 40% red clay, 40% rice husk ash, and 20% glass powder were used as a replacement for lightweight aggregate. Zhao et al., (2018) studied the compressive strength of concrete incorporating waste clay bricks as a replacement the fine and coarse aggregates to produce lightweight concrete. They obtained a concrete with a dry density of 1850 kg/m^3 and a 28-day compressive strength of 40 MPa.

Izzati et al., (2019) performed an experimental study on the unit weight, compressive strength, and water absorption properties of bricks where the sand was partially replaced with expanded polystyrene. The replacement percentages of expanded polystyrene were 0%, 20%, 30%, 40% and 50%. The unit weight and the compressive strength of the bricks were 1680kg/m^3 and 11.7 MPa respectively at a 50% replacement. From the results, the water absorption of the bricks was found to decrease as the percentage of expanded polystyrene was increased. Wu et al., (2017) studied the effect of lightweight aggregate type on the mechanical properties of concrete. They compared the properties of concrete made from three different lightweight aggregates which included; expanded clay, sintered fly ash, and expanded shale. The lightweight aggregates concrete comprising expanded shale exhibited higher compressive strength than the compressive strength of concrete produced using the other two lightweight aggregates.

2.7 Internal Curing on Lightweight Concrete

Internal curing (IC) is defined as a process by which the hydration of cement continues in concrete because of the availability of internal water in the pore structure of aggregate particles which does not form the portion of the mixing water. IC is curing of concrete from the inside out. A study done by Vosoughi, Taylor, & Ceylan, (2017) while investigating the effects of internal curing on jointed plain concrete pavements confirmed a reduction in plastic shrinkage and warping on the pavements done with lightweight fine aggregate as a replacement of river sand. The internally cured concrete recorded an increase in the compressive strength by 15% while the modulus of elasticity was decreased by 15%. The expanded shale, clay, or slate (ESCS) was utilized by the USA government to provide internal curing in concrete for the construction of ships (Holm & Ries, 2006). The same material has been adopted extensively throughout Russia, Japan, Europe, North and South America. IC is achieved by presoaking expanded shale, clay or slate (ESCS) lightweight aggregate before the mix is carried out. ESCS in the US is used to produce lightweight concrete for making green roofs. IC improves the hydration in concrete and subsequently contributes to the extension of the concrete's service life. Internal curing helps prevent, or reduce early age shrinkage and cracking of concrete by providing additional internal water (Vosoughi, Taylor, & Ceylan, 2017). Mousa et al., (2015) found that lightweight expanded clay aggregate (LECA) could be utilized as a self-curing agent to provide internal curing in concrete which cannot easily be achieved by conventional curing methods. The study found that internal curing of concrete using LECA helped counter self-desiccation and autogenous shrinkage in concrete.

2.8 The Unit Weight of Concrete

The densities of fresh and hardened concrete are of great interest as they affect the durability, strength, and resistance to permeability of concrete in structures. It affects the self-weight of structures significantly. The density of lightweight concrete is affected by components which include; air content, water-cement ratio, particle relative density, and absorbed moisture content by the lightweight aggregates. In the study of lightweight oil palm shell (OPS) concrete, Shafigh, et al., (2018) obtained a lightweight concrete with a dry unit density of 1950 kg/m^3 and compressive strength of 26 MPa after 28 days. The results obtained by Shafigh, et al., (2018) were almost similar to the results obtained in the current study. Grabois et al., (2016) assessed the properties of self-compacting lightweight concrete and found out that all concrete samples produced 28-day compressive strengths above 30 MPa and unit densities in the range of 1700–1900 kg/m^3. Nadesan and Dinakar (2017) studied the properties of lightweight concrete produced from incorporating fly ash waste and found that the structural lightweight aggregate concretes developed achieved 28-day compressive strengths between 28 and 70 MPa and air dry unit densities below 2000 kg/m^3. Babu and Thenmozhi (2018), on the other hand, obtained a lightweight concrete with a compressive strength of 42.6 MPa when they replaced steel fibers with a 40% of sintered fly ash lightweight aggregate in concrete. The addition of sintered fly ash aggregate showed an increase in the workability of the concrete but a decline in the mechanical properties such as compressive strength, split tensile strength, flexural strength, and elastic modulus.

2.9 Water absorption and durability of lightweight concrete

A study carried out by Ibrahim et al., (2020) on the durability of lightweight concrete comprising expanded perlite aggregate found out that the concrete had good durability properties. The water absorption was established to be in the range of 4.10% to 7.22%, while the chloride permeability was determined to be 354 to 844 coulombs and the chloride migration coefficient was in the range 11.90 to 17.07 ($\times 10-12$) m^2 /s. A research done by Andi (2017) on the amount of water absorbed by Styrofoam cube concrete specimen established that the concrete had water absorption of 11.97% while the water absorption test result on mortar cube specimen was 10.77%. Therefore, Styrofoam concrete was found to have a waterproof capability.

Youm et al., (2016) studied the mechanical properties and durability performance of lightweight aggregate concrete made with silica fume. The results of the research indicate that the durability against chemical deterioration of lightweight aggregate concrete incorporating silica fume depends largely on the compositions of hardened cement pastes in the concrete, whereas the durability against physical attacks depends on the types of aggregates used in mixing the concrete.

2.10 Tensile Strength of Concrete

Timoshenko and Goodier (1970) obtained a formula for determining the tensile strength of concrete according to the Equation (2.1) below.

$$\sigma_x = \frac{2P}{\pi L D} \tag{2.1}$$

Where;

σ_x; represents the splitting tensile strength, MPa or N/mm^2

P; is the maximum applied point load, N

L; is the length of the specimen, mm

D; is the diameter of the specimen, mm

According to this formula, the compressive force P is distributed along a loading strip, of a width equal to d, on both sides of the cylindrical specimen. A point load, P, in the plane corresponds to a load per unit length of the cylindrical specimen. They noted that the splitting tensile strength had a major impact on shear strength, crack control, concrete cover, and the spacing of reinforcements in concrete. Furthermore, Tunc, et al., (2020) developed an estimation method of measuring the tensile strength of lightweight concrete using a software program and found that it was consistent with the experimental results obtained during the study.

2.11 Static Elastic Modulus of Concrete in Compression

Elastic modulus of concrete, Ec, is defined as the slope of the line drawn from stress of zero to a compressive stress of 0.40f'c in a stress-strain diagram. Elastic modulus is an important property in determining the deflection and cracks in a structure. Ec is the ratio of stress to corresponding strain/deformation within the limit of proportionality of stress to strain. The stress is directly proportional to the strain within the limit of elasticity. American Society for Testing and Materials C469 (2010) defines the static modulus of concrete under static uniaxial compression according to Equation (2.2) below.

$$\text{Modulus of elasticity,} \quad E_C = \frac{0.4f_{c-\sigma(\epsilon_1)}}{\epsilon(0.4f_c)-\epsilon_1} \qquad (2.2)$$

Where;

Ec; is the modulus of elasticity in N/mm^2.

σ (ε_1); refers to compressive stress corresponding to strain ε_1 in N/mm^2.

ε ($0.4f_c$); refers to the strain corresponding to stress $0.4f_c$.

$f_{c;}$ is the ultimate or maximum stress in N/mm^2.

$\varepsilon_{1;}$ is the concrete strain of 0.00005.

A study by Szydlowki and Mieszcak, (2017) on the effect of lightweight concrete on post-tensioned long-span slabs showed that, despite having a lower modulus of elasticity, the lightweight concrete with fly ash gave better deflection values of 25.2 mm compared to 40.1 mm for normal-weight concrete. The lightweight concrete gave a density of $1710kg/m^3$ while the compressive and elastic modulus strengths were 33.5 MPa and 33 GPa respectively after 28 days.

2.12 Summary of Literature Review and Research Gap

A large number of research studies involving lightweight concrete revolve around the use of artificial aggregates either wholly or as partial replacement of conventional concrete materials. However, there are a few studies where natural lightweight materials have been used to produce structural lightweight concrete without blending with other artificial materials. Artificial lightweight concrete materials are very expensive to produce as they involve a lot of heating and sintering although they produce concretes with good mechanical properties. Water absorption in lightweight concrete is a common phenomenon that can affect negatively the mechanical properties of concrete if not carefully addressed. However, internal curing due to absorbed water into the pores of aggregates is beneficial to lightweight concrete. Lightweight concrete is an innovative material useful in high-rise buildings and long-span bridges. Extensive research on suitable natural materials to produce lightweight concrete is therefore paramount in minimizing overreliance on conventional concrete. In particular, the various researchers did not explore the use of tuff aggregates to produce lightweight concrete or conventional concrete. The researchers also failed to explore the use of natural lightweight aggregates for coarse or fine aggregate for any major structures constructed with lightweight concrete yet the material is less costly to extract and crush compared to normal-weight aggregates and artificial lightweight concretes. They emphasized more on the partial replacement of natural normal-weight aggregates with artificial aggregates.

From the studies reviewed, the use of lightweight concrete in high-rise buildings cannot be over-emphasized especially as a floor-filling material. A study done by Zareef (2010), Tunç et al., (2018) and others were in agreement that lightweight concrete is a good material for floor slabs in high-rise buildings. They found that the floor systems of the 28-storey Park

Plaza Hotel in St. Louis and Standard Bank building in Johannesburg were done using structural infra lightweight concrete. Similarly, the floor systems of the South Western Bell Telephone Office in Kansas City and the Bank of America Corporate Center were done using lightweight concrete were constructed using lightweight concrete.

2.13 Conceptual Research Framework

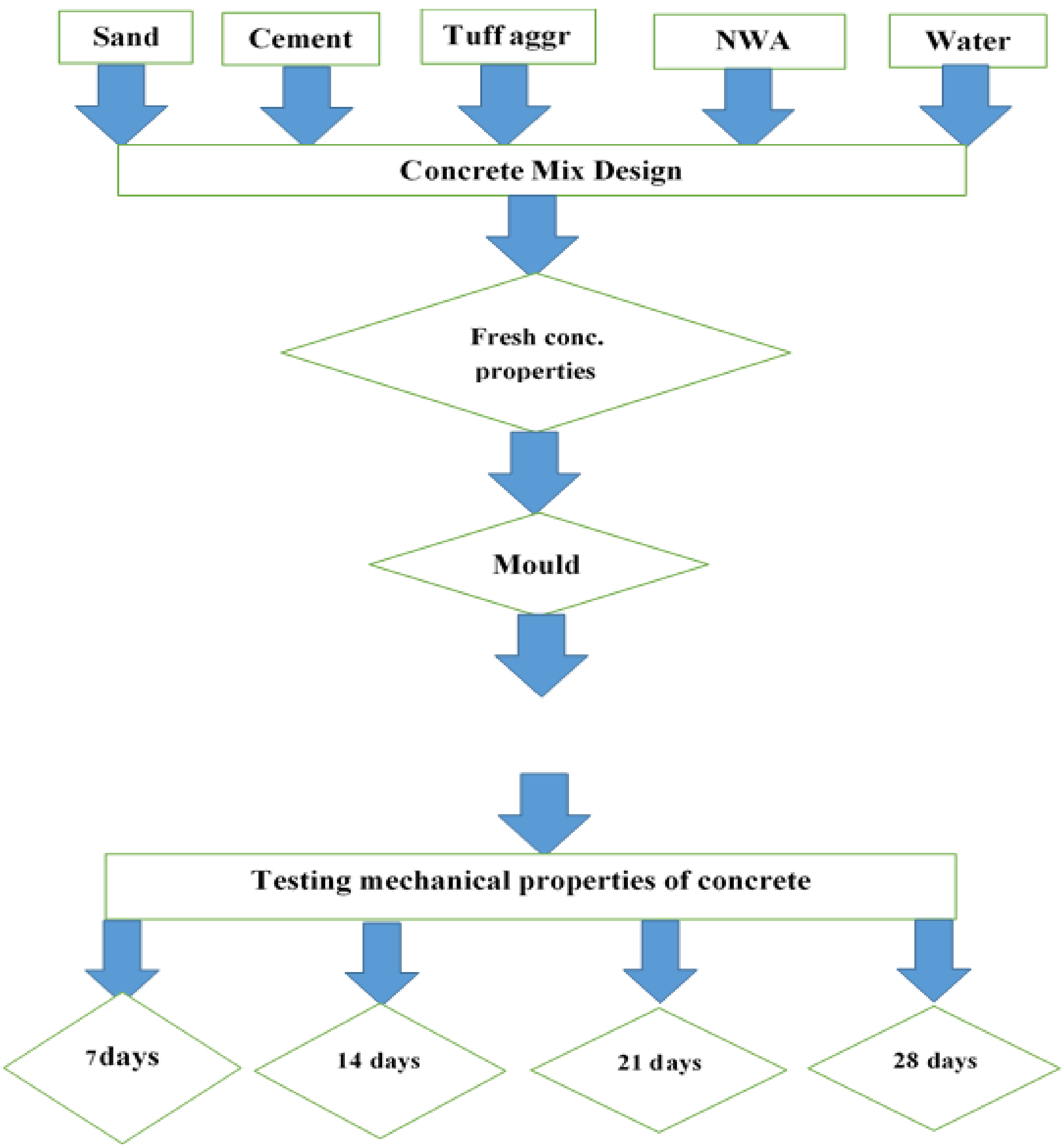

Figure 2.1: Conceptual Framework of the Research

CHAPTER THREE

MATERIALS AND METHODS

3.1 Introduction

The materials used for tuff lightweight concrete production include; water, Type 1 cement, sand, and crushed tuff aggregate while crushed granite aggregate was used in place of crushed tuff aggregate to produce normal-weight concrete. A superplasticizer was incorporated in some concrete mixtures to assess the effect on workability and strength properties.

3.1.1 Materials

3.1.1.1 Water

Any water suitable for consumption is recommended for concrete mixing purposes. Clean and potable water was obtained from the University of Nairobi laboratory for mixing and curing concrete. The water is supplied by the Nairobi Water and Sewerage company after the treatment process in compliance with BS 8550.

3.1.1.2 Cement

Ordinary Portland cement Type 1 with class strength of 32.5N complying with EN 197 -1 (2000) was used for this investigation. This cement is used for general constructions and is available in the local market hardware. Other types of cement available in the market include; Type 2 which is used for marine structures, Type 3 is used for high early strength, Type 4 is used when the amount and rate of heat generated must be kept low. Type 5 is a sulfate-resistant cement used in high alkaline soil or water.

3.1.1.3 Aggregates

The borrow site for tuff aggregate material was in Bahati area in Nakuru County, Kenya. Nakuru County is in Rift Valley Province located between longitudes 35°28' and 35° East and Latitude 0°13' and 10°10' South at an altitude of about 1912 meters above the sea level. Masonry blocks are normally extracted from tuff rock deposit formation for use in constructing walls of buildings in the area. The borrow site for the conventional aggregate material was Ongata Rongai quarry located in Kajiado County, Kenya. Kajiado is situated 17 km south of Nairobi Central Business District at an altitude of 1,731 above sea level. Fine aggregate for use during the study was river sand obtained from Kajiado river beds. Kajiado is a town in Kajiado County, Kenya which is located 80 km south of Nairobi, along the Nairobi – Arusha Highway. The samples were taken using gunny bags to the university of Nairobi laboratory for storage, analysis, and testing. Plate 3.1 shows the sample of sand harvested from Kajiado River which was used as fine aggregate. Plates 3.1 and 3. 2 show the tuff aggregate borrow site and tuff aggregate sample collected from Bahati area in Nakuru. Tuff rock formation in the site is found at an average depth of 1.5m beneath the ground level and extends several kilometers in the area of coverage. Plates 3.3 and 3.4 show the borrow site and sample of conventional aggregate material respectively. Coarse tuff aggregate was sourced from Bahati quarry in Nakuru town while granite coarse aggregate was obtained from Ongata Rongai quarry. The river sand was obtained from riverbeds of Ongata Rongai river streams in Kajiado County in Kenya.

Plate 3.1: River sand from Kajiado County.
(Source: Author)

Plate 3.2: Tuff aggregate heap and borrow site in Bahati quarry.
(Source: Author)

Plate 3.3: A sample of tuff aggregates from Bahati quarry.
(Source: Author)

Plate 3.4: Conventional aggregate borrow site from Ongata Rongai.
(Source: Author)

Plate 3.5: A sample of conventional aggregates from Ongata Rongai quarry.
(Source: Author)

3.1.1.4 Super Plasticizer

Commercially available polycarboxylic ether based super plasticizer complying with requirements of ASTM C-494 Type F and G (1999) was incorporated into concrete mixtures to assess the effect on workability and compressive strength properties of concretes**.** This superplasticizer is also referred to as polycarboxylate. This superplasticizer contains polycarboxylic ether as active ingredients. It is an innovative latest generation superplasticizer based on polycarboxylic ether (PCE) polymers and is specially engineered for ready-mix concrete where the consistency of aggregates may be variable. The recommended dosage rate of the superplasticizer is 0.60 to 2.00 litres per 100 kg of total cementitious material. This is in the range of 0.6% to 2.0% by weight of cement content in the mixture. It is used for the production of high-quality ready-mix concrete. It is whitish to light brown liquid with a specific gravity of 1.073 and pH value 5.0 – 7.0. This superplasticizer is available in the local market.

3.1.2 Experimental Program

Various tests on concrete materials and concrete specimens were carried out following various standards which include; ACI, ASTM and BS Standards. Some of the experimental tests and processes carried out include; grading and determination of physical properties of aggregates, concrete mix design, investigation of mechanical properties of fresh and hardened concrete.

3.1.2.1 Grading of Aggregates

The process of grading fine and coarse aggregates was done as per procedures set out in BS 812 (1995). Good grading of aggregates results in the economical use of cement material and provides concrete with better properties.

3.1.2.1.1 Grading of Fine Aggregate

Grading of fine aggregate was carried out following the procedures set out in BS 812 (1995). The grading of sand was carried out using test sieves of sizes: 10 mm, 5 mm, 2.36 mm, 1.18 mm, 0.6mm, 0.3 mm and 0.15 mm.

3.1.2.1.2 Determination of Fineness Modulus of Sand

This was done according to procedures set out in BS 812 (1995). The Fineness Modulus (FM) is an empirical figure obtained by adding the total percentage of the sample of an aggregate retained on each of a specified series of sieves and dividing the sum by 100.

3.1.2.1.3 Grading of Coarse Aggregates

The process of grading aggregates was done following the guidelines in BS 812 (1995) using BS test sieves of sizes 50 mm, 37.5 mm, 20 mm, 14 mm, 10 mm, 5 mm and 2.36 mm.

3.1.2.2 Determination of Physical Properties of Aggregates

3.1.2.2.1 The Specific Gravity of Sand

The specific gravity was done according to BS 812 (1995). A sample of about 15gms of sand was thoroughly washed to remove material finer than 0.075 mm. The washed sample was transferred to a tray and oven-dried to 105ºC for 24 hrs. The weight of pycnometer was measured and recorded as w_1. A part of the sample was placed in the pycnometer and weighed (w_2), the pycnometer containing sample was then filled with water and weighed (w_3). The pycnometer was then emptied and refilled with water and weighed (w_4). The specific gravity of sand, G_S ,was calculated using Equation (3.1) below.

$$G_S = \frac{(w_2 - w_1)}{(w_4 - w_1 - w_3 + w_2)} \tag{3.1}$$

3.1.2.2.2 The Rodded Unit Weight of Coarse Aggregate

This process involved the determination of the unit weight of coarse aggregate according to ASTM-C29 (2009) in a compacted condition. This ensures the aggregates settle and fill the whole volume of the container. The rodded unit weight of saturated surface dry aggregate, M_{SSD}, was calculated using Equation (3.2) below.

$$\text{Rodded unit weight, } M_{SSD} = \frac{(G-T)(1+\frac{A}{100})}{V} \tag{3.2}$$

Where;

$M_{SSD;}$ is the rodded unit weight of the saturated surface dry aggregate, kg/m^3.

