University of Nairobi

Institute of Nuclear Science and Technology

NON-DESTRUCTIVE TESTING OF CONCRETE STRUCTURES USING SCHMIDT HAMMER AND PROFOMETER 5+

By

KAIRU, WILSON MACHARIA

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A thesis submitted in partial fulfilment for the degree of Master of Science in Nuclear Science in the Institute of Nuclear Science and Technology, University of Nairobi.

Declaration

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

Signature .... Date ....

Kairu, Wilson Macharia (S56/74431/2012)

This thesis has been submitted with the knowledge of the supervisors.

Supervisors

Mr. David M. Maina, Sign……………………..
Institute of Nuclear Science and Technology, Date……………………..
University of Nairobi

Prof. Michael J. Gatari Sign……………………..
Institute of Nuclear Science and Technology, Date……………………..
University of Nairobi

Prof. Michael L. Muia Sign……………………..
Faculty of Science Date……………………..
Technical University of Kenya

Dedication

This research work is dedicated to my wife Kate and my family for their immeasurable support during my study period.

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Abstract

Kenya has been facing various and sometimes serious durability problems in concrete buildings. In the past ten years alone, more than fourteen buildings have collapsed. The causes of these failures have been in most cases attributed to poor workmanship, as well as poor quality of the materials used during the construction of these buildings. This in turn results in poor quality structures that eventually fail leading to injuries, deaths and loss of money. The long term development goals for Kenya, under Vision 2030, feature a key component on infrastructure development. The infrastructure will require a high degree of structural safety, longevity and performance. To guarantee safety and durability in these developments, an effective technique for early and frequent structural assessments, is both dependable and reliable, is required to provide assessment and quality assurance of these structures. Non-destructive techniques possess a tremendous potential to be part of that system.

This study reports non-destructive testing measurements of concrete compressive strength using a laboratory pre-calibrated Schmidt rebound hammer, while rebar diameter, location and cover were measured using a Profometer 5+ Covermeter, as well as test reliability of this tools to the task. Laboratory calibration involved preparation of 60 cubes of concrete specimens at 5 different classes of concrete mix proportions. The compressive strengths of these cubes were assessed using a Denison Compressive Testing Machine. Data obtained from hammer measurements were plotted against related Denison data, and a calibration equation $y=0.9x$ generated using least square fitting at a correlation coefficient of 0.8. This equation was the used to obtain in-situ strength properties of new and old concrete structures.

The Profometer was able to locate the exact position of the bar with a high level of accuracy. The equipment was also able to measure the size of the reinforcement bars and its cover, provided that they are located at 60 mm depth or less from the measuring surface. These methods were later used to conduct structural test in two residential and three commercial buildings within Nairobi city. Higher compressive strengths were recorded in commercial buildings as compared to the residential ones. The obtained data can be compared with the structural design to check for compliance.

In conclusion, the two methods proved to be fast, inexpensive and non-destructive in nature, and can be an important tool for monitoring the integrity of concrete structures

throughout their design life to assure their safety and durability. However, the two tools had their own shortcomings. The performance of Schmidt hammer was found to be affected by plastering. To obtain more reliable results, it is recommended to remove the plaster coating prior to testing. As for Profometer 5+, its efficiency was found to decline at cover depths > 60 mm, and when the rebars are closely spaced.

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CHAPTER 1

INTRODUCTION

1.1 Background

Concrete is one of the commonly used materials worldwide in the construction industry due to its durability. It is continuously being used in the construction of countless number of public structures and buildings. These structures include; deep foundations, high rise buildings, earthquake-proof bridges, dams, among others. The Burj Khalifa skyscraper in Dubai, United Arab Emirates, the tallest man-made structure in the world (829.8 m) was constructed using a highly flowable concrete mixture that does not solidify before reaching at the top, but still result into a robust final product (Crow, 2008). Burj Mubarak al Kabir skyscraper in Kuwait which is scheduled for completion in 2016 with a height of 1,001m, that will be able to withstand wind speeds of over 240 Km/h, uses high performance concrete with a mix designed to provide low permeability and high durability. In Japan, a special ultra-strength variety of concrete has been manufactured and used to construct earthquake-proof bridges.

Human beings have been using concrete in their pioneering architectural feats for millennia. The first recorded concrete-based structures were constructed in some parts of Jordan and Syria, around 6500 BC by the Nabataea traders (Gromicko et al., 2013). Around 3000 BC, the ancient Egyptians used lime mortars and gypsum in constructing the Pyramids. The Pantheon, Rome, is the world's largest non-reinforced concrete structure and is more than 2000 years old (constructed in 126 AD) and stands at a height of 46 m.

Modern concrete is normally composed of aggregate, sand, cement, water, mineral admixtures and chemical admixtures. The work of the admixtures is to improve the behavior of concrete under several conditions (Merin et al., 2014). Mineral admixtures make the concrete economical, increase strength, reduce permeability and influence other concrete properties. Chemical admixtures bring down the cost of construction, alter properties of hardened concrete, and ensure quality of concrete during mixing, transferring, placing and hardening.

When subjected to compressive forces, concrete is very strong but is relatively weak when tensional forces are involved. In order to set up concrete constructions which can withhold tensile stresses, it must be reinforced with steel to form what is commonly known as reinforced concrete (RC). In reinforced concrete, the steel reinforcement and the concrete form a firm bond, the combination acting as a single material providing a high tensile compressive and shear strength. In addition to increasing the strength of concrete, reinforcements also aids in preventing growth of cracks caused by shrinkage or surface cracking in the concrete construction (IAEA, 2002).

The strength of concrete and the position of the reinforcements are critical for the durability of the structures, especially those subjected to a degrading environment during their service life (Gjørv, 2011). Concrete with high compressive strength will have better durability performance since it can be able to withstand exposure to severe environmental condition. The positioning of the reinforcement is also important to the durability of the concrete structures. If its placement is too close to the surface, it is usually vulnerable to corrosion that is induced by chloride ion ingress, which leads to a reduced service life of the reinforced concrete constructions (Farid et al., 2010). It is therefore important that the specified concrete cover is reached, so that the reinforcement is protected from external agents that may cause corrosion.

It is important to have a regular schedule for testing concrete structures, in order to have a safe and efficient operation of these structures. This schedule may vary depending on the structure use, and also on the policies regulating such test. For instance, the concrete containment building of nuclear power plant structures may require a more regular testing schedule than a commercial or residential building. This is because the structures are designed to retain radioactive material should an accident occur, and it is therefore highly crucial to guarantee the structural capacity and leak tightness of the structure. These assessments are meant to give data on the integrity of the structure, and hence its performance over the time. Essentially, these assessments ought to be carried out devoid of any damages to the concrete structure.

The available tests for concrete testing can either be destructive, semi-destructive or non-destructive in nature (Hola and Schabowicz, 2010). Destructive tests results in the samples being destroyed after testing. As a result, only a few sample representatives can be subjected to these tests. Semi-destructive tests on the other hand involve a small (often superficial) intrusion into the concrete structure, leading to localized loss of service properties which can easily be fixed (Hola and Schabowicz, 2010). Non-destructive test allows inspection of the concrete component or structure without interfering on its service properties or final use (IAEA, 2001). As a result, they can be used to carry out test and examination the same components and structures numerous times and at different times without any worry of change in their properties (Hola and Schabowicz, 2010).

The primary goal of non-destructive testing is to predict or assess the service life and performance of concrete structure at different stages of its service cycles (IAEA, 2005). The performances as well as the service life of a reinforced concrete structure are governed by several parameters such as strength, quality of concrete, concrete cover, age, and most significantly by exposure conditions (Sanjeev et al., 2014). Non-destructive testing provides us with information on these parameters which enable us to assess both the performance and the service life of any structure.

NDT methods can be put into two main categories. The first category consists of methods which are used in estimation of the strength of concrete, while the second category comprise of methods whose aim is to evaluate integrity (Carino, 1997). The strength of concrete in RC structures is one of the most crucial properties and constitutes the principal parameter used while designing these structures (Pereira and Mederies, 2012). Several methods can be used to assess concrete strength in finished structures or structures in use. These methods include rebound hammer test, pull-out test, penetration resistance, pull off tests, break-off methods and internal fracture test (Bungey, 1994). Integrity tests are employed in flaw detection and condition assessment. Several flaws are likely to occur in concrete and includes; honeycombing, voids, de-lamination, cracks and deficiency of sub-base support. The flaw detection techniques are based on the principle that any internal anomaly in a component or structure will always interfere with the propagation of certain types of waves. Techniques used to carry out an integrity test include; visual inspection, ground penetrating radar, stress-wave propagation method, electrical/ magnetic methods, infrared thermography, and nuclear methods.

Non-destructive testing may be employed to inspect both fresh built and ageing RC structures. The primary application in new structures is quality control. This is done to ensure that the structures adhere to a defined set of quality criteria, or meets the requirement of a client or customer. Traditionally, quality control in new structures has been done mostly through visual inspection, and by taking concrete specimens for standard laboratory tests on both fresh and hardened specimens (Davis, 1998). This approach is not reliable because it does not provide information on the concrete in situ properties. The in-place concrete properties is dependent on many other factors such as aggregates type, type of cement, the ratio of water to cement, hardening and the surrounding environmental conditions (Al-Mishhadani et al., 2012). In addition, the control exercised during construction and proper compaction, for instance, contributes a great deal to the accomplishment of the desired quality. Thus this approach is not sufficient for quality control of new concrete structures (Mahmood, 2008).

The examination of existent and ageing concrete construction is usually carried out to assess their structural integrity (IAEA, 2002). It has traditionally been done by obtaining core samples from structures and testing them in the laboratory. This is usually complicated since it involves specimen removal which is expensive and may result in some damage on the structure. Moreover the cost of coring and assessments could possibly allow a few tests to be done on a big structure. Thus, both the quality and the quantity of the resulting information might be inadequate, inconsistent and misleading (IAEA, 2002). NDT may be employed in such circumstances as a preliminary to consequent coring.

Different concrete properties can be obtained using non-destructive testing methods (BS103, 2009). These properties include resistance to penetration, resonance frequency, hardness, rebound number, and the capacity to permit ultrasonic waves to pass through concrete. Once these properties are obtained, they are applied to assess the concrete structure condition (ACI Committee, 2013).

Quality evaluations may be made with NDT techniques to offer invaluable data and information on the performance of RC concrete structures. NDT techniques are able to provide dimensions of structures, identify areas where there is delamination, cracking, and debonding, provide information on the degree of consolidation and presence of voids and honeycomb, measurements of size and location of steel reinforcement, corrosion action on the reinforcement, and extent of damage caused by chemical exposure, accidental fire, or freezing and thawing. With this kind of information, it is possible to easily locate suspected areas, thus decreasing the period and budget of inspecting a big mass of concrete.

1.2 Statement of the Problem

Failure and collapse of both residential and commercial buildings is a key challenge facing the housing industry in Kenya. In the last ten years, the country has experienced a series of disasters, with over fourteen buildings collapsing claiming over 50 lives and more than 120 injured as shown in Table 1.1 (Ngugi, 2013). These tragedies have been mostly attributed to poor supervision, poor workmanship, as well as poor quality of the materials used during the construction of these buildings (Machuki, 2012; Ayedeji, 2011). Due to high demand for housing in the country, some property developers often bypass building regulation to cut cost and maximize profits. As a result, they fail to adhere to the basic laws and regulation governing infrastructural development. This results in poor quality of the materials being supplied and used during the construction of these buildings, which in turn results in poor quality structures which eventually fails leading to injuries, deaths and loss of money to the investor and the nation at large. The culture of using non-destructive methods in inspection of reinforced concrete structures is not anchored in any piece of registration in Kenya. Non-destructive testing can help in determining whether these structures are being set up without following the code of practice

Table 1.1: Reported cases of collapsed buildings in Kenya from 2003 to 2013 (Ngugi et al., 2014)

	Location	**Building Description**	**Date**	**No. of Reported Deaths**	**No. of injured**
1	Ronald Ngala Street, Nairobi CBD, Nyamakima	Five storey commercial building	24th June 2006	20	35
2	Kiambu Town	Five storey commercial building	19th October 2009	11	14
3	Kiambu Town	Rental Residential Building	January 2010	3	4
4	Mulolongo, Nairobi	Six storey building	9th June 2011	4	15
5	Langata, Southern bypass, Nairobi	Langata Southern Bypass building	20th June 2011	None	6
6	Mosocho in Kisii County	One-storey building	7th May 2012	None	3
7	Ngara, Nairobi County	One Storey building	30th July 2011	None	5
8	Makupa, Mombasa County	Four storey building	April 09,2009	3	7
9	Luanda, Vihiga, Western Kenya	Three storey building	September 2011	3	5
10	Westlands, Nairobi	Seven storey building	May 2012	Unknown	2
11	Kasarani, Nairobi	Residential buildings	5th February 2012	None	6
12	Embakasi, Pipeline estate	Six storey building	June 2011	2	6
13	Matigari Building Mathare North	Not reported	9th Sept 2011	Not reported	Not reported
14	Kisumu	Six storey building	16th Jan 2014	7	35
16	Makongeni, Nairobi	Five storey residential	17thDec 2014	1	6
15	Huruma, Nairobi	Seven Storey	5th Jan 2015	4	38

