

**Laterized Quarry Dust and Recycled Concrete as Alternative Building  
Materials**

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Civil Engineering in the Jomo Kenyatta University of Agriculture and  
Technology**

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## DECLARATION

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

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## **DEDICATION**

This research work is dedicated to my lovely wife Christine and our charming  
daughter Bright

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## LIST OF ABBREVIATIONS

|             |   |
|-------------|---|
| <b>BS</b>   | British standards                               |
| <b>ECCO</b> | Environmental Council of Concrete Organizations |
| <b>QD</b>   | Quarry dust                                     |
| <b>NCA</b>  | Natural crushed aggregates                      |
| <b>RCA</b>  | Recycled concrete aggregates                    |
| <b>RS</b>   | River sand                                      |

## ABSTRACT

The use of conventional materials is facing two main challenges of high cost and large-scale depletion of the sources thus creating environmental problems. These challenges demand that alternative materials be explored that are not only affordable but are also environmentally friendly. In this regard, laterized quarry dust and recycled concrete are proposed as possible alternative building material.

To date, extensive studies have been done on laterite, quarry dust and recycled aggregates, separately. However, there is lack of data on performance of blended materials as well as large scale tests on structural elements made from the alternative materials. This research therefore seeks to assess the performance of blended alternative materials when used in concrete and blocks. In the study, samples of materials were investigated to determine basic properties following which the optimum proportions of ingredient materials were determined for concrete and block mix. Finally, the viability of using the materials was assessed for large scale concrete beams and wall panels.

The results demonstrate that there is great potential of laterized quarry dust concrete and blocks. When recycled concrete aggregates are used, 30% of river sand can be replaced with laterized quarry dust and still attain compressive strength of 20 N/mm<sup>2</sup>. In addition, the flexural capacities of alternative as well as conventional concrete beams were found to be within 5%. The findings are intended to contribute to sustainable construction of low cost housing and development.

## CHAPTER ONE

### 1.0 INTRODUCTION

#### **1.1 Background**

In Kenya, good quality concrete is manufactured from natural river sand and natural crushed aggregates satisfying the grading requirements of British standards or Kenya standards (Arai, 1986). River sand is harvested from river beds while the coarse aggregate is obtained by crushing natural rock in quarries. The manufacture of conventional concrete is governed by the Kenya or British standards. The availability of the conventional aggregates for concrete manufacture has become scarce in some areas due to; limited occurrence brought by over-exploitation in harvesting, excessive cost of the material on the market, reduced supply due to increasing demand and transportation difficulties due to the poor state of the roads (Mostafa, 1990).

Besides concrete construction, most of the housing construction in Kenya is done in masonry. The main materials used are; burnt ceramic bricks, concrete blocks and machine dressed natural stones. The use of burnt ceramic bricks is increasingly discouraged due to the high cost of energy requirements and negative environmental impacts such as air pollution. The use of concrete blocks as well as dressed natural stones faces similar challenges as the conventional concrete.

There is also growing emphasis on sustainability in construction industry. This has led to a paradigm shift in the building industry with emphasis on sustainable construction. Sustainability is defined as the 'need to ensure that development meets

the needs of the present without compromising on the ability of the future generations to meet their own needs' (Brundtland, 1987). Sustainability has three main dimensions namely; ecological sustainability, economic sustainability and social sustainability. Ecological sustainability in construction is achieved when the materials used produce low emissions, pollutants and have both recycling economy and resource efficiency. Economic sustainability is ensured by optimisation in low cost construction techniques, maintenance of the engineering systems and construction techniques which have low life-cycle cost. Social sustainability on the other hand focuses on social satisfaction of the construction from material harvesting, production and final structure. Most of the conventional construction materials such as steel and concrete have low rating as far as sustainability is concerned when compared to the traditional earth based construction material.

Emission of toxic substances and energy requirements of construction material during manufacture has been used as a measure of ecologic sustainability of materials. Of importance are the carbon dioxide and sulphur dioxide gases that are emitted by materials during manufacture. Table 1.1 shows the equivalent gases emitted during the manufacture of some of the most common construction materials.

**Table 1.1:** Gaseous emission of various construction materials

| <b>Material</b>  | <b>Equivalent carbon dioxide emission gCO<sub>2</sub>/kg</b> | <b>Equivalent sulphur dioxide emission gSO<sub>2</sub>/kg</b> |
|------------------|--|---|
| Aluminium        | 11800  | 95  |
| Steel (profiles) | 2540   | 11.2  |
| Steel galvanized | 4080   | 21  |
| Paint synthetics | 1700   | 6   |
| PVC              | 2050   | 14.3  |
| Cement           | 1000   | -   |
| Clay tile fired  | 295  | 2.3   |
| Concrete         | 123  | 0.4   |
| Brick            | 225  | 0.8   |
| Concrete tile    | 220  | 0.7   |
| Wood             | 774  | 1.6   |

It is seen from the table that aluminium produces the highest amount of carbon dioxide. This is followed by galvanized steel. Concrete production produces the least amount of gaseous emissions. Earth-based materials generally have lower emission values as compared to the conventional concrete production. In Table 1.2, energy requirements for construction material during manufacture have been summarised. Here again aluminium and steel have the highest energy requirements for their manufacture. Stabilized earth, natural stone and aggregates have the lowest energy requirements. Stabilized earth requires between 0.3 and 0.8MJ of energy for every kg produced. Timber has lower energy requirements of between 0.1 and 5MJ/kg. However, since cutting down of trees is detrimental to the environment, the use of timber in many countries is discouraged.

**Table 1.2:** Energy requirements of various construction materials

| <b>Material</b>                           | <b>Energy requirements MJ/kg</b> |
|---|----------------------------------|
| Aluminium                                 | 130-270                          |
| Steel                                     | 20-60                            |
| Steel galvanized                          | >60                              |
| PVC                                       | 50-90                            |
| Concrete-in-situ                          | 0.6-2                            |
| Cement                                    | 4-8                              |
| Fibre cement                              | Average 4.8                      |
| Lime                                      | 3-10                             |
| Lime/cement mortar                        | 0.5-1                            |
| Stabilised earth                          | 0.3-0.8                          |
| Sand-lime bricks                          | 0.7-1.2                          |
| Clay bricks                               | Average 2.5                      |
| Concrete blocks and tiles                 | 0.9-1.6                          |
| Natural stone, sand, aggregates, soil     | <0.3                             |
| Ceramics, bricks and tiles                | 1.5-8                            |
| Corrugated galvanized steel sheets, 0.4mm | 60                               |
| Ceramic roofing tiles, 12mm               | 4                                |
| Concrete roofing tiles, 12mm              | 1.3                              |
| Micro concrete roofing tiles, 8mm         | 1.3                              |
| Gypsum plaster                            | 1-4                              |
| Timber                                    | 0.1-5                            |

Due to the numerous challenges and shortcomings of conventional construction materials, the search for alternative sustainable construction material has been seriously considered by several researchers. In Kenya, research work has been done on several local materials in order to establish their potential use in construction. For example; quarry dust (Kioko, 1996; Mulu *et al.*, 1998), low cost masonry blocks (Oyawa, 2004; Karanja, 2003). On the global scale, the same trend has been experienced particularly in India, Malaysia and Nigeria. In India, more research is being carried out on quarry dust also called manufactured sand (Babu *et al.*, 1997; Nagaraj *et al.*, 1996; Narasimahan *et al.*, 1999). In Nigeria there is growing interest in researching on the possible use of laterite, a common and abundant material in

African continent, in concrete production as river sand substitute. Results obtained so far are encouraging.

Although several research activities have been carried out on sustainable material such as wastes and recycled material, the necessary information that can aid in design with the material is still limited. In addition, most of the waste and recycled material have been found to possess some undesirable engineering properties that hinder their application in practice. No data is available on the effects of blending of the materials which could modify and improve some of the undesirable properties of the materials when applied separately. This research aims at resolving some of these issues by evaluating the potential of laterite soil, quarry dust and recycled concrete as supplementary or complementary materials for eco-concrete production.

## **1.2 Problem statement and justification**

Conventional building materials like sand and crushed aggregates are expensive due to excessive cost of extraction and transportation from natural sources. Moreover large-scale extraction of these sources creates environmental problems in form of resource depletion and toxic emissions. As environmental issues, transportation challenges and other constraints make the availability and use of conventional material less attractive, alternative supplementary or complementary materials need to be found. Presently in Kenya, river sand poses the problem of acute shortage in many areas, leading to serious problems with respect to its availability, cost and environmental impact. In such a situation lateritized quarry dust is foreseen to be an economic alternative to the river sand. Laterite soil is readily available in Kenya

while quarry sites have heaps of quarry dust, a waste product in the production of crushed coarse aggregates.

Another material that could be investigated as a supplementary material is recycled concrete derived from demolition of old buildings. Many of the existing buildings in most of the towns have been in existence for many years. The value of land has increased a lot favouring high rise structures to meet the high demand for space. Furthermore there is growing demand for change of use of the existing buildings. Due to these reasons many buildings are already being demolished. Some demolition of buildings, by the government, previously built on land set aside for road expansion has increased waste material specifically concrete which pose disposal challenges. It is expected that demolitions will increase as demand for new facilities that meet the current society's needs also increase. Recycling has many advantages. In this time of increasing attention to the environmental impact and sustainable development, recycled Portland cement concrete has much to offer: it is resource efficient thus minimizing depletion of our natural resources; it is energy efficient in all phases of production.

In summary and in view of prevailing environmental, economic and social challenges in the construction industry, this research aims at studying the performance of recycled concrete aggregates and laterized quarry dust in concrete and masonry. It is believed that blending these materials together in concrete will also combine their advantages in the resulting applications.

### **1.3 Objectives of the study**

#### ***General objective:***

The main objective is to study the performance of recycled concrete and laterized quarry dust as aggregates in concrete and filler in masonry for sustainable construction.

#### ***Specific objectives:***

1. To determine and compare the main material properties of the alternative and conventional materials.
2. To evaluate concrete mixes and hence determine the optimum proportions of constituent materials in alternative sustainable concrete.
3. To evaluate building block mixes and hence determine the optimum proportions of constituent materials in alternative sustainable blocks.
4. To determine the structural behavior in bending, under static loading, of reinforced concrete beams made from the alternative sustainable concrete viz-a-viz conventional concrete.
5. To establish the comparative structural behavior, under static axial compression, of masonry wall panels made from the alternative blocks and conventional machine dressed blocks.

## CHAPTER TWO

### 2.0 LITERATURE REVIEW

#### **2.1 Introduction**

Research work has generally been carried out on all the proposed alternative materials with a view of utilising the materials in construction. Studies on the alternative materials have however been done in different parts of the world. For instance, much of the data on laterized concrete is as a result of studies that have been carried out by researchers in Nigeria (Udoeyo *et al.*, 2006; Osunade *et al.*, 1994; Ata *et al.*, 2005) while studies on quarry dust have been extensively done in India and Australia (Babu *et al.*, 1997; Nagaraj *et al.*, 1996; Narasimahan *et al.*, 1999). Up to date, there are generally no guidelines for practical application of the proposed alternative materials.