G; is the combined mass of the oven-dry aggregate and the bucket, kg

T; is the mass of the bucket alone, kg.

V; is the volume of the bucket, m^3.

A; is the percentage absorption.

3.1.2.2.3 The Specific Gravity of Coarse Aggregate in SSD Condition

It is the ratio of the mass of aggregate to the mass of a volume of water equal to the volume of the aggregate sample. The specific gravity was done as per the procedures set out in BS 882 (1992). The specific gravity, $Gs_{SSD},$ on a saturated surface-dried condition was calculated using Equation (3.3) below.

$$Gs_{SSD} = \frac{m_1}{(m_1 - (m_2 - m_3\,)} \tag{3.3}$$

Where;

m_1; is the mass of the saturated surface dried aggregate in the air (in g).

m_2; is the mass of container with water and saturated aggregate sample (in g).

m_3; is the mass of empty container (in g).

Plate 3.6: Sample of SSD tuff aggregate sample.
(Source: Author)

3.1.2.2.4 The Specific Gravity of Coarse Aggregate in Oven-Dry Condition

The specific gravity of aggregate in oven-dry condition, Gs_{OD}, was determined following the guidelines set out in BS 812 (1995) as per Equation (3.4) below.

$$G_S OD = \frac{m_4}{[m_1 - (m_2 - m_3)]} \tag{3.4}$$

Where;

m_1, m_2, and m_3 are as defined in equation (3.3) above.

m_4; is the mass of oven-dried aggregate in the air (in g).

3.1.2.2.5 The Aggregate Water Absorption

The water absorption test was carried out according to BS 812-2 (1995). To reach this condition, the aggregates were kept in water for at least 24 hours. The water was allowed to drain to dry up the surface of the aggregate. The water absorption (as % of dry mass) was calculated using Equation (3.5) below.

$$A = \frac{100(m_1 - m_4)}{m_4} \tag{3.5}$$

Where;

A; is the percentage amount of water absorbed by aggregate.

m_1 and m_4 are as defined earlier in equation (3.4).

3.1.2.3 Concrete Mix Design

The most common methods of concrete mix designs include ACI and BS methods. The concrete mix design process for tuff and normal-weight concrete was done according to American Concrete Institute (1998) while targeting 28-day compressive strengths of 25 N/mm^2 with a w/c ratio range from 0.4 to 0.6. The process involved the batching by weight of aggregates, cement, and water ingredients to produce 1 m^3 of concrete in the mixture. This process is very important in the preparation of concrete to meet the specific needs of the construction. It is important to note that concrete samples either of good or bad quality are made of the same ingredients. Bad quality concrete results in a substance of poor consistency. The resulting concrete is honeycombed, nonhomogeneous and weak. Poor quality concrete is made simply by incorporating cement, aggregate, and water. These are the same ingredients used for making quality concrete. The knowledge applied for mixing the concrete is responsible for the big difference in quality. Concrete mix design involves the process of choosing suitable proportions of materials that will not only allow for a high degree of workability but also allow for convenient transportation, placement, and compaction of concrete. Furthermore, a good mix design should have adequate strength and durability to allow it to withstand the loadings imposed during its design life, without experiencing significant distortions. Thus, to attain the desired strength and durability, proper proportioning of concrete constituent materials is important. This process was intended to obtain a practical combination of components in the mix for lightweight structural concrete. The mixtures were prepared to produce a unit volume of concrete for each type of concrete as per the respective weights of constituent materials according to Table A.10 in the Appendix Section.

3.2.3.1 Mixing Water and Air Content Estimation

For concrete mixes with slump values of 25- 75 mm and typical entrapped air content of 2%, Table 3.2 of the ACI (1998) was used. The recommended volume of mixing water in one cubic meter of PCC as per the code is 190 kg/m^3. However, the actual amount of mixing water required was determined through trial mixes.

3.2.3.2 Determination of Water-Cement Ratio

The water-cement ratio was determined in accordance with procedures set in America Concrete Institute (1998). However, workable water-cement ratios that could give desirable results of unit weight and strength tests were determined through trial mixes.

3.2.3.3 Determination of Cement Content

The amount of cement was obtained by dividing the water content by the w/c ratio. The cement content is equaled to; 190 /0.57 =333 kg/m^3

The recommended volume of Portland cement in one cubic meter of PCC was computed as follows;

Volume of cement in the mix=333 kg/m^3/ (1000 kg/m^3x3.15) =0.106 m^3

3.2.3.4 Coarse Aggregate Content

Using the coarse aggregate content as provided by the ACI code (1998) and given the nominal maximum aggregate size of 19 mm and fineness moduli of fine aggregates of 2.70, the recommended volume fraction of coarse aggregate in the mix was found to be 0.63. This means the coarse aggregate would occupy 63 percent of the total volume. However, this volume of aggregate included the volume of air between the aggregate particles. Therefore, 63 percent volume was converted to the weight of aggregate in saturated surface dry (SSD) condition. Given the rodded unit weight of 1180 kg/m^3, the quantity of coarse aggregate component in a unit volume of concrete was 0.63x1 m^3. Therefore 0.63 m^3x1180 kg/m^3 was the weight of coarse aggregate in 1 m^3 of the mix. The mass of coarse aggregate in SSD in 1 m^3 of concrete, therefore, was 743.0 kg.

Now, the recommended volume of coarse aggregate (which was the solid volume of coarse aggregate) in one cubic meter of PCC was calculated as follows;

Volume of aggregate in the mix=743 kg/ (1000 kg/m^3x1.89) =0.392 m^3.

3.2.3.5 Fine Aggregate Content

The fine aggregate content by volume was found by subtracting the other constituent volumes from the unit volume and the result compared with that from the estimated unit weight of fresh concrete in Table 3.6 of the ACI code (1998). The volume of fine aggregate in the mix was obtained by subtracting all the other fractions from 1.0 m^3 as follows;

$1.000-0.190-0.02-0.106-0.392=0.292\ m^3$

The mass of fine aggregate in a $1m^3$ of concrete=$0.292\ m^3 x1000\ kg/m^3 x2.63=768\ kg/m^3$.

The estimated density of tuff fresh concrete from the weights of constituent components was **2031** kg/m^3; this was less than the maximum density of LWC of **2200** kg/m^3 as defined by Euro Code 2 (2008). The density of normal weight concrete was **2510** kg/m^3. Usually, to make trial batches, a sample less than the unit volume is made – a typical trial batch size was 0.03 m^3. Once the trial batch is made, it is tested for the slump, compressive strength, and any other required concrete property.

Plate 3.7: Mixing of concrete materials in a rotating mixer in the lab.
(Source: Author)

3.2.4 Investigation of Workability Properties of Fresh Concretes

3.2.4.1 Slump Test

The slump test was performed to determine the workability of fresh concrete mix. The test was determined as per the procedures set out in BS EN 12350-2 (2009). The tests were run during the first 2-minute resting period after the mixing process.

3.2.4.1.1 Effect of Water-Cement Ratio on the Slump of Concrete Mix

Different concrete batches for tuff and granite concrete were prepared with varied w/c ratios. The determination was made with concrete samples with w/c ratios varied from 0.40, 0.45, 0.50, 0.55 and 0.60 in turn in the mix. The slump samples and values were denoted S1, S2, S3, S4, and S5. Three samples for each mix with a given water /cement ratio were taken for test and the averaged results recorded.

3.2.4.1.2 Effect of Superplasticizer on the Slump Test of Concrete Mix

At least six separate mixtures of concrete were prepared while varying the dosages of SP from 0%, 0.5%, 1.0%, 1.5%, 2.0% and 2.5% by weight of cement. A water-cement ratio of 0.4 was applied to all the concrete samples. This procedure was done for both tuff and normal weight concrete samples. Three samples for each mix with a given SP dose were taken for test and the averaged result recorded.

3.2.4.1.3 Effect of Superplasticizer on the Compaction Factor of Concrete

The test was carried out according to BS EN 12350-2 (2009). Eighteen concrete samples from each concrete were made with w/c ratio of 0.4 while varying the superplasticizer dosages from 0%, 0.5%, 1.0%, 1.5%, 2.0% and 2.5%. Concrete mixtures were denoted S1, S2, S3, S4, S5, and S6 in the order of increasing dose of superplasticizer. Three samples for each mix with a given SP dose in each case were taken for test and the averaged result recorded. For this test, the top hopper was filled with concrete. The trap door was then opened to allow the concrete to fall into the lower hopper. The trap door of the lower hopper was then opened to allow the concrete to fall into the cylinder. The surface of the concrete in the cylinder was then struck and the cylinder full of concrete weighed. The amount of concrete was then compared with the amount that filled the cylinder when the concrete was compacted in layers. The compaction factor was computed as per Equation (3.6) as follows.

$$Compaction\ factor, cf = \frac{mass\ of\ free\ fall\ concrete}{mass\ of\ compacted\ concrete} \tag{3.6}$$

3.2.5 Determination of Physical Properties of Hardened Concrete

3.2.5.1 Unit Weight of Concrete

The average unit weights of concrete mixtures were measured according to procedures set out in ASTM, C 29 (2009). The unit weights of concrete were taken for fresh and hardened state. Density test for 14 and 28-day old concrete 150 mm cube samples were calculated using Equation (3.7) below.

$$\text{Dry density (bulk density)}, g_1 = \frac{A\rho}{(C - D)} \tag{3.7}$$

Where;

A; is the mass of oven-dried concrete sample in air, (gm).

C; is the mass of saturated surface-dry concrete sample in air, (gm).

D; is the mass of sample in water after immersion, (gm).

ρ; is the density of water (kg/m^3).

The unit densities of fresh concrete cube samples were taken by getting the mass of each cube and dividing by the volume of the cube sample. Three samples for each type of concrete were taken for test and the averaged result recorded.

3.2.5.2 Investigation of Compressive Strength of Concrete

Compressive strength is defined as the maximum stress a concrete specimen can withstand when loaded axially. Concrete mixtures with a water-cement ratio of 0.4 were prepared and cast in 150 mm cubes. Three concrete cube specimens for each category were tested after 7, 14, 21, 28, 35 and 42 days. The current research focused on the determination of the mechanical properties of concrete after 28 days of curing. However, internal curing is expected to take place after 28 days owing to the presence of water in the pores of tuff concrete. Therefore, the compressive strength of concrete was tested after 35 and 42 days to check the effect of internal curing on the strength of concrete. The average compressive strength for each day was recorded. Three concrete cubes of the same age for each type of concrete were tested using universal test machine and the average result recorded. The test was carried out according to BS 1881-part 116 (1983). The actual compressive strength of concrete was computed according to Equation (3.8) below.

$$f_c = \frac{P}{A} \tag{3.8}$$

Where;

f_c; is the actual compressive strength in N/mm^2.

P; is the failure load in N.

A; is the area of the cube specimen in mm^2.

The effects of varying the w/c ratio on the 28-day compressive strength were determined. The other test involved the investigation of the effect of varying the superplasticizer dosage on the strength of concrete. Another test involved assessing the rate of development of compressive strength of concrete with age.

Plate 3.8: Cube specimen being tested for compressive strength.
(Source: Author)

3.2.5.2.1 Determination of Compressive Strength of Concrete with Age

Concrete mixtures with a water-cement ratio of 0.4 were prepared and cast in 150 mm cubes. Three concrete cube specimens for each category were tested after 7, 14, 21, 28, 35 and 42 days. The average compressive strength for each day was recorded. Three concrete cubes of the same age for each type of concrete were tested using universal test machine and the average result recorded.

3.2.5.2.2 Effect of W/C Ratio on Compressive Strength of Concrete

The determination was made while varying w/c ratios from 0.40, 0.45, 0.50, 0.55,0.60, and 0.65. Concrete mixtures for each w/c ratio were cast into 150 mm cube moulds. Three samples for each mix were taken for test and the averaged result recorded. The average 28-day compressive strengths of the sampled cubes were taken.

3.2.5.2.3 Effect of Superplasticizer on Compressive Strength of Concrete

Concrete cube specimens with w/c of 0.4 were prepared for this test. The superplasticizer dosages were varied from 0.0%, 0.5%, 1.0%, 1.5%, 2.0% and 2.5%. Three cube specimens for each dosage in each category were prepared, tested and the average results recorded for compressive strength after 28 days.

3.2.5.3 Determination of Splitting Tensile Strength of Concrete

Plate 3.9: Cylindrical specimen being tested for splitting tensile strength. (Source: Author)

Testing of splitting tensile strength of concrete was done in accordance with BS 8110 (1983). The test involved the application of a compression line load at a constant rate along a concrete cylinder specimen placed with its axis horizontal between two compressive platens as shown in Plate (3.9) above. Four concrete sample specimens for each type of concrete were cast on 150 mm x 300 mm size cylinder moulds. The w/c ratio for the mixtures was kept constant at 0.4. There was no addition of SP to the mix samples. The specimens were cured and tested after 28 days. The average test results for tuff aggregate concrete were compared with those of granite aggregate concrete. The tensile strengths were determined according to Equation (3.9) below which was formulated by Timoshenko and Goodier (1970).

$$\sigma_x = \frac{2P}{\pi LD} \tag{3.9}$$

Where;

σ_x; represents the splitting tensile strength in MPa or N/mm^2.

P; is the maximum applied point load in N.

L; is the length of the specimen in mm.

D; is the diameter of the specimen in mm.

3.2.5.4 Determination of Static Elastic Modulus of Concrete

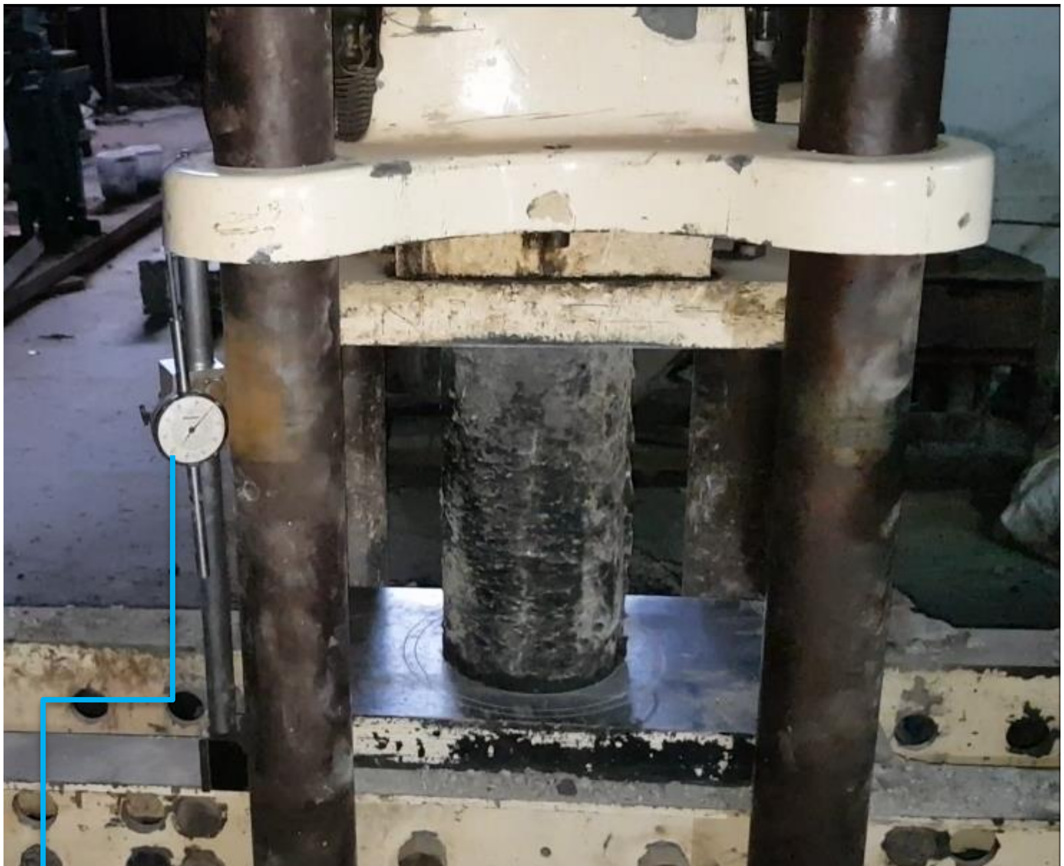

Plate 3.10: Cylindrical specimen being tested for elastic modulus.
(Source: Author)

Elastic modulus test involves the determination of a gradient of a line drawn from the point of origin in a stress-strain diagram to a point of compressive stress of 0.4f'c. Concrete specimens were prepared and tested for elastic modulus according to ASTM C469 (2010). Three cylindrical specimens for each category of tuff and granite aggregate concrete were prepared while maintaining the water-cement ratio constant at 0.4. There was no addition of the superplasticizer to both concrete samples for this test. A strain gauge was positioned longitudinally on the compression-testing machine and set to a zero mark as shown in Plate (3.10) above. Starting from zero, a continuous compression load was axially applied on the specimen. The compression loading was increased at a constant rate of 10 kN and each time the strain gauge deflections were taken. The deflections were converted to strain readings by multiplying them by 0.0254. As the specimen deformed, the strain gauge deflected until the maximum ultimate load was reached. All the concrete specimens were tested after 28 days. By averaging the values of strains and stress for specimen per loading cycle in each case, representation graphs were plotted for comparison and elastic moduli results for concrete samples were determined using Equation (3.9) below.

$$\text{Modulus of elasticity,}\quad E_C = \frac{0.4f_{c-\sigma(\epsilon_1)}}{\epsilon(0.4f_C)-\epsilon_1} \qquad (3.10)$$

Where;

Ec is the modulus of elasticity in N/mm^2.

σ (ε_1); refers to compressive stress corresponding to strain ε_1 in N/mm^2.

ε ($0.4f_c$); refers to the strain corresponding to stress $0.4f_c$.

$f_{c,}$ is the ultimate or maximum stress in N/mm^2.

ε_1, is the strain of 0.00005.

CHAPTER FOUR

RESULTS AND DISCUSSION

4.0 Introduction

This chapter presents the results of grading of aggregates; mix design of concrete, properties of concrete in fresh and hardened conditions. The results of the study have also been discussed in this chapter.