1.3 Justification

The demand for housing and the development of infrastructure is expected to increase even more as the country endeavors to achieve its development blueprint under Vision 2030. Numerous buildings, both residential and commercial, are expected to be set up to meet housing demand by year 2030. It is highly important to ensure that these developments meet structural safety requirements and durability by employing an efficient system of structural assessment. The system should be able to provide structural performance of these developments during construction and on a regular basis during the structure's lifetime. Non-destructive testing (NDT) techniques have a tremendous capacity of being employed in carrying out these assessments. This is because nondestructive examination is capable of providing useful data on the condition of a concrete structure without causing any damage. Such information can be invaluable help when planning on maintenance these developments. Nondestructive testing could also provide quality assurance during construction as concealed flaws can be discovered early and remedied, when access to the construction is still possible without significant inconvenience. Special inspections consisting of a visual survey and NDT techniques can determine fault locations at an early stage and hence provide useful information of the structure actual condition. These techniques are comparatively fast, easy to employ, inexpensive and provide an overall indication of the necessary concrete property

NDT methods have been available for metallic and composite materials for over a century now. Aashish et al., (2014), gives the historical events in the development nondestructive testing. The first nondestructive testing was the Oil and Whiting technique (a precursor to modern liquid penetrant test) which was used to locate cracks in steel metals used in the railroad industry between 1880 and 1920. This was followed by successful use of radiography to examine casting set up in a steam pressure power station in 1924. In 1926, the first electromagnetic eddy current device was designed to determine material thickness. Ultrasonic test method was later developed in the period 1940-1944 by Dr. Floyd Firestone.

The development of non-destructive test methods for concrete testing has advanced at a much slower pace as compared to non-destructive test methods for steel structures. This is mainly due to heterogeneous nature of concrete which makes it harder to test than steel (Carino, 1997). The first NDT method for testing concrete, the Schmidt

Hammer, was invented in 1950 (Malhotra, 1976). It was until 1980's that innovative NDT techniques that could be employed for the evaluation of existent structures became available (Carino, 1997). Even so, they are still not fully established for regular inspections and examination and this work aims to investigate whether these methods can actually be used to carry out these regular structural checks.

1.4 Objectives of the study

1.4.1 Main Objective

The primary objective of this study was to investigate the applicability of Schmidt Rebound Hammer and Profometer 5+ Covermeter in assessing the performance of reinforced concrete structures.

1.4.2 Specific Objectives

The specific objectives were;

- Develop a co-relation between Schmidt hammer rebound index and compressive strength of concrete structures.
- Use the obtained co-relation to assess the compressive strength of existing concrete structures.
- To determine the accuracy, performance and applicability of the Profometer 5^+ covermeter in measuring the reinforcement size and cover in concrete structures.
- Use the Profometer 5^+ covermeter to obtain the reinforcement bar parameters in an existing structure.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Before 1980's, conferences, symposia and workshops organized by the NDT community rarely included sessions dealing with civil structures (Carino, 1997). The growing demand for characterization of ruined concrete structures, together with the crisis of "aging infrastructure" has highlighted the need for reliable NDT techniques that can be employed to evaluate the quality of reinforced concrete constructions. This has led to a growing attention to the subject of non-destructive testing (NDT) of concrete in recent years (Turgut, 2004; Bilgehan and Turgut, 2010; Akash et al., 2013). As a result, several methods have been developed for investigation and assessment of various aspects associated with durability, strength and quality of concrete.

Carino (1997) presents a brief history for the non-destructive techniques commonly employed in testing and evaluation of concrete. The author reviews the underlying principle of each method, the inherent limitation, and presents a historical perspective of its development. According to Carino (1997), there is no standard definition for non-destructive test as employed to concrete testing. To some, non-destructive testing comprises of any technique that does not alter the concrete, while to other people they are assessment that cause less damage than that caused by drilling of cores in their application.

The first non-destructive test on concrete according to Carino (1997) was developed way back in 1934 by Prof. K. Gaede. Gaede reported on usage of a spring-driven impactor to provide force that drives a steel ball into the concrete. A non-linear, empirical relationship was noted between indentation diameter and cube strength. Several other methods would later be developed but at a much slower pace as compared to the non-destructive test in metals. Carino (1997) attributes the slow pace to the heterogeneous nature of concrete, making it difficult to carry-over the NDT technology designed for steel testing to inspection of concrete structures.

There are several benefits of applying non-destructive tests such as reduction in labor, prospect of examining strength of concrete structures even in areas where it is impractical to drill cores, and the fact that it is relatively cheaper as compared to core testing (Leshchinsky, 1991). It is important to note that the non-destructive test results must be reliable and representative, otherwise these benefits are of no value (Akash et al., 2013; Turgut, 2004).

2.2 Schmidt Hammer Test

The Schmidt Hammer is a suitable tool in estimation and prediction of concrete strength. This is because a satisfactory degree of accuracy can be attained for strength approximation of concrete using a suitable regression model. As a result, it can be employed safely for estimation of concrete strength in reinforced concrete structure examination (Shang et al., 2012).

The Cemex (2013) technical standard gives the appropriate way to operate the Schmidt hammer with updates to reflect the modifications to ASTM C805 standard. According to this standard, the Schmidt Hammer test can be applied in the following functions: to evaluate in-place uniformity of concrete, to delineate areas in a structure which are of low quality, and finally to estimate in-place strength in case a correlation is established. The Schmidt hammer could be a valuable device for examining the uniformity of concrete in situ, on condition that the concrete is constructed under similar conditions in relation to surface carbonation, moisture content, age, and temperature.

There are so many factors affecting the Schmidt hammer test which can give rise to up to a 70% error when predicting the concrete strength (Antonio et al., 2013). These factors are moisture content, maturity of concrete, and stress state. During strength estimation, it is necessary to take into account these factors, failure to which the results of the measurement will not be reliable. According to Antonio et al., 2013, this particular observation might be as a result of restricted portion of the concrete on which the test is conducted, thus enabling minor sample inhomogeneity to strongly influence the test. Consequently, the authors conclude that this tool is not very effective in the estimation of compressive strength, and can merely be used as an instrument for conducting homogeneity tests in certain concrete types. These results are in contrary to those of Hamidian et al., (2012), hence further investigations are required.

According to Hamidian et al., (2012), Schmidt hammer proved to be a simple, fast and cost effective technique of determining concrete strengths, which can readily be put to use on concrete specimens and also on existing structures. The author carried out a structural health monitoring using Schmidt Rebound Hammer. The results revealed that a strong positive correlation exists between the reading from Schmidt hammer (Rebound number) and compressive strength.

A case study by Mahmoudipour (2009) on the Schmidt hammer and ultrasonic test methods derived a unique equation for combined methods. He observed that Schmidt hammer test and ultrasonic test are very convenient and can be executed anywhere. Furthermore, when using ultrasonic tests the compression strength can predicted more precisely than with Schmidt hammer test. By combining both test methods, the strength estimation is greatly is enhanced.

2.3 Electromagnetic Methods of Testing Concrete

According to Barnes and Zheng, 2008, reinforcement bar size as well as detectable range setting in the covermeter has a considerable influence on its reliability and accuracy. The author made this observation after carrying out a research on factors affecting concrete cover measurements. The covermeter probe detectable range setting i.e. high or low ranges, the bar size setting, and also the scan position relative to a secondary bar were investigated. The authors observed that it is advisable to get the information on bar size in the concrete prior to obtaining the cover measurement, so as to attain better results. In addition, lower values were recorded at “High” range settings, thus recommended for use only if the “low” range probe is unable to detect the reinforcing bars. In case cover measurement is needed for structures with deep concrete cover, then an appropriate calibration must be performed before undertaking the actual measurement.

Nyim (2006) developed an experimental calibration equation for electromagnetic covermeter test, which found that it was adequate and reliable enough to use an electromagnetic covermeter to undertake tests on reinforced concrete structures. He further observed that its usage can play a vital role in assuring product quality, thus avoid the loss of life and property. He proposed the incorporation of NDT methods in early stage of construction to ensure quality, safety and minimize cost that may arise as a result of failures.

According to Subramanian et al., (2013), the performance of covermeters is highly overrated by the manufacturers. They investigated the accuracy and reliability of Proceq Profometer and found that it was only rebars with diameter 16-32 mm in the cover depth range of 34-42 mm that could be detected effectively. The Profometer was unable to provide rebar sizes beyond this cover range.

2.4 Discontinuities and defects in concrete structures

Concrete structures are susceptible to different types of defects and discontinuities during their lifetime. These flaws in concrete structures can lead to direct or indirect expenses with regards to rectification and construction time. It is therefore crucial to make sure that there is proper handling during construction to reduce occurrences of defects in the structure. The most prevalent type of defects in concrete structures and probable causes are as discussed in the following sections.

Cracking of concrete

Cracking has an effect on the visual appeal of the concrete. Occasionally, it also affects the structural durability and strength (Kashinath and Gupta, 2015). In reinforced concrete, cracking enables air and moisture to reach the reinforcement bars causing steel to corrode, and consequently weakening the entire structure. Cracking either occurs before the concrete hardens or after hardening. Cracks occurring before hardening are as a result of movement of concrete before the concrete have set. These types of cracks fall into three categories; first, plastic shrinkage cracks which is common in hot and windy environments and usually develop as straight lines, either parallel or pattern; second, plastic settlement cracks that tends to follow the lines of reinforcement and often appear while concrete is still plastic and; and third, cracks caused by movement of the formwork which occur during positioning and compaction and is caused by movement of a weak formwork (IAEA, 2002). On the other hand, cracks may appear after hardening and are caused by settlement, drying shrinkage, as well as structural cracks. These cracks may necessitate structural renovation using high pressure epoxy.

Spalling

Spalling occurs when the edges or surfaces of concrete blocks chip off or break from the main element (plate 2.2). Normally, this is due to a combination of several factors such as poor installation and environmental aspects like freezing temperatures which stress the concrete, inducing some damages. Failure to timely repair it, spalling can occasionally prompt structural damages like rusting of reinforcing bars located in the concrete (PCA, 2001). Spalling can easily be fixed by breaking out the affected area, followed by wetting and refilling of the affected area to its original form. Spalling being a visual defect does not require non-destructive testing as a repair technique.

Plate 2.1: A cracked concrete surface

Honeycombing

Honeycombing is a phrase which is used to describe parts of the concrete exterior which are rough and stony (plate 2.3). It is usually attributed to inadequate fine material in the concrete mix, possibly as a result of wrong aggregate grading or even poor blending. Some of the proposed remedial measures include boosting the cement and sand content in the concrete mix and also through adequate blending, placing as well as compaction.

Honeycombing could also be as a result of leakage of grout or mortar portion of the concrete during formwork joints or construction, which may be averted by making sure that the joints are well sealed and leak-free. Deep honeycombing areas may lower the protection ability of the concrete cover to the reinforcement bars, which could in turn result in to possibly durability problems in the future.

Plate 2.2: A Spalled Concrete Surface

Plate 2.3: Honeycombing on a concrete surface

Dusting

Dusting is manifested as a fine, powder-like substance that readily rubs off the concrete surface (plate 2.4). It may either occur indoors or outdoors, though it is more probable to become an issue when it develops indoors. Dusting develops as a result of a thin, weak layer, referred to as laitance, made up of cement, aggregates and water. Fresh concrete is reasonably cohesive mass, with the cement, water and aggregates being evenly distributed all over. A specified period of time must elapse to allow water and cement to react thoroughly to produce hardened concrete. In this time period, the aggregate and cement particles tend to be suspended in water (PCA, 2001). However, with time, the aggregates and cement tend to sink since they are denser than water. As they settle at the bottom, the displaced water moves to the top and appears at the surface as bleed water, which results in more water near and at the surface in comparison to the bottom section of the concrete. Consequently, the laitance, which is weak, more permeable and easily worn out is at the top, precisely where the strongest, impermeable, and most wear-resistant concrete is required.

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Plate 2.4: Dusting on Concrete Surfaces

Crazing

Crazing is a systematic pattern of tiny cracks which often does not permeate deeper into the surface (plate 2.5), which is caused by minor surface shrinkage. The cracking looks like a map design and they only run through the concrete surface. It is attributed to slight surface shrinkage due to the drying conditions. One way to avoid crazing is by finishing and curing at the earliest opportunity after the concrete has been poured. However, these cracks normally do not trigger any further destruction of the concrete structure (PCA, 2001).