#### **2.2 Quarry wastes**

In Kenya, there are two common types of quarry dust wastes; one from natural aggregate crushing quarries (QDNbi) and the other from mechanical stone dressing quarries (QDNdar). Quarry dust is defined as a residue, tailing or other non-voluble waste material after the extraction and processing of rock to form fine particles less than 4.75mm (Ilangovan *et al.*, 2007). The utilization of quarry dust has been accepted as a building material in the industrially advanced countries of the west for the past three decades (Nisnevich *et al.*, 2007). This is as a result of sustained research and development work undertaken with respect to increasing application of this industrial waste. The level of utilization of quarry dust in industrialized nations

like Australia, France, Germany and UK has reached more than 60% of its total production (Ilangovan *et al.*, 2007). The use of quarry dust in Kenya has not been much as compared to some advanced countries due to lack of sufficient experimental data and guidelines on utilisation of the material in construction. Usually, quarry dust is used in large scale in highways as a surfacing material and in manufacturing the hollow blocks and lightweight prefabricated elements.

Previous work by Mulu *et al* (1998) indicates that crushed rock quarry dust has physical properties comparable to natural sand and hence the desirable properties needed for the concrete manufacture except for its fineness which requires high cement content. Ilangovan *et al* (2007) investigated the performance of quarry dust in concrete. In their study, concrete with 100% replacement of river sand was prepared and used to cast test cubes and concrete beams that were tested for compressive strength, tensile strength and durability behaviour. The results showed that the compressive, flexural strength and durability studies of concrete made of Quarry Rock Dust were nearly 10% more than the conventional concrete.

Raman *et al* (2007) carried out non-destructive evaluation of flowing concrete incorporating quarry dust. In their research, flowing concrete was produced using quarry dust as partial replacement of natural mining sand. Non-destructive properties such as dynamic modulus of elasticity, ultrasonic pulse velocity and initial surface absorption were investigated. The slump, slump flow, V-funnel flow, air content and compressive strength were also observed. The results indicated that

quarry dust does not significantly affect the non-destructive properties of concrete except the initial surface absorption.

The characteristics of quarry dust as a low cost construction material in Kenya was investigated by Mulu *et al* (2003). The physical properties and grading characteristics of typical quarry dust were investigated and compared with those of natural river sand. The engineering properties of hardened concrete samples were also determined. The results of this study indicated that, generally an increase in quarry dust content in concrete lead to decrease in compressive strength. However, it was found that when 25% of quarry dust replaces river sand in concrete, the compressive strength of the concrete was higher than concrete utilizing river sand.

Generally there is lack of adequate information on quarry dust from machine dressing quarries. Much research has focused on the waste from aggregate crushing quarries. Mulu *et al* (2003) reported that silt content and water absorption of this material is very high as compared to conventional river sand. This research aimed at investigating the engineering properties of the material and its potential use in low cost construction. Musiomi *et al* (2007) while investigating the effect of mortar types on the strength of masonry walls made of stabilized quarry dust blocks found that the dust performed adequately both in blocks and mortar. In the study, six wall panels were built from laterized quarry dust blocks and different mortar joint conditions. The walls were then tested on a loading frame while monitoring the surface strains on blocks and mortar. Both vertical and horizontal surface strains were monitored. The blocks that were used contained quarry dust from machine

dressing plants. The results indicated that the wall panels with mortar joint containing the quarry dust as the blocks had compressive strength comparable to conventional walls. Ductility of alternative walls was however, higher by 30%.

From previous work, it can be deduced that quarry dust can be used to replace river sand in concrete without adversely affecting the mechanical properties. The present study seeks to establish the optimum proportions of quarry dust required to replace sand in concrete. It is expected that addition of quarry dust will reduce the workability of the concrete since the dust contain a lot of fine material as compared to the conventional river sand. In this study, quarry dust from machine dressing plant is limited to the production of masonry blocks. The dust is produced from soft rock with high water absorption which makes the material unsuitable for concrete production.

### **2.3 Lateritic material**

Laterite is a pedogenic and highly weathered natural material formed by the concentration of hydrated oxides of iron and aluminium, further oxidized to form an insoluble precipitate of fine particles. Further concentration and dehydration and subsequent cementation forms hard concretionary nodules or the coalescence of particles into a hard vesicular mass of honeycomb structure where cavities may contain the host soil (Aleva, 1994; Bardosy *et al.*, 1990).

The soluble hydrated ferrous oxide (FeO) dissolves in water and is leached from parent rock together with aluminium oxide into a host soil. Further oxidation occurs

to the ferrous oxide resulting in ferric oxide ( $\text{Fe}_2\text{O}_3$ ), which is insoluble and precipitates into fine particles. Concentration of the oxides is either by residual accumulation or by solution, movement and chemical precipitation. Increased concentration due to loss of moisture results in the formation of discrete soft nodules of soil cemented with the precipitate. This process and the subsequent hardening of the nodules are referred to as concretionary development. The presence of oxides of iron and aluminium together with silica and kaolinite clay minerals in various different proportions gives laterite the distinct ochre, yellow, purple or red colour of which red is the most predominant emanating from the red iron oxide (Aleva, 1994; Bardosy *et al.*, 1990; Schellman, 1983).

Depending on the degree of concretionary development, the physical properties of laterite varies widely from soil to rock-like material. In order for laterite to form, there should be parent rock rich in iron, aluminium and silica and the climatic conditions should include moderate to high temperatures and moderate to high rainfall with distinct dry and wet season. Laterite occurs mostly in the tropical and sub-tropical regions with lowest maximum temperatures above  $18^\circ\text{C}$  and the average temperature over  $24^\circ\text{C}$  (Aleva, 1994; Bardosy *et al.*, 1990; Schellman, 1983). The other conditions that are a prerequisite for the formation of laterite are summarized in Table 2.1:

**Table 2.1:** Theoretical conditions for concrectionary laterite development

|                         |               |                                  |  |
|-------------------------|---------------|----------------------------------|--|
| Annual rainfall<br>(mm) | 750 - 1000    | 1000 - 1500                      | 1500 - 2000  |
| Length of Dry<br>Season | 7             | 6                                | 5  |
| Type of Laterite        | Rock laterite | Hard<br>concretionary<br>gravels | Min. requirements<br>for concretions to<br>develop |

The abundance of the laterite in tropical region has generated research interest in the material. There is an attempt to determine the performance of laterite in concrete production regardless of its great deviation from the conventional concrete material in terms of physical properties (Adepegda, 1975; Balogun, 1982; Osunade, 1994). The effect of using laterite as fine aggregate in concrete was first studied by Adepegda (1975). He compared the properties of concrete made with conventional aggregates and the laterized concrete.

The possibility of replacing sand in concrete with laterite was also investigated by Balogun (1982). In another study, Balogun and Adepegda (1982) found that the most suitable mix of laterized concrete for structural purposes is 1:1.5:3, using batching by weight with water/cement ratio of 0.65, provided that the laterite content is kept below 50% of the fine aggregate content. In a study by Osunade (1994), it was found that shear and tensile strength of laterized concrete increased with curing duration. Laterite containing wide range of particle size also resulted to higher strength.

In a recent study by Ata *et al* (2005), it was established that Poisson's ratio of laterized concrete ranges between 0.25 and 0.35 and increases with age. In addition, it was noted that methods of curing, compaction and water/cement ratio have little influence on Poisson's ratio. Poisson's ratio of laterized concrete increases as the mix becomes less rich.

Investigation by Udoeyo *et al* (2006) show that concrete with up to 40% replacement level of sand by laterite attained compressive strength of 20 N/mm<sup>2</sup>. This indicates that it is possible to use laterite as partial replacement for sand up to this level. Additionally, the results showed that the compressive, split tensile and flexural strengths and the percentage water absorption of the concrete decrease with increase in the replacement level of sand. In this study, the viability of replacing river sand with laterite mixed with quarry dust is investigated. The amount of laterite was generally kept low since too much of laterite has adverse effects on the engineering properties of concrete.

#### **2.4 Recycled concrete aggregates**

Recycled concrete aggregate (RCA) is obtained by crushing old concrete from demolished concrete structures. This material is being applied to a large scale in construction work in the developed countries such as the USA, Europe and Japan. In the USA, studies have reported that there is no longer any barriers to the use of recycled concrete as aggregates in new concrete (ECCO, 1999). The technology of concrete recycling is well established in the USA where the recycled concrete aggregate is used for new roads, streets and highway construction. Studies report

that concrete recycling to produce structural grade concrete for non-pavement uses is technically feasible with certain precautions such as avoiding contaminated materials (ECCO, 1999). For example, it is generally accepted that when natural sand is used, up to 30% of natural crushed aggregate can be replaced with coarse recycled concrete aggregate without significantly affecting the mechanical properties of the concrete (Hansen, 1992).

Limbachiya *et al* (2004) carried out an extensive study on the performance of recycled aggregate concrete. The effects of up to 100% coarse recycled concrete aggregate on fresh, engineering and durability properties were established and its suitability for use in a series of designated applications was assessed. The study demonstrated that up to 30% coarse recycled concrete aggregates can be used, without any modification in the mix design, in concrete construction with performance similar to natural aggregate concrete. The results also indicates that for recycled concrete aggregate samples obtained from four different sources, there was no significant variation in strength of concrete at a given recycled concrete aggregate content. This indicates that there is no significant effect of the properties of parent materials on compressive strength of concrete. The RCA concrete mixes were found to possess bulk engineering and durability properties similar to the corresponding natural aggregate concrete, providing they were designed to have equal strength.

The influence of recycled aggregate on the stress-strain relation of concrete was investigated by Marcus *et al* (2008). The results of their study indicated that by

using recycled concrete and recycled clay for concrete aggregate, a gain of deformation has to be accepted. Hence, in construction where deformations have to be considered, the smaller elastic modulus, resulting from the use of recycled aggregate, is noticed. The study also confirmed that there is no decrease in the compressive strength, when aggregate derived from recycled concrete or clay is used. From these results, the components made of concrete with recycled aggregate can be designed with the same characteristic values as components of concrete made with natural aggregates.

In the present study, recycled aggregates from concrete with varying compressive strength is investigated because previous studies (Limbachiya *et al.*, 2004) demonstrate that the properties of the parent concrete do not influence the new concrete utilizing recycled concrete aggregates. The study also seeks to evaluate concrete with 100% replacement of natural with recycled aggregates from demolished concrete. Recycled aggregates from burnt clay were found to increase the elastic modulus and hence deformability of hardened concrete (Marcus *et al.*, 2008).

## **2.5 Stabilized earth blocks**

Recently, there has been renewed interest by researchers on the application of earth based construction techniques (Venkatarama *et al.*, 2007; Alutu *et al.*, 2006). This is particularly so in the developing countries where there is shortage of adequate housing for the ever increasing poor population. Stabilized soil blocks and earth rammed walls are examples of earth based construction. Venkatarama *et al.* (2007)

studied the influence of soil grading on the characteristics of soil-cement blocks and shear-bond strength of soil-cement block masonry triplets. They examined the influence of clay content of soil-cement block on strength, absorption and durability characteristics and interfacial mortar-block bond shear strength. The results indicated that the optimum clay content leading to maximum strength is in the range of 14-16%. The results also show that the initial rate of water absorption decreases with increase in clay content of the block.