4.1 Grading of Aggregates

Figures 4.1, 4.2, and 4.3 below show the different grading curves for sand, tuff, and granite aggregates respectively. The grading curves were found to fall within the acceptable limits of the BS 812-102 (1989). The effective sizes of the particle size distribution (PSD) (D_{10}), (D_{30}), and (D_{60}) passing were 0.27 mm, 0.50 mm and 0.90 mm respectively for fine aggregates (sand). Similarly, the effective sizes (D_{10}), (D_{30}) and (D_{60}) passing were 7.0 mm, 10.10 mm and 10.50 mm respectively for tuff coarse aggregates. The effective sizes (D_{10}), (D_{30}) and (D_{60}) passing were 6.0 mm, 10.10 mm and 10.70 mm respectively for conventional aggregates. The coefficients of uniformity, Cu, which is given by the ratio $\frac{D_{60}}{D_{10}}$ for fine, tuff and granite aggregates were determined to be 3.3, 1.5 and 1.8 respectively. The coefficients of curvature, C_C, which is calculated from equation (4.1) below, were found to be 1.02, 1.38 and 1.6 for sand, tuff and granite aggregates respectively. These results were checked against

AASHTO classification which provides that C_U should be greater than 4, i.e. $C_U > 4$ while C_C should be greater than 1 but less than 3, i.e. $3 > C_C > 1$ for aggregates to considered well-graded. It follows therefore, that C_U for all the aggregates, being lower than 4 implies that they were not well graded. The coefficients of curvature C_C however were within the limits for well-graded aggregates. Any classification outside the parameters provided by AASHTO is uniformly or poorly graded aggregates. Save for the co-efficient of curvature, C_C, the aggregates were not well-graded but they were uniformly graded according to AASHTO specifications. This means the cement paste and fine aggregate required to fill the void spaces between aggregates in the mix were more in quantity and could have led to an increase in the cost of producing the concrete than the case would be if well-graded aggregates were used.

Well-graded aggregates enhance the workability, placement, degree of compaction, and durability of concrete. In contrast, aggregates that are poorly graded result in the under-filling of voids in particles resulting in entrapped air and can negatively affect the workability of the concrete mix. A concrete made from poorly graded materials does not protect the reinforcement from corrosion due to possible leakages. Results of sieve analysis were presented in Tables B.1, B.2, and B.3 of the Appendix. The coefficient of curvature, C_c, of aggregate is given by Equation (4.1) below;

$$Coefficient\ of\ curvature, C_c = \frac{((D_{30})^2)}{(D_{10})(D_{60})} \tag{4.1}$$

Where;

D_{10}, is the effective size of particle size distribution at 10%

D_{30}, is the effective size of particle size distribution at 30%

D_{60} is the effective size of particle size distribution at 60%

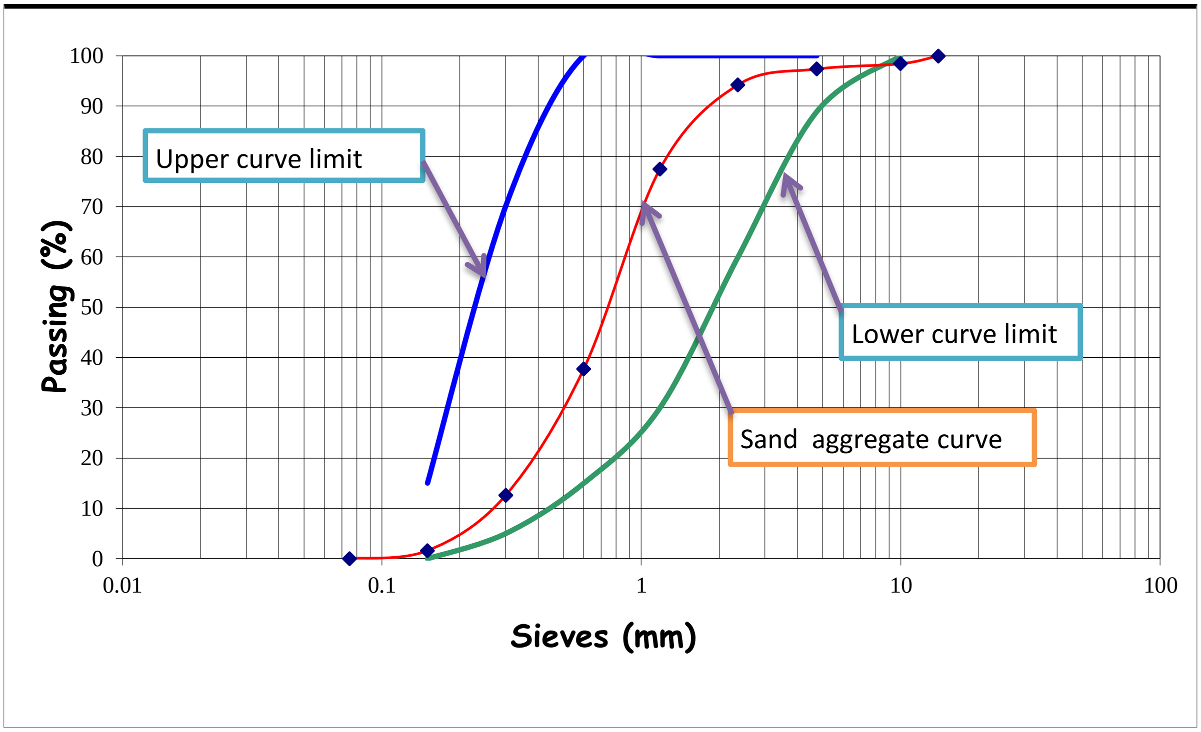

Figure 4.1: Grading Curve for Sand

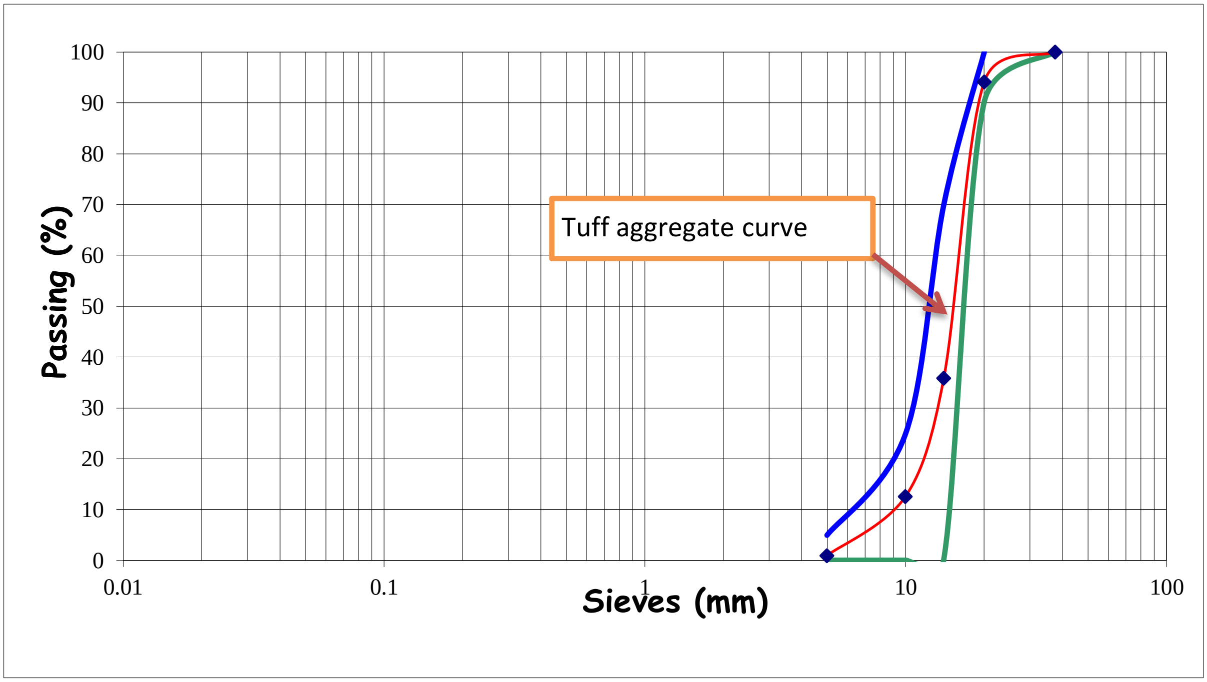

Figure 4.2: Grading Curve for Tuff Aggregate from Bahati area in Nakuru

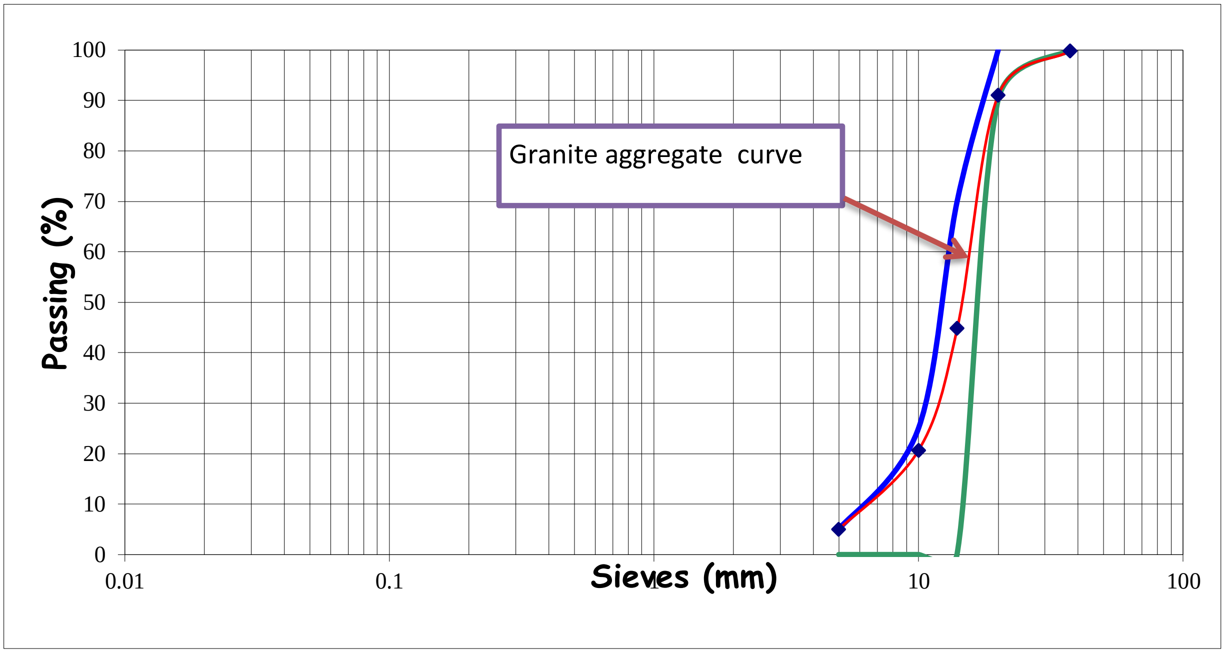

Figure 4.3: Grading Curve for Coarse Granite from Birika in Kajiado County

4.2 Physical Properties of Aggregates

4.2.1 The Fineness Modulus of Sand

The fineness modulus of the sand was found to be 2.70. A fineness modulus of 2.70 indicates that the sand was of medium-range and zone II according to BS 812-part 103. It was not too fine nor too coarse. Medium sand has fineness modulus ranging from 2.6 – 2.9. The fineness modulus values in the range of 2.40 to 3.00 are common in Portland cement concrete (PCC) mixtures. A fineness modulus value of 2.7 is fairly good for workability and finishing of concrete. It is important for good texture and finishes in concrete works. Fineness Modulus (FM) of sand is also used to determine the degree of uniformity of aggregates in PCC mix designs. It is also significant in determining the relative coarseness or fineness of the aggregate in a mixture. Tables of fineness moduli and zones of sand are presented in Appendix Tables B.4 and B.5 respectively.

4.2.2 The Specific Gravity of Sand

The specific gravity of sand was obtained by substituting the values in Equation (3.1) of Chapter 3. The average specific gravity was determined to be 2.63. Sand particles are composed of quartz and iron minerals. The specific gravity value of 2.63 is slightly lower than a value of 2.65 which is provided in design manuals for sand aggregate used in the concrete mix design. The sand could be having slightly less composition of quartz and iron minerals. The sand grains could also be slightly porous making the specific gravity lower than 2.65 which is normally attributed to good quality sand. This slightly low value affects the fresh and hardened concrete properties by lowering the density of concrete. The aggregate specific gravity was used in determining the weight-to-volume of concrete mix design according to BS 882.

4.2.3 The Specific Gravity of Coarse Aggregate in OD Condition

The specific gravity (SG) of a substance is a measure of its density. The specific gravity of coarse aggregate on the oven-dried condition was calculated using Equation (3.3). The results revealed an average specific gravity of 1.55 and 2.65 for tuff and granite aggregates respectively. The specific gravity of 2.65 obtained for granite aggregate was within the range of 2.6-2.7 for granite rock while that of tuff rock was within 1.4-1.7. Lower specific gravity for tuff aggregates was because they have more pores in their cellular structure than granite aggregates. A lower specific gravity of tuff aggregates indicates that these rocks are lighter in weight than conventional aggregates. Furthermore, these aggregates absorb more water into the pores than granite aggregates. Conventional aggregates are denser since the particles are tightly packed within the cellular structure with minimal or tiny pore spaces left. The specific gravity of aggregate affects the proportioning of concrete materials as given in ACI (1998) when carrying out mix designs. The source of data is found in Table B.7.

4.2.4 The Specific Gravity of Coarse Aggregate in SSD Condition

The average specific gravity of tuff aggregate was found to be 1.89 while that of granite aggregate was as high as 2.66. When compared, the specific gravity of tuff aggregate increased from 1.55 to 1.89 when tuff aggregate was soaked in water for 24 hours representing a 21.9% increase in specific gravity while that of granite aggregate increased slightly from 2.65 to 2.66 representing an increase of less than 1.0%. This means tuff aggregates absorb more water than granite aggregate since they have numerous pores in their cellular structures. The tendency of tuff aggregates to absorb more water than granite aggregates affects the water-cement ratio and the workability of fresh concrete. Due to its high absorptive nature, the water in the aggregates contributes to internal curing in tuff concrete than in conventional concrete while in service. The reason for pre-wetting tuff aggregates before mixing is to prevent absorption of water by aggregates which can affect the workability of concrete. The specific gravity of aggregate helps in establishing the weight-volume relationship of concrete mix design. The source of data is found in Table B.8.

4.2.5 Coarse Aggregate Water Absorption

The water absorption of aggregate was carried out as per the procedures set out in BS 812 (1995). The average water absorption test on tuff aggregate samples was higher reaching 21.6% while that of granite samples was as low as 0.25%. The aggregate samples were presoaked for 24 hours to saturation condition. From the results of water absorption, tuff aggregates were found to absorb more water than granite aggregates due to numerous larger sizes of pores within their cellular structure. As a result of large pores, tuff rocks have high permeability and the ability to store water which explains the high rate of water absorption. The crystal structures forming granite rocks are tightly interlocked leaving tiny pore spaces to store water. The absorption capacity and rate of absorption are especially important for mix design calculations to correctly establish the water-cement ratio which controls the workability, strength, and permeability characteristics of concrete. Due to high water absorption of tuff aggregate, the actual and effective water-cement ratio required to produce high workability mix is usually lowered. Many factors affect the actual water-cement ratio of lightweight concrete mixtures. The most important ones include; water absorption, the state of moisture content, and the amount of porous aggregate in a concrete mix. The reduction of the water-cement ratio in fresh concrete is usually considered a negative phenomenon that can lead to loss of concrete workability. As a result, the aggregate to be used in a lightweight concrete mix should be saturated with water to protect the fresh concrete from the water-cement ratio reduction and loss of mixture workability.

4.2.6 Results of Rodded Unit Weight of Coarse Aggregate

The results revealed an average rodded unit weight, M_{SSD}, of 1180 kg/m^3 for tuff aggregate in saturated surface dry conditions. A similar test for granite aggregate produced a rodded unit weight of 1600 kg/m^3. The bulk density of granite aggregate was found to be in the range of 1520-1680 kg/m^3 which is normally specified in most design standards for conventional aggregates. The results were checked against the requirements set in BS 812 part 2. The difference in the unit weights was 420 kg/m3. This means concrete produced from tuff concrete could well be lighter in weight than conventional concrete by about 26%. This difference in unit weight of coarse aggregate significantly affects the overall weight of a concrete structure. The presence of numerous pores in the cellular structure of tuff aggregates reduces the rodded unit weight as compared to granite aggregates. In granite aggregates, the particles are interlocked leaving tiny pore spaces between the particles. The composition of these rocks also contributes greatly to their varying bulk densities. Tuff rocks contain fragments of bedrocks, tephra, volcanic ash, magma, and other materials which are less dense. Granite rocks are hard, granular, and crystalline consisting mainly of quartz, mica, and feldspar particles which are responsible for higher unit weight. The bulk density is required for mixture proportioning. The results of unit weights were presented in Appendix Tables B.9 and B.10 respectively.

4.3 The Specific Gravity of Portland cement

Ordinary Portland cement (type one) was used throughout this research. The cement used had a specific gravity value of 3.15 as per the manufacturer’s specification. The amount of cement was obtained by dividing the water content by the w/c ratio. The cement content is equaled to; 190 /0.57 =333 kg/m^3

The recommended volume of Portland cement in one cubic meter of PCC was computed as follows;

Volume of cement in the mix=333 kg/m^3/ (1000 kg/m^{3}x3.15) =0.106 m^3

4.4 Coarse Aggregate Content

Using the coarse aggregate content as provided by the ACI code (1998) and given the nominal maximum aggregate size of 19 mm and fineness moduli of fine aggregates of 2.70, the recommended volume fraction of coarse aggregate in the mix was found to be 0.63. This means the coarse aggregate would occupy 63 percent of the total volume. However, this volume of aggregate included the volume of air between the aggregate particles. Therefore, 63 percent volume was converted to the weight of aggregate in saturated surface dry (SSD) condition. Given the rodded unit weight of 1180 kg/m^3, the quantity of coarse aggregate component in a unit volume of concrete was 0.63x1 m^3. Therefore 0.63 m^{3}x1180 kg/m^3 was the weight of coarse aggregate in 1 m^3 of the mix. The mass of coarse aggregate in SSD in 1 m^3 of concrete, therefore, was 743.0 kg.

Now, the recommended volume of coarse aggregate (which was the solid volume of coarse aggregate) in one cubic meter of PCC was calculated as follows;

Volume of aggregate in the mix=743 kg/ (1000 kg/m^{3}x1.89) =0.392 m^3.

4.5 Fine Aggregate Content

The fine aggregate content by volume was found by subtracting the other constituent volumes from the unit volume and the result compared with that from the estimated unit weight of fresh concrete in Table 3.6 of the ACI code (1998). The volume of fine aggregate in the mix was obtained by subtracting all the other fractions from 1.0 m^3 as follows;

$1.000-0.190-0.02-0.106-0.392=0.292\ m^3$

The mass of fine aggregate in a $1m^3$ of concrete=$0.292\ m^3 x1000\ kg/m^3 x2.63=768\ kg/m^3$.

The estimated density of tuff fresh concrete from the weights of constituent components was **2031** kg/m^3; this was less than the maximum density of LWC of **2200** kg/m^3 as defined by Euro Code 2 (2008). The density of normal weight concrete was **2510** kg/m^3. Usually, to make trial batches, a sample less than the unit volume is made – a typical trial batch size was 0.03 m^3. Once the trial batch is made, it is tested for the slump, compressive strength, and any other required concrete property.

4.6 Concrete Mix Design Results

The results were presented in Table B.21 of the appendix section. The results of the mix design carried out in this study produced unit densities of 2031 kg/m^3 and 2510 kg/m^3 for tuff and normal-weight concretes respectively when estimated using the weights of constituent materials. The results of the mix proportions of cement, sand, and coarse material by weight were determined to be 1: 2.3: 2.2 for tuff concrete and 1: 2.3: 3.7 for the conventional concrete while maintaining a range of water-cement ratio from 0.40 to 0.60 depending the type of workability desired and the particular application of concrete.