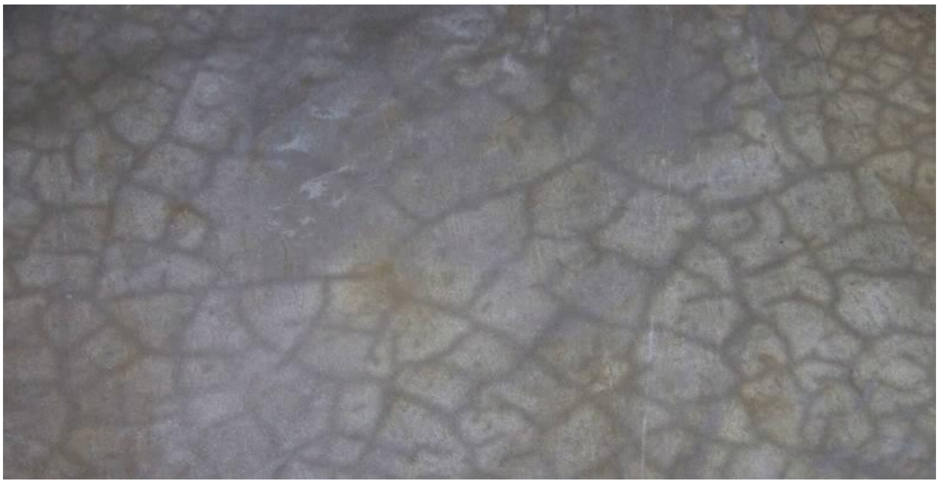

Plate 2.5: Crazing on concrete surface

Rain damage

Heavy rain may erode the surface of newly placed concrete (plate 2.6). It can easily be averted through covering freshly set concrete using plastic or polythene sheeting whenever there is a down pour. Rain damage on the concrete surface that is yet to harden may be reworked on or refurbished (IAEA, 2002).

Efflorescence

Efflorescence is a whitish crystalline deposits resulting from water-soluble salts that are left on the concrete surface during drying process (plate 2.7). These salt deposits often appear several weeks after the construction, and sometimes even a year after completion.

Plate 2.6: Concrete surface which has been damaged by rain

For efflorescence to occur, the following three conditions must exist. First, there has to be water-soluble salts existing somewhere within the structure. Second, there needs to be adequate moisture within the structure in order to dissolve the salts into a solution, and lastly, there has to be a pathway for the dissolved salts to seep through to the exterior where the moisture can vaporize, hence depositing these salts which in turn crystallizes to cause efflorescence (PCA, 2001). However, efflorescence does not have any impact on the concrete structural performance and can easily be cleaned up by dry scrubbing and flushing with water (IAEA, 2002).

Plate 2.7: Efflorescence on concrete surface

Blistering

Blistering takes place when air gets trapped within the concrete and fails to break off the seal that was developed in the course of finishing operations or else prompted by a quickly setting concrete surface. The air accumulates in spots beneath the airtight surface seal, consequently creating blisters (plate 2.8). It can be averted simply by delayed trowelling and also by covering the surface to minimize evaporation (IAEA 2002).

Plate 2.8: Blistering on concrete surface

Corrosion of reinforcing bars

Corrosion is described as a thermodynamically spontaneous and inevitable reaction of metals, which is adverse to the metallurgical process. Most metals, particularly steel which is iron based, are extremely prone to corrosion (plate 2.9). The rate of corrosion varies widely depending on various factors such as steel properties and environmental medium. With regard to reinforced concrete structures, the high alkalinity levels of the pore solution, as well as the barrier offered by the concrete cover from outside environmental conditions, then the rate of corrosion of the steel reinforcement bars is expected to be too slow to raise any concern. However, over time, certain concrete cover is unable to provide sufficient protection to these bars as a result of deterioration of concrete and also the infiltration of corrosive elements from the surroundings. Corrosion happen when either the cracking in concrete surface permit water to enter, or when water gets into the concrete through diffusion as a result of carbonation. The reinforcement bars tend to expand due to the formation of rust (iron oxide), which in turn leads to the concrete cover over affected bars to spall off (Guangling and Ahmad 1998). Consequently, corrosion of the reinforcement bars reduces the durability of concrete structures, leading to loss of money in the repairs and rehabilitation of damaged structures.

Plate 2.9: Corrosion of reinforcements

These defects and discontinuities can affect the integrity and strength of the concrete structure. The techniques used to carry out assessment of these flaws include; visual inspection, infrared thermography, stress-wave propagation method, electrical/ magnetic methods, ground penetrating radar and nuclear methods. In addition, the efficiency and performance of Schmidt hammer has been found to be affected in areas with these flaws (Brozovsky, 2011).

CHAPTER 3

METHODOLOGY

3.0 Introduction

This section presents the operation principles of the Schmidt hammer and the Profometer 5+ covermeter. This is followed by a presentation on the methods used to develop a correlation between the Schmidt hammer rebound index and the compressive strength, and also on determination of the accuracy of the Profometer 5+ in the measuring the reinforcement size and cover. While the Schmidt hammer is a surface hardness testing tool, the Profometer 5+ is used in locating rebars, obtaining the rebar size and the concrete cover over the rebar. The parameters obtained from these two equipment are important when determining the quality and durability of any reinforced concrete structure.

3.1 Operation Principles

3.1.1 Schmidt rebound hammer method

Schmidt rebound hammer (Figure 3.1), invented by Ernst Schmidt in 1948, is primarily a surface hardness tester. Its working principle is based on the fact that a Schmidt of an elastic mass depends on the hardness of the surface against which the mass impinges (Malhotra, 1976). There is an empirical correlation between the strength properties and the Schmidt number for its use in concrete strength evaluation.

The Schmidt rebound hammer weighs about 1.8 kg and is made up of a plunger rod (11), an internal spring loaded steel hammer (10), and a latching mechanism. Figure 3.2 shows how measurements are done using the Schmidt rebound hammer. The hammer is pressed hard towards the concrete surface and its body let to move far from the concrete surface till the latch connects the hammer mass to the plunger (Figure 3.2 a). Then, the body is pressed towards the concrete, while holding the plunger in a perpendicular position to the surface (Figure 3.2 b). This kind of movement stretches the spring holding the mass to the body. Upon reaching the maximum extension of the spring, the latch lets out and then the mass is drawn to the surface by the spring (Figure 3.2 c). The mass strikes the shoulder of the plunger rod and rebounds since the rod is pressed hard against the concrete (Figure 3.2 d). As it rebounds, the slide indicator on the exterior of the device moves with the hammer mass, and halts at when the optimum distance is reached by the mass after the rebound. This indication is referred to as the rebound number (R-number). The R-number can be obtained from the graduated scale by simply pushing the button on the side of the device to lock the plunger in the retracted position. The higher the R-number the greater the hardness of the concrete surface, indicating a higher compressive strength. The tests can be executed either in a horizontal, vertically downward, vertically upward or any inclined positions relative to the surface (Cemex, 2013). The Schmidt rebound hammer measurements are based on the standards ASTM C805 and BS 4408 part 4 (Mindes et al., 1981, Kumar et al., 1987).

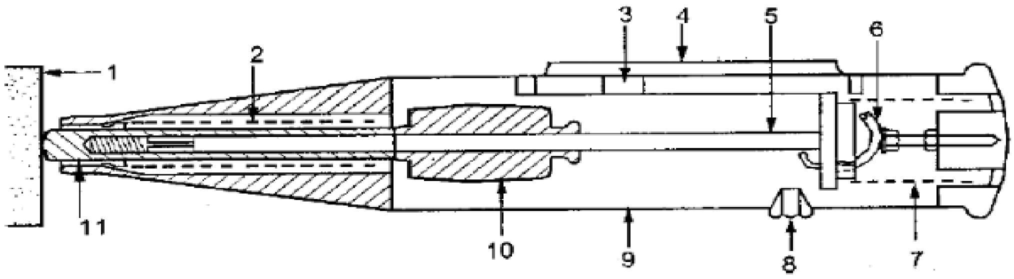

Figure 3.1: Components of a Schmidt Hammer (Mahmood, 2008): 1 is concrete; 2 is impact spring; 3 is rider on guide rod; 4 is window and scale; 5 is hammer guide; 6 is release catch; 7 is compressive strength; 8 is locking button; 9 is housing; 10 is hammer mass and 11 is plunger

The vertical positioning of the hammer in relation to the surface, however, affects the R-number because of gravity. Therefore the R-number of the floor is likely to be small compared to that which is acquired in a vertically upwards position, whereas inclined and vertical orientation would give intermediate results. Though a high R-number signifies concrete having a higher compressive strength than one having a low R-number, the test is just valuable only if a relationship is established between the R-number and concrete constructed using similar coarse aggregate as the one being tested. Over reliance on the calibration curve provided by the manufacturer with the hammer has been discouraged since the manufacturer generates this calibration curve by use of standard cube specimens, and the used mix could be totally different from the one being tested.

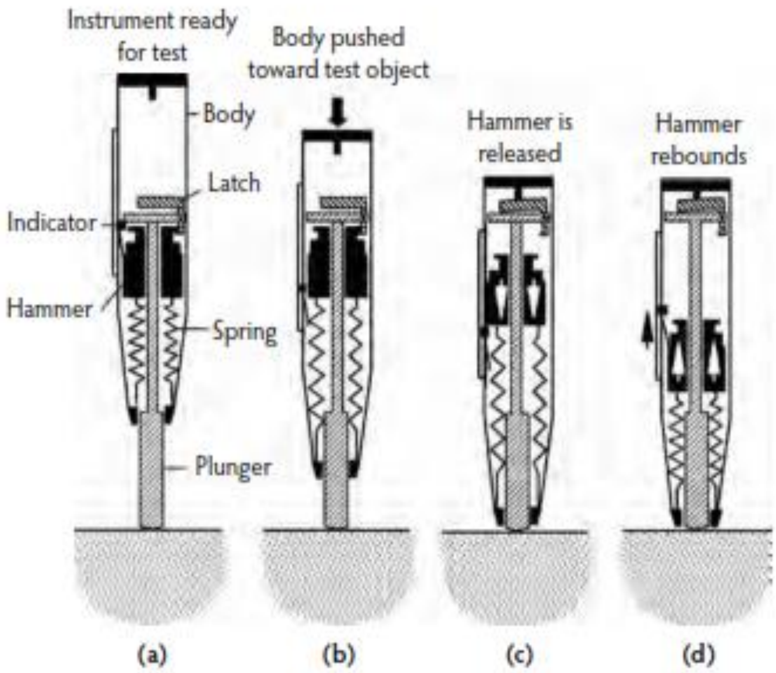

Figure 3.2: Schematic cross section of Schmidt hammer with illustration of how measurements are carried out

The results of the Schmidt rebound hammer are influenced by the texture of the test surface. Better results are obtained when the hammer is used against a smooth surface (Brozovsky, 2011). Very coarse or soft surfaces or even surfaces having loose mortar need to be rubbed smooth using an abrasive carborundum stone. The abrasive stone is made of medium-grain texture silicon carbide or equivalent material (Figure 3.3). Any water present on the surface of the concrete should be removed before measurements and testing since this may result in a lower R-number value and in turn a lower compressive strength recorded. Additionally, areas with rough texture, honeycombing, scaling, or high porosity should be rectified or avoided prior to measurements.

It is necessary to carry out a periodic calibration of the Schmidt hammer using standard anvil (Figure 3.4). This is done to ensure that the rebound test mechanism is functioning properly. In event that values above the tolerance (caused by contamination by very fine cement, defects or wear), inspection or cleaning is necessary. The test anvil is comprised of a high cylinder made of tool steel with an impact area hardened to 66 ± 2 HRC, which stands for Rockwell Hardness. An instrument guide is provided to center the Schmidt hammer over the impact area and maintain the hammer perpendicular to the test surface (WSDOT, 2013).