Alutu *et al* (2006) in their study on the strength, durability and cost effectiveness of cement-stabilized laterite hollow blocks, reported that for 7% cement content and 13.8 N/mm<sup>2</sup> compactive pressure, blocks of compressive strength of at least 2.0 N/mm<sup>2</sup> at 28 days are produced. The blocks showed no features of wear after exposure to rain with weight losses within 2.4% after 12-cycled of wetting-brushing-drying. A study on the ecological housing material in Kenya was done by Sakata *et al* (2005). In their study they investigated the fundamental properties of stabilized soil blocks. Stabilized soil blocks were made from various soil types including laterite and red coffee soil. The stabilized soil blocks utilizing laterite material attained compressive strength of 2.8 N/mm<sup>2</sup> which is adequate for single storey residential house. In another study by Musiomi *et al* (2007), the viability of utilising quarry dust in making eco-blocks was investigated. Masonry walls prepared from stabilised quarry dust blocks were tested in compression while monitoring strains in blocks and mortar. The study established that, the walls had compressive strength comparable to conventional walls.

To date, a number of studies have been carried out on earth based construction. Various researchers have given some guidelines on the properties of the ingredient materials as well as the proportions of the mix. However, building standards with guidelines that are based on extensive studies are generally lacking. This has limited the application of the earth based materials in construction because the designers lack confidence in the limited available data.

The present study focuses on the feasibility of using stabilised laterized quarry dust blocks in construction. The amount of cement required to stabilise the material is limited to a range of 0 to 13% of the materials because red soil and murrum have been found to make satisfactory blocks when stabilised with 5-11% of Portland cement.

## **2.6 Outline of concrete mix design**

Mix design is defined as the process of selecting suitable ingredients of concrete and determining their relative quantities with the object of producing concrete of certain minimum properties, notably consistence, strength, and durability (Neville, 1993). The concrete mix design is performed to ensure that the concrete mix formulation meets or exceeds the specification requirements. Aggregates takes up to 60-90% of the total volume of concrete. Selection of aggregate type and particle size distribution greatly affects the workability of concrete as well as the mechanical strength, permeability, durability and the total cost of hardened concrete (Genadij *et al.*, 1998). Therefore, aggregate mix design is an essential part of the concrete mix design and optimisation.

Neville (1993) reviewed various studies on concrete mix design and established that although there are certain desires for good grading curves for aggregates, no ideal grading exist, and excellent concrete can be made with a wide range of aggregate grading. This conclusion forms the basis upon which unconventional aggregates, which do not meet the standard grading curves, can be investigated for their viability in concrete production. The alternative aggregates that are investigated in this study are unconventional and hence their particle size distribution differs from the standard curves specified in codes of practice. For instance, recycled concrete aggregates are generally large as compared to the conventional natural crushed aggregates and hence their respective gradation curves are different. Therefore, gradation curves for the alternative materials are developed in this study so as to compare with the existing standards.

## CHAPTER THREE

### 3.0 MATERIALS AND METHODS

#### **3.1 Introduction**

In order to achieve the objectives of the study, each of the materials was investigated to establish the fundamental material properties following which the optimum proportions of ingredient materials were determined for concrete mix and masonry block mix. Finally the viability of using the materials was assessed for large scale concrete beams and masonry wall panels. Reinforced concrete beams were cast and tested in bending to assess flexural response, while wall panels were tested in compression in the laboratory to evaluate compressive behaviour.

#### **3.2 Material sampling and preparation**

##### ***3.2.1 Cement***

Cement was used in the production of concrete and as binder material in masonry blocks and mortar. The cement used was Portland pozzolana cement of normal strength of  $32.5 \text{ N/mm}^2$  as per the Kenya Standards (KS-18-1:2001). In this study cement type is not a variable hence was obtained from one source.

##### ***3.2.2 River sand***

River sand was required for the production of concrete. River sand from Machakos, which is commonly used in Nairobi, was used in the study. The material was obtained from a dealer in Nairobi. The river sand satisfying British Standards (BS 882) and American Standards (ASTM C33-78) grading requirements for fine aggregates was used in the study.

### ***3.2.3 Natural crushed aggregates***

Natural crushed coarse aggregate was required for the production of concrete. The material was obtained from Athi River situated to the south of Nairobi. The natural crushed aggregates satisfying both the BS 882 and ASTM C33-78 grading requirements for coarse aggregates was used in the study.

### ***3.2.4 Quarry dust wastes***

Two common types of quarry dust wastes found in Kenya were used in this study. Quarry dust from aggregate crushing plant (QDNbi) was obtained from Athi River.

On the other hand quarry dust from stone dressing quarries (QDNdar) was obtained from Juja in Thika District. Plate 3.1 shows quarry dust material in a typical quarry. In this study, QDNbi was used in both the production of concrete and block making while QDNdar due to its high water absorption was used in making masonry block only.



**Plate 3.1:** Quarry dust in a quarry site

### ***3.2.5 Laterite***

Since in this study, laterite was intended for use with quarry dust, laterite found near Nairobi, where there is plenty of quarry dust was used. The material was obtained from a suitable quarry in Nairobi. The role of the laterite material was to modify the quarry dust component to be utilized in production of concrete and masonry blocks.

### ***3.2.6 Recycled concrete aggregates***

From the literature, it has been reported that the grade of the parent concrete does not significantly influence the mechanical properties of new concrete manufactured from recycled concrete aggregates (ECCO, 1999). Therefore, concrete of varying compressive strength was used as parent material for the production of recycled concrete aggregates.

Recycled concrete aggregates were obtained by manually crushing laboratory tested concrete cylinders and cubes some as old as over 15 years. The old concrete was sorted out to remove all foreign unwanted materials before crushing. The material was then manually crushed by use of sledge hammer. A 5 mm sieve was then used to separate the fine dust from the aggregates. Only material retained on the 5mm sieve was used for concrete production as coarse aggregates. Confirmatory test was also set up to confirm that the properties of parent concrete do not influence the performance of the new concrete. To carry out this conventional concrete of known compressive strength was made in the laboratory. The conventional concrete was then used in casting small sized slabs. The concrete slabs were then cured using standard procedure for 28 days. After this period, the concrete slabs were manually crushed and the material separated using the 5 mm sieve. New concrete was then produced for the recycled concrete aggregates of known strength and compared to concrete utilizing the old parent concrete of unknown strength.

### ***3.2.7 Conventional building blocks***

Conventional building blocks were obtained from a typical quarry at Ndarugu, Thika district. A photo of conventional machine dressed blocks is shown in Plate 3.2. The blocks were reduced in size to match the size of the



stabilized quarry dust blocks using a **Plate 3.2:** Machine dressed blocks, bench mounted cutter. This was done in order to allow comparison of the results to be made.

### **3.3 Material physical properties test**

The main material physical properties tests carried out on the aggregates included density, silt content, water absorption and gradation tests. Specific gravity data is used in describing the aggregate while bulk density is used for proportioning material in mix design. Silt content information is important in determining the suitability of a particular material in concrete production and the water requirement of the concrete mix. Finally, gradation information is useful in establishing workability requirement of the concrete mix.

#### ***3.3.1 Density tests***

Bulk density test was carried out according to the standard procedures required by BS812: Part 2 of 1996. The bulk density measurements were done in two states of

the aggregates: loose and compacted state. In both cases, the aggregates were oven dried. Apparent specific gravity test was carried out according to the standard procedures required by BS812: Part 2 of 1996. Apparent specific gravity is defined as the ratio of the weight of the aggregate dried in an oven at 100 to 110°C for 24 hours to the weight of water occupying a volume equal to that of the solid including the impermeable pores (Neville, 1981). Bulk and apparent densities for both the fine and coarse aggregates were determined.

### ***3.3.2 Silt content test***

Silt content in fine aggregates was determined according to the standard procedures required by BS812: Part 2 of 1996 and KS-02-95 of 1984.

### ***3.3.3 Water absorption test***

Water absorption tests were conducted on all the fine and coarse aggregates. The water absorption of the aggregate was determined by measuring the increase in weight of an oven-dried sample when immersed in water for 24 hours. The water absorption was defined as the ratio of the increase in weight to the weight of the dry sample, expressed as a percentage. The test was carried out according to BS 812: Part 2 of 1975 requirements

### ***3.3.4 Sieve analysis or gradation test***

The sieve analysis test involves dividing a sample of aggregate into fractions which contain particles between specific size limits, these being the openings of standard

sieves. Gradation test was carried out using sieves to the requirements of BS812: Part 1; 1975. This was done for all the fine and coarse aggregates used in this study.

### **3.4 Studies on concrete mixes and optimal material combination**

#### ***3.4.1 Optimum QDNbi content***

The optimum QDNbi content test was aimed at determining the optimum fine aggregate combination of river sand and quarry dust to give concrete with the best performance. In the conventional concrete of nominal mix 1:1.5:3 for cement, river sand and natural crushed aggregates, respectively, river sand was partially replaced with 0, 25, 50, 75 and 100% of quarry dust (crushed rock quarry dust). Test cubes were then prepared and tested for compressive strength at the age of 7, 28 and 90 days using standard procedures.

#### ***3.4.2 Optimum RCA content***

In this study, only the coarse part of recycled concrete aggregate is proposed for use in concrete production. In the conventional concrete of nominal mix 1:1.5:3 for cement, river sand and natural crushed aggregates respectively, natural crushed coarse aggregates was partially replaced with recycled concrete. Test cubes were then be prepared and tested for compressive strength at the age of 7, 28 and 90 days using standard procedures.

#### ***3.4.3 Optimum laterized quarry dust content***

The performance of laterite as fine material replacement in concrete has been studied and found to produce good results. Udoeyo *et al* (2006) demonstrated that

concrete with up to 40% replacement level of sand by laterite attained the designed strength of 20 N/mm<sup>2</sup>. This research aims at utilizing laterized quarry dust as river sand replacement in concrete. In concrete utilizing natural crushed aggregates and an optimum combination of river sand and quarry dust as determined above, the amount of quarry dust was partially replaced with 0, 12.5, 25, 37.5 and 50% of laterite material. This test was aimed at evaluating the behaviour of the combined material when natural crushed aggregates are used. The effect of partially replacing quarry dust with laterite when recycled concrete aggregates are used as coarse aggregates was also investigated. In this case, the amount of quarry dust was partially replaced with 0, 20, 40 and 60% of laterite material. Test cubes were then be prepared and tested for compressive strength at the age of 7, 28 and 90 days using standard procedures.

#### ***3.4.4 Workability tests***

The most common workability test methods are the slump test, the compacting factor test and the Vebe test. Since the materials used in the concrete production are not conventional, two methods; the slump test and the compacting factor test were used. Slump test was carried out on samples of fresh concrete as per the requirements of BS1881: Part 102 of 1983. Compacting factor test was also carried out on samples of fresh concrete as per the requirements of BS1881: Part 103 of 1983. Both tests were carried out immediately after mixing the concrete.

### ***3.4.5 Compressive strength test***

Laboratory compressive strength test on concrete cubes is important because it gives an indication of the mechanical strength of the concrete which is a very important parameter in the design of any concrete structure. The concrete test cubes were prepared according to BS1881: Part 108 of 1983 requirements. After 7, 28 and 90 days, the cured cubes and cylinders were tested according to BS1881: Part 116 of 1983 requirements.