4.7 Physical Properties of Fresh Concrete

4.7.1 Effect of W/C Ratio on the Workability of Concrete Mix

Since the target tuff concrete compressive strength was 25 MPa, the America Concrete Institute (1998) estimates 0.57 as the w/c ratio from America Concrete Institute Code (1998) however, the values were determined experimentally to establish a practical w/c ratio. From Figure 4.4 below, the workability of both tuff and granite concrete was enhanced when the w/c ratio in the mix was increased. However, conventional concrete exhibited higher workability than tuff concrete. This is because tuff aggregates absorbed more water leaving little water for hydration of cement paste and lubrication of aggregates resulting in lower slumps. However, the slump values of both concrete samples increased with an increase in the mixing water. Generally, normal-weight concrete revealed higher workability than tuff aggregate concrete for a given water-cement ratio. In the experiment, the cement content was kept constant while varying the water content throughout all the mixtures. The aggregates were dried in the air before mixing. From the results, tuff concrete mix with a w/c ratio of 0.4 produced a slump value of 12 mm while normal-weight concrete had a slump of 27 mm. A tuff concrete sample with a w/c ratio of 0.4 could not mix thoroughly well as more water was absorbed by porous aggregates making the mixing process difficult and resulting in the concrete of low consistency. This is because the cement particles failed to fully dissolve and interact with fines into cement paste. It was noted that an increase in w/c ratio from 0.55 to 0.60 produced a slump of 40 mm from 30 mm while normal-weight concrete produced a slump of 75 mm from 60 mm. The increase in the slump of normal-weight concrete was higher as seen in the sharp rise in the slope of the curve especially when the w/c ratio was increased from 0.5 to 0.6. Increasing the w/c ratio to 0.55 resulted in slump values of 30 mm and 60 mm for tuff and normal-weight concrete respectively. A water-cement ratio of 0.6 resulted in slump values of 40 mm and 75 mm for tuff and granite concretes respectively. Slump values of tuff aggregate concrete were generally lower than those of normal-weight concrete because tuff aggregates absorbed more water into the pores leaving little water necessary for increased workability of the concrete. Water meant for the slump is the water remaining after water was absorbed by the aggregates. High water absorption contributed to slump loss in tuff concrete subsequently leading to reduction of workability, unlike granite concrete where the effective water for workability remains unabsorbed. However, too much water in the mix resulted in the segregation of concrete. Too much water is therefore not recommended as it reduces the compaction of concrete and increase the chances of concrete bleeding and segregation. This results in the formation of voids and reduction of concrete strength. The segregation occurred when the sand and coarse aggregate components settled at the bottom while the cement paste formed at the top of the concrete mass. Slump values from 25 mm to 75 mm are normally specified for concrete used for floor slabs in ACI Code (1998) where the compaction of concrete is necessary. The tabulated results are in Appendix Table B.13.

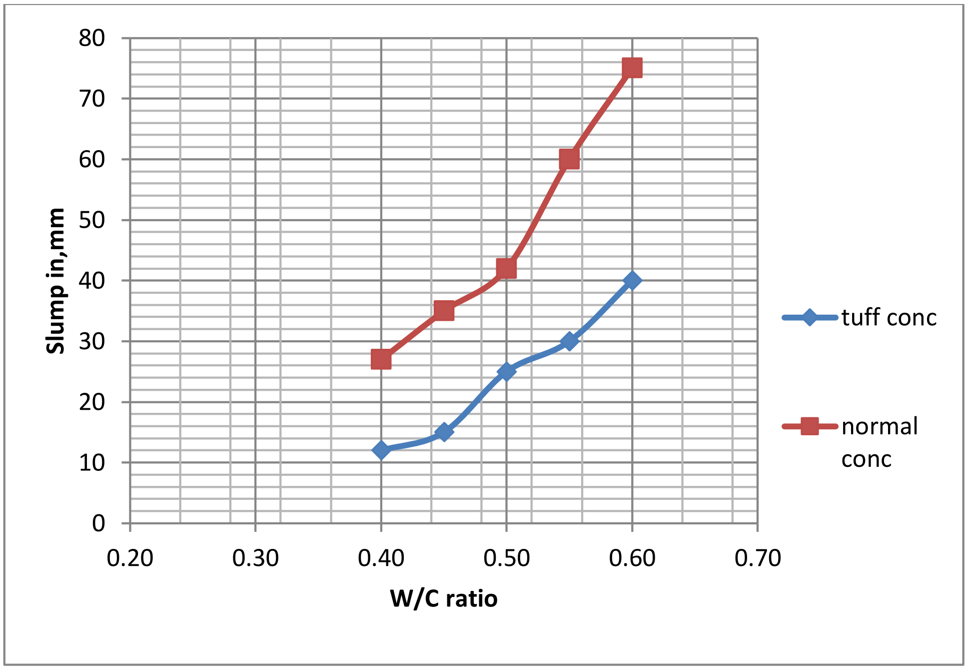

Figure 4.4: Effect of w/c ratio on the slump of the concrete mix

4.7.2 Effect of Superplasticizer on the Slump of Concrete Mix (W/C 0.4)

From the results in Figure 4.5 below, the addition of superplasticizer to concrete was observed to greatly enhance the workability of both concrete samples. Both tuff and normal-weight concrete samples without superplasticizer exhibited low slump values of 15 mm and 26 mm respectively, with the workability of tuff aggregate concrete being the lowest. This suffices to show that tuff aggregate concrete has generally lower workability than normal-weight when superplasticizer is not incorporated during mixing. The tuff aggregate samples in the mix absorbed more water into the pores of the aggregates leaving little water for mixing and production of a cement paste available to coat the surfaces of aggregates and fill the voids thus making the concrete stiff within a few minutes of mixing. Unlike the conventional aggregates, a small percentage of water was absorbed by the aggregates leaving more water for paste formation and lubrication of aggregates resulting in the initial increase in a slump. The formation of more cement paste in conventional concrete was responsible for filling the voids and lubricating the aggregates to make it more consistent. Upon the addition of 0.5% SP dose, the slump of tuff concrete increased to 25 mm while that of normal-weight concrete increased to 30 mm. Superplasticizers cause the transformation of stiff, low-slump concrete into flowing, pourable, and easily placed concrete. The gap between the two curves however kept reducing with the addition of superplasticizer until the workability of the two concrete samples converged to 55 mm when 1.2% superplasticizer was added. On reaching a 1.2% SP dosage, the tuff aggregates were possibly wet enough to absorb more water and the superplasticizer. The superplasticizer was increasingly becoming effective in the dispersal and de-flocculation of cement particles by creating like charges on the solid surfaces of the cement particles increasing the concentration of cement paste responsible for the lubrication of aggregates causing an increase in the workability of tuff concrete. The increase in workability of tuff concrete was more rapid surpassing that of normal-weight concrete when the SP dosage was increased from 1.2% to 2.5%. The gap between the two curves widened when SP dosage was increased from 2.0% to 2.5% registering slump values from 70 mm to 100 mm for tuff aggregate concrete and 55 mm to 70 mm for normal-weight concrete. More concentration of superplasticizer and water in the tuff concrete mix could have possibly caused the weakening and lowering of bonding effect between the interlocked aggregates and cement paste. However, segregation and bleeding of tuff concrete were noted to occur when 2.5% SP dosage was added. This is because excess unabsorbed water and superplasticizer by aggregates which was not consumed during the hydration process caused bleeding and segregation of concrete by increasing the formation of voids and lowering the compaction of the wet concrete. When cement particles absorbed excess superplasticizer, they formed too many like charges which caused them to repel and disperse each other resulting in segregation and bleeding.

The addition of 2.5% SP to a normal-weight concrete sample resulted in increased workability with the formation of a true slump of 90 mm. The workability of the tuff concrete increased more than that of normal-weight concrete when the SP dosage was increased from 1.0% to 1.5%. Plate 4.2 below shows the formation true slump of tuff aggregate concrete mix with SP dosage of 1.5%. The gap between the two curves started widening when SP dosage was increased from 1.5% to 2.0% registering improved workability from 70 mm to 100 mm for tuff aggregate concrete and 55 mm to 70 mm for normal-weight concrete. There was a more rapid increase in the workability of the tuff concrete than the normal-weight concrete sample when the superplasticizer was increased from 1.5% to 2.0%. The addition of 2.0% to 2.5% SP dosage produced concrete having good workability tuff aggregate mix with some samples occasionally resulting in collapse slumps. The reason for higher slump values associated with tuff concrete could be because more water and superplasticizer created by the weakened bonding effect responsible for the interlocking of aggregates and filling of voids by the cement paste. However, segregation and bleeding of tuff concrete were noted to occur when 2.5% SP dosage was added. The segregation and bleeding of tuff concrete with 2.5% SP dosage was the main reason for collapse slump formation. The concrete lost its compact-ability due to the formation of voids between coarse aggregates resulting from finer aggregates and cement paste separating to the top of the concrete. The addition of 2.5% SP to the normal-weight concrete sample resulted in enhanced workability with the formation of a true slump of 90 mm. In this experiment, the aggregates were presoaked to ensure minimal absorption of water and superplasticizer occurred during mixing.

What happens to the concrete mixture without a superplasticizer is that the flocculation of cementitious materials occurs in a concrete mixture and leads to a reduction in the workability of the mix. However, for the same amount of water in the mix, the addition of a superplasticizer causes a uniform distribution of cement particles in a concrete mix. Concrete mix with high workability is suitable to use in areas of closely spaced and congested reinforcing steel members where compaction of concrete is difficult. It is also useful in instances where the setting rate of concrete is required to be retarded especially in hot areas. High workability concrete can also be used to prevent cold joint formations in successive lifts during construction. The use of superplasticizer reduces slump loss in concrete and becomes useful where concrete is expected to be transported through long distances or to be used in areas where the transportation of concrete is affected by delays due to long traffic jams. High dosage of superplasticizer above 2.0%, however, impairs the cohesiveness of concrete causing segregation and bleeding resulting in collapse and shear slumps as shown in Plate (4.1) below. It can also push the cost of construction up.

The superplasticizer used was polycarboxylate ether which is composed of a methoxy-polyethylene glycol copolymer side-chain joint with the methacrylic acid copolymer. The dispersion of cement particles in a mix occurs due to a steric hindrance. Poly (carboxylate ether)-based superplasticizers, (PCEs), have acrylate groups in the backbone and also contain side chains (i.e., poly (ethylene oxide)) that protrude from the cement surface into the pore solution to produce a steric hindrance effect. The mechanism of superplasticizer is through giving the cement particles a highly negative charge so that they can repel each other due to the same electrostatic charge. The free water enveloped by the flocculent structure is released as a result of the destruction of the flocculent structure. The amount of water that contributes to the mixture fluidity is increased. Therefore, with the increase of dosage of SP, the fluidity of the mixes increase. Concrete samples containing too much water with a w/c ratio of 0.5 or higher superplasticizer dosage of over 1.5% resulted in high degree workability concrete samples. The concrete samples segregated and took a long time to set and form.

Concrete must be of desirable workability to achieve a maximum bulk density with a reasonable amount of compaction effort. If concrete is of low workability, it will not be easy to compact to its desired density resulting in porous and low strength concrete. If concrete is of low workability, it will be difficult to place it in a formwork. More details of the slump are in Appendix Table B.14.

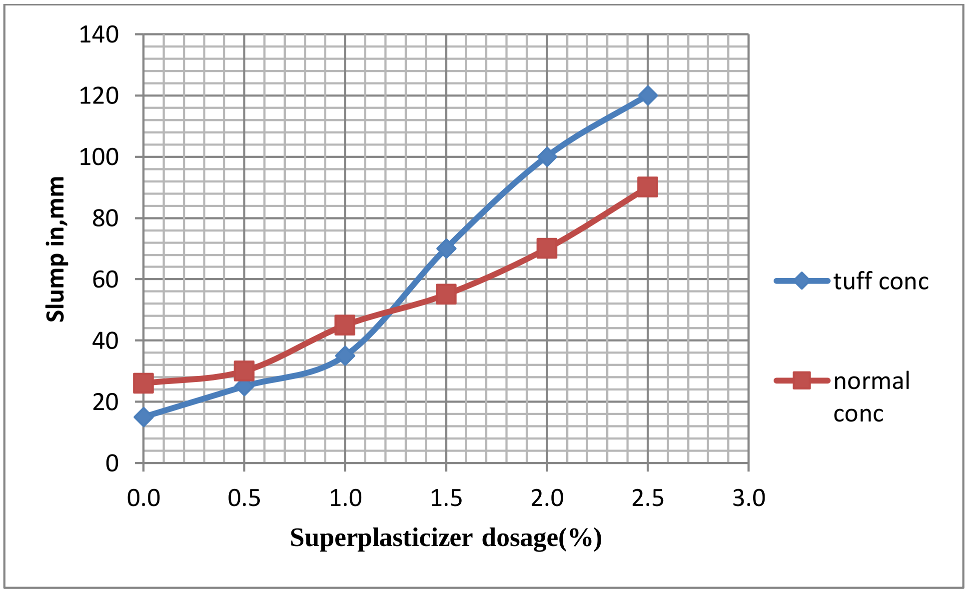

Figure 4.5: Effect of superplasticizer on the slump of concrete

Plate 4.1: Collapse slump of a concrete mix
(Source: Author)

Plate 4.2: True slump of a concrete mix
(Source: Author)

4.5.3 Effect of Superplasticizer on the Compaction Factor of Concrete

From Figure 4.6 below, the compaction factors for both concrete samples improved generally when dosages of superplasticizer were increased in the mixes. The aggregates used for this test were presoaked before mixing to minimize the absorption of water and the superplasticizer, particularly by the tuff aggregates. The compaction factors for tuff concrete, however, were noted to increase more than granite concrete with an increase in SP dosage. This was seen as the gap between the two curves reduced as superplasticizer dosage was increased gradually until the two curves almost converged. While the compaction factors increased from 0.88 to 0.96 for normal-weight concrete with SP dosage of 0% to 1.5%, the compaction factors increased from 0.83 to 0.93 for tuff concrete. What happened to the concrete is that the superplasticizer dispersed more cement particles to form cement paste which expelled entrapped air from the concrete. The cement paste and the aggregates interlock after entrapped air is expelled from the voids in the process of compaction producing denser concretes. However, when 2.0% of SP was added to the mix, the compaction factor for tuff concrete reduced to 0.92 while that of granite concrete dropped to 0.95. At 2.5% SP dosage, the tuff aggregate compaction factor slightly increased to 0.93 while that of granite aggregate reduced to 0.93. From these results, an economical optimal percentage dosage of 1.5% to achieve high compaction factors to save on power and cost during mixing and placing can be recommended for the two types of concrete. Since tuff concrete contains air in the pores and between the particles, the superplasticizer disperses more films of water and cement paste to expel the air from the pores and between the particles causing an increase in compaction factor. Superplasticizer dispersed more cement paste to fill the pores and voids between the particles resulting in an increase in compaction of tuff concrete than conventional concrete as the dosage of superplasticizer was increased from 0.0% to 1.5%. However, the compaction of both concrete mixes declined when more superplasticizer dosage was increased from 1.5% to 2.5% while keeping the w/c ratio constant. That means more free water was left in the mixes causing segregation and bleeding of concretes and a decline in the compaction factors was noted. However, normal-weight concrete showed higher compaction factors than tuff concrete in general. The higher bulk density of granite aggregates in conventional concrete contributed to higher compaction factors than tuff aggregates. Optimum superplasticizer dosage of 1.5% is therefore recommended for tuff and normal-weight concrete to achieve high compaction factors of 0.93 and 0.96 respectively. More details were presented in Appendix Table B.14.

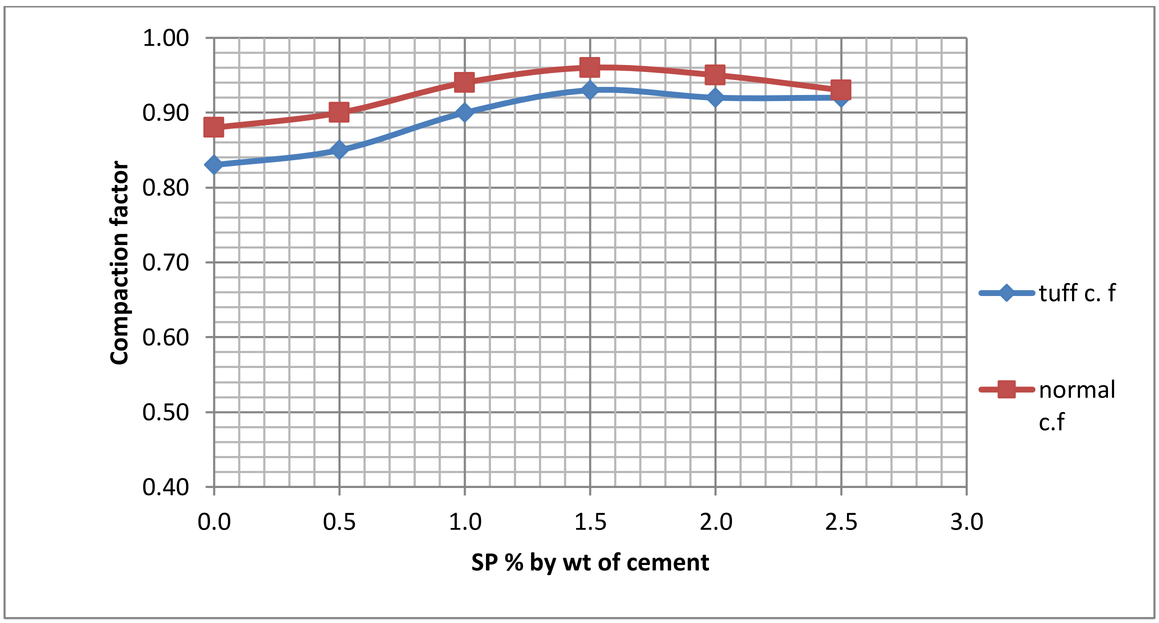

Figure 4.6: Effect of SP on compaction factors of concrete mix with w/c 0.4

4.8 Physical Properties of Hardened Concrete

4.8.1 Change in Unit Weights of Concrete with Age

From Table B.15 in the Appendix section, the densities of tuff and granite concrete when determined in fresh states were 2031 kg/m^3 and 2510 kg/m^3 respectively. The water-cement ratio of 0.4 was maintained for all the mixtures. When measured at the ages of 14 days, the densities were 2036 kg/m^3 and 2515 kg/m^3 respectively. When measured after 28 days, the density of tuff concrete increased by 2 kg/m^3 to 2038 kg/m^3 while granite concrete density increased by 5 kg/m^3 to 2527 kg/m^3. The densities of conventional concrete ranged from 2510 kg/m^3 to 2527 kg/m^3. The 28-day density of normal-weight concrete, in particular, was found to lie within the range of 2200 kg/m^3 to 2600 kg/m^3 as specified by Neville (2000). The 28-day density of tuff concrete was less than 2200 kg/m^3 being the maximum density beyond which lightweight concrete should not exceed as per the definition of Euro Code 2 (2008). The 28-day unit weight of tuff concrete was 2038kg/m^3 while that of conventional concrete was 2527 kg/m^3. This means a structure made with tuff concrete will be lighter in weight by 19% than that one constructed with conventional concrete. Consequently, the earthquake forces affecting the structure will be reduced by 19% if tuff concrete was used to construct it instead of conventional concrete. The bulk density of the tuff coarse aggregate in the mix contributed to the low density of tuff concrete whereas higher density of the conventional concrete was due to the high bulk density of the granite aggregates in the mix. Proper curing provides a moist environment for the development of hydration products. This reduces the voids in hydrated cement paste increasing the density of micro-structure in concrete. The hydration products extend from the surfaces of cement grains reducing the volume of voids.