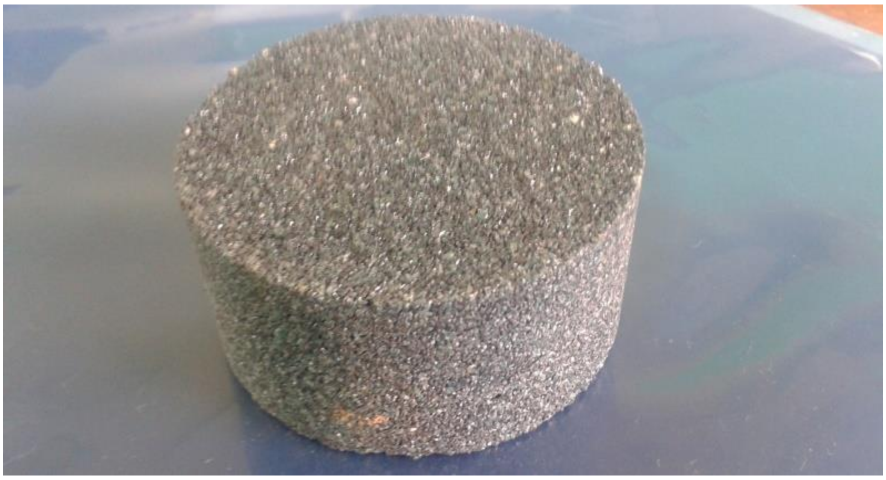

Figure 3.3: Picture showing an abrasive stone used to prepare course or soft surface for Schmidt hammer measurements

Figure 3.4: A picture showing a test anvil for periodic calibration of the Schmidt hammer

3.1.2 Profometer 5$^+$ Covermeter

The Profometer 5+ covermeter is a device used in locating the rebars and also to determine the depth of the concrete cover (Figure 3.5). It has two key components; a meter and a locating probe. The meter has a display that gives the size of concrete cover over the reinforcement bars, or a signal strength reading (Figure 3.6). The locating probe of the covermeter is a rectangular encapsulated unit that contains a directional search coil. An audible indicator is also given by the meter that guides someone on signal strength and in locating the embedded steel.

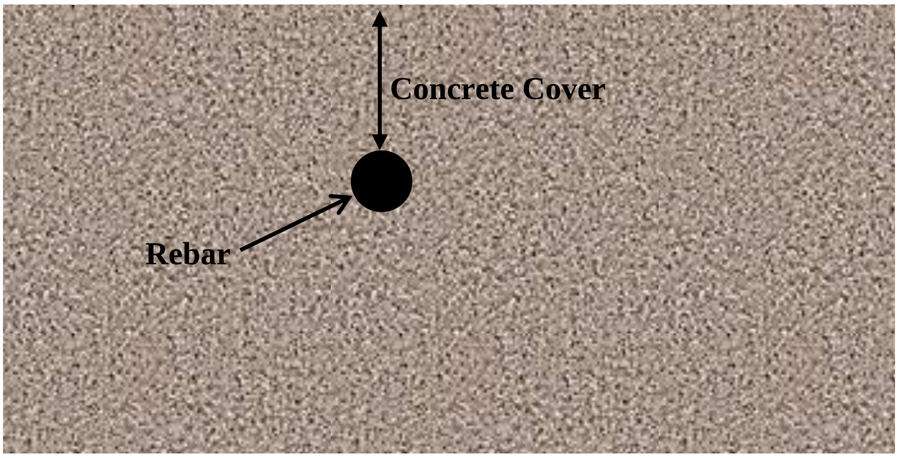

Figure 3.5: A diagram displaying the concrete cover and reinforcement position in concrete

Figure 3.6: Picture of a Profometer 5+ covermeter

There are two possible physical principle involved; either by using magnetic induction effects or eddy current effects. For the covermeter utilizing the effect of eddy current, the current passing through a coil generates eddy currents around the steel reinforcement which causes a difference in the impedance measurement of the search coil. These covermeters are operated at frequencies more than 1 kHz. They are therefore very sensitive to the existence of any kind of conducting material which is near the search coil. On the other hand, for covermeters utilizing magnetic induction employs multiple coil search head operating at a lower working frequency compared to the one utilizing eddy current (normally less than 90 Hz). As such they are less sensitive to components which are not magnetic when compared to those utilizing the eddy current principle. A change in the quantity of ferromagnetic material underneath the search coil, for example, by the existence of reinforcement bar or any other metallic object, results in a rise in the strength of the field. Consequently, this leads to an increment in the voltage observed from the secondary coil, which can be displayed by a meter after amplification.

The direction of orientation relative to the search head and the distance of the reinforcement from the search head influence the meter reading regardless of whether the instrument utilizes eddy current or magnetic induction effects. Therefore, it is possible to locate reinforcement bars and other metallic object inside the concrete and establish their orientation as well. The concrete cover can also be established by use of appropriate calibration. Most covermeter come with a process on how estimates can be made for both the size and cover to the bar when neither is known.

The search head has a primary coil that carries time-varying electric current which in turn produces a time-varying magnetic field. The generated field advances towards the metallic target besides other directions, as it reacts with the magnetic and/ or electrical properties of the metal target. Then, the target responds through generation of a secondary magnetic field which links back to the coils in the search head inducing an electrical voltage in the receiver coil.

3.2 Samples Preparation

The samples used in this work were made from locally available materials which include Bamburi Nguvu cement (CEM IV/ B(P) 32,5N), fine aggregates obtained from natural river sand, and crushed course aggregate of diameters ranging from 10 mm to 20 mm as per KS 95 (5). The fine aggregates were obtained from Mlolongo, one of the main supply points in Nairobi and its environs.

3.3 Development of the correlation between the Schmidt hammer rebound index and the compressive strength

A total of 60 cubes samples of various grades were prepared for the Schmidt hammer measurements using five different grades of concrete. The five grades of mix proportions are listed in Table 3.1. Each grade of specimens consisted of 12 cubes of dimensions 150 mm × 150 mm × 150 mm.

Table 3.1: The mix ratio of materials for the five grades used for sample/ specimen preparation of reinforced concrete

Grade	Cement	Sand	Course Aggregate	Water/Cement
C15	1	3	6	0.5
C20	1	2	4	0.5
C25	1	1.5	3	0.4
C30	1	2	3	0.4
C35	1	1.5	2.6	0.4

A set of 3 cubes were tested at 7 and 14 days intervals with the remaining cubes being tested at 28 days in accordance with ASTM C 805. Two opposite surfaces of the cubes were prepared for the horizontal measurements by the Schmidt hammer by wiping with a dry cloth. A load of 7 N mm^{-2} was applied on the specimen cubes to prevent specimen motion during the hammer measurements. The measurements involved obtaining a horizontal rebound number from each of the two prepared surfaces by pushing the hammer against the surface at a fixed amount of energy in accordance with procedures described in ASTM C 805-85 (1993) and BS: 1881 (1986). The Hammer was held firmly so that the plunger is perpendicular to the test surface. It was then gradually pushed toward the test surface until the hammer impacted. After impact, the pressure was maintained and the lock button depressed. Twenty-four (24) readings at selected points on the surfaces, 12 on each side, were obtained, and an average value (f_a) calculated in accordance with ASTM C 805. If any single reading differed from the average by more than seven units, it was discarded and the average recalculated using the remaining readings. Thereafter the cube was loaded until failure on a Denison Compressive testing machine at Department of Civil Engineering, College of Architecture and Engineering and the failure loading recorded (Figure 3.8). The standard deviations for the hammer measurements were calculated using equation (3.1).

$$S = \sqrt{\frac{1}{n}\sum_{x=1}^{n}(f_i - X_b)^2} \quad \text{.............................. (3.1)}$$

Where, S is the standard deviation, n is rebound numbers measured on the each specimen, X_b is the average of the measured rebound numbers in each specimen, equation (3.2), and f_i is the measured rebound number.

$$X_b = \frac{1}{n}\sum_{i=1}^{n} f_i \qquad (3.2)$$

A regression curve of the average rebound number (X_b) versus the maximum compressive strength to failure was then drawn from which a regression equation was generated for use in calculating the compressive strength for rebound numbers obtained in the field measurements of the building columns.

Figure 3.7: Taking the Rebound measurements before loading the cubes to failure

3.4 Determination of the accuracy of the Profometer 5^+ Measurements

For measurements with the Profometer 5+ five concrete blocks measuring 500 mm × 300 mm × 150 mm were cast using the mix ratio of grade C25. This involved making moulds from block boards with holes at 40 mm, 60 mm, 80 mm and 100 mm from the surface (Figure 3.9) and inserting reinforcement steel rebar's in those holes. The concrete mix was poured carefully to ensure the position the rebars did not move out the specified depth (Figure 3.10). The blocks were then compacted through external vibration, de-moulded after 24 hours and cured for 28 days before measurements could be made.

Figure 3.8: Picture of the Denison machine for testing concrete compressive strength

The accuracy of the covermeter probe was then assessed in the laboratory using these blocks. This was done by obtaining the average of four measurements from the prepared specimen and comparing it to the actual rebars sizes which had been casted into these blocks. It was then used in the field to obtain the reinforcements cover and sizes.

Figure 3.9: A picture of the block board mould prepared for specimen casting.

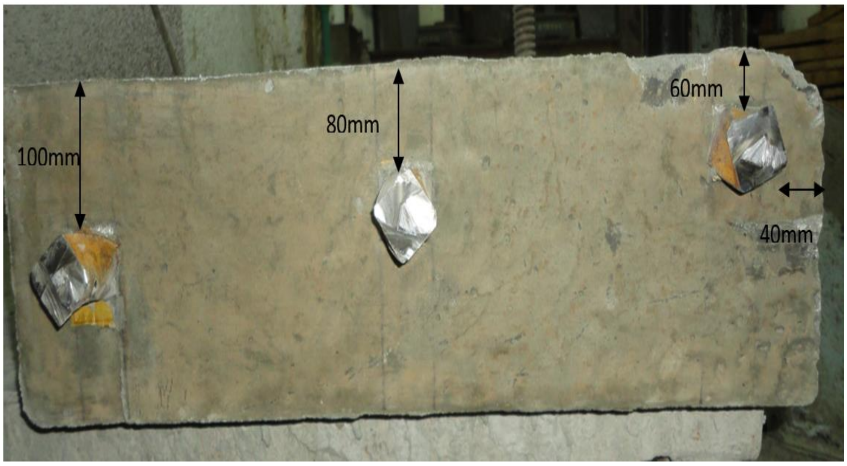

Figure 3.10: A picture showing the position of the rebars and concrete cover depth on the casted reinforced concrete block.

To locate the reinforcement bar, the probe was placed on the concrete surface (Figure 3.11) and parallel orientation to the bar established by holding it at the position of maximum indicator signal. The probe was then moved from one end to other in a direction perpendicular to the rebar while observing the signal value on the covermeter screen. At the position of maximum signal, the cover and rebar diameter value were recorded, where the latter was obtained by pressing an activation button.

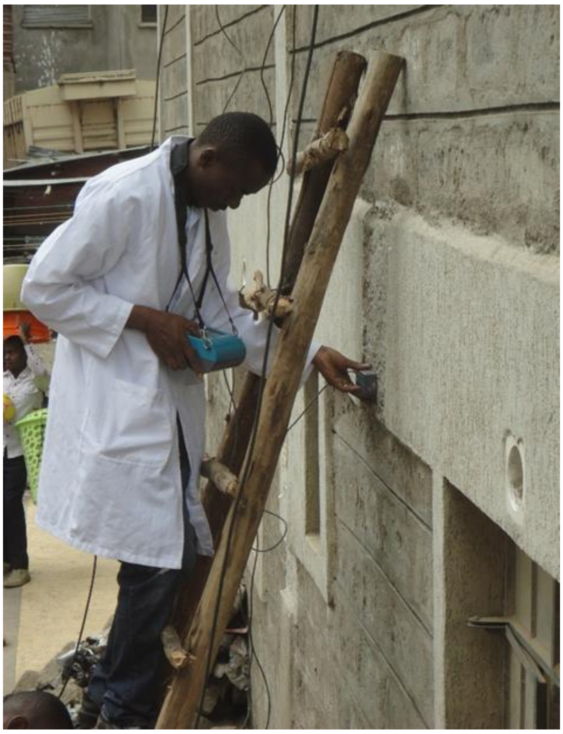

Figure 3.11: Undertaking structural tests using Profometer 5+ covermeter

Field measurements were conducted on five buildings within Nairobi metropolis. The selection of these buildings was determined mainly by accessibility of the columns, beams and slab, and most importantly, authorisation from the management. Schmidt hammer measurements were conducted on the columns (Figure 3.12) because the empirical formula generated in the Denison Compressive testing machine was based on horizontal orientation. The measurements were also conducted on the beams where it was possible to make horizontal measurements. In total, five buildings were selected for the study, and below is a brief description;

i. A five storey building in Westlands area. This is an old structure, with year of completion as 1994. For this structure, columns were only accessible from the basement, fourth and fifth floor which were unoccupied.
ii. A four storey building located along Uhuru Highway, which was undergoing construction, in its finishing stage at the time of study. Columns were selected from all the floors i.e. lower ground floor, ground floor, mezzanine floor, as well as first to fourth floor.

Figure 3.12: Undertaking surface hardness tests using Schmidt rebound hammer

iii. A three storey residential building in Kasarani estate. The building structural health was determined using columns in the first, second and third floor.
iv. Sstructural tests were also carried out in a residential eight storey building located in Eastleigh estate, columns in the ground, fifth and eighth floor were assessed.
v. A three storey building in Kenyatta National Hospital, hosting the Institute of Tropical and Infectious Diseases. It was constructed in 2004, currently being renovated. Compressive strength was determined for all the floors.