## **3.5 Studies on masonry block mixes and optimal material combination**

### ***3.5.1 Optimum cement content***

To determine the optimum cement content, laterized quarry dust of mix ratio 1:1:2 for QDNbi, QDNdar and laterite, respectively, was stabilized with cement content varying between 3 to 14%. The stabilized quarry dust block mixes were then used to make machine pressed blocks which were cured at room temperature for 7 days and let to dry in a shed.

### ***3.5.2 Optimum quarry dust combination***

The optimum quarry dust combination was determined by evaluating block mixes containing various proportions of quarry waste from machine dressed blocks. In a block mix containing 7% of Portland cement, QDNbi was partially replaced with 0, 25, 50, 75 and 100% of QDNdar. The laterite content was kept equal to the quarry dust combined content.

### ***3.5.3 Optimum laterite content***

To determine the optimum laterite content in block mix, quarry dust material was partially replaced with 0, 25, 50, 75 and 100% of laterite. The quarry dust material in the mix consisted of quarry dust from aggregates crushing plant and quarry waste from machine dressed stone quarry in equal proportions.

### ***3.5.4 Compressive strength test of masonry blocks***

Cured masonry blocks were tested for compression strength on a universal testing machine (Plate 3.3) after 7, 28 and 90 days from the day of casing. Three blocks were tested for each case and the average compressive strength reported as the compressive strength of the blocks at that specified age.



**Plate 3.3** Experimental set up for testing blocks on a UTM.

### ***3.5.5 Water absorption test of blocks***

The water absorption test was carried out on blocks after curing and subsequent drying at the age of 28 days from the date of casting. The test was determined by measuring the increase in weight of dried sample of blocks when immersed in water for 24 hours (the source of the water being removed). The ratio of the increase in weight to the weight of the dry sample, expressed as a percentage, is termed absorption.

### **3.6 Reinforced concrete beam testing**

#### ***3.6.1 Specimen preparation***

Beams measuring 1200 mm length x 150 mm wide x 200 mm depth were tested to determine the flexural behaviour when subjected to a central point load. The beams were reinforced with varying amount of reinforcement with cover to steel reinforcement being maintained at 30 mm. Shear reinforcement in form of links were also provided to enhance shear capacity of the beam and ensure the beams fail in bending and not shear mode. First, reinforcement cage was prepared for each beam. The cage was then placed in prepared formwork in which spacer blocks were used to position the reinforcement in order to maintain cover to the reinforcement bars at 30 mm. Concrete of various material compositions was prepared and the beams cast by placing the concrete in the mould and vibrating using poker vibrator. After 24 hours the beams were demoulded and rapped in wet cloth for curing. Water was used for curing the beams for 7 days. The beams were then let to harden up to an age of 28 days. A total of 8 beams were tested in this study. During casting of the beams, standard concrete cubes were also cast and cured for testing after 28 days. Table 3.1 shows details of the beam specimen.

**Table 3.1** Beam specimen details

| Beam                | Material              | Concrete Mix | Bottom reinforcement |             |
|---------------------|-----------------------|--------------|----------------------|-------------|
|                     |                       |              | Rebars               | % age steel |
| ConvConc1-2-4       | Conventional concrete | 1:2:4        | 2Y10                 | 0.523       |
| ConvConc1-1.5-3(8)  | Conventional concrete | 1:1.5:3      | 2Y8                  | 0.333       |
| ConvConc1-1.5-3(10) | Conventional concrete | 1:1.5:3      | 2Y10                 | 0.523       |
| ConvConc1-1-2       | Conventional concrete | 1:1:2        | 2Y10                 | 0.523       |
| AltConc1-2-4        | Alternative concrete  | 1:2:4        | 2Y10                 | 0.523       |
| AltConc1-1.5-3(8)   | Alternative concrete  | 1:1.5:3      | 2Y8                  | 0.333       |
| AltConc1-1.5-3(10)  | Alternative concrete  | 1:1.5:3      | 2Y10                 | 0.523       |
| AltConc1-1-2        | Alternative concrete  | 1:1:2        | 2Y10                 | 0.523       |

### ***3.6.2 Testing procedure and instrumentation***

Well cured beam was placed on a steel beam on which roller supports had been mounted by use of clamps in order to prevent movement of supports during testing. The beam was checked for verticality using a spirit level. An electrical resistance strain gauge was then pasted at the centre bottom face of the beam. The strain gauge was used to measure the strain of concrete surface of the beam. Linear Variable Displacement Transducer (LVDT) to measure beams central displacement was positioned at the central point of the beam near the bottom. LVDT and electrical resistance strain gauge were connected to a data logger (TDS 302) which was in turn connected to a computer to automatically take measurements. Load applied was

measured by the Universal Testing Machine (UTM) and recorded by the data logger. During testing, settlement load was first applied on the beams. This was done in order to check that the entire system was working well. The beam was then loaded to failure while monitoring the formation and general pattern of crack. Beams failed by breaking of steel reinforcement which was accompanied by loud noise. The procedure was repeated for all the other seven beams. Plate 3.2 shows the concrete beam set-up ready for testing.



**Plate 3.4** Experimental set-up of concrete beam ready for testing

### **3.7 Masonry wall panel testing**

#### ***3.7.1 Specimen preparation***

##### ***3.7.1.1 Stabilized laterized quarry dust masonry blocks***

Stabilised laterized quarry dust masonry blocks were made by mixing cement, QDNbi, QDNdar and laterite in appropriate proportions. After mixing the materials in dry state, water was added to the material then it was thoroughly mixed.

Thereafter the mixture was pressed using a standard block pressing machine to make blocks of size 290 x 140 x 120 mm. The block was then removed and stored in a shed. This was repeated for each masonry block mix prepared. Curing was done after 24 hours for a period of 7 days by sprinkling water on the blocks. Plate 3.5 shows the stabilized quarry dust masonry block being removed from the machine.



**Plate 3.5** Laterized quarry dust block being removed from the machine

#### *3.7.1.2 Wall panels*

Wall panels of varying dimensions were built on steel plates fitted with handles for ease of lifting the walls to the testing machine. Details of wall panels tested in this study are shown in Table 3.2. The variables investigated included the strength of mortar joint, the effect of reinforcement on wall behaviour and the effect of size of the panel on wall behaviour. A total of 10 wall panels were tested.

**Table 3.2** Wall panel details

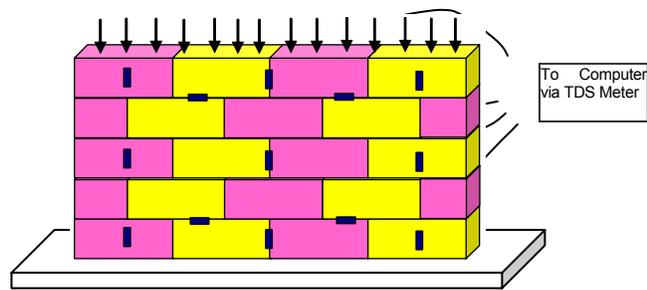
| Wall           | Material     | Size   |        | Mortar ratio | reinforcement |
|----------------|--------------|--------|--------|--------------|---------------|
|                |              | H (mm) | B (mm) |              |               |
| ConvBL 1:3(R)  | Conventional | 520    | 600    | 1:3          | Reinforced    |
| ConvBL 1:3     | Conventional | 520    | 600    | 1:3          | None          |
| ConvBL 1:3L    | Conventional | 520    | 900    | 1:3          | None          |
| ConvBL 1:4     | Conventional | 520    | 600    | 1:4          | None          |
| ConvBL 1:3(R)x | Conventional | 620    | 760    | 1:3          | Reinforced    |
| ConvBL 1:3x    | Conventional | 620    | 760    | 1:3          | None          |
| AltBL 1:3(R)   | Alternative  | 520    | 600    | 1:3          | Reinforced    |
| AltBL 1:3      | Alternative  | 520    | 600    | 1:3          | None          |
| AltBL 1:3L     | Alternative  | 520    | 900    | 1:3          | None          |
| AltBL 1:4      | Alternative  | 520    | 600    | 1:4          | None          |

**H**, Height of the wall panel; **B**, Breadth of the wall panel

### ***3.7.2 Testing procedure and instrumentation***

The laterized quarry dust blocks designed above and conventional blocks were used to build masonry wall panels. The wall was lifted manually and placed in position on a UTM. Electrical resistance strain gauges were then pasted on blocks and mortar joints on the surface of the walls. The strain gauges were then connected on a data logger strain meter which was in turn connected to a computer. Timber parking was then placed on top of the wall panel in order to distribute the applied load on the wall panel. Loading was applied by the UTM and recorded by the data logger. The

wall panel was then loaded to failure with the surface strains and load being automatically recorded by the assembly. This was repeated for all the other wall panels. The experimental set-up for wall testing is shown in Plate 3.6.



(a) Schematic representation



(b) Photo of experimental set-up for wall panels

**Plate 3.6** Experimental set up for wall panels

## CHAPTER FOUR

### 4.0 MATERIAL PHYSICAL PROPERTIES

#### 4.1 Introduction

In this chapter, material properties for all the aggregates investigated are evaluated. The main material properties that have been found to influence the material behaviour include gradation, clay or silt content, water absorption and density (which is primarily used in determining batching proportions in mix design). Other mechanical properties also exist but their influence is application dependent hence have not been the focus in this study. The gradation of aggregates, clay or silt content, water absorption and density of the material were determined according to the requirements of BS812: Part 2 of 1996 and KS-02-95 of 1984.

#### 4.2 Results and discussion

##### 4.2.1 Relative density

Both the bulk and the specific relative densities were used to describe the aggregates in this study. The bulk density of material was determined both in the loose and dense state. The values for bulk and specific densities of river sand, QDNbi, QDNdar, laterite, natural crushed aggregates and recycled concrete aggregates are shown in Table 4.1. From the table, the bulk densities of river sand and QDNbi in the loose state are 1405 and 1354 kg/m<sup>3</sup>, respectively. Loose state densities for QDNdar and laterite material are 950 and 1162 kg/m<sup>3</sup>, respectively. These values compare well with the results reported by Raman *et al* (2007) and Mulu *et al* (1998).