The slight gain in the densities of the two concretes, therefore, was partially because of absorption of water for the hydration process and possibly because of the expulsion of air from the voids making the concretes denser as they hardened. The increase in the unit weight of tuff aggregate concrete was 0.34% while that of normal weight concrete was 0.67% within 28 days. These increments were, however, insignificant since most structural designs consider unit weights of concretes taken after 28 days. The lightweight dead load reduces the failure of structures due to earthquake forces. It also reduces the sizes of structural members and the amount of reinforcement required in the structure compared with conventional concrete.

4.8.2 Results of Compressive Strength of Concrete

4.8.2.1 Effect of Water- Cement Ratio on Compressive Strength of Concretes with Presoaked Aggregates

When the amount of mixing water was increased in the concrete mixes, there was a reduction in the compressive strength of both concrete. The amount of cement content was kept constant throughout all the samples. The compressive strengths shown were taken on 28-day old concrete cube specimens. It was observed that the compressive strength of tuff concrete was reduced from 25.0 N/mm^2 to 24.4 N/mm^2 when the w/c ratio was varied from 0.4 to 0.45. When the w/c ratio was further increased from 0.45 to 0.50 the compressive strength was reduced from 24.4 N/mm^2 to 23.8 N/mm^2. A reduction in compressive strength from 23.8 N/mm^2 to 19.2 N/mm^2 was noted when the w/c ratio was varied from 0.50 to 0.55. A further reduction of compressive strength from 19.2 N/mm^2 to 17.0 N/mm^2 was recorded when the w/c ratio was increased from 0.55 to 0.60. By increasing the w/c ratio from 0.40 to 0.60, there was an overall reduction of 32% in the compressive strength of tuff concrete. Any amount of excess water in the concrete that was not absorbed by the aggregates nor used for the hydration of cement was responsible for the decrease in the compressive strength of the concretes. All the excess water that is in the concrete mix must leave the concrete in the form of what is called bleed water. Bleed water leaves voids and increases the drying shrinkage in concrete. This causes cracks in concrete and leads to a reduction in compressive strength. Secondly, the excess water in concrete impedes the compaction of concrete and results in the formation of voids that traps air pockets. This further contributes to a reduction in the compressive strength of concrete.

Similarly, there was an overall reduction in compressive strength of normal-weight concrete from 29.0 N/mm^2 to 21.0 N/mm^2 when the water-cement was increased from 0.4 to 0.6 representing 27%. A reduction in the compressive strength of tuff concrete from 25.0 N/mm^2 to 17.0 N/mm^2 was noted when the w/c ratio was increased from 0.4 to 0.6. This represents a 32% drop in strength. The compressive strength of tuff concrete seemed to be affected more adversely by the increase in mixing water than granite concrete. The reason why the compressive strength of tuff concrete was generally low compared to conventional concrete was mainly because tuff aggregates are more porous and less rigid compared to granite aggregates. Tuff aggregate concrete absorbs more water than granite aggregate concrete leaving little water for the hydration process resulting in the loss of bonding between cement paste and the aggregates eventually lowering the strength.

Too much-mixing water in a concrete mix adversely affects the hydration process leading to weaker concrete. This is because the air voids in concrete tend to increase with the increase in the amount of water. This results in a decline in the compressive strength of concrete. Excess mixing water also contributes to segregation. This affects the homogeneity and leads to an uneven hydration process thus a loss in compressive strength of concrete. This means the cement paste and fine aggregate required to fill the void spaces between aggregates in the mix separate to form a top layer while the coarse aggregate settles at the bottom. As a result, the concrete ends up with voids filled with air and becomes weak in strength. Furthermore, the water that is not consumed by the hydration reaction process evaporates as the concrete hardens leaving microscopic pores that contribute to a reduction in strength. A concrete mix with too much water also tends to experience drying shrinkage as excess water evaporates. This results in the formation of internal cracks which again reduce the compressive strength of concrete.

In concrete, the mixing water is available in three different forms, namely the chemically bonded water, the physically bonded water, and the free water. The chemically bonded water is utilized during the hydration process. The physically bonded water is the water bonded to the solid concrete materials by adhesive forces. Free water is that water which is beyond the range of solid surface forces and is considered to behave like bulk water. The chemically bonded water is not lost in drying. It can only be released out when the hydrates decompose on heating up to 1,000°C. The distribution of the physically bonded water and the free water in porous materials strongly depends on the moisture content. Appendix Table B.16 has tabulated values.

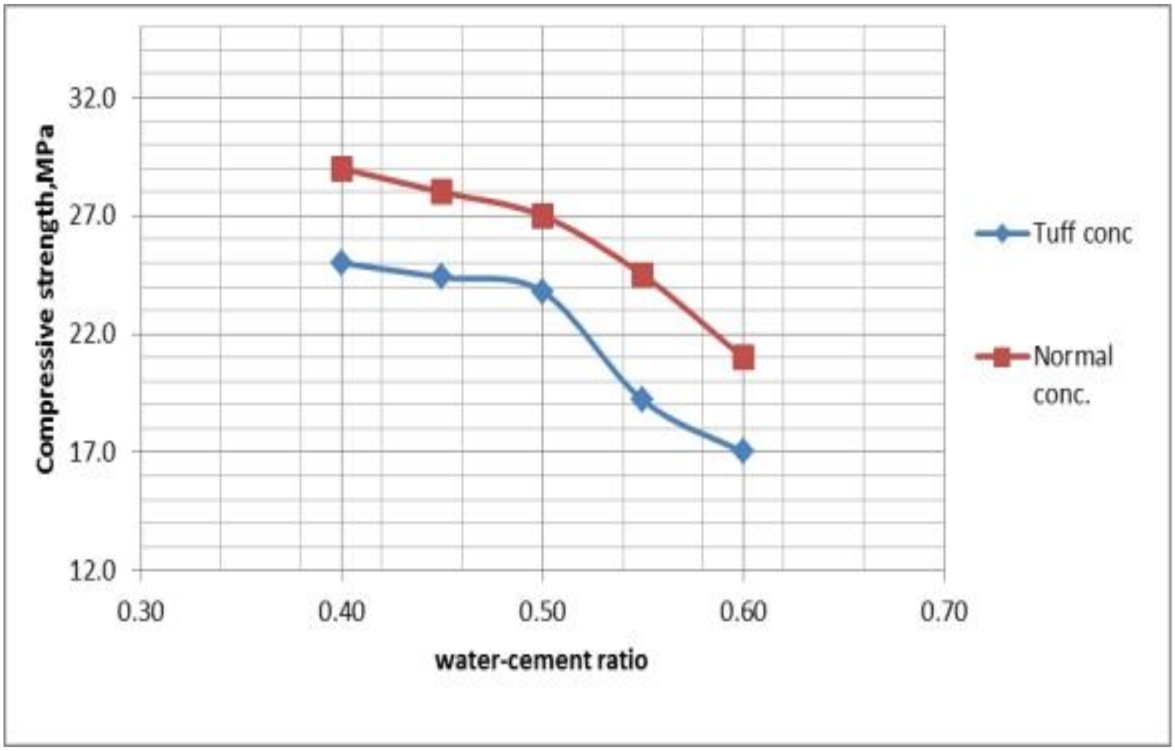

Figure 4.7: Effect of w/c ratio on compressive strength of concrete

4.8.2.2 Effect of W/C Ratio on Compressive Strength of Concrete with Oven Dry (OD) Aggregates

When the amount of mixing water was increased from 0.4 to 0.6 as noted in Figure 4.8 below, there was a reduction in the compressive strength of the conventional concrete as the curve kept falling. The compressive strength of conventional concrete reduced from 31.4 N/mm^2 to 19.1 N/mm^2 representing a drop in strength by 39%. However, there was an unusual increase in compressive strength of tuff concrete from 23.0 N/mm^2 to 25.2 N/mm^2 when the water-cement ratio was increased from 0.40 to 0.55 representing an increase in strength by 9.5% before it started declining. The compressive strength increased sharply from 23.2 N/mm^2 to 25.2 N/mm^2 when the water-cement ratio was increased from 0.50 to 0.55. The strength further increased from 25.2 N/mm^2 to 25.5 N/mm^2 as the w/c ratio was increased from 0.55

to 0.60. The curve started leveling when water-cement ratio was further increased from 0.55 to 0.60 before declining as the water-cement ratio was increased to 0.65. The amount of cement content was kept constant throughout all the samples. As can be noted, there was a striking difference in the behaviors of the two concretes from the two curves. What happened is that any amount of water that was not used for hydration nor absorbed by conventional aggregates contributed to the reduction in compressive strength of the concrete. At a water-cement ratio of 0.4, the conventional concrete exhibited higher compressive strength. This implies that the available water was just adequate to fully hydrate the cement and lubricate the aggregates. The cement paste formed was adequate to bind all the aggregates together into a concrete mass. This is because conventional aggregates absorbed very little water during mixing as compared with the tuff aggregates. However, excess water in the concrete evaporates and leaves voids that are filled with air as the concrete hardens. This creates weak linkages and bonds between the cement paste and the aggregates. The same concrete becomes difficult to compact due to the existence of voids. As excess water evaporates, drying shrinkage occurs followed by superficial and internal cracks as the concrete hardens. The excess water started hurting the concrete. This is what caused the loss of strength in concrete. So the more the water content is increased in a concrete mixture the more the compressive strength is adversely affected particularly for the conventional concrete.

Unlike the conventional aggregate concrete, at a water-cement ratio was 0.4, tuff aggregates absorbed most of the water into the pores of aggregates leaving little water for hydration and lubrication of aggregates. The resulting concrete was stiff with the aggregates failing to conglomerate. As a result of the low water-cement ratio, the cement in the mix failed to hydrate fully leaving the aggregates to segregate and lose the bonding effect. The formation of voids occurred in the concrete contributing to a reduction in compaction and compressive strength. However, as the water-cement ratio was increased from 0.40 to 0.55 gained strength. Possibly, the aggregates absorbed enough water and left sufficient water for mixing and hydration of cement particles. Adequate water for hydration improved the workability as most of the aggregates were lubricated and more cement paste was formed to aid the interlocking between aggregates. Consequently, the air entrapped in the concrete was expelled during compaction resulting in denser concrete. This gave rise to an increase in compressive strength of tuff concrete to 25.5 N/mm^2. Therefore, a good balance of water-cement ratio plays an important role in producing lightweight structural concrete from tuff aggregates. However, with a further increase in the water-cement ratio to 0.65 there was a drop in the compressive strength. This means the excess water in the mix contributed to the formation of voids and segregation of concrete. The compaction of concrete was reduced leading reduction of compressive strength in tuff concrete. The compressive strengths were taken on 28-day old concrete cube specimens. Results can be found in the appendix Table 16.

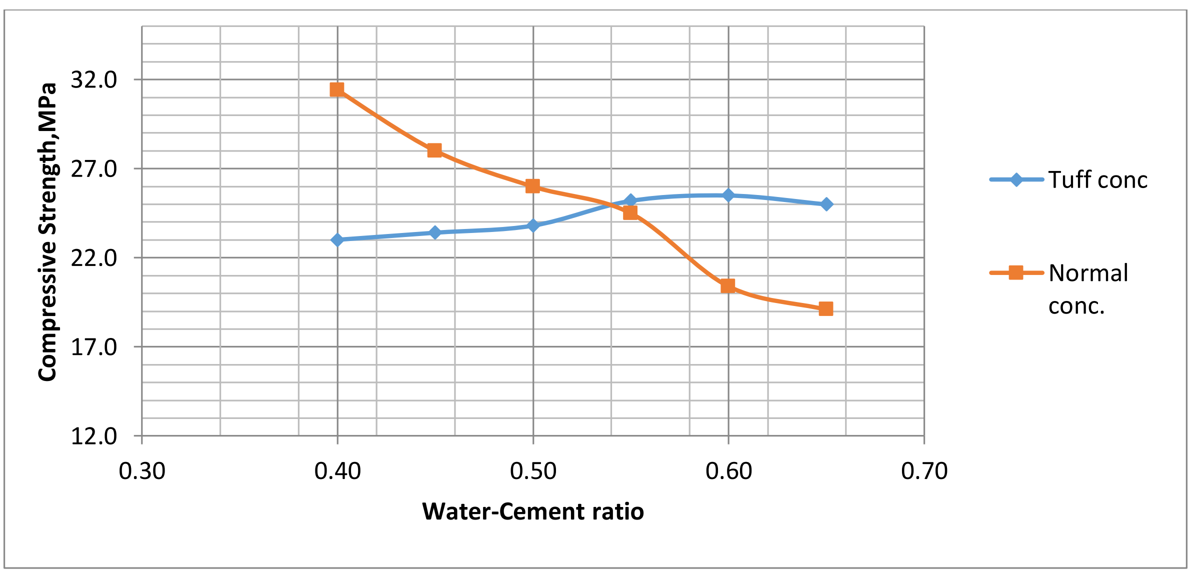

Figure 4.8: Effect of w/c ratio on 28-day compressive strength of concrete

4.8.2.3 Development of Compressive Strength of Concrete with Age

From Figure 4.9 below, the two concretes registered a sharp increase in compressive strengths within the first 7 days of curing but the rate of increase in strength started slowing as the concretes matured in age. This is because the hydration process is normally faster at early ages of moist cured concrete due to the high concentration of tricalcium silicate (Ca_3SiO_5) which is responsible for early strength development in concrete. Tricalcium silicate reacts rapidly with water to release calcium ions (Ca^{2+}), hydroxide ions (OH^-), and hydro silicate ions ($H_2SiO_4^{2-}$). A large amount of heat is produced during the process. This explains why the rate of strength development was higher within the first seven days of the two concretes. At surface temperatures or just above, the silicates react with water to form an amorphous calcium silicate hydrate and calcium hydroxide. Dicalcium silicate (Ca_2SiO_4) reacts more slowly with water and contributes mainly to strength development after 7 days. The process is slower as compared to hydration caused by tricalcium silicate. The hydration of dicalcium silicate leads to the formation of amorphous calcium silicate hydrate and calcium hydroxide. The average compressive strength of tuff aggregate concrete after 7 days was 14.0 N/mm^2 representing a strength gain of 54 % with respect to the 28-day strength while that of normal-weight concrete was 17.0 N/mm^2 representing a gain of 57% with respect to the 28-day strength. For structural normal-weight concrete, the strength at 7 days for well-cured specimen should be between 60% to 65% of the 28-day compressive strength depending on the type of cement used. The slow gain in strength of conventional concrete could be because the aggregates used in the mix were not well graded. In particular, the aggregates were uniformly graded, elongated, and flaky. This could have contributed to low strength gain with the age of the concrete. At the age of 14 days, the compressive strength of tuff concrete increased to 20.0 N/mm^2 while that of normal-weight concrete increased to 23.0 N/mm^2. After 21 days the compressive strength of tuff concrete was 24.0 N/mm^2 while that of normal-weight concrete was 27.0 N/mm^2. After 28 days, the strength of tuff concrete had increased to 26.0 N/mm^2 while normal-weight concrete strength was 30.0 N/mm^2. A 28-day compressive strength which is in excess of 25.0 N/mm^2 makes tuff concrete a competitive concrete compared to most lightweight concretes produced from natural materials studied by many researchers. Most high-strength lightweight concretes are mainly those produced from artificial lightweight materials but they are expensive to produce because they involve burning in a kiln. When measured after 35 days, the strength of tuff concrete was 26.8 N/mm^2 while that of normal-weight concrete was 30.7 N/mm^2. Furthermore, the compressive strength of tuff concrete was noted to increase from 26.8 N/mm^2 to 27.7 N/mm^2 while that of conventional concrete from 30.7 N/mm^2 to 31.0N/mm^2 after 42 days. Although the curing of the concrete samples was stopped after 28 days, it was noted that there was a slight gain in compressive strength of both concretes when tested after 35 and 42 days most likely due to internal curing in the concretes.

When compared, the increments in the compressive strengths of the tuff concrete were higher than those of the conventional concrete after 28 days. For example, at 42 days, the strength of tuff concrete sample increased by 1.7 N/mm^2 to 27.7 N/mm^2 with respect to the 28-day strength representing an increase of 6.5% in strength while normal-weight concrete strength increased by 1.0N/mm^2 to 31.0N/mm^{2}representing an increase of 3.3% in strength. These increments are attributed to the continuous hydration of dicalcium silicate with water in the micro-structure of the concretes. The higher increment of strength in tuff concrete than in conventional concrete is attributed to the presence of more water in the pores of tuff concrete which facilitated a better hydration process in the tuff aggregate concrete than in conventional concrete. Thus the presence of water in the pores of tuff aggregates facilitated better internal curing process to take place in tuff concrete even after curing was stopped. The process of internal curing was more pronounced in tuff concrete than conventional concrete due to the existence of numerous pores in the micro-structure of tuff aggregates which could store water which facilitated continuous hydration of cement particles in the concrete to take place.

Although the strength of tuff aggregate concrete was generally lower than that of conventional concrete, it was nonetheless found to be structurally efficient concrete compared to normal-weight concrete. Structural efficiency is determined by dividing the compressive strength by the unit weight of concrete. Therefore, the structural efficiency of tuff aggregate concrete after 28 days was 1.3% while that of conventional concrete was 1.2%. This makes tuff concrete competitive for use in various applications including floor-filling for high-rise buildings especially those in earthquake-prone areas. It also makes it suitable for use in buildings congested urban areas where the sizes of foundations and other structural members may need to be reduced.

The low rigidity of tuff aggregates coupled with numerous pores in their cellular structure contributes to the lower strength of tuff concrete. The reason for lower compressive strength could also be because of the presence of either interconnected or disconnected pores within the tuff aggregates that serve as weak spots that allow for the initiation and propagation of cracks directly through the aggregate particles when the concrete is loaded. Granite aggregates on the other hand are more rigid and denser than tuff aggregates. This made the compaction of conventional concrete better than that of tuff concrete. This resulted in higher compaction results and higher compressive strengths of conventional concrete than tuff aggregate concrete. Secondly, the rigidity of conventional aggregates helped the concrete to have better compressive strength. More details of the results can be found in Appendix Table B.27.

Vu et al., (2000) suggested a formula to predict the compressive strength of concrete according to Equation (4.2) below;

$$(f_c')_t = \frac{t}{4+0.85t} \tag{4.2}$$

Where;

t in days is the age of concrete.

$(f'_c)_{28}$ is the average 28-day compressive strength of concrete.