In-situ rebound measurements were conducted for the five structures. The compressive strength, concrete cover and size of the reinforcement bars were determined. For each column, at least five measurements were recorded to determine the average for that column.

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 Introduction

In this section the results of this study are reported and discussed. The laboratory calibration results for the Schmidt hammer are presented first followed by accuracy measurements of the Profometer 5+ covermeter which is then compared to the manufacturers accuracy charts Finally, the insitu measurement results on existing buildings using the Schmidt hammer and Profometer 5+ covermeter are presented and discussed.

4.2 Schmidt hammer rebound index and the compressive strength correlation

4.2.1 Concrete Grade 15

The results of concrete grade 15 are shown in Table 4.1. The compressive strength of concrete increases with age, with 99% of the maximum strength possible being achieved at 28 days (Table 4.2). For concrete grade 15, the minimum compressive strength after curing for 7, 14 and 28 days should be 9.8, 13.5 and 15 N mm^{-2} respectively. However, at age of 7 days, it was impossible to obtain compressive strength of concrete grade 15 cubes specimens with an initial holding load of 7 N mm^{-2}. The first two cubes failed while setting up the Denison compressive testing machine to this holding load. The failure was attributed to high holding force, and on reducing the initial load to 2.2 N mm^{-2}, the third cube yielded a compressive strength of 3.6 N mm^{-2} which is way below what is expected at 7 days. All cubes tested at 14 and 28 days failed to achieve the minimum compressive strength (Table 4.1). As a result, this concrete grade was not used to obtain the correlation curve.

Table 4.1: Table of compressive strength (N mm^{-2}) of concrete grade 15 at various ages

	Compressive strength		
Cube no.	**7 days**	**14 days**	**28 days**
1	3.6	7.6	11.6
2	-	10.2	9.8
3	-	7.1	8.2
4	-	-	8.6
5	-	-	8.2
Expected value	9.8	13.5	15.0

Table 4.2: Expected strength at various ages (The Constructor, 2014).

Age (days)	Strength (%)	Grade 15	Grade 20	Grade 25	Grade 30	Grade 35
1	16	2.4	3.2	4.0	4.8	5.6
3	40	6.0	8.0	10.0	12.0	14.0
7	65	9.8	13.0	16.3	19.5	22.8
14	90	13.5	18.0	22.5	27.0	31.5
28	99	14.9	19.8	24.8	29.7	34.7

4.2.2 Concrete Grade 20

The results of measurements for the grade 20 concrete at the ages of 14 and 28 days are shown in Table 4.3. The expected compressive strength at 14 and 28 days is 18 and 20 N mm^{-2} respectively. All the cubes tested in this grade achieved the minimum compressive strength.

Table 4.3: Compressive strength of concrete grade 20 (N mm^{-2}) at various ages

	Compressive Strength		**Average R-Number**	
Cube no.	**14 days**	**28 days**	**14 days**	**28 days**
1	18.7	24.0	25.85	25.21
2	21.3	23.3	24.65	27.54
3	20.4	23.6	23.46	27.08

4.3.3 Concrete Grade 25

Table 4.4 gives the results of compressive strength and rebound number (R- Number) obtained from the Denison compressive testing machine and the Schmidt hammer for grade 25 concrete. The expected compressive strength at the age of 7, 14 and 28 days is 17, 22.5 and 25 N mm^{-2} respectively. At the age of 7, 14 and 28 days all the cubes achieved the expected compressive strength. One of the cubes tested at day 7 showed an extremely high value of compressive strength (32.4 N mm^{-2}) which could not be explained, thus treated as an outlier.

4.2.4 Concrete Grade 30

Results of measurements for the grade 30 concrete at the ages of 7and 28 days are presented in Table 4.5. All the cubes tested at 7 and 28 days achieved the minimum strength of 19.5 and 30 N mm^{-2} respectively.

Table 4.4: Results of compressive strength (N mm^{-2}) and rebound number

	Compressive Strength			Average R-Number		
Cube no.	**7 days**	**14 days**	**28 days**	**7 days**	**14 days**	**28 days**
1	26.2	33.8	25.3	27.96	28.28	33.50
2	21.8	26.7	32.0	26.17	29.60	31.88
3	25.3	31.1	29.8	25.42	29.52	34.21
4	22.2	-	32.4	39.91	-	30.75
5	32.4	-	38.2	38.30	-	34.67
6	-	-	33.8	-	-	36.08

Table 4.5: Results of measurements for the grade 30 concrete (N mm^{-2}) at the ages of 7 and 28 days

	Compressive Strength		Average R-Number	
Cube no.	**7 days**	**28 days**	**7 days**	**28 days**
1	24.0	37.3	26.71	39.79
2	20.9	34.2	25.29	36.79
3	23.6	35.6	27.71	38.25
4	-	37.8	-	41.38
5	-	42.1	-	40.88
6	-	35.1	-	41.21

4.2.5 Concrete Grade 35

Table 4.6 presents the results of compressive strength and R-number obtained from the Denison compressive testing machine and the Schmidt hammer for grade 35 concrete. The strength gained at 7, 14 and 28 days is expected to be over 22.5, 31.5 and 35 N mm^{-2} respectively. At 7 days, only one of the three cubes tested attained the minimum expected strength. All cubes tested at 14 days failed to attain the minimum strength of

31.7 N mm^{-2}. This was attributed to the presence of honeycombing which was observed on the surface of the cubes. The remaining six cubes which were tested after 28 days of curing achieved the required minimum strength.

Table 4.6: Results of compressive strength and rebound number at 7, 14 and 28 days

	Compressive Strength			Average R-Number		
Cube no.	**7 days**	**14 days**	**28 days**	**7 days**	**14 days**	**28 days**
1	23.1	24.9	37.6	25.79	33.50	34.83
2	20.4	26.7	43.4	25.63	31.88	35.42
3	20.9	24.0	33.2	25.17	34.21	35.25
4	-	-	35.7	-	-	34.87
5	-	-	40.3	-	-	36.08
6	-	-	38.1	-	-	35.63

Measurements of rebound numbers and the compressive strength were used to develop the best fit equation (Equation 4.1), which was used to estimate the compressive strength of two sampled buildings, using in situ rebound measurements.

$$y = 0.92x \qquad \text{……… (Equation 4.1)}$$

Figure 4.1 show the line of best fit with Schmidt rebound value as the independent variable (x-axis) and the compressive strength as the dependent variable (y-axis), where y is the compressive strength, and x is the average rebound number of a specimen. The coefficient of correlation was 0.8 which shows a fairly good correlation between these two variables.

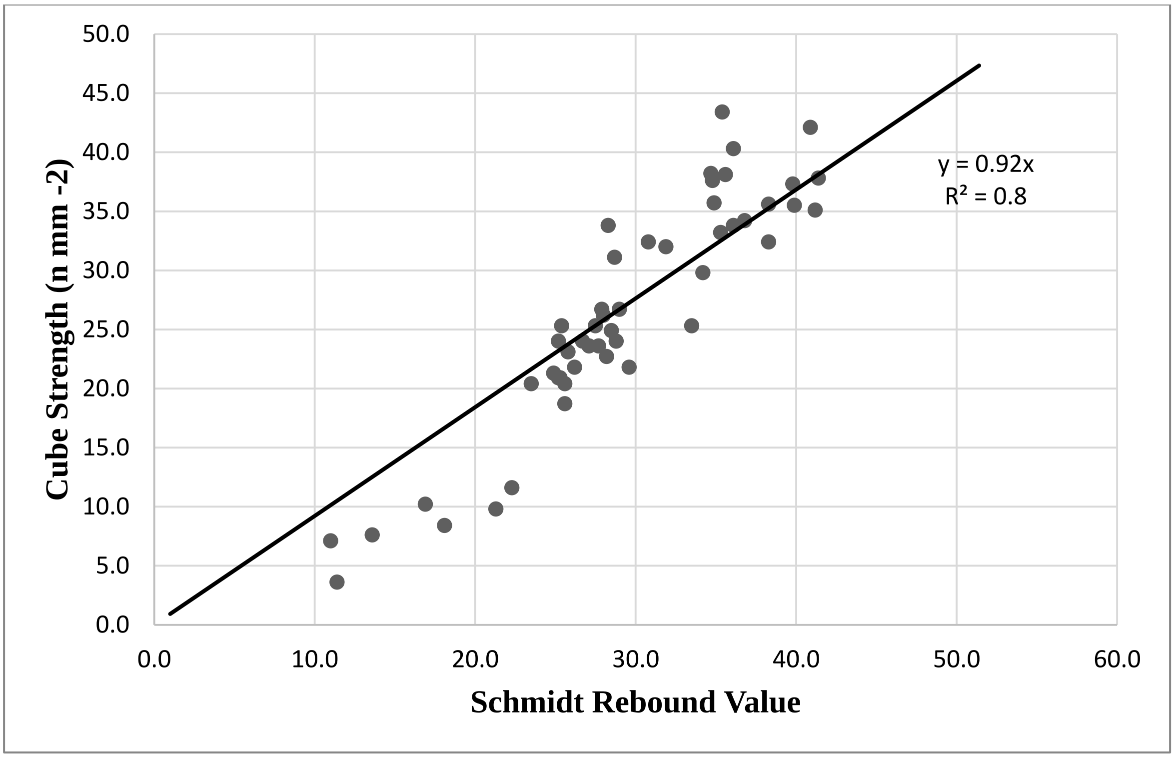

Figure 4.1: A graph of best line of fit for Schmidt rebound number against compressive strength

4.2.6 Compressive strength estimation

Equation 4.1 was used to estimate the compressive structural strength of five selected buildings. For the five storey commercial building in Westlands, it was only possible to obtain measurements from the basement, fourth and fifth floor which were unoccupied at the time of this study. The compressive strength in the basement ranged from 40.8 - 51.3 N mm^{-2} (Table 4.7), which would be the likely compressive strength to be obtained in a concrete grade 40. For columns in the same floor, it would be expected that the concrete grade used is the same. The variation in the value of the estimated compressive strength would most likely be as a result of any of the following reasons: concrete inhomogeneity, curing condition, water to cement ratio, environmental conditions, plastering thickness, among other factors. The ranges for the compressive strength measurements in the fourth and fifth floor were 29.2 - 38.1 N mm^{-2} and 28.5 - 33.0 N mm^{-2} (Table 4.7) indicating that that the floors were constructed using concrete grade 30. As one moves from the basement upwards, there is a decrease in the compressive strength of the concrete used. For instance, exceptionally high values were recorded in the basement. This is contrary to the hypothetical situation, whereby concrete of same grade is used for the entire building. However, it is worth noting that in the basement, the columns were also reinforced with steel casing since it was being used as a parking which could explain the high recorded values. Therefore, the values were could not be relied on for the purposes of the study.

Table 4.7: Estimated Compressive strength in N mm^{-2} of a five storey building in Westlands, Nairobi

Basement		4th Floor		5th floor	
Rebound Value	Estimated Compressive Strength	Rebound Value	Estimated Compressive Strength	Rebound Value	Estimated Compressive Strength
44.3	40.8	31.7	29.2	30.7	28.2
50.8	46.8	34.7	31.9	35.8	33.0
50.	46.4	34.0	31.3	28.8	27.6
50.0	46.0	36.3	33.5	32.0	29.5
50.	46.4	37.7	34.7	27.7	28.5
55.6	51.3	41.3	38.1	30.5	28.1
51.3	47.3	39.3	36.2	30.2	28.8
46.8	43.1	38.7	35.6	34.2	31.5
-	-	35.7	32.8	30.7	28.2
-	-	-	-	29.3	27.0
-	-	-	-	29.0	28.9

Note: The columns were not equally accessible in all the floors.

Table 4.8 presents the estimated compressive strength in a four storey building along Uhuru Highway. Similar to the observation in the building in Westlands, a decreasing trend in compressive strength towards the upper floors was recorded. The compressive strength in the lower ground was 40.8 N mm^{-2}, while that in the fourth floor was 21.9 N mm^{-2}. This variation may be caused by use of poor quality of materials, poor supervision and poor workmanship.

Table 4.8: Estimated Compressive strength in N mm^{-2} of a new four storey building along Uhuru highway

	Lower Ground	Ground floor	Mezzanine	First Floor	Second Floor	Third floor	Fourth Floor
	45	40	34	33	35	30	26
	43	40	45	36	34	27	24
	44	44	39	34	33	29	23
	46	42	42	35	33	26	25
	43	42	42	30	33	30	22
	45	43	41	36	30	30	23
Average R Number	44.3	41.8	40.5	34.0	33.0	28.7	23.8
Compressive Strength (N mm^{-2})	40.8	38.5	37.3	31.3	30.4	26.4	21.9

In a residential building in Eastleigh, Schmidt hammer was used to estimate the structural strength. Compressive strength of the columns and beams in the ground, fifth and eighth floor was determined. The results of the estimated strength are presented in table 4.9. In general, a uniform strength was recorded from ground to the eighth floor, where an average compressive strength of 23.7, 24.1 and 24.3 N mm^{-2}was recorded in the ground floor, fifth floor and eighth floor respectively. Slightly higher strength values were recorded for the beams at 27.3, 32.6 and 29.8 N mm^{-2} for the ground, fifth and eighth floor respectively. This could be attributed to the use of a lighter plastering coating on the beams as compared to the columns.