**Table 4.1:** Basic material properties

| Physical properties               |                 | River sand | QDNbi    | QDNdar    | Laterite | NCA  | RCA  |
|-----------------------------------|-----------------|------------|----------|-----------|----------|------|------|
| Specific density                  |                 | 2.6        | 2.63     | –         | –        | 2.55 | 2.57 |
| Bulk density (kg/m <sup>3</sup> ) | Loose condition | 1405       | 1354     | 950       | 1162     | 1303 | 1152 |
|                                   | Dense condition | 1547       | 1496     | 1050      | 1294     | 1395 | 1253 |
| Water absorption in %             |                 | 1.3        | 2.0      | 6.5       | 6.8      | 1.4  | 5.6  |
| Silt content %                    |                 | 2.7        | 5.5      | 12.5      | 14.0     | –    | –    |
| Fineness modulus                  |                 | 2.27       | 4.46     | 1.78      | 5.46     | –    | –    |
| Avg. sieve size(mm)               |                 | 0.3-0.6    | 1.18-2.0 | 0.15-0.30 | 2.36     | –    | –    |
| Max. Aggregate Size (mm)          |                 | 9.52       | 9.52     | 9.52      | 9.52     | 31.7 | 31.7 |

The bulk densities of quarry dust (QDNbi) in dense and loose state were reported as 1360 and 1510 kg/m<sup>3</sup>, respectively (Raman *et al.*, 2007). Quarry dust from building stone (QDNdar) on the other hand, was reported to have bulk densities of 940 and 1160 kg/m<sup>3</sup> in loose and dense state, respectively (Mulu *et al.*, 1998). Quarry dust from stone dressing plant is usually softer than aggregate crushing plant quarry waste. The dust contains a lot of fine material because it is produced when the machine saw blade moves, cutting the rock to shape the building blocks (Musiyomi *et al.*, 2007). The dust can be considered to fall in lightweight aggregates due to low bulk density. The role of laterite in this study is to modify the quarry dust material to be used for both concrete production and masonry block manufacture. Compared to river sand, laterite material has lower bulk density. It is less dense than the quarry

dust from aggregate crushing plant but denser than the dust from stone dressing quarries.

In the coarse aggregates, the loose state bulk densities of natural crushed and recycled concrete aggregates were 1303 and 1152 kg/m<sup>3</sup>, respectively. The corresponding values for dense state density were 1395 and 1253 kg/m<sup>3</sup>, respectively. Recycled concrete aggregate had lower bulk density than conventional aggregates. Limbachiya *et al* (2004) while investigating the performance of recycled aggregate concrete reported bulk density in loose state for RCA and natural crushed aggregates (NCA) as 1190 and 1340 kg/m<sup>3</sup>, respectively. These values are generally comparable to the results obtained in this study. The use of recycled concrete aggregate as course aggregates in concrete production has been accepted in industrially advanced countries such as the USA, Japan and the UK (ECCO, 1999). It is generally accepted that when river sand is used, up to 30% of natural aggregates can be replaced with recycled concrete aggregates without affecting properties of concrete (ECCO, 1999).

#### **4.2.2 Silt content**

The determination of silt content was carried out according to the standard procedures required by BS812; Part2, 1996: Methods of Sampling and Testing of Mineral Aggregates, Sands and Fillers. The silt content for river sand, QDNbi, QDNdar and laterite material is also shown in Table 4.1. From the table, the silt content of 2.7 and 5.5% were present in river sand and QDNbi, respectively. The values for QDNdar and laterite were 12.5 and 14%, respectively. Laterite material

contain the highest amount of silt or clay content followed by the QDNdar. River sand recorded the lowest silt content. BS 882:1973 specifies maximum silt content of 15% in crushed stone sand and 3% in natural or crushed gravel sand. River sand and laterite material are obtained naturally without any specific processing. Laterite has silt content of 14% and hence does not meet the minimum requirements for silt content according to BS 882:1973. River sand on the other hand has silt content of 2.7% which is less than the maximum value of 3% specified in BS 882:1973. High silt content in construction material has the effect of increasing the water requirement of concrete needed to achieve a specified level of workability.

#### ***4.2.3 Water absorption***

Water absorption was determined for all the fine and coarse aggregates by measuring the increase in weight of an oven-dried sample when immersed in water for 24 hours. Table 4.1 shows water absorption of all the fine and coarse aggregates. In the fine material, laterite recorded the highest absorption followed by QDNdar. River sand had absorption of 1.3% which was the lowest in fine aggregates.

Quarry dust (QDNbi) was found to have higher water absorption than river sand because the quarry dust contains crusher dust which has the effect of increasing the water absorption of the material. The values of water absorption quoted for river sand and quarry dust as 1.3 and 2%, respectively, are generally within the limits proposed by Neville (2007) for commonly used fine aggregates. Quarry dust from stone quarries (QDNdar) has even higher water absorption than QDNbi because it is produced from soft rock. During processing of the two materials, QDNdar is

produced with high contents of fine material than QDNbi. Due to high fine material content, the dust has high water absorption and silt content than the conventional river sand and the quarry dust from the aggregate crushing plant. The high water absorption of the dust restricts its use as fine aggregates in concrete production. In this study, the dust is recommended for use in making of masonry blocks only since it has been demonstrated to perform well in cement stabilized blocks. The results in Table 4.1 shows that laterite material has the highest water absorption and silt content than all the other fine aggregates. The high content of fine material in laterite leads to the high water absorption of the material.

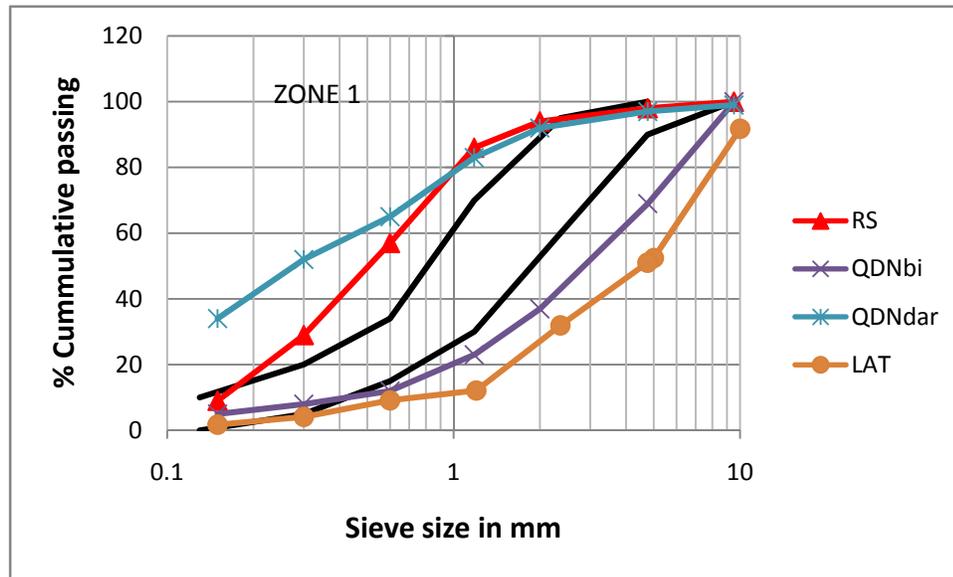
Natural crushed aggregates had absorption of 1.4% while recycled concrete aggregates recorded a value of 5.6%. The high water absorption for recycled concrete aggregates is due to the cement paste attached to aggregates in the material (Limbachiya *et al.*, 2004). The values of water absorption reported by Limbachiya *et al* (2004) in their study are 5.5 and 2.5% for RCA and NCA, respectively. The findings in this study therefore, confirm earlier research work on the materials.

#### ***4.2.4 Gradation of aggregates***

Gradation of aggregates was obtained by dividing samples of material using standard sieves according to the standard procedures required by BS812; Part2, 1996: Methods of Sampling and Testing of Mineral Aggregates, Sands and Fillers. The standard sieves that were used for gradation test of coarse aggregates include; 2.36, 4.75, 9.5, 12.5, 19.0, 25.0, 38.1 and 50.0 mm, while fine materials were analysed using standard sieves of sizes 0.15, 0.30, 0.60, 1.18, 2.36, 4.75 and 9.5

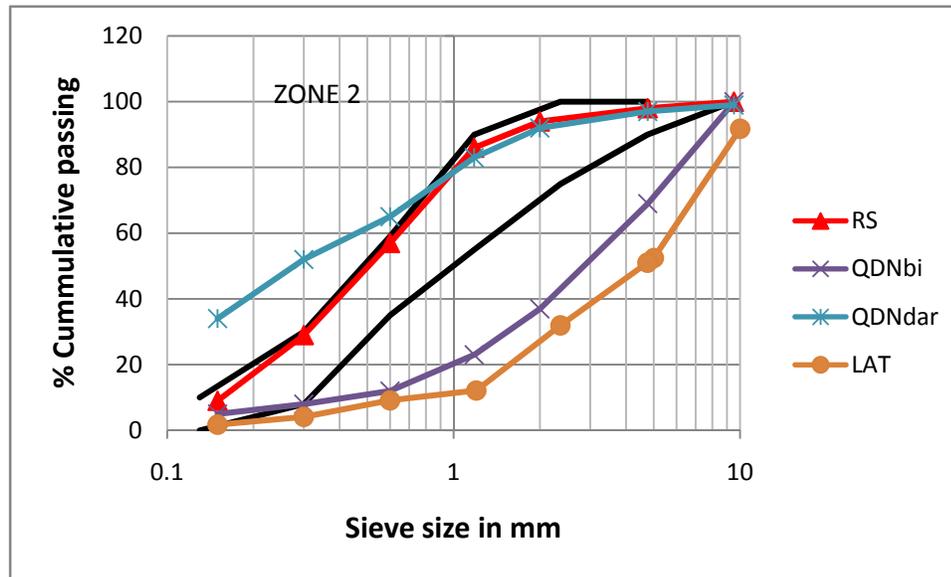
mm. It has been reported that compressive strength of fully compacted concrete with a given water/cement ratio is independent of the grading of the aggregates. Hence grading is of importance only as far as it affects workability (Neville, 1981).

The gradation curves for the river sand, quarry dust and laterite are shown in Figure 4.1. In the figure, the gradation curves are plotted together with the standard limits of Zone 1 for fine aggregates according to BS 882:1973. The figure shows that all the fine materials investigated fall outside the standard limits specified for Zone 1 fine aggregates as per the British standards. River sand and QDNdar are finer than the requirements of Zone 1 while QDNbi and laterite are coarser. The trend in the gradation curves is evident in the values of fineness modulus of the respective materials. Fineness modulus is a single parameter used to give an indication of the particle size distribution in fine aggregates. In Table 4.1, laterite has the largest fineness modulus of 5.46 and then followed by QDNbi, river sand and QDNdar in that order. The fineness modulus for the QDNbi, river sand and QDNdar were 4.46, 2.27 and 1.78, respectively. Therefore, the fineness modulus and the gradation curves show that laterite contain the largest amount of coarse material and then followed by QDNbi, river sand and QDNdar in that order.