Therefore, from the above equation, the estimated compressive strength equation for tuff concrete can written as follows;

$$(f_c')_t = \frac{t}{4+0.85t} \cdot 26 \tag{4.3}$$

The estimated compressive strength equation for conventional concrete can written as follows;

$$(f_c')_t = \frac{t}{4+0.85t} \cdot 30 \tag{4.4}$$

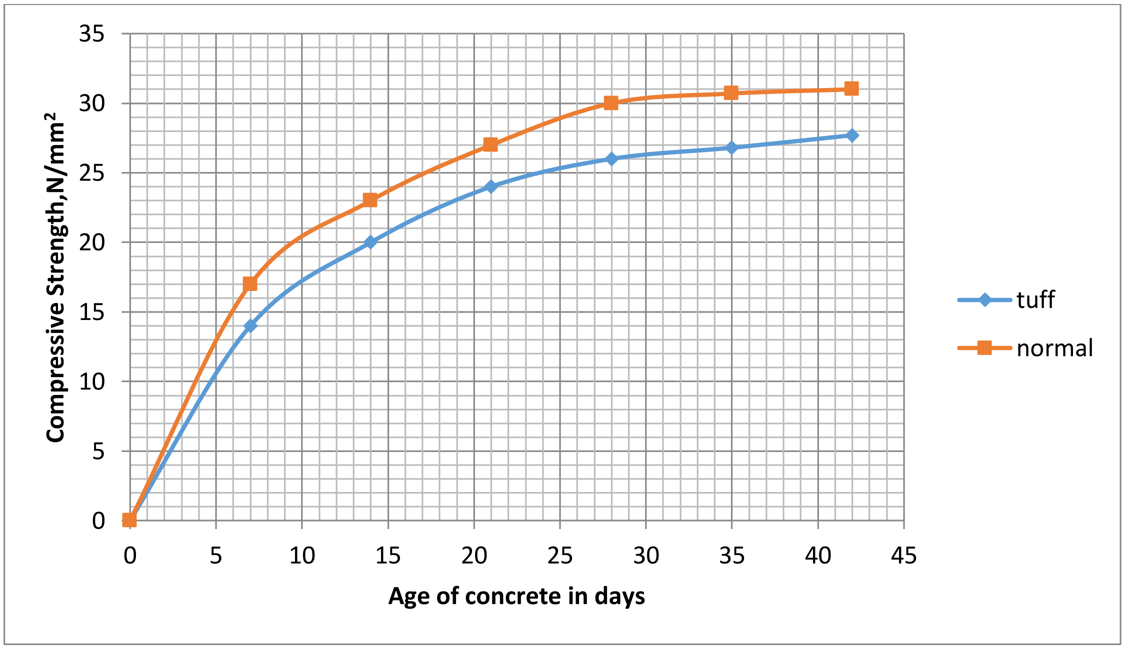

Figure 4.9: Compressive strength of tuff and normal concrete with age (w/c 0.4)

4:8:2:4 Failure Modes for Concrete Cube Samples for Tuff and Conventional Concretes

In Plate 4.3 below, the concrete cube sample for tuff aggregate concrete displayed satisfactory failure mode category. This was a non-explosive failure type. Concrete cracking occurs in three stages. In stage one, as the concrete specimen is loaded, the localized cracks are initiated at the microscopic level at isolated points throughout the specimen where the tensile strain concentration is the largest. In stage two, the crack system multiplies and propagates but in a slow and stable manner. The final stage involves crack system developing into a stage that becomes unstable and the release of strain energy is sufficient to make the cracks to undergo self-propagation until a complete disruption and failure occurs in concrete.

The reason for such a failure of tuff concrete can be attributed to the presence of interconnected pores within the tuff lightweight aggregates that served as weak spots for the initiation of micro cracks within the concrete. As the applied stress was gradually increased, the micro cracks extended in length and width until they formed major failure cracks passing through the aggregates and thus causing cracking of the specimen along two planes. The concrete cube specimen developed macroscopic cracks which split the specimen into three sections as loading of the cube was increased gradually. However, the crack on the left of the specimen was wider than the crack on the right side of the specimen. This side of the specimen was weak most probably due to uneven compaction which could have caused some element of segregation moving the cement paste to the top of the specimen which could have been the right hand side.

Plate 4.3:Failure mode of tuff aggregate concrete cube specimen

In Plate 4.4, the failure of the conventional concrete specimen resulted in an unsatisfactory failure mode with tensile cracks forming near the extreme edges of the left and right-hand side faces of the cube. As the loading was increased on the specimen, the formation of microscopic cracks in the concrete developed. These cracks enlarged into macroscopic tensile cracks which propagated into tensile cracks causing failure of the specimen as loading was gradually increased in the specimen. The failure was semi-explosive with an accompanying loud sound being produced as the specimen failed in compression. The cracks sizes were smaller in width than those in a failed tuff cube specimen. This is because of the high individual elastic moduli of the aggregates and the paste component in the normal-weight cube specimen. The tensile crack on the left face is smaller than the crack on the right side face of the cube. This means the compressive stresses were more concentrated on the right side edge than on the left side edge of the specimen. The crack on the right-hand side edge of the concrete cube was thicker in width than on the left-hand edge of the specimen most probably due to an uneven cement paste and aggregate distribution on the lower portion of the specimen during mixing and compaction of concrete.

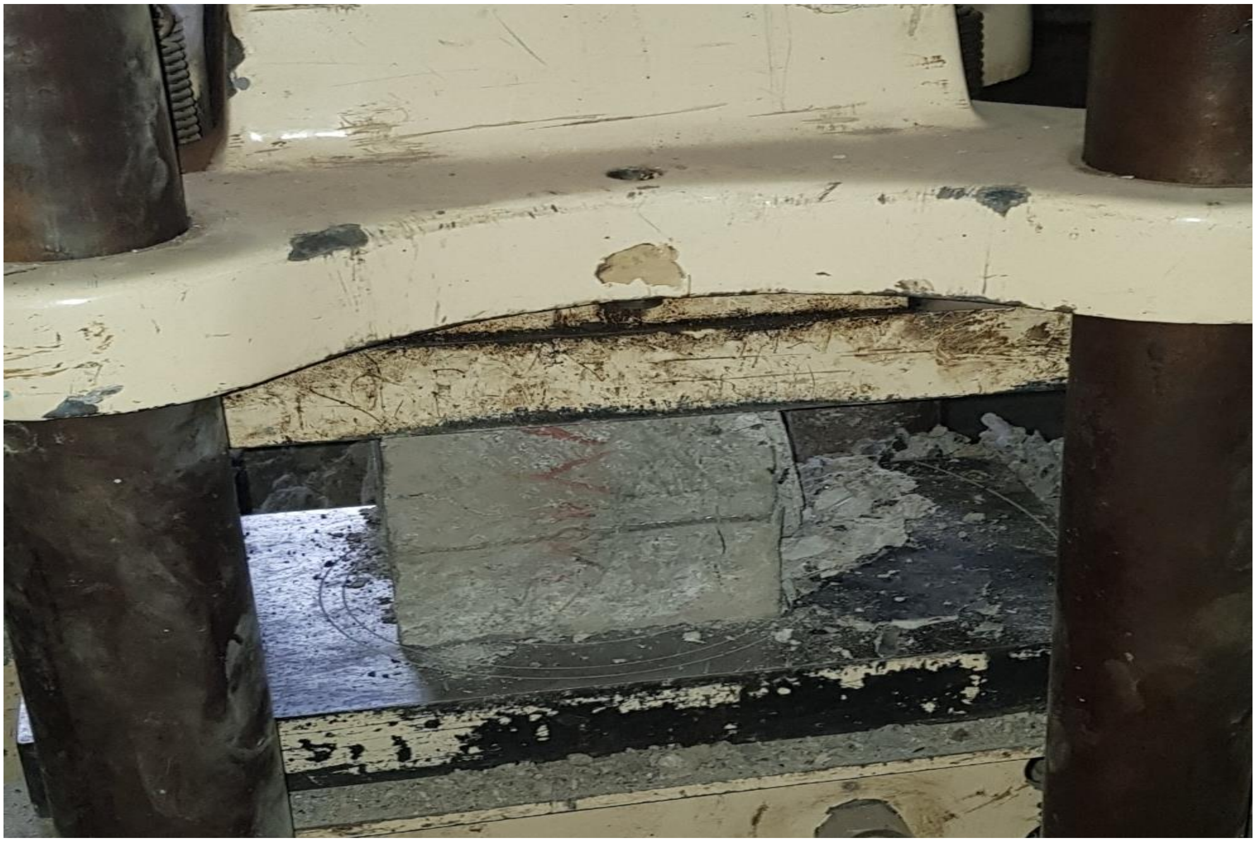

Plate 4.4:Failure mode of normal -weight concrete cube specimen

4.8.3 Results of Splitting Tension Test of Concrete

The minimum splitting tensile strength of a structural lightweight concrete specified by ASTM: C330(2006) is 2.0 MPa. The splitting tensile strengths of 28-day old tuff concrete specimens were compared having been prepared from the concretes samples with the application of 1.5% superplasticizer and w/c of 0.40. Splitting tensile strengths obtained from three cylindrical concrete specimens were 2.7 N/mm^2, 2.8 N/mm^2, and 3.0N/mm^2. The average splitting tensile strength for tuff concrete, therefore, was 2.9 N/mm^2. The corresponding 28-day compressive cube strengths from the respective concrete were 24.9 N/mm^2, 25.2 N/mm^2, and 26.8 N/mm^2. The average 28-day compressive strength of tuff concrete was 25.6 N/mm^2. The ratio of the tensile stress to the compressive strength of tuff concrete was found to be $\frac{1}{9}$ or 11%. More details are in Appendix Table B.18.

Plate 4.5: Cylindrical tuff concrete specimen tested for splitting tensile strength (Source: Author)

The splitting tensile strengths from three samples of conventional concrete were 3.5 N/mm^2, 3.6 N/mm^2, and 3.7 N/mm^2. The average splitting tensile strength for the concrete samples was 3.6 N/mm^2. The corresponding 28-day compressive cube strengths from the respective concrete samples were 33.7 N/mm^2, 34.2 N/mm^2, and 36.5 N/mm^2. The average 28-day compressive strength of conventional concrete was 34.8 N/mm^2. More details are in Appendix Table B.19

The ratio of the tensile stress to the compressive strength of conventional concrete was found to be $\frac{1}{10}$ or 10%. One common characteristic of conventional concrete is its increased brittleness more than tuff concrete. The ratio of tensile strength to compressive strength is one of the methods used to judge the brittleness of the material. The lower the ratio, the more brittle the material. Conventional concrete therefore from this study was observed to be more brittle. The splitting tensile values for both concretes were however found to fall within the range of 2.2 - 4.2 MPa specified in most design standards as acceptable for the structural design of concretes. However; the tensile strengths for tuff concrete specimens were lower than those of normal-weight concrete. The reason can be attributed to the presence of more interconnected pores within the tuff lightweight aggregates that served as weak spots for the initiation of cracks within the tuff concrete. That is why the tuff concrete specimen developed wider plane cracks than conventional concrete. Although the load at which NWC failed in tension was higher, its failure mode was sudden and explosive depicting a more brittle behavior than tuff aggregate concrete. It can therefore be deduced that conventional concrete has a poor capacity to resist vibrational loads and may not be suitable for earthquake structures compared to tuff lightweight concrete. Tuff concrete was observed to develop more irregular internal cracks which appeared to spread to the surface while NWC developed fewer plane cracks with the primary crack developing along the plane of loading. The conventional concrete sample failed suddenly producing explosive sound into two parts through the middle of the cross-section as shown in Plate 4.6. The formation of two major cracks in the case of tuff concrete allows the tensile stresses to be distributed in the cross-section of the concrete. This causes the transfer of tensile stresses to the steel in the case of reinforced concrete. This is because at a cracked section, concrete stress is zero but the steel stress is maximum for a reinforced section. However, more cracks in concrete allow the ingress of water into the concrete causing damage to reinforcement by corrosion and eventual failure to the structure.

The higher tensile strength of the conventional concrete compared with tuff concrete could be because tuff aggregates were more porous and less rigid. To produce a good quality tuff concrete, a lot of mixing water, superplasticizer, and cement were required to ensure proper bonding of aggregates and cement paste.

Plate 4.6: Conventional concrete specimen tested for splitting tensile strength. (Source: Author)

4.8.4 Results of Static Elastic Modulus of Concrete in Compression

The static elastic modulus of concrete, Ec, is defined as the slope of the line drawn from a stress-strain graph from zero to a compressive stress of 0.40f'c. From the above results, the conventional concrete depicted two types of elastic moduli. One type was obtained by determining the gradient of a tangent line on the curve drawn from zero stress. This elastic modulus is termed as the initial tangent modulus. This modulus is approximately equal to the dynamic modulus; E_d of concrete. Dynamic modulus is determined by using ultrasonic measuring techniques by getting the resonant frequency of the concrete prism specimen. This method is used to assess the elastic modulus of concrete of an actual structure in service.

Dynamic modulus is used to determine the long-term elastic modulus of concrete and the creep effects on the structure under sustained load. If there is a creep effect due to a particular loading on a concrete member, it is considered the long-term modulus of elasticity and can be calculated in accordance with Equation (4.5) below.

$$E_{Long} = \frac{E_{Short}}{(1+\theta)} \tag{4.5}$$

Where;

$E_{long;}$ is the long-term modulus of elasticity.

E_{short}; is the short-term modulus of elasticity.

θ; is the creep coefficient; the ratio of creep strain to the elastic strain of concrete.

The other modulus which is referred to as secant modulus, E_c, was obtained by determining the gradient of a line drawn from zero stress to coincide with a line drawn from the value of stress equivalent to 40% of ultimate stress. The ultimate stress values of 18.1 N/mm^2 and 19.8 N/mm^2 for tuff and conventional concrete samples respectively were determined. The values of compressive stresses equivalent to 40% f'c were 7.9 N/mm^2 and 7.24 N/mm^2 for conventional and tuff aggregate concrete respectively. The elastic moduli of conventional concrete were determined from Equation (3.10) as follows;

The initial tangent modulus for conventional concrete was calculated as follows;

E_d = (6.2-0.0)/ (0.0002-0.00005) =41.0 GPa.

The secant modulus of elasticity of conventional concrete was determined as follows;

E_C = (7.9-0.0)/ (0.0006-0.00005) =14.3GPa.

Both the initial and secant moduli of tuff concrete coincided in one line and were computed as follows;

E_{ct} (tuff aggregate concrete) = (7.24-0.0)/ (0.00068-0.00005) =11.5 GPa.

Secant modulus of elasticity is useful when determining short-term member stiffness whereas initial tangent modulus and dynamic moduli of elasticity are used for determining the long-term stiffness of a member. The curves indicate that concrete is not an elastic material but heterogeneous material and therefore does not have a fixed value of elastic modulus.

The elastic modulus of tuff aggregate concrete, Ect, was determined to be 11.5 GPa while that of conventional concrete, Ec, was found to be 14.3 GPa. The modulus of elasticity of conventional concrete was higher than tuff concrete because the normal-weight aggregates were more stiff and rigid than tuff aggregates. The initial slope of the stress-strain diagram revealed a steady rise in elastic modulus of the concrete samples with the stress-strain graph nearly linear from zero stress to the stress of 6.2 N/mm^2 and 8.5 N/mm^2 for conventional and tuff concrete respectively. This elastic deformation of conventional concrete happened between the stress of zero to the stress of 6.2 N/mm^2 and the corresponding strain of 0.0002 while that of tuff aggregate concrete was between a point of zero stress to a point of compressive stress of 8.5 N/mm^2 and a corresponding strain of 0.0008. Within this range, the stress was proportional to strain giving a linear graph. However; the elastic modulus of conventional concrete was higher than that of tuff aggregate concrete as shown by a sharp rise in its slope during the initial loading. Although the elastic strain of tuff aggregate concrete was higher, its elastic modulus was lower than that of conventional concrete. On reaching the yield point and upon exceeding the compression stress of 6.2 N/mm^2, the normal-weight concrete deformed permanently losing its elastic properties transitioning into plastic state material. There was a decline in its elastic modulus as noted from the decrease in the gradient of the slope from the stress of 10.2 N/mm^2. In the case of tuff aggregate concrete, the specimen yielded after attaining a compression stress of 8.5 N/mm^2 in which the concrete deformed permanently. As such, the compressive stresses were not proportional to the strains in the specimen. However, the elastic modulus of tuff aggregate concrete started increasing more than that of normal-weight concrete when the compressive stress on the specimen reached 10.2 N/mm^2. As the compression load in the specimen increased, the concrete deformed until the ultimate compressive stress of 18.1 N/mm^2 was reached. At this point, the specimen failed by rupture just after attaining a maximum strain of 0.0036. Similarly, as the normal-weight concrete specimen deformed under loading at a continuous rate, the concrete deformed plastically until compressive stress of 19.8 N/mm^2 was reached. It then failed by rupture as large cracks formed which then propagated from inside and spread outwardly. The fact that tuff aggregate concrete exhibited a higher range of compressive strain in its elastic deformation state than conventional concrete is beneficial and desirable in buildings susceptible to earthquakes.

Because of the ability of tuff concrete to hold more water within the pores of aggregates, the presence of water creates a more continuous contact zone between the aggregate and the paste and continued internal curing. These properties tend to reduce cracking in the concrete. Generally, the elastic modulus of lightweight concrete was lower than that of normal-weight concrete, mainly because of the lower rigidity and stiffness of tuff aggregates. The elastic modulus of concrete is normally between 10-30 GPa. The elastic modulus of concrete is a very important mechanical parameter reflecting the ability of the concrete to deform elastically. For example, in prestressed concrete structures, elastic shortening of prestressed concrete is one of the main factors that contribute to prestress losses. More details are in Appendix Table B.20-Table B.23.

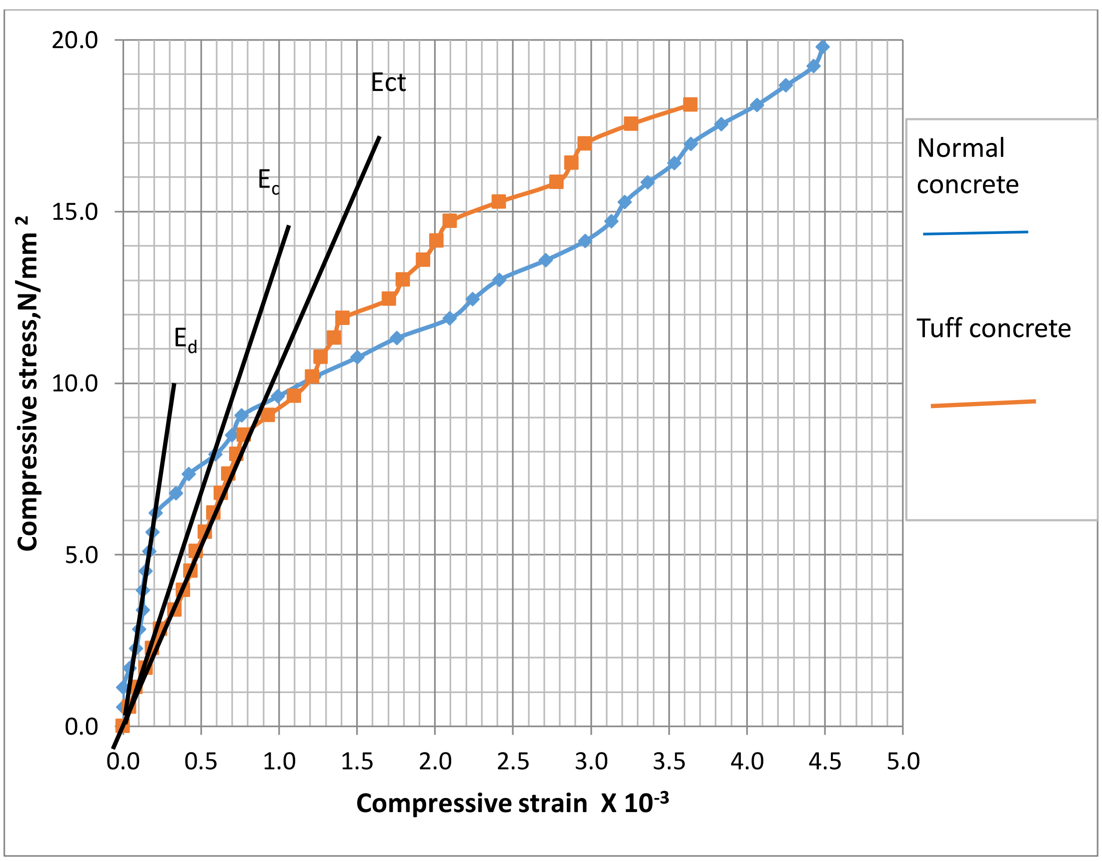

Figure 4.10: 28 -day average modulus of elasticity curves of concretes.