Table 4.9: Estimated compressive strength in a residential building in Eastleigh

	Ground floor		5th Floor		8th Floor	
	Column	Beams	Column	Beams	Column	Beams
	25	24	28	36	29	33
	26	31	28	39	24	35
	26	24	26	34	26	32
	23	32	24	36	28	32
	23	32	26	34	26	28
	30	33	28	32	25	34
	28	32	-	-	29	33
Average R-Number	25.8	29.7	26.3	34.5	26.7	32.4
Compressive Strength ($N\ mm^{-2}$)	23.7	27.3	24.1	32.6	24.3	29.8

Table 4.10 shows the estimated structural strength of a building located at Kenyatta National Hospital (KNH). This building was being renovated as it had cracked walls and leaking floors. The average compressive strength for the columns in the ground floor and first floor was determined at 29.1 and 29.4 $N\ mm^{-2}$ respectively, while at the second and the third floor, the compressive strength was estimated at 28.4 and 28.3 respectively. The data from this particular building was in complete agreement with the hypothetical situation whereby the strength of concrete is not expected to vary as one moves from one floor to another,

Table 4.10: The estimated compressive strength of a three storey building at KNH

	Ground floor	First floor	Second floor	Third floor
	28	35	28	30
	34	29	29	27
	35	30	33	27
	32	34	36	29
	37	30	32	37
	26	32	29	35
	30	34	30	31
Average R – Number	31.7	32.0	31.0	30.8
Estimated Compressive strength (N mm^{-2})	29.1	29.4	28.4	28.3

4.3 Profometer 5^+ accuracy determination

4.3.1 Rebar Diameter Measurements

Table 4.9 shows the measurements results of the Profometer 5+ for different cover depth and rebar diameter. The accuracy of the Profometer was found to vary with the cover depth and rebar diameter. The level of accuracy in the determination of the concrete cover decreased with increase in the cover depth. For instance, at 40 mm, it was possible to obtain the cover depth at an error of 2.5 % compared to an error of 17 % at a 100 mm cover for the block containing the 12 mm diameter.

Table 4.11: Measured bar cover and bar diameters from the prepared moulds.

	Bar diameter = 12 mm		Bar diameter = 16 mm		Bar diameter = 20 mm		Bar diameter = 25 mm		Bar diameter = 32 mm	
Bar cover (mm)	Observed cover (mm)	Detected bar diameter (mm)	Observed cover (mm)	Detected bar diameter (mm)	Observed cover (mm)	Detected bar diameter (mm)	Observed cover (mm)	Detected bar diameter (mm)	Observed cover (mm)	Detected bar diameter (mm)
40	41	13.2	43	16.6	39	18.8	42	26.2	38	32.8
60	59	*	57	16.9	58	20	65	24.2	61	31.4
80	76	*	78	*	78	*	85	*	80	*
100	83	*	90	*	104	*	98	*	98	*

The * indicated covermeter error message where the cover was too thick for rebar size measurements.

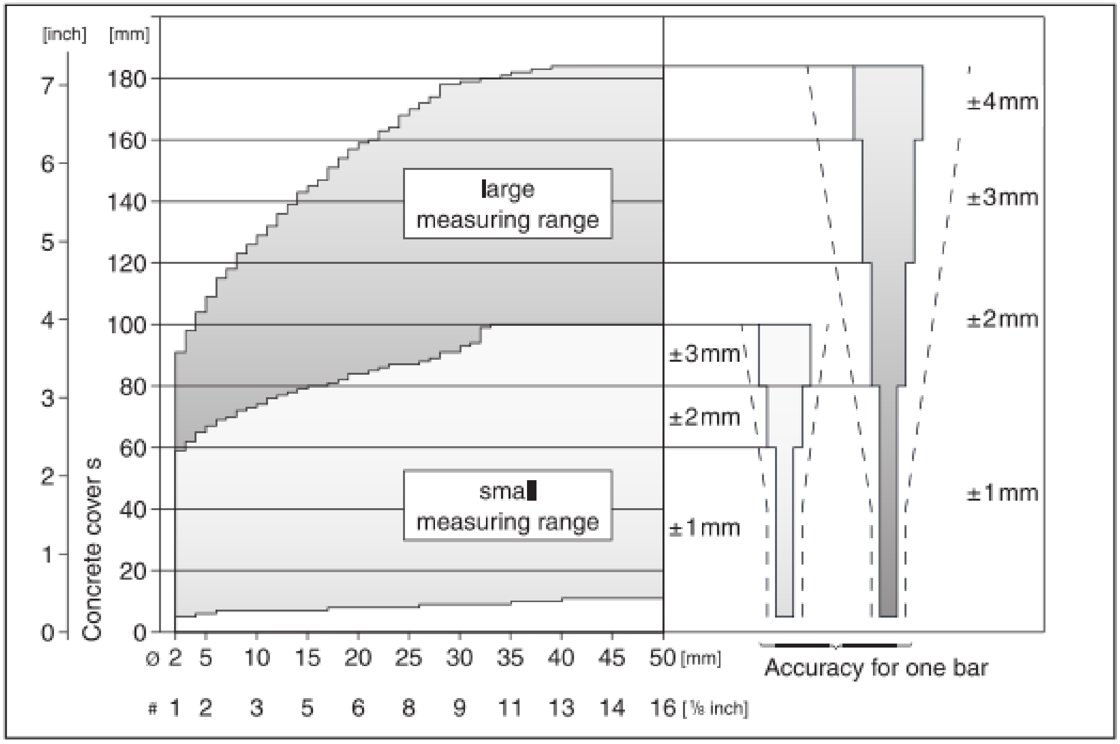

Figure 4.2: Measuring ranges and accuracy of Profometer 5 as provided by the manufacturer

For depths less than 60 mm, the covermeter was observed to be efficient in locating the rebars, and measuring their cover and diameters within an error of <10% in both cases in small measuring range. The level of accuracy in measuring the rebar diameter was best when measuring the specimen casted with the 12 mm diameter reinforcement bar. For the subject diameter, it was not possible to measure the size of the rebar at ≥ 60 mm cover depth. The accuracy was found to decrease with increase in depth of the cover and it was not possible to measure the rebar diameter at 80 and 100 mm. The manufacturer chart of accuracy (Figure 4.2) indicates that it is possible to get bar diameters for 32 mm rebar at cover depth of 180 mm in the long measuring range which was not possible in this work.

The covermeter was then used to make field measurements (Table 4.11) for both cover and rebar sizes in a commercial building in Westlands area. Field measurements were a bit challenging because of the existence of bars which are close to each other. This was overcome by identifying and taking measurements in areas on the columns where the reinforcement bars were not closely spaced. In addition, failure to position the search probe in an orientation parallel to the reinforcement gave a bigger error in the rebar diameter measurement. It was observed that the size of reinforcement used in the basement during construction was 32 mm and that of fourth and the fifth floor was 16 mm for the columns measured.

Table 4.12: Diameters of rebars and their cover from a five storey building in Westlands, Nairobi

Basement			Fourth floor			Fifth floor		
Column Code	Rebar Diameter (mm)	Measured cover	Column Code	Rebar Diameter (mm)	Measured cover	Column Code	Rebar Diameter (mm)	Measured cover
C18	31.0	46	C18	16.7	40	C18	17.3	52
C21	32	33	C26	17	46	C26	17.7	49
C22	32.3	59	C28	17	36	C28	17	47
C32	30.3	31	C35	16.7	46	C31	16.3	48

Table 4.13 shows the estimated rebar diameter and cover in a residential building in Eastleigh, in the three sampled floors i.e. ground floor, fifth floor and eighth floor. The average size and cover of the rebar in the columns on the Ground Floor is 21.7-23.7 mm (Y20) and 41.3 - 51.9 mm respectively. These results correspond to five columns accessible in the ground floor during the exercise. In the fifth floor, only three columns were accessible. On average, the rebar size and cover for the columns was in the range of 20.2 - 23.4 mm (Y20) and 38.6 - 57.3 mm respectively. Finally, an average of 19.9mm (Y20) and 26.5 mm for rebar size and cover respectively, was recorded for one column in the Eighth Floor. Therefore, from these measurements, we can conclude that a 20mm (Y20) rebar, was used for the columns in the entire building.

Table 4.13: Estimated rebar diameter and cover in a residential building in Eastleigh

	Ground floor			Fifth floor			Eighth Floor	
Column	Rebar size (mm)	Cover (mm)	Column	Rebar size (mm)	Cover (mm)	Column	Rebar size (mm)	Cover (mm)
B	23.4	51.9	C	20.6	57.3	F	19.9	26.8
D	23.7	37.9	D	20.2	52.7			
E	23.5	45.1	F	23.4	38.6			
F	21.7	32.2						
G	25.1	41.3						

In a three storey residential building in Kasarani, the size and cover of the reinforcement bars were measured in the first, second and third floor. It is observed from the results (Table 4.13) that 16 mm (Y16) rebars were used in the columns in all the floors. The average cover of the rebar for the measured columns was found to be 45 mm, 34 mm and 45 mm for the first, second and third floor respectively. The difference in cover depth measurements could either be caused by movement of the reinforcement during casting due to poor placement or by variation in plaster thickness.

Table 4.14: Estimated rebar diameter and cover in a residential building in Kasarani

	First	floor		Second	floor		Third	Floor
Column code	Rebar Size	cover	Column Code	Rebar Size	cover	Column Code	Rebar Size	Size cover
C11	18.5	40	C11	17.9	33	C11	17.7	51
C12	19.6	41	C12	16.5	36	C12	17.4	41
C13	17.9	47	C13	16.7	36	C13	12.9	52
C14	17.4	47	C14	18.4	38	C14	12.7	47
C15	16.8	52	C15	18.6	35	C15	23.4	38
Average	**17.9**	**45**		**17.2**	**35**		**16.8**	**45**

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusion

A study, using non-destructive methods, has been achieved highlighting the applicability of the Schmidt rebound hammer and Profometer 5+ covermeter in the assessment the performance of reinforced concrete structures. It was established that despite the high inhomogeneity in concrete, there exist a correlation between the rebound number and the compressive strength. An equation showing the relationship between Schmidt hammer rebound index and compressive strength of concrete structures was developed and used to estimate the compressive strength of existing concrete structures from rebound values obtained from the Schmidt hammer.

The Schmidt hammer produced results that are reliable and as such, the resulting regression model for strength evaluation could be used for concrete strength estimation in engineering investigations. This would significantly reduce the number of cores to be obtained from the structures thus reducing the amount of damage caused by the removal of these cores.

The Profometer was able to locate the exact position of the bar with a high level of accuracy. The equipment was also able to measure the size and cover of the reinforcement provided the reinforcement was located at 60 mm or less from the measuring surface.

The Schmidt hammer and covermeter proved to be versatile instruments for assessment of concrete strength in structural development and may assist the Kenyan contractors, planners and safety enforcement institutions in ascertaining the structures were built according to specifications. It can also help in monitoring quality deterioration of concrete under environmental stress.

5.2 Recommendation

From the measurement taken, it was evident that some structures had deviation from what is expected. For instance in the Schmidt hammer measurements taken from a new building along Uhuru highway, there was a decrease in compressive strength as one moved up the floors. This might have been caused by any of the following reasons: poor quality of materials, poor supervision and poor workmanship. More research work need to be carried out to identify the exact cause of this reduction in strength.

There is also a need for the development of procedure and regulations for regular inspections concrete structures using non-destructive methods before, during and after constructions in order to prevent unnecessary injury or death caused by failure of these structures. This can help the government and contractors to save money, material, time and life as the country endeavors to achieve the vision 2030.

While carrying out the Schmidt hammer measurements in existing structures, there were no core specimens which were obtained from the structures assessed. It would be important to compare the results of the cores that were obtained from the Schmidt hammer test to see if there is any significant variation of the estimated compressive strength from the actual values obtained from crushing the cores.

I recommend that more work need to be done in order to determine ways to improve the efficiency of Profometer 5+. In addition, the effect of close parallel bars on the accuracy of the Profometer should be investigated.

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Appendices

Appendix 1: Data for various classes of concrete at different age.