**Figure 4.1** Gradation curves for river sand, QDNbi, QDNdar and Laterite-BS 882:1973 Grading Zone 1.

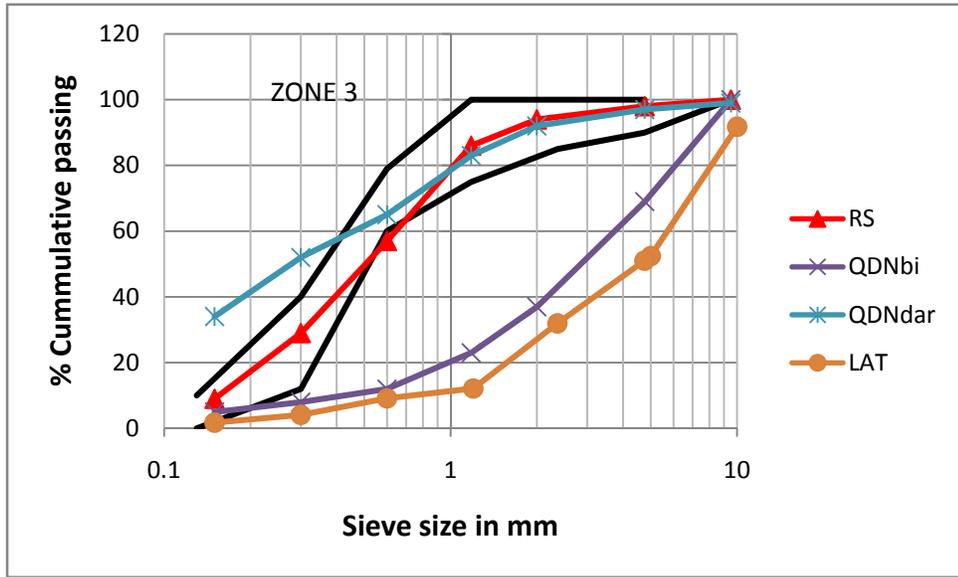
In Figure 4.2, gradation curves for river sand, quarry dust and laterite are shown. The figure shows that out of all the fine aggregates investigated, it is only river sand that satisfies the requirements of Zone 2 according to BS 882:1973. QDNdar contain large amount of fine material that make its gradation curve to cross the lower boundary line that defines Zone 2. Laterite and QDNbi contain large amount of coarse material and hence they fall outside the specified limits for Zone 2. Generally, Zone 2 defines fine aggregates that are finer than the aggregates that meet Zone 1 of BS 882:1973 requirements.



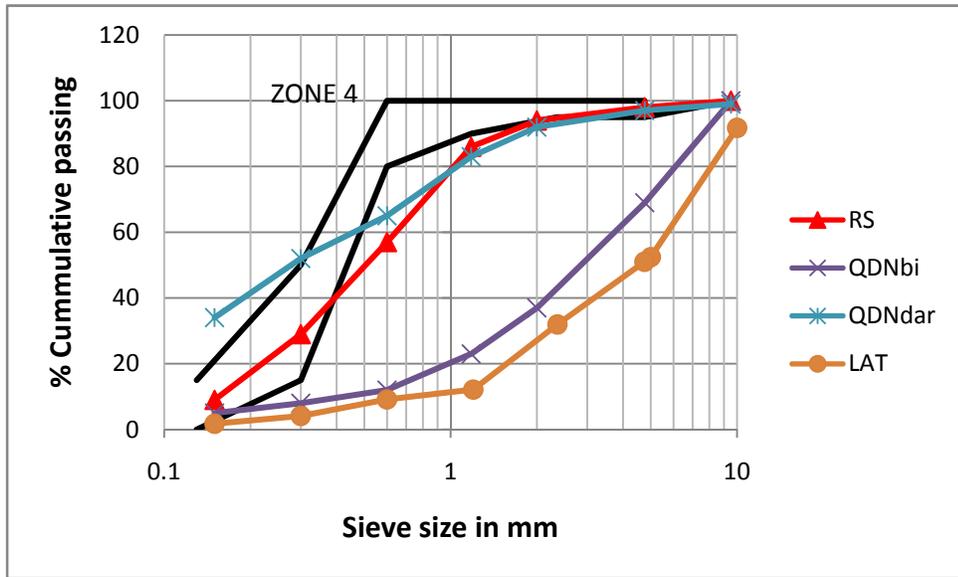
**Figure 4.2** Particle size distribution for river sand, QDNbi, QDNdar and Laterite-BS 882:1973 Grading Zone 2

Gradation curves for river sand, quarry dust and laterite, plotted with Zone 3 limits of the British standards, are shown in Figure 4.3. Out of all the fine aggregates, only river sand satisfies the particle size distribution requirements of Zone 3 according to BS882 of 1973. QDNdar, due to high silt content, crosses the boundary line for Zone 3 and hence does not satisfy the requirements of this zone. QDNbi and laterite on the other hand, contain a lot of coarse material and hence do not meet the requirements for Zone 3 as per BS882 of 1973.

In Figure 4.4, particle size distribution curves for fine aggregates are shown plotted on Zone 4 limits of British standards. Zone 4 defines aggregates that fall in finest category of fine aggregates. Aggregates that fall in this category usually require special consideration to workability as well as compressive strength since.



**Figure 4.3** Gradation curves for river sand, QDNbi, QDNdar and Laterite-BS 882:1973 Grading Zone 3



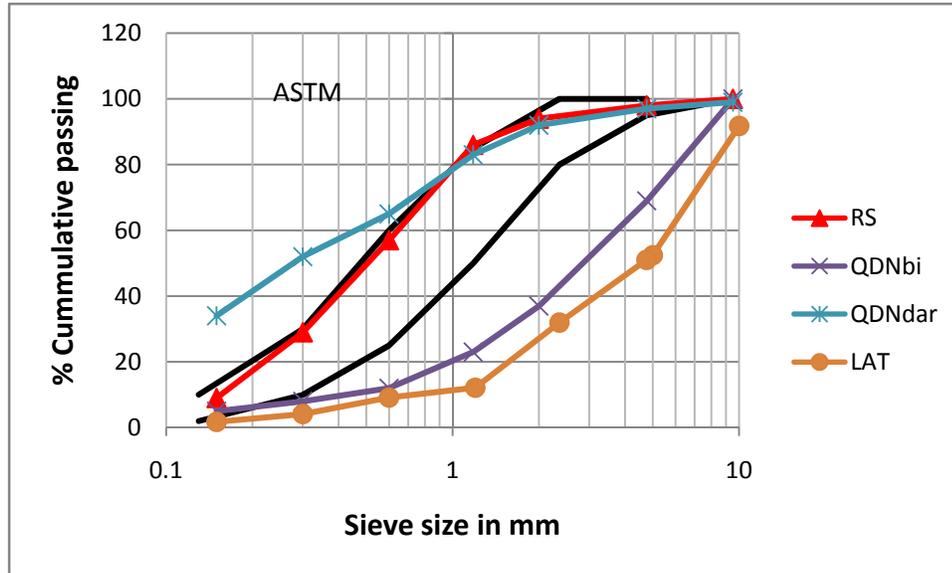
**Figure 4.4** Particle size distribution for river sand, QDNbi, QDNdar and Laterite-BS 882:1973 Grading Zone 4

Generally, all the fine aggregates investigated are coarser than the requirements for Zone 4 aggregates (Figure 4.4). River sand crosses into the limits while QDNdar crosses the boundary lines in the lower end of the curves which indicates that the quarry dust contains more silt content than the requirements for Zone 4 according to BS882 of 1973.

Unlike the British standards which classify fine aggregates for concrete production into four zones, the American standards has one zone that all the fine aggregates must satisfy. Gradation curves for river sand, quarry dust and laterite are shown in Figure 4.5. The figure shows that river sand satisfies the requirements of this standard. Quarry dusts and laterite on the hand, fall outside the limits for fine aggregates according to ASTM Standard C33-78. QDNdar is generally finer than the requirements of the American standards while QDNbi and laterite contain a lot of coarse materials than required by ASTM Standard C33-78.

River sand had a gradation curve fitting the grading Zones 2 and 3 of the BS 882: 1973 and also the ASTM Standard C33-78 grading limits. This put the sand in an average category of fine aggregates. Quarry dust on the other hand did not fit any of the standard grading zones. The material contains a lot of coarse particles which makes the aggregates to be coarser than conventional fine aggregates. This is revealed further in the examination of the fineness modulus of the material. Quarry dust has a higher fineness modulus than river sand. Safiudin *et al* (2007) while researching on the performance of quarry dust in concrete reported the fineness modulus of mining sand and quarry dust to be 3.01 and 3.2 respectively. The same

values were also quoted by Raman *et al* (2007) in their study on quarry dust although he used pit sand as opposed to mining sand.



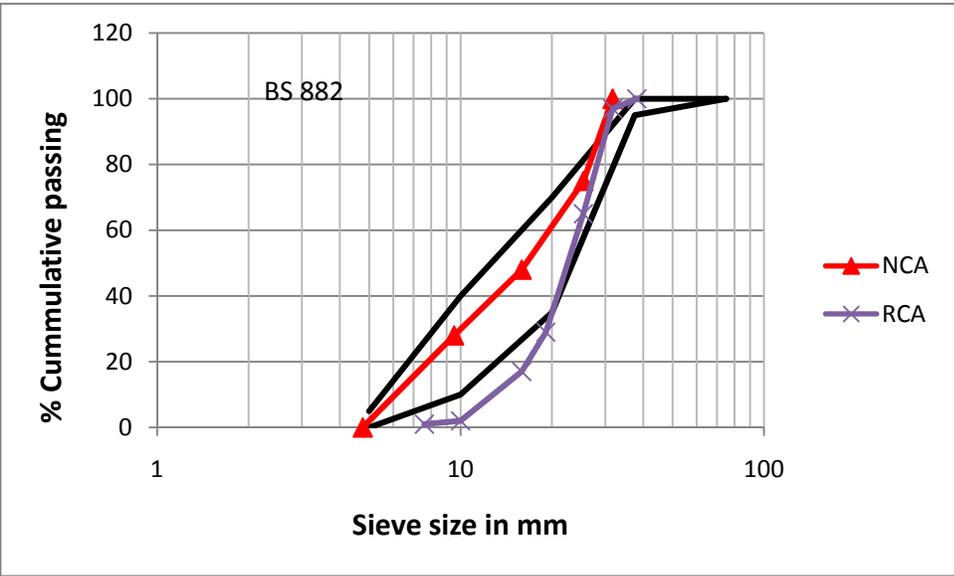
**Figure 4.5** Gradation curves for river sand, QDNbi, QDNdar and Laterite- ASTM Standard C33-78

Quarry dust from stone dressing plant approximate the grading Zone 3 of BS 882:1973 but contain a lot of fine material that makes it deviate from the standard limits for conventional material. River sand on the other hand fit the grading Zones 2 and 3 of BS 882:1973 and ASTM Standard C33-78 grading. The fineness modulus of river sand and dust are 2.27 and 1.78, respectively. If used for concrete production, the dust would increase the water demand of the concrete hence negatively affect the compressive strength of the concrete.