4.9 Comparative Analysis of the Benefits of using tuff concrete over NWC

There are many aspects of comparison of cost-benefit nature that can be cited in this study. The concrete mixes used crushed granite stones as the coarse aggregate while incorporating Portland cement, water, and sand for the conventional concrete. From the current study, natural granite stone is relatively denser than tuff aggregates adding to the weight of the conventional concrete mix. For instance, if floor slabs of a multistory building are made from tuff concrete, the self-weights of slabs will be reduced by 19%. The sizes of beams, columns, and foundations can therefore be reduced in sizes. Since billing of concrete in the construction industry is done by volume, it therefore follows that the cost of concrete required for the structural members supporting tuff concrete floor slabs such as beams, columns, and foundation would cost less than that required for structural members supporting conventional concrete floor slabs. Another area of comparison is the amount of reinforcement required during the construction of the two concrete slabs. For example, a builder would buy more reinforcements to do a conventional concrete slab element with a thickness of say 220 mm supported between two beams which are 5.0 m apart as observed from the analysis below. A builder will provide rebars of area 347 mm^2/m for tuff concrete as compared to 411 mm^2/m required for conventional concrete as top rebars on the supports of a concrete slab.

Assume a live load of 3.0 kN/m^2 is to applied on the slab element.

Assume the conditions of exposure as mild.

f_{cu} = 26.0 N/mm^2 for tuff concrete (from the results)

fcu = 30.0 N/mm^2 for tuff concrete (from the results)

f_y=460 N/mm^2 for steel

Comparative analysis based on the amount of Reinforcement

- **Tuff concrete slab element**

=≫Self weight for tuff concrete slab=0.22 mx1mx20.38 kN/m^2

=4.484 kN/m^2

=>Live load is given as 3.0 kN/m^2

Design load

Ld= 1.4gk+1.6qk (4.6)

Design load=1.4x4.484+1.6x3.0=11.078 kN/m^2

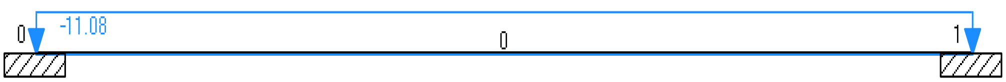

Figure 4.11: Free body diagram of the tuff concrete slab element.

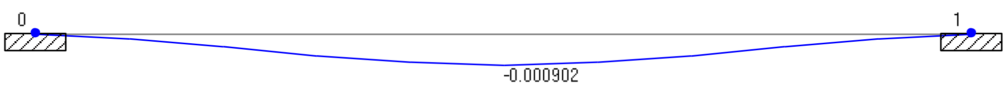

Figure 4.12: Deflection line diagram of the slab element.

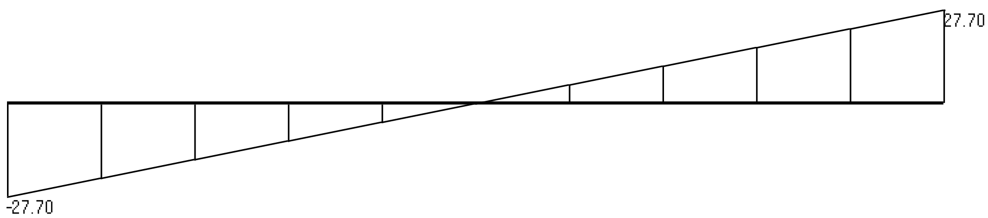

Figure 4.13: Shear force diagram of the slab element.

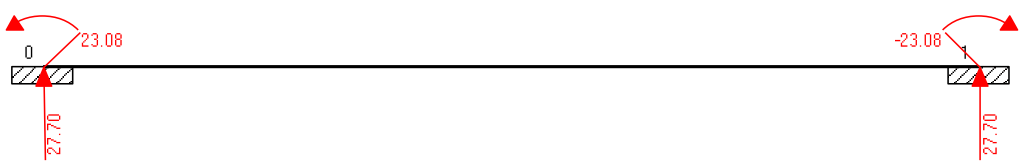

Figure 4.14: Reactions and end moment diagram of the slab element.

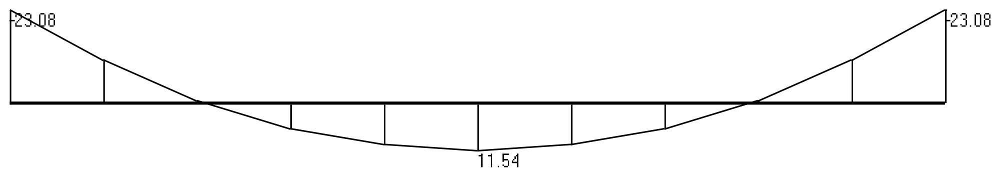

Figure 4.15: Bending moment diagram of the slab element.

Areas of reinforcement

At supports (due to hogging moments)

Main rebars;

k=$\frac{M}{f_{cu}bd^2}$ (4.7)

k=$\frac{M}{f_{cu}bd^2}$=$\frac{23.08x10^6}{26x1000x190^2}$ =0.025< 0.156

z=$\left\{\left[0.5+\sqrt{}(0.25-\frac{0.025}{0.9})\right]\middle|d\right\}$

z=$\left\{\left[0.5+\sqrt{}(0.25-\frac{0.025}{0.9})\right]\middle|d\right\}$=0.85d

BS 8110-1 1997, Table 3.3

Cover=25 mm, assumed diameter of bars 10 mm

d=220-5-25=190 mm

A_S=$\frac{M}{0.95Zf_y}$ (4.8)

A_S=$\frac{M}{0.95Zf_y}$=$\frac{23.08X10^6}{0.95x0.85x190x460}$=327 mm^2/meter run

Provide Y10-225 mm center to center (As provided=347 mm^2/m)

Distribution rebars;

A_{Smin}=$\frac{0.13bh}{100}$ (4.9)

A_{Smin}=$\frac{0.13bh}{100}$=$\frac{0.13x1000x220}{100}$=286 mm^2

Provide Y10-250 mm c/c (As provided=312 mm^2/m)

At the bottom (due to sagging moments)

Main rebars

M=11.54 kN-m

k=$\frac{M}{f_{cu}bd^2}$=$\frac{11.54x10^6}{26x1000x190^2}$ =0.012$<$ 0.156

z=$\left\{\left[0.5+\sqrt{(}0.25-\frac{0.012}{0.9})\right]\middle|d\right\}$=0.986, but $z \leq 0.95d$

BS 8110-1 1997, Table 3.3

Cover=25mm, assumed diameter of bars 10 mm

d=220-5-25=190 mm

A_S=$\frac{M}{0.95Zf_y}$=$\frac{11.54X10^6}{0.95x0.95x190x460}$=146 mm^2/ meter

But A_{Smin}=$\frac{0.13bh}{100}$=$\frac{0.13x1000x220}{100}$=286 mm^2

Provide Y10-250 mm c/c (As provided=312 mm^2/m)

Distribution bars

A_{Smin}=$\frac{0.13bh}{100}$=$\frac{0.13x1000x220}{100}$=286 mm^2

Provided Y10-250 mm c/c (As provided=312 mm^2/m)

- **Conventional concrete slab element**

=≫Self weight for NWC concrete slab=0.22mx1mx25.27kN/m^2

=5.560kN/m^2

=>Live load is given as 3.0 kN/m^2

Design load

Given by 1.4gk+1.6qk

Design load=1.4x5.56+1.6x3.0=12.583 kN/m^2

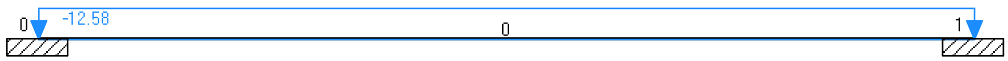

Figure 4.16: Free body diagram of the slab element.

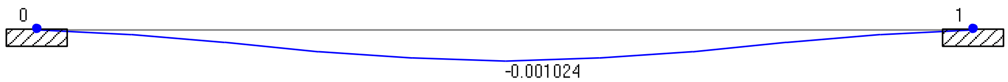

Figure 4.17: Free body diagram of the slab element.

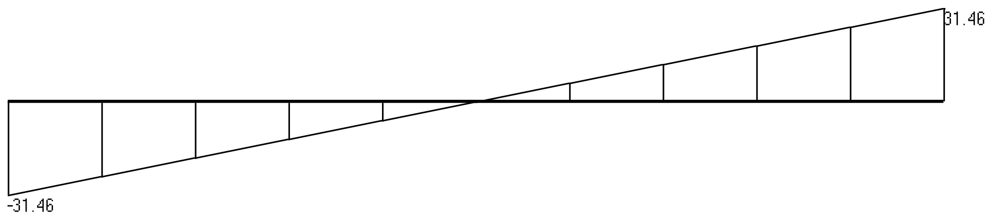

Figure 4.18: Shear force diagram of the slab element.

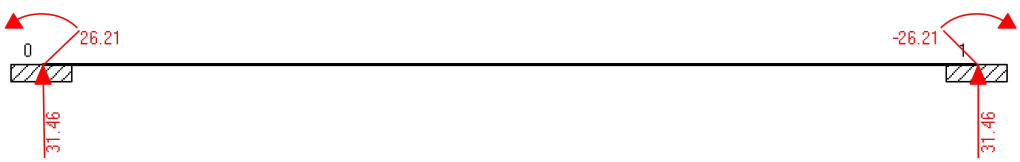

Figure 4.19: Reactions and end moment diagram of the slab element.

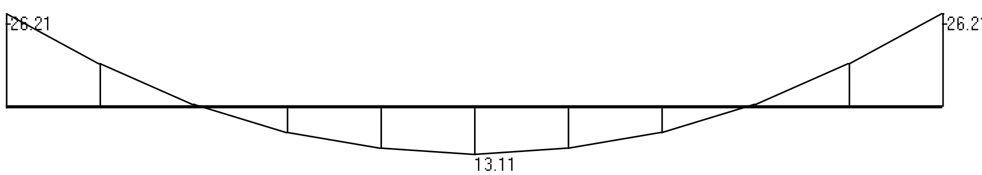

Figure 4.20: Bending moment diagram of the slab element.

Areas of reinforcement

At supports (due to hogging moments)

$$k=\frac{M}{f_{cu}bd^2}=\frac{26.21x10^6}{30x1000x189^2}=0.024< 0.156$$

$$z=\left\{\left[0.5+\sqrt{(0.25-\frac{0.024}{0.9})}\right]\middle|d\right\}=0.85d< 0.95d$$

BS 8110-1 1997, Table 3.3

Cover=25mm, assumed diameter of bars 12 mm

d=220-6-25=189 mm

A_S=$\frac{M}{0.95Zf_y}$=$\frac{26.21X10^6}{0.95x0.85x189x460}$=373 mm^2/meter run

Provide Y12-275 mm center to center (As provided=411 mm^2/m)

Distribution bars

A_{Smin}=$\frac{0.13bh}{100}$=$\frac{0.13x1000x220}{100}$=286 mm^2

Provided Y10-250 mm c/c (As provided=312 mm^2/m)

At the bottom (due to sagging moments)

M=13.11 kN-m

k=$\frac{M}{f_{cu}bd^2}$=$\frac{13.11x10^6}{30x1000x190^2}$ =0.012$<$ 0.156

z=$\left\{\left[0.5+\sqrt{}(0.25-\frac{0.012}{0.9})\right]\middle|d\right\}$=0.98d, $z = 0.95d$

BS 8110-1 1997, Table 3.3

Cover=25mm, assumed diameter of bars 10 mm

d=220-5-25=190 mm

A_S=$\frac{M}{0.95Zf_y}$=$\frac{13.11X10^6}{0.95x0.95x190x460}$=166 mm^2/ meter

But , A_{Smin}=$\frac{0.13bh}{100}$=$\frac{0.13x1000x220}{100}$=286 mm^2

Provide Y10-250 mm c/c (As provided=312 mm^2/m)

Distribution bars

A_{Smin}=$\frac{0.13bh}{100}$=$\frac{0.13x1000x220}{100}$=286 mm^2

Provide Y10-250 mm c/c (As provided=312 mm^2/m)

4.10 Summary of the Results

The results revealed an average rodded unit weight, M_{SSD}, of 1180 kg/m^3 for tuff aggregate SSD condition. A similar test for granite aggregate produced a rodded unit weight of 1600 kg/m^3. The average water absorption for tuff aggregate samples was higher reaching 21.6% while that of granite samples was as low as 0.25%. It was noted that tuff concrete mix with a w/c ratio of 0.4 can produce a slump value of 12 mm while normal-weight concrete produces a slump of 27 mm this was the case when air-dried coarse aggregates were used. However, the slump values increase as the water-cement ratio is increased for both concretes. While the compaction factors increase from 0.88 to 0.96 for normal-weight concrete with SP dosage of 0% to 1.5%, the compaction factors increase from 0.83 to 0.93 for tuff concrete having a similar dosage of SP. The compressive strength increases from 23.0 MPa to 25.5 MPa as the water-cement ratio is increased in the range 0.40 up to 0.60 for tuff concrete whereas the compressive strength of conventional concrete declines from 31.4 to 20.4 MPa with a similar w/c ratio variation.

The average 28-day unit densities of 2038 kg/m^3 and 2527 kg/m^3 for tuff and conventional concretes respectively were obtained during the current study. The average 28-day compressive strengths of 26.0 MPa and 30.0 MPa were obtained for tuff and conventional concretes respectively when presoaked aggregates were used with a water-cement ratio of 0.4. However, there was a slight increment in the compressive strength of tuff concrete noted after 35 and 42 days. This can be attributed to internal curing taking place in the concrete. The average tensile stresses of 2.9 N/mm^2 and 3.6 N/mm^2 for tuff and conventional concretes respectively were determined. The elastic moduli were 11.5Gpa and 14.3GPa for tuff and conventional concrete respectively. The elastic modulus of structural concrete is in the range of 10GPa to 30GPa. It was established that a conventional concrete member would require more reinforcement during construction than a similar member made from tuff concrete, making the production of tuff concrete less costly and economical in terms of rebars.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Introduction

This section has both conclusions as well as recommendations drawn from the study. It also has recommendations for further study.

5.1.1 Conclusions

The following conclusions can be drawn based on the results of the study;

i. The results of the investigation suggest that tuff aggregates are suitable natural materials for use in the production of structural low-density concrete in the mixture proportions of 1:2.3:2.2 for cement, sand, and tuff aggregates respectively.

ii. The workability of both concretes improves as the water-cement ratio in the mixes is increased.

iii. Inclusion of superplasticizer enhances the slump of both fresh tuff and conventional concrete from low to high and medium workability properties of 120 mm and 90 mm respectively.

iv. The compressive strength of tuff concrete increases from 23.0 MPa to 25.5 MPa as the water-cement ratio is increased from 0.40 up to 0.60 while that of the conventional concrete declines from 31.4 to 20.4 MPa with a similar w/c ratio variation.

v. Tuff aggregate concrete has a 28-day average compressive strength of 26.0 N/mm^2 and unit weight of 2038 kg/m^3 while a similar mixture for the conventional concrete produces a 28-day unit weight of 2527 kg/m^3 and compressive strength of 30.0 N/mm^2.

vi. The 28-day tensile strength of tuff aggregate concrete is 2.9 N/mm^2 while that of normal-weight concrete is 3.6 N/mm^2.

vii. The 28-day static elastic modulus of tuff aggregate concrete is 11.5 Gpa and that of conventional aggregate concrete of the same specific strength is 14.3 Gpa.

viii. The self-weight of structure constructed with tuff aggregate concrete will be approximately 19% lighter in weight than the one with normal-weight granite concrete.

ix. A concrete structure would cost less if constructed with tuff concrete than conventional concrete.

5.1.2 Recommendations

5.1.2.1 Recommendations Resulting from the Research

As evidenced by the findings, the following recommendations can be made;

i. Given the compressive strength of 26.0 N/mm^2, tuff concrete should be used for the construction of high-rise buildings where a reduction in self-weight of structures and sizes of the structural members may be necessary.

ii. Presoaking tuff aggregates before mixing concrete is a very important step in reducing the amount of mixing water required as well as in improving the workability of tuff concrete.

iii. Tuff aggregates absorb more water than conventional aggregates necessitating the use of superplasticizers to improve the workability of tuff concrete.

5.1.2.2 Recommendations for Further Research

The following recommendations can be made for further research;

i. Further research on change in unit weight should be carried out on tuff aggregate concrete to establish the equilibrium density which is usually measured after 90 days.
ii. Further study on tuff concrete to establish the positive effects of internal curing on concrete properties especially compressive strength development should be conducted for at least 90 days.
iii. Further research on the durability characteristics of tuff lightweight concretes containing different binder contents and water-cement ratios should be done.
iv. Further research should be carried out on the behavior and performance of tuff concrete in water retaining and prestressed structures with respect to tensile stresses, deflections, and permeability.

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APPENDICES

Appendix A

Table A.1: Concrete Mix Proportions to Produce 1m^3 of Concrete

Type of concrete	Tuff concrete	Granite concrete
Mixing water for concrete	133kg/m^3	133kg/m^3
Cement content	333kg/m^3	333kg/m^3
Fine aggregate content	768kg/m^3	768kg/m^3
Coarse aggregate content	740kg/m^3	1219kg/m^3
Air content	2%	2%
Water cement ratio	0.40	0.40

Appendix B

Table B.1: Sieve Analysis of Sand Aggregate (FA)

A	**FINE AGGREGATE/SAND**		
Sample source	**KAJIADO COUNTY**		
CLIENT		**Project**	**Research Thesis**
Test date:	**04-Jun-17**		

Specification	BS882:1992 TABLE 4

Pan mass (gm)	100		
Initial dry sample mass + pan (gm)	291		
Initial dry sample mass (gm)	191	Fine mass (gm)	0.5
Washed dry sample mass + pan (gm)	291	Fine percent (%)	0.3
Washed dry sample mass (gm)	191	Criteria (%)	

Total mass=191g

Sieve size (mm)	Retained mass (gm)	% Retained	Cumulative Retained mass	Cumulative passed percentage	Acceptance Criteria Min (%)	Max (%)
14	0	0.0	0.0	100.0		

10	3	1.6	3.0	98.4	100	
4.76	2	1.0	5.0	97.4	89	100
2.36	6	3.1	11.0	94.2	60	100
1.18	32	16.8	43.0	77.5	30	100
0.6	76	39.8	119.0	37.7	15	100
0.3	48	25.1	167.0	12.6	5	70
0.15	21	11.0	188.0	1.6	0	15
0.075	3	1.6	191.0	0.0		
	191	100.0				

Table B.2: Sieve Analysis of Tuff Coarse Aggregate/CA

Sample Type	**Coarse Aggregates**		
Sample source	**Bahati Area in Nakuru County**		
Client	**Department of Civil UON**	**Project**	**Lightweight aggregates**
Test date:	**16-Jun-17**		
Specification	BS882:1992 TABLE 3 20mm graded aggregates		

Dry sample mass		(gm)	10300		
				Acceptance Criteria	
Sieve size (mm)	Retained mass (gm)	% Retained (%)	Cumulative passed percentage	Min (%)	Max (%)
37.5	0	0.0	100.0	100	
20	600	5.8	94.2	90	100
14	6000	58.3	35.9	0	70
10	2400	23.3	12.6	0	25
5	1200	11.7	1.0	0	5
2.36	100	1.0			
	10300				

Table B.3: Sieve Analysis of Granite Coarse Aggregate/CA

Sample Type	**COARSE AGGREGATES**		
Sample source	**Kajiado Birika**		
Client	**DEPARTMENT OF CIVIL UON**	**Project**	**Granite aggregates**

Test date:	**16-Jun-17**
Specification	BS882:1992 TABLE 3 20mm graded aggregates

Dry sample mass (gm) 6436

Sieve size (mm)	Retained mass (gm)	% Retained (%)	Cumulative passed percentage (%)	Acceptance Criteria Min(%)	Acceptance Criteria Max (%)
37.5	10	0.2	99.8	100	
20	567	8.8	91.0	90	100
14	2975	46.2	44.8	0	70
10	1554	24.1	20.7	0	25
5	1005	15.6	5.0	0	5
2.36	325	5.0			
	6436				

Table B.4: Fineness Modulus of Sand

	Retained	Cumulative

Sieve size (mm)	mass (gm)	Mass retained	% passing spec'd sieve	% Retained spec'd sieve
10	3	3	98.4	1.5
4.76	2	5	97.4	2.5
2.36	6	11	94.2	5.5
1.18	32	43	77.5	21.5
0.6	76	119	37.7	59.5
0.3	48	167	12.6	83.5
0.15	21	188	1.6	94
0.075	3	191	98.5	1.5
				=270/100
			FM	=2.7

Table B.5: Zones of Sand According to IS 383-1970.