CONCRETE GRADE: C35 CONCRETE AGE: 28 DAYS Date of Testing: 01 September 2014 Date of Casting: 02 August 2014						
	CUBE 1	CUBE 2	CUBE 3	CUBE 4	CUBE 5	CUBE 6
	R NO.	R NO.	R NO.	R NO.	R NO.	R.NO
	36	30	36	33	37	38
	35	28	39	34	36	37
	40	36	33	35	34	33
	35	35	31	36	36	38
	35	30	38	34	34	34
	32	40	41	34	34	41
	36	41	38	35	34	38
	35	40	34	34	36	34
	28	41	35	34	37	42
	34	40	34	32	33	38
	40	36	28	36	28	39
	38	34	38	35	36	36
	37	36	34	35	37	30
	33	37	34	33	37	28
	33	40	38	39	40	40
	37	35	34	37	34	29
	33	37	32	33	39	34
	38	30	33	39	39	38
	32	32	36	38	34	36
	36	35	36	37	42	32
	35	35	36	31	34	36
	38	30	38	32	40	30
	28	36	39	36	38	38
	32	36	31	35	37	36
AVG R NO.	34.8	35.4	35.3	34.9	36.1	35.6
MAX LOAD (KN)	847	977	746	803	906	857
MAX LOAD (N/MM2)	37.6	43.4	33.2	35.7	40.3	38.1
WEIGHT (KG)	7.4	7.6	8	8	8.4	8.2
DENSITY (KG M-3)	2193	2252	2370	2370	2489	2430

CONCRETE GRADE: C35 CONCRETE AGE: 14 DAYS Date of Testing: 17 August 2014 Date of Casting: 02 August 2014			
	CUBE 1 R NUMBER	CUBE 2 R NUMBER	CUBE 3 R NUMBER
	27	32	34
	30	32	30
	26	31	28
	29	27	26
	27	26	20
	30	22	30
	33	28	26
	29	26	28
	28	30	30
	30	28	33
	25	30	28
	30	27	26
	32	26	30
	30	29	28
	30	29	30
	27	26	30
	26	26	28
	25	28	30
	29	26	33
	28	30	28
	30	28	26
	25	30	30
	30	27	28
	27	26	30
AVG	28.5	27.9	28.8
MAX LOAD (KN)	560	600	540
MAX LOAD (N/MM2)	24.9	26.7	24.0
WEIGHT (KG)	8.5	8.7	8.7
DENSITY (KG M-3)	2519	2578	2578

CONCRETE GRADE: C35 CONCRETE AGE: 7 DAYS Date of Testing: 11 August 2014 Date of Casting: 02 August 2014			
	CUBE 1 R NUMBER	CUBE 2 R NUMBER	CUBE 3 R NUMBER
	26	24	28
	26	27	24
	26	24	28
	27	30	26
	25	26	26
	27	26	26
	27	26	22
	26	28	22
	26	26	20
	27	20	27
	22	28	29
	20	22	26
	26	30	25
	25	24	22
	27	27	22
	25	23	30
	24	26	26
	28	28	24
	27	26	27
	27	20	29
	25	28	26
	27	22	25
	27	30	22
	26	24	22
AVG	25.8	25.6	25.2
MAX LOAD KN	520	460	470
LOAD (N/MM2)	23.1	20.4	20.9
WEIGHT	8	8.8	8.5
DENSITY (KG M-3)	2370	2607	2519

CONCRETE GRADE: C30
CONCRETE AGE: 28 DAYS
Date of Testing: 30 August 2014
Date of Casting: 01 Aug 2014

	CUBE 1	CUBE 2	CUBE 3	CUBE 4	CUBE 5	CUBE 6
	R NO.	R NO.	R NO.	R NUMBER	R NUMBER	R NUMBER
	40	45	43	39	41	44
	40	38	42	46	47	44
	45	35	36	44	40	42
	44	30	34	40	43	38
	37	36	28	46	46	43
	37	37	35	42	49	40
	39	39	43	45	47	41
	36	35	37	45	44	44
	43	28	30	42	44	38
	45	36	39	35	36	42
	38	38	42	43	42	40
	38	39	39	39	41	44
	33	33	39	43	40	44
	36	37	42	40	40	40
	42	33	36	42	42	41
	43	40	40	45	36	37
	43	41	32	45	38	38
	34	41	42	45	39	36
	42	42	32	39	32	44
	42	40	36	40	42	44
	42	36	46	32	42	40
	38	30	42	42	42	38
	36	36	38	34	36	45
	42	38	45	40	32	42
AVG	39.8	36.8	38.3	41.4	40.9	41.2
MAX LOAD (KN)	840	770	800	850	947	790
MAX LOAD (N/MM2)	37.3	34.2	35.6	37.8	42.1	35.1
WEIGHT (KG)	8.4	8.4	8.4	8	8.4	8
DENSITY (KG M-3)	2489	2489	2489	2370	2489	2370

CONCRETE GRADE: C30 CONCRETE AGE: 7 DAYS Date of Testing: 08 August 2014 Date of Casting: 01 Aug 2014			
	CUBE 1 R NUMBER	CUBE 2 R NUMBER	CUBE 3 R NUMBER
	30	24	30
	20	28	30
	28	29	24
	24	25	26
	30	29	30
	29	24	28
	25	28	30
	30	26	32
	26	20	26
	24	23	26
	26	22	28
	27	22	24
	27	26	24
	29	22	30
	30	24	26
	20	28	26
	28	24	28
	24	26	31
	30	29	30
	29	20	29
	25	25	26
	30	27	31
	26	31	24
	24	25	26
AVG	26.7	25.3	27.7
MAX LOAD KN	540	470	530
LOAD IN N/MM2	24.0	20.9	23.6
WEIGHT (KG)	8	8.2	8
DENSITY (KG M-3)	2370	2430	2370

CONCRETE GRADE: C25 CONCRETE AGE: 28 DAYS Date of Testing: 25 August 2014 Date of Casting: 23 July 2014						
	CUBE 1	CUBE 2	CUBE 3	CUBE 4	CUBE 5	CUBE 6
	R NUMBER	R NUMBER	R NUMBER	R NUMBER	R NUMBER	R NUMBER
	27	34	38	32	36	38
	37	31	38	30	32	40
	39	35	30	37	35	38
	39	26	40	30	32	30
	35	35	38	26	36	34
	36	34	30	30	36	38
	32	30	30	35	30	38
	33	27	36	26	38	36
	36	26	32	30	36	32
	30	30	35	32	37	38
	34	30	40	36	32	32
	30	34	38	30	36	38
	34	36	34	30	36	38
	34	31	36	27	38	38
	36	35	32	32	36	36
	34	35	30	30	36	30
	30	36	34	35	34	40
	30	28	34	26	33	36
	34	35	34	30	30	36
	34	26	36	30	32	34
	36	30	32	28	37	35
	34	30	30	34	32	38
	30	35	30	32	36	35
	30	36	34	30	36	38
AVG	33.5	31.9	34.2	30.8	34.7	36.1
MAX LOAD (KN)	570	720	670	730	860	760
MAX LOAD (N/MM2)	25.3	32.0	29.8	32.4	38.2	33.8
WEIGHT (KG)	7.5	8.4	7.7	7.6	7.2	8
DENSITY (KG M-3)	2222	2489	2281	2252	2133	2370

CONCRETE GRADE: C25 CONCRETE AGE: 14 DAYS Date of Testing: 07 August 2014 Date of Casting: 23 July 2014			
	CUBE 1 R NUMBER	CUBE 2 R NUMBER	CUBE 3 R NUMBER
	30	32	30
	31	31	28
	23	23	20
	30	30	20
	30	22	30
	32	30	32
	31	28	28
	26	35	33
	27	35	30
	30	26	28
	30	26	35
	23	30	30
	25	25	30
	23	33	32
	30	30	28
	30	28	33
	32	35	30
	27	26	28
	30	33	30
	25	30	28
	23	33	35
	30	30	30
	30	28	30
	32	35	32
	27	26	28
AVG	28.3	29.6	29.5
MAX LOAD (KN)	760	600	700
MAX LOAD (N/MM2)	33.8	26.7	31.1
WEIGHT	8.2	8.4	8
DENSITY (KG M-3)	2430	2489	2370

CONCRETE GRADE: C25 CONCRETE AGE: 7 DAYS Date of Testing: 29 July 2014 Date of Casting: 23 July 2014					
	CUBE 1 R NUMBER	CUBE 2 R NUMBER	CUBE 3 R NUMBER	CUBE 4 R NUMBER	CUBE 5 R NUMBER
	28	22	30	34	39
	22	30	20	33	41
	27	25	26	40	39
	28	22	28	40	33
	27	32	26	35	40
	27	30	23	40	42
	28	21	22	38	34
	29	24	30	41	38
	30	18	28	42	32
	27	28	28	40	41
	24	32	26	45	41
	29	30	24	44	40
	30	34	26	37	38
	30	23	28	42	40
	25	30	27	39	36
	30	25	24	42	36
	26	22	23	40	34
	26	32	20	43	43
	26	24	27	42	38
	32	18	26	42	36
	36	28	22	38	40
	28	23	26	41	38
	28	30	20	40	42
	28	25	30		
AVG	28.0	26.2	25.4	39.9	38.3
MAX LOAD (KN)	590	490	570	500	730
MAX LOAD (N/MM2)	26.2	21.8	25.3	22.2	32.4
WEIGHT (KG)	8.1	7.9	8.4	8.7	8.2
DENSITY (KG M-3)	2400	2341	2489	2578	2430

CONCRETE GRADE: C20 CONCRETE AGE: 28 DAYS Date of Testing: 19 August 2014 Date of Casting: 22 July 2014			
	CUBE 1 R NUMBER	CUBE 2 R NUMBER	CUBE 3 R NUMBER
	30	30	30
	30	28	22
	30	28	26
	24	26	30
	26	35	28
	26	28	30
	26	26	20
	25	24	20
	28	30	26
	29	26	28
	20	30	28
	22	22	24
	22	30	32
	27	36	28
	22	22	26
	24	30	26
	24	26	26
	24	26	34
	24	28	26
	26	22	28
	26	28	28
	22	26	24
	24	28	32
	24	26	28
AVG	25.2	27.5	27.1
MAX LOAD (KN)	540	570	530
MAX LOAD (N/MM2)	24.0	25.3	23.6
WEIGHT (KG)	8	8	8.2
DENSITY (KG M-3)	2370	2370	2430

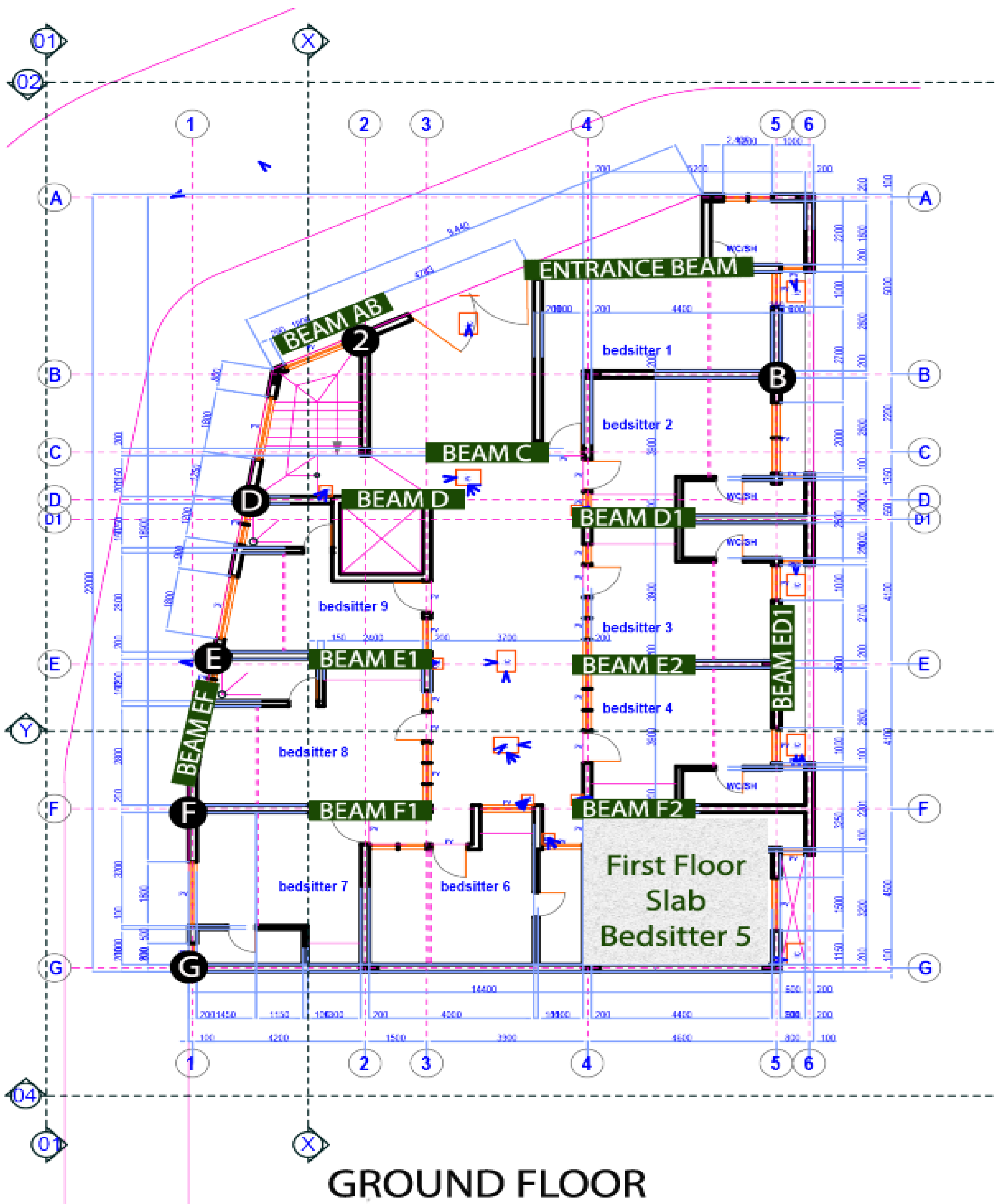
CONCRETE GRADE: C20 CONCRETE AGE: 14 DAYS Date of Testing: 06 August 2014 Date of Casting: 22 July 2014			
	CUBE 1 R NUMBER	CUBE 2 R NUMBER	CUBE 3 R NUMBER
	28	29	22
	24	24	22
	26	22	23
	28	25	27
	20	28	22
	22	27	24
	22	23	25
	28	20	22
	24	33	23
	24	26	24
	22	23	25
	30	32	26
	22	25	26
	29	24	24
	24	27	22
	26	29	20
	28	24	26
	25	24	26
	26	24	22
	25	23	24
	30	20	25
	26	23	20
	30	20	24
	25	22	20
	30	24	24
	28	20	22
AVG	25.6	24.9	23.5
MAX LOAD	420	480	460
MAX LOAD (N/MM2)	18.7	21.3	20.4
WEIGHT (KG)	8.4	8.8	8.1
DENSITY (KG M-3)	2489	2607	2400

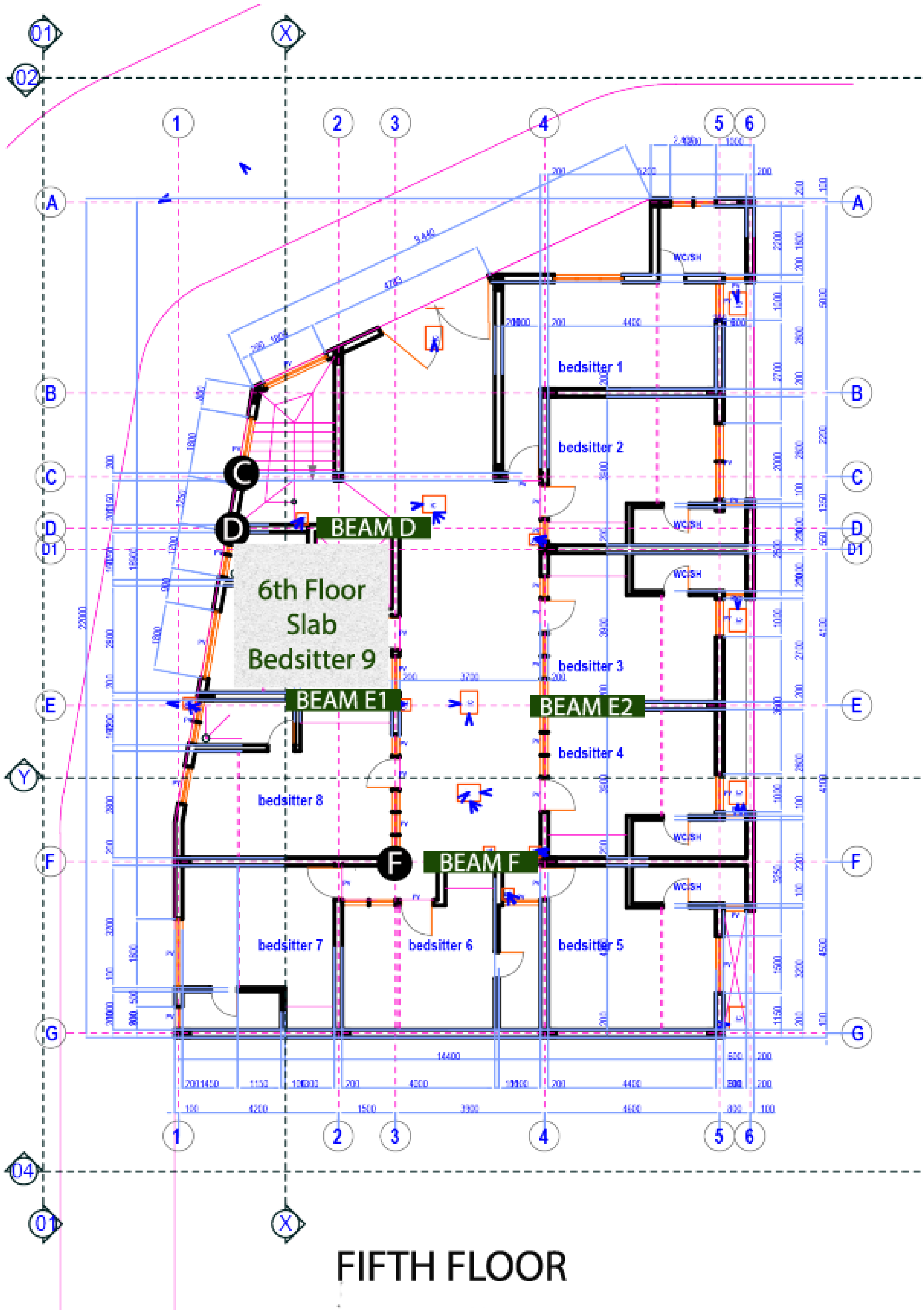
CONCRETE GRADE: C 15 CONCRETE AGE: 28 DAYS Date of Testing: 26 August 2014 Date of Casting: 24 July 2014					
	CUBE 1 R NUMBER	CUBE 2 R NUMBER	CUBE 3 R NUMBER	CUBE 4 R NUMBER	CUBE 5 R NUMBER
	25	21	16	28	27
	18	24	16	28	29
	30	22	16	32	25
	20	24	18	27	22
	22	23	25	32	30
	22	25	20	26	20
	18	23	26	29	32
	20	20	18	34	24
	25	18	16	30	33
	26	15	18	27	30
	28	16	20	27	24
	22	25	20	32	30
	20	23	16	30	38
	20	20	16	30	30
	23	18	18	30	29
	27	15	16	30	23
	24	22	18	32	33
	20	24	20	24	20
	18	23	22	30	30
	16	25	16	32	28
	22	23	16	28	38
	26	20	14	32	30
	24	18	18	30	29
	20	25	16	30	23
AVG	22.3	21.3	18.1	29.6	28.2
MAX LOAD (KN)	260	220	190	490	510
MAX LOAD (N/MM2)	11.6	9.8	8.4	21.8	22.7
WEIGHT (KG)	8.4	8.6	8.2	8.6	8.2
DENSITY (KG M-3)	2489	2548	2430	2548	2430

CONCRETE GRADE: C15 CONCRETE AGE: 14 DAYS Date of Testing: 08 August 2014 Date of Casting: 24 July 2014			
	CUBE 1 R NUMBER	CUBE 2 R NUMBER	CUBE 3 R NUMBER
	15	22	10
	14	16	12
	15	16	12
	14	16	10
	14	16	11
	13	14	10
	10	20	10
	13	18	10
	13	15	10
	17	12	12
	15	18	12
	12	17	10
	14	18	12
	13	15	12
	10	22	10
	17	18	10
	12	12	10
	14	17	12
	13	18	12
	10	15	10
	17	22	12
	15	18	10
	12	12	12
	15	18	12
AVG	13.6	16.9	11.0
MAX LOAD (KN)	170	230	160
MAX LOAD (N/MM2)	7.6	10.2	7.1
WEIGHT (KG)	7.5	8.2	7.9
DENSITY (KG M-3)	2222	2430	2341

CONCRETE GRADE: C15 CONCRETE AGE: 7 DAYS Date of Testing: 31 July 2014 Date of Casting: 24 July 2014	
	CUBE 1
	R NUMBER
	16 12 13 10 10 12 10 10 17 11 16 11 10 12 12 10 10 10 10 10 10 10 10
AVG	11.3913
MAX LOAD (KN)	80
MAX LOAD (N/MM2)	3.6
WEIGHT (KG)	8
DENSITY	2370.37

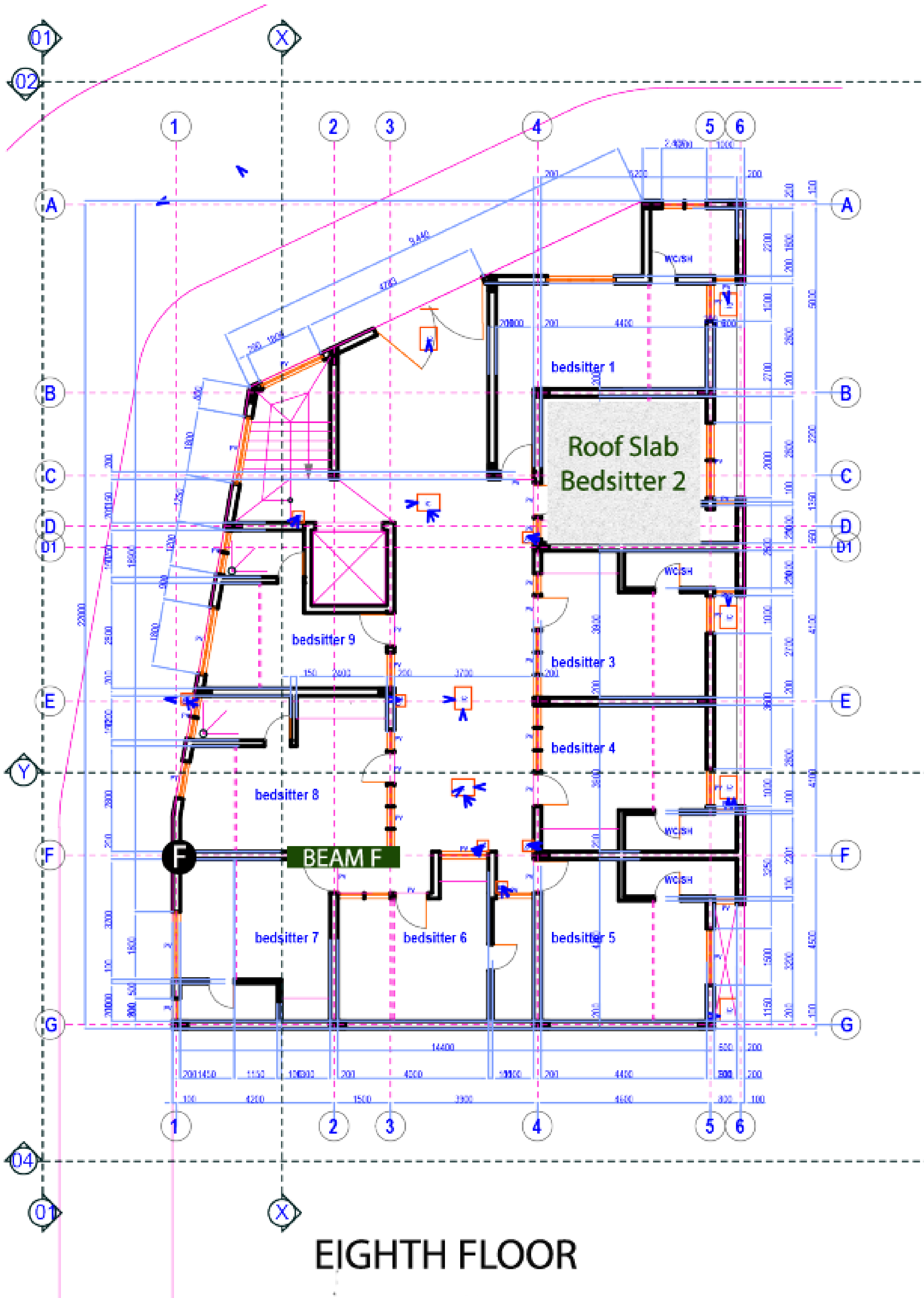
Appendix 2: Architectural drawings of the sampled floors in a residential building in Eastleigh, showing the sampled columns.





6th Floor
Slab
Bedsitter 9
BEAM D
BEAM E1
BEAM E2
BEAM F
bedsitter 1
bedsitter 2
bedsitter 3
bedsitter 4
bedsitter 5
bedsitter 6
bedsitter 7
bedsitter 8
WC/SH

FIFTH FLOOR



EIGHTH FLOOR