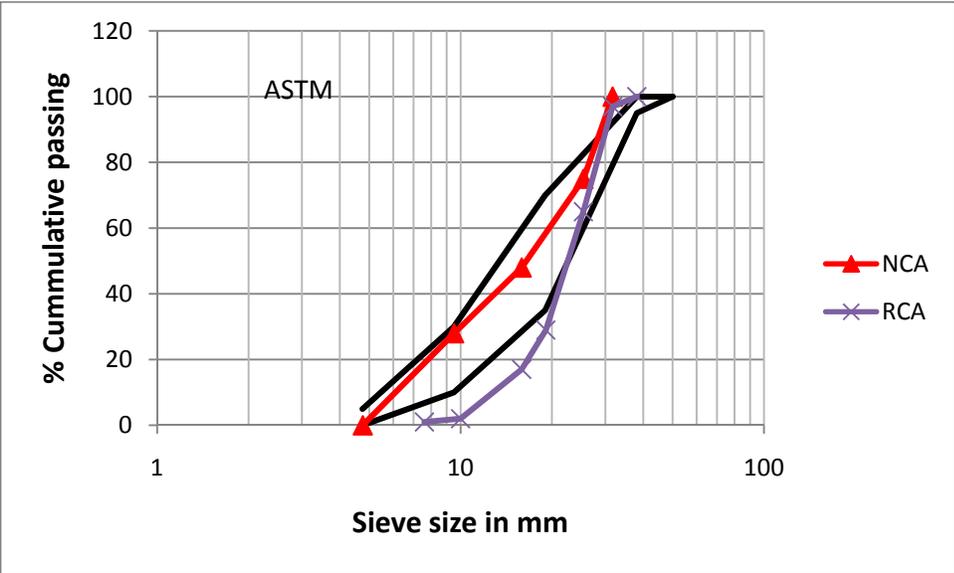
Sieve analysis results indicated that laterite contain coarser particles than the other fine aggregate materials. Therefore, the gradation curve of laterite is not fitting any of the standard grading limits for fine aggregates. The material, however, contains

more clay or silt than the rest of the fine aggregates. Laterite material recorded the highest fineness modulus in the fine aggregate category. The fineness modulus of laterite was 5.46 corresponding to average sieve size of 2.36 mm.

Figure 4.6 presents the gradation curves for natural crushed aggregates and recycled concrete aggregates plotted with the British standard specification for coarse aggregates. Particle size distribution of the natural crushed aggregates as seen from the gradation curve satisfies the requirements of BS 882:1973 for coarse aggregates. On the other hand, recycled concrete aggregates falls outside these limits. The aggregates have more coarse material than the natural crushed aggregates. Similar trend was seen from Figure 4.7 which shows the gradation curves for the aggregates plotted with the ASTM Standard C33-78 requirements for the coarse aggregates. Therefore, natural crushed aggregates fit well within the BS 882:1973 and ASTM Standard C33-78 grading limits for coarse aggregates while recycled concrete aggregates do not fit in any of the standards. Generally, the RCA contain coarser particles than the NCA material, because in RCA, cement paste is attached on all the parent aggregates. The presence of cement paste modifies the gradation properties of the aggregates.



**Figure 4.6** Gradation curves for natural crushed aggregates (NCA) and recycled concrete aggregates (RCA)-BS 882:1973



**Figure 4.7** Gradation curve for natural crushed aggregates (NCA) and recycled concrete aggregates (RCA)- ASTM Standard C33-78

### **4.3 Summary and conclusion**

The results of material physical properties for all the aggregates and masonry filler materials have been presented in this chapter. The main material properties for the alternative materials have also been compared with those of the conventional aggregates. In evaluating the gradation properties of aggregates, both the British and American standards were used.

The results have established that alternative fine material have properties different from the conventional river sand. All the fine aggregates and filler materials contain high silt content and water absorption as compared to conventional river sand. Laterite for instance contains the highest silt content as well as water absorption. The gradation curves for the alternative materials do not meet any of the standard requirements for both the British and the American standards. The physical properties for recycled concrete aggregates are different from those of conventional crushed aggregates. The grading of recycled aggregates contains material generally coarser than the conventional natural aggregates.

Therefore, based on this study, the alternative materials generally differ from the conventional aggregate material.

## CHAPTER FIVE

### 5.0 STUDIES ON LATERIZED QUARRY DUST CONCRETE MIXES

#### 5.1 Introduction

In this chapter, concrete mixes utilizing various combinations of materials are evaluated. The main aim is to determine the optimum combination of the materials to be used in making of concrete beams for further testing and investigation. Results of flexural test on reinforced concrete beam are presented in chapter seven. Starting with the conventional concrete, river sand and natural crushed aggregates are separately replaced with quarry dust and laterite for the fine materials and recycled concrete aggregates for the coarse aggregates, respectively. The quarry dust proposed for concrete production is the waste material that is produced when natural crushed aggregates are manufactured. Properties of concrete used in evaluating the effect of these alternative concrete materials were the wet concrete workability properties, i.e. the slump test and the compacting factor test and compressive strength test on the hardened concrete. The physical properties of all the materials used in the study have been discussed in chapter three. Both the slump and compacting factor test were used in order to allow comparison of the results. The materials under investigation are unconventional and hence there was need to use two different methods one as a confirmation test.

The use of quarry dust in concrete has been investigated and found to produce good results (Nisnevich *et al.*, 2007). Udoeyo *et al* (2006) demonstrated that concrete with up to 40% replacement level of sand by laterite attained the designed strength

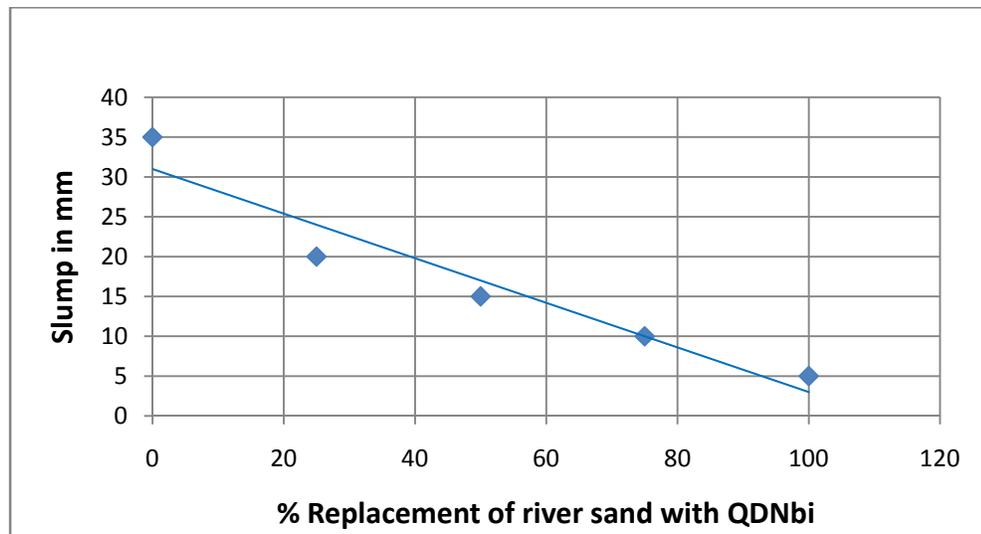
of 20 N/mm<sup>2</sup>. It is also generally accepted that when natural sand is used, up to 30% of natural crushed aggregate can be replaced with coarse recycled concrete aggregate without significantly affecting the mechanical properties of the concrete (Hansen, 1992). So far no attempt has been made to study the performance of a combination of these materials. No data is also available on the performance of concrete beams utilizing these alternative concrete materials. This chapter therefore presents data on the properties of concrete incorporating these alternative materials.

## **5.2 Results and discussion**

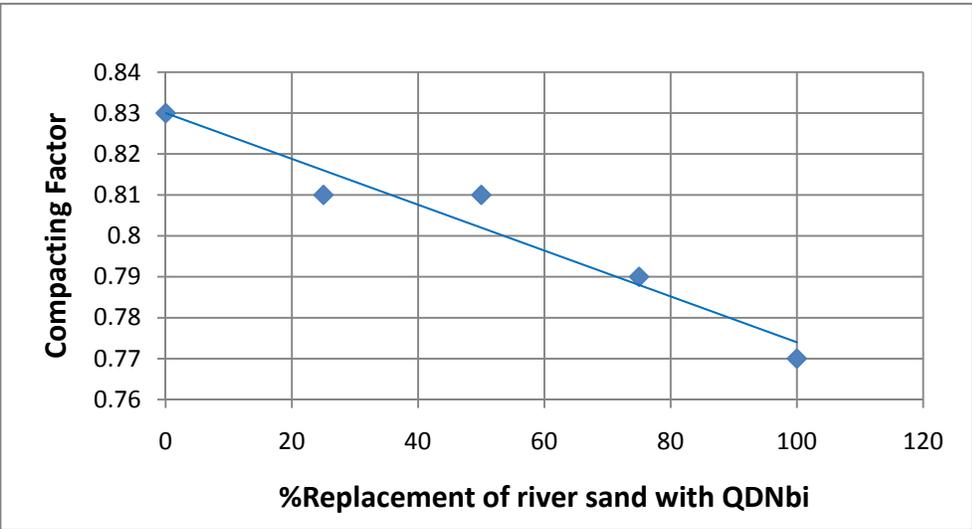
### ***5.2.1 Optimum quarry (QDNbi) dust content***

To carry out this test, river sand in conventional concrete of mix ratio 1:1.5:3 for Portland cement, river sand and natural aggregates respectively, was partially replaced with 0, 25, 50, 75 and 100% of quarry dust. Figure 5.1 shows the The effect of QDNbi content on slump in a concrete mix. From the figure it can be shown that the slump of fresh concrete was reducing with increasing amount of QDNbi in the mix. This was due to the large amount of fine material present in the quarry dust as compared to the river sand (Mulu *et al.*, 2003). Fine materials generally require much water to coat the individual particles of the materials in order to provide the necessary lubrication between the particles that is needed to allow easy movement during placing of the concrete. This trend in the slump test was confirmed by the compacting factor test results (Figure 5.2). The results showed that there was a reduction in compacting factor of concrete with increase in the quarry dust content.

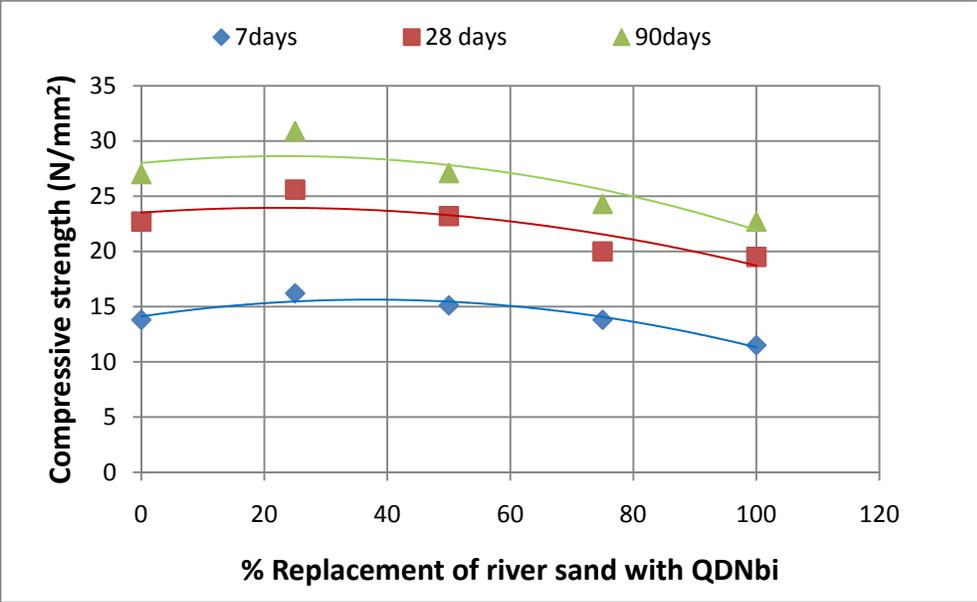
Figure 5.3 on the other hand shows the variation of compressive strength of concrete with replacement levels of quarry dust. From the figure, compressive strength of concrete initially increased to a peak value at about 30% replacement levels. The compressive strength then reduced again to a minimum value at 100% replacement. This was evident at all the three ages of testing. The maximum compressive strength attained at the age of 28 days was 24 N/mm<sup>2</sup> at a replacement level of 30%. At this replacement value, the slump of the wet concrete was 22 mm for water/cement ratio of 0.65. Ilangoan *et al* (2007) quoted an optimum value occurring when 40% of river sand is replaced with quarry dust.



**Figure 5.1** The effect of QDNbi content on slump



**Figure 5.2** Variation of compacting factor with QDNbi content



**Figure 5.3** Variation of compressive strength with QDNbi content

The results also reveal that increase in the amount of quarry dust in concrete mix leads to reduction in workability of wet concrete. The quarry dust material as used in this study, contain a lot of finer materials as compared to the natural river sand. The presence of fine material in the concrete, demands for extra water content in order to

achieve a specified level of workability (Mulu *et al.*, 2003). At low replacement levels of river sand with quarry dust, the fine material introduced in the concrete mix has the effect of filling the small void spaces in the concrete which remain when conventional materials are used.

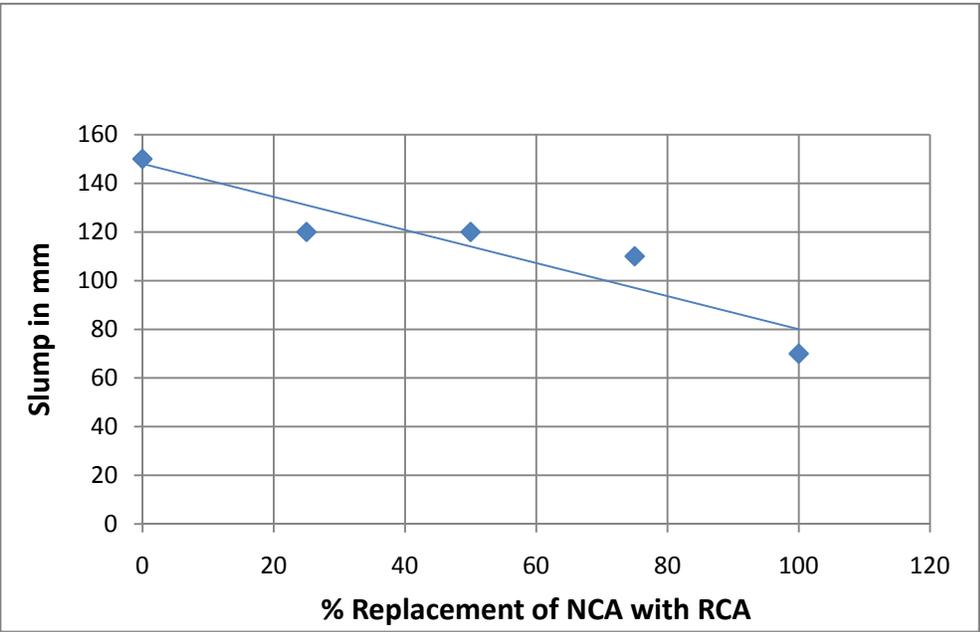
Researchers of high performance concrete have discovered that one of the methods of achieving high strength in concrete is to ensure that the fine void in concrete are filled as much as possible (Ilangovan *et al.*, 2007). This has led to introduction of fine waste material into high performance concrete. Although material such as ground granulated blast furnace slag and silica fumes have cementitious properties, they have an effect of filling any fine pores in concrete hence leading to a very compact high strength concrete. Therefore, the consequence of introducing quarry dust in small amounts is to increase the compressive strength of concrete. At high replacement, however, there is a reduction in compressive strength of concrete as the amount of quarry dust increases. Concrete containing large amount of quarry dust have low workability which consequently affects placing and compaction of the concrete. Compaction of concrete has great influence on compressive strength of concrete (Neville, 1993).

### ***5.2.2 Optimum recycled concrete aggregates (RCA) content***

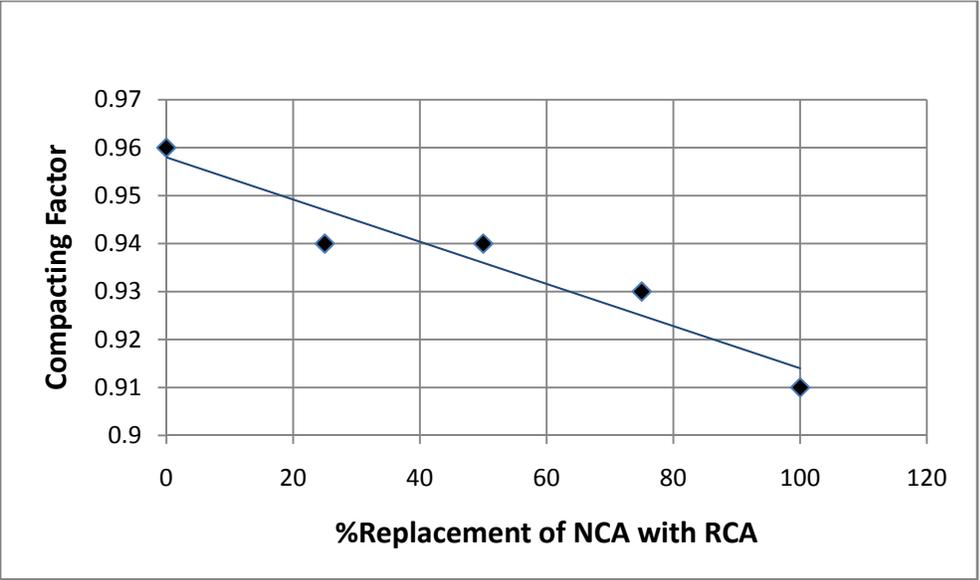
To determine the optimum RCA, the natural crushed aggregate in conventional concrete of mix ratio 1:1.5:3 for cement, river sand and natural crushed aggregates, respectively, was partially replaced with 0, 25, 50, 75 and 100% of recycled concrete aggregates.

The relationship between the slump of wet concrete and recycled concrete aggregates content is shown in Figure 5.4. The figure shows that the slump of wet concrete was reducing with increase in replacement of natural crushed aggregates with recycled concrete aggregates. This trend in concrete slump was confirmed by the results from compacting factor test as shown in Figure 5.5. The reduction of workability of fresh concrete is due to the high water absorption behaviour of the recycled concrete aggregates (Limbachiya *et al.*, 2004). Figure 5.6 shows the effect of recycled concrete aggregates content on the compressive strength of concrete with. The figure shows that compressive strength of concrete is not significantly affected by the change in the amount of recycled concrete aggregates in the mix.

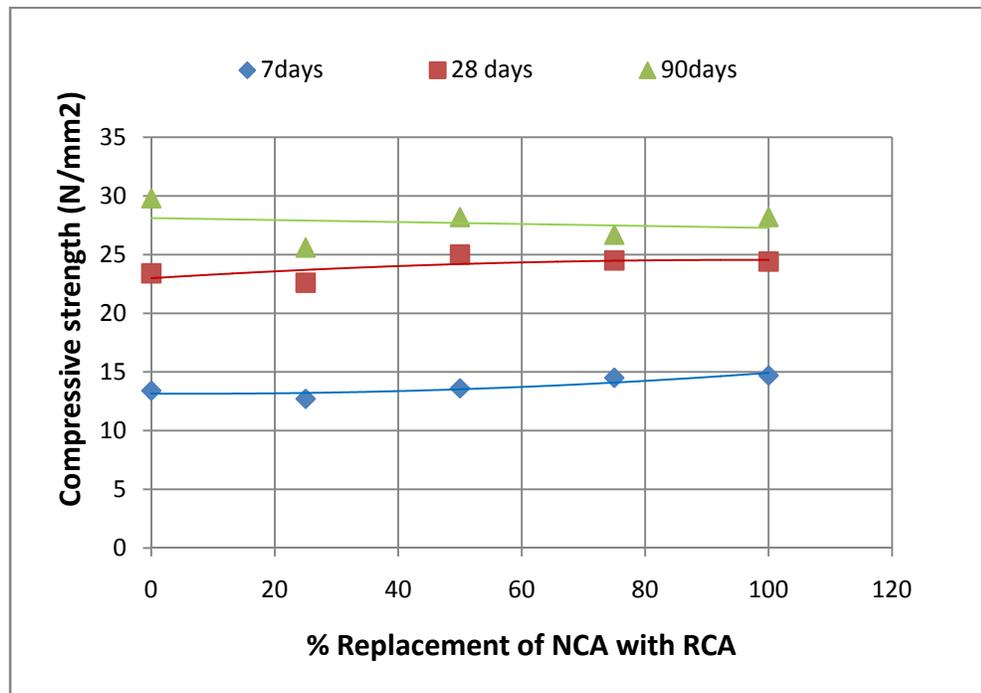
It has been reported that when river sand is used, up to 30% of natural crushed aggregates can be replaced with recycled concrete aggregates (ECCO, 1999). Since there is no significant effect of the recycled concrete aggregates on the compressive strength of concrete, total replacement of natural crushed aggregates by recycled concrete aggregates is adopted in this study. At 100% replacement level, the workability of concrete is practically suitable. The slump recorded at this level was 80 mm. The alternative concrete used to cast beams contains recycled concrete aggregates as coarse aggregates.



**Figure 5.4** The effect of RCA content on slump



**Figure 5.5** The effect of RCA content the compacting factor



**Figure 5.6** Variation of compressive strength with RCA content

Recycled concrete aggregates contain natural crushed aggregates and cement sand paste used in the parent concrete material. The use of recycled concrete aggregates in industrially advanced countries has been accepted. The results have indicated that the workability of fresh concrete reduces with increase in the amount of recycled concrete aggregates in the concrete mix. The cement sand paste attached on the natural aggregates has high water absorption. Some of the mixing water is therefore held up by the recycled concrete aggregates and is not available for mixing the concrete materials. Water absorption of recycled concrete aggregates was reported to be 2 to 3 times higher than the corresponding natural aggregates (Limbachiya *et al.*, 2004).

The results reveal that compressive strength of concrete is not affected by increase in amount of recycled concrete aggregates in the concrete. The recycled concrete aggregates have a gradation curve suitable for good quality concrete. Neville (1993) has demonstrated that the strength properties of concrete greatly depend on the strength and gradation of coarse aggregates.

Coarse aggregates account for the largest proportion of concrete mix. Concrete with high content of recycled concrete aggregates has lower density than when low amount of the recycled aggregates are used. Recycled concrete aggregates have lower density than the natural crushed aggregates. The loose bulk density of recycled concrete aggregates and natural crushed aggregates were 1150 and 1300 kg/m<sup>3</sup>, respectively. These results are in agreement with earlier work of Limbachiya *et al* (2004). In their study, while investigating on the performance of recycled concrete aggregates, Limbachiya *et al.* (2004) reported the loose bulk density for recycled concrete aggregates and natural crushed aggregates as 1190 and 1340 kg/m<sup>3</sup>, respectively. In this work, various concrete mix proportions targeting specific compressive strength were investigated. For the target strength ranging from 0 to 35 N/mm<sup>2</sup>, there was