Sieve size	**Zone-1**	**Zone-2**	**Zone-3**	**Zone-4**
10mm	100	100	100	100
4.75mm	90-100	90-100	90-100	95-100
2.36mm	60-95	75-100	85-100	95-100
1.18mm	30-70	55-90	75-100	90-100
0.6mm	15-34	35-59	60-79	80-100

0.3mm	5-20	8-30	12-40	15-50
0.15mm	0-10	0-10	0-10	0-15
F.M	4.0-2.71	3.37-2.1	2.78-1.71	2.25-1.35

Table B.6: Specific Gravity Test Results of Sand

Contents	**Mass of Samples in gms**	
Samples	Sample 1	Sample 2
Mass of pycnometer,w1	35.82	35.86
Mass of pycnometer +sand,w2	51.49	51.51
Mass of pycnometer filled with water + sand, w3	97.18	97.29
Mass of pycnometer filled with water,w4	87.53	87.53
Specific gravity, GS	2.60	2.65

Table B.7: The Specific Gravity of Oven Dried and SSD Tuff Aggregate

Contents	Mass in grams		
Samples	Sample 1	Sample 2	Sample 3

Mass of SSD sample,m1	474.7	524.9	504.2
Mass of bottle with sample filled with water, m2	1488.8	1537.5	1519.3
Mass of bottle filled with water, m3	1253.8	1302.3	1283.8
Mass of oven dried sample,m4	390.25	431.30	414.68
Specific Gravity, OD	1.62	1.49	1.54
Specific Gravity, SSD	1.98	1.81	1.88

Table B.8: Specific Gravity of Oven Dried and SSD Coarse Aggregate (Granite)

Contents	Mass in grams		
Samples	Sample 1	Sample 2	Sample 3
Mass of SSD sample,m1	704.7	787.1	804.2
Mass of bottle with sample filled with water, m2	1693.3	1742.4	1758.7
Mass of bottle filled with water, m3	1253.8	1253.8	1253.8
Mass of oven dried sample,m4	702.9	785.14	802.2
Specific Gravity, OD	2.65	2.63	2.68
Specific Gravity, SSD	2.66	2.64	2.69

Table B.9: Results of the Rodded Unit Weight of Tuff Aggregate

Contents	Mass of samples		
Samples	Sample 1	Sample 2	Sample 2
Mass of bucket, T(g)	2582	2582	2582
Volume of the bucket, V(cm^3)	2759	2759	2759
Mass of bucket +aggregates, G(g)	5255	5260	5258
Rodded unit weight ,M_D(kg/m^3)	1178	1184	1179

Table B.10: Results of the Rodded Unit Weight of Granite Aggregate

Contents	Mass of samples		
Samples	Sample 1	Sample 2	Sample 2
Mass of bucket, T(g)	2582	2582	2582
The volume of the bucket, V(cm^3)	2759	2759	2759
Mass of bucket +aggregates, G(g)	4411	4406	4400
Rodded unit weight, M_D(kg/m^3)	1602	1600	1598

Table B.11: Physical Properties of Coarse Aggregates (Tuff and Granite)

Physical Properties	Tuff aggregate	Granite aggregate
Water absorption (%)	21.6	0.25
Specific gravity(OD)	1.55	2.65
Specific gravity(SSD)	1.89	2.66
Bulk density, kg/m^3	1180	1600

Table B.12: Mix Proportion per Cubic Meter of Concrete Grade M25

Component type	Volume of mix materials,m^3	Density of tuff concrete materials kg/m^3	Density of NWC materials, kg/m^3
Entrapped air	0.02		
Mixing water	0.190	190	190
Portland cement	0.106	333	333
Fine aggregate	0.292	768	768
Coarse aggregate	0.392	740	1219
Unit volume,m^3	1.000	2031	2510

Table B.13: Effect of W/C Ratio on a Slump of Concrete Mix Grade M25

Mix number	S1	S2	S3	S4	S5
Cement content, kg	4	4	4	4	4
Sand, kg	11.5	11.5	11.5	11.5	11.5
Tuff aggregate, kg	11.1	11.1	11.1	11.1	11.1

Granite aggregate, kg	18.3	18.3	18.3	18.3	18.3
Water content, kg	1.6	1.8	2	2.2	2.4
Water-cement ratio	0.4	0.45	0.5	0.55	0.60
Tuff concrete slump, in mm	12	15	25	30	45
Normal concrete slump, in mm	20	27	38	55	75

Table B.14: Effect of SP on Fresh Properties of Concrete Mix Grade M25

Mix number	S1	S2	S3	S4	S5	S6
Cement content, kg	4	4	4	4	4	4
Sand, kg	11.5	11.5	11.5	11.5	11.5	11.5
Tuff aggregate, kg	11.1	11.1	11.1	11.1	11.1	11.1
Granite aggregate, kg	18.3	18.3	18.3	18.3	18.3	18.3
Water content, kg	1.6	1.6	1.6	1.6	1.6	1.6
Water-cement ratio	0.40	0.40	0.40	0.40	0.40	0.40
Superplasticizer,%	0	0.5	1.0	1.5	2.0	2.5
Tuff concrete slump, in mm	12	25	35	70	100	120
Compaction factor	0.83	0.85	0.90	0.93	0.92	0.92
NWC slump, in mm	26	30	45	50	75	87
Compaction factor	0.88	0.90	0.94	0.96	0.95	0.93

Table B.15: Unit weights of tuff and conventional concretes

Days or age of concrete in days	Unit weight of tuff concrete in kg/m^3	Unit weight of Conventional concrete in kg/m^3
1	2031	2510
14	2036	2515
28	2038	2527

Table B.16: Effect of W/C Ratio on Compressive Strength of Concrete presoaked aggregates

Mix number	C1	C2	C3	C4	C5
Cement content, kg	4	4	4	4	4
Sand, kg	11.5	11.5	11.5	11.5	11.5
Tuff aggregate, kg	11.1	11.1	11.1	11.1	11.1
Granite aggregate, kg	18.3	18.3	18.3	18.3	18.3
Water content, kg	1.6	1.8	2.0	2.2	2.4
Water-cement ratio	0.4	0.45	0.5	0.55	0.60
Tuff concrete strength, N/ mm^2	25.0	24.5	24.0	19.2	17.0
Normal-weight concrete strength, N/ mm^2	29.0	27.8	23.7	24.5	21.0

Table B.17: Effect of W/C Ratio on Compressive Strength of Concrete

Mix number	C1	C2	C3	C4	C5	C6
Cement content, kg	4	4	4	4	4	4
Sand, kg	11.5	11.5	11.5	11.5	11.5	11.5
Tuff aggregate, kg	11.1	11.1	11.1	11.1	11.1	11.1
Granite aggregate, kg	18.3	18.3	18.3	18.3	18.3	18.3
Water content, kg	1.6	1.8	2.0	2.2	2.4	2.4
Water-cement ratio	0.4	0.45	0.5	0.55	0.60	0.65
Tuff concrete strength, N/ mm^2	23.0	23.4	23.8	25.2	25.5	25.0
Normal-weight concrete strength, N/ mm^2	31.4	28.0	26.0	24.5	20.0	19.0

Table B.18: Results of Compressive Strength of Concrete with Age

Mix number	C1	C2	C3	C4	C5	C6	C7
Cement content, kg	4	4	4	4	4	4	4
Sand, kg	11.5	11.5	11.5	11.5	11.5	11.5	11.5

Normal aggregate, kg	11.1	11.1	11.1	11.1	11.1	11.1	11.1
Water content, kg	1.6	1.6	1.6	1.6	1.6	1.6	1.6
Water-cement ratio	0.40	0.40	0.40	0.40	0.40	0.40	0.4
Age of concrete	0	7	14	21	28	35	42
Compressive strength, N/mm^2 (Granite)	0	17.0	23.0	27.0	30.0	31.0	31
Compressive strength, N/mm^2 (tuff)	0	14.0	20.0	24.0	26.0	27.0	28

Table B.18: Results of 28-day Splitting Tensile Strength of Tuff Concrete

Specimen no	Applied force, P (N)	Splitting tensile strength (N/mm^2)	Average splitting tensile strength (N/mm^2)
1	200,000	2.8	2.9

2	190,000	2.7	
3	210,000	3.0	

Table B.19: Results of 28-day Splitting Tensile Strength of NWC

Specimen no	Applied force, P (N)	Splitting tensile strength (N/mm^2)	Average tensile strength (N/mm^2)
4	250,000	3.5	
5	260,000	3.7	3.6
6	255,000	3.6	

Table B.20: Modulus of Elasticity of Tuff Concrete Specimen E1

SPECIMEN 1 | Date Tested: 21.07.18 | Density: 2031 Kg/m^3

Mean Diameter Do: 150 mm | The height of Specimen Lo:300 mm | Mean Area Ao: 17671 mm^2

Deformation Gauge Factor = 0.025mm/Division | The volume of Specimen: 5301437 mm^3

Deformation Gauge Reading	Compression of Specimen ΔL (mm)	Strain $\epsilon = \Delta L/Lo$	Axial force P (KN)	Specimen Area A = Ao mm^2	Axial Stress $\sigma 1 = P/A$
0.0	0.00	0.0000	0	17671	0
1.0	0.03	0.0001	10	17671	566
1.0	0.03	0.0001	20	17671	1132
1.5	0.04	0.0001	30	17671	1698
2.3	0.06	0.0002	40	17671	2264
3.0	0.08	0.0003	50	17671	2829
3.9	0.10	0.0003	60	17671	3395
4.5	0.11	0.0004	70	17671	3961
5.0	0.13	0.0004	80	17671	4527
5.3	0.13	0.0004	90	17671	5093
6.0	0.15	0.0005	100	17671	5659
6.7	0.17	0.0006	110	17671	6225
7.2	0.18	0.0006	120	17671	6791

7.8	0.20	0.0007	130	17671	7357
8.4	0.21	0.0007	140	17671	7923
9.0	0.23	0.0008	150	17671	8488
10.0	0.25	0.0008	160	17671	9054
13.0	0.33	0.0011	170	17671	9620
15.0	0.38	0.0013	180	17671	10186
16.0	0.41	0.0014	190	17671	10752
16.0	0.41	0.0014	200	17671	11318
17.0	0.43	0.0014	210	17671	11884
23.0	0.58	0.0019	220	17671	12450
24.0	0.61	0.0020	230	17671	13016
26.0	0.66	0.0022	240	17671	13582
27.0	0.69	0.0023	250	17671	14147
28.0	0.71	0.0024	260	17671	14713
30.0	0.76	0.0025	270	17671	15279
32.0	0.81	0.0027	280	17671	15845
34.0	0.86	0.0029	290	17671	16411
35.0	0.89	0.0030	300	17671	16977
37.0	0.94	0.0031	310	17671	17543
43.0	1.09	0.0036	320	17671	18109

Table B.21: Modulus of Elasticity of Tuff Concrete Specimen E2

The height of Specimen Lo:300 mm	Mean Diameter Do: 150 mm Mean Area Ao: 17671 mm^2	The volume of Specimen:5301437 mm^3 SPECIMEN 2		Density: 2031 Kg/m^3	
Deformation Gauge Reading	Compression of Specimen ΔL (mm)	Strain $\epsilon = \Delta L/Lo$	Axial force P (KN)	Specimen Area A = Ao mm^2	Axial Stress $\sigma 1$ = P/A (kN/m^2)
0.0	0.00	0.0000	0	17671	0
0.0	0.03	0.0001	10	17671	566
1.0	0.03	0.0001	20	17671	1132
2.0	0.04	0.0001	30	17671	1698
2.2	0.06	0.0002	40	17671	2264
2.7	0.08	0.0003	50	17671	2829
4.0	0.10	0.0003	60	17671	3395
4.7	0.11	0.0004	70	17671	3961
5.3	0.13	0.0004	80	17671	4527
5.8	0.13	0.0004	90	17671	5093
6.5	0.15	0.0005	100	17671	5659
7.1	0.17	0.0006	110	17671	6225
7.7	0.18	0.0006	120	17671	6791
8.3	0.20	0.0007	130	17671	7357
8.9	0.21	0.0007	140	17671	7923

9.5	0.23	0.0008	150	17671	8488
12.0	0.25	0.0008	160	17671	9054
13.0	0.33	0.0011	170	17671	9620
13.7	0.38	0.0013	180	17671	10186
14.0	0.41	0.0014	190	17671	10752
16.0	0.41	0.0014	200	17671	11318
16.3	0.43	0.0014	210	17671	11884
17.4	0.58	0.0019	220	17671	12450
18.4	0.61	0.0020	230	17671	13016
19.5	0.66	0.0022	240	17671	13582
21.0	0.69	0.0023	250	17671	14147
23.0	0.71	0.0024	260	17671	14713
25.0	0.76	0.0025	270	17671	15279
29.0	0.81	0.0027	280	17671	15845
34.0	0.86	0.0029	290	17671	16411
37.0	0.89	0.0030	300	17671	16977
41.0	0.94	0.0031	310	17671	17543

Table B.22: Modulus of Elasticity of Normal Weight Concrete Specimen E3

SPECIMEN 1 Mean Area Ao: 17671 mm^2

Deformation Gauge Factor =0.025mm/Division Density:2551 Kg/m^3

Deformation Gauge Reading	Compression of Specimen ΔL (mm)	Strain $Є = ΔL/Lo$	Axial force P (KN)	Specimen Area	Axial stress
0.0	0.00	0.0000	0	17671	0
0.0	0.00	0.0000	10	17671	566
0.0	0.00	0.0000	20	17671	1132
0.0	0.00	0.0000	30	17671	1698
0.0	0.00	0.0000	40	17671	2264
0.0	0.00	0.0000	50	17671	2829
0.0	0.00	0.0000	60	17671	3395
0.0	0.00	0.0000	70	17671	3961
0.0	0.00	0.0000	80	17671	4527
0.0	0.00	0.0000	90	17671	5093
0.0	0.00	0.0000	100	17671	5659
0.0	0.00	0.0000	110	17671	6225
2.0	0.05	0.0002	120	17671	6791
3.0	0.08	0.0003	130	17671	7357
4.0	0.10	0.0003	140	17671	7923
5.5	0.14	0.0005	150	17671	8488
6.0	0.15	0.0005	160	17671	9054

9.5	0.24	0.0008	170	17671	9620
12.0	0.30	0.0010	180	17671	10186
15.0	0.38	0.0013	190	17671	10752
19.0	0.48	0.0016	200	17671	11318
24.5	0.62	0.0021	210	17671	11884
27.0	0.69	0.0023	220	17671	12450
30.0	0.76	0.0025	230	17671	13016
33.0	0.84	0.0028	240	17671	13582
37.0	0.94	0.0031	250	17671	14147
40.0	1.02	0.0034	260	17671	14713
41.0	1.04	0.0035	270	17671	15279
43.0	1.09	0.0036	280	17671	15845
45.5	1.16	0.0039	290	17671	16411
46.0	1.17	0.0039	300	17671	16977
46.6	1.18	0.0039	310	17671	17543
49.0	1.24	0.0041	320	17671	18109
50.0	1.27	0.0042	330	17671	18675
52.0	1.32	0.0044	340	17671	19241
53.0	1.35	0.0045	350	17671	19806

Table B.23: Modulus of Elasticity of Normal Weight Concrete Specimen E4

The height of Specimen Lo: 300 mm	Date Tested: 21.07.18	SPECIMEN 2
Mean Diameter Do: 150 mm	Mean Area Ao: 17671 mm^2	Density: 2551 Kg/m^3
The volume of Specimen: 5301437 mm^3		Deformation Gauge Factor = 0.025mm/Division

Deformation Gauge Reading	Compression of Specimen ΔL	Strain $Є = ΔL/Lo$	Axial force P (KN)	Specimen Area $A = Ao\ mm^2$	Axial stress $σ1 = P/A$ (kN/m^2)
0.0	0.00	0.0000	0	17671	0
0.0	0.00	0.0000	10	17671	566
0.0	0.00	0.0000	20	17671	1132
1.0	0.03	0.0001	30	17671	1698
2.0	0.05	0.0002	40	17671	2264
2.5	0.06	0.0002	50	17671	2829
3.0	0.08	0.0003	60	17671	3395
3.0	0.08	0.0003	70	17671	3961
3.4	0.09	0.0003	80	17671	4527
4.0	0.10	0.0003	90	17671	5093
4.5	0.11	0.0004	100	17671	5659
5.0	0.13	0.0004	110	17671	6225
6.0	0.15	0.0005	120	17671	6791
7.0	0.18	0.0006	130	17671	7357

10.0	0.25	0.0008	140	17671	7923
11.0	0.28	0.0009	150	17671	8488
12.0	0.30	0.0010	160	17671	9054
14.0	0.36	0.0012	170	17671	9620
17.0	0.43	0.0014	180	17671	10186
20.0	0.51	0.0017	190	17671	10752
22.5	0.57	0.0019	200	17671	11318
25.0	0.64	0.0021	210	17671	11884
26.0	0.66	0.0022	220	17671	12450
27.0	0.69	0.0023	230	17671	13016
31.0	0.79	0.0026	240	17671	13582
33.0	0.84	0.0028	250	17671	14147
34.0	0.86	0.0029	260	17671	14713
35.0	0.89	0.0030	270	17671	15279
36.5	0.93	0.0031	280	17671	15845
38.0	0.97	0.0032	290	17671	16411
40.0	1.02	0.0034	300	17671	16977
44.0	1.12	0.0037	310	17671	17543
47.0	1.19	0.0040	320	17671	18109
50.4	1.28	0.0043	330	17671	18675
52.6	1.34	0.0045	340	17671	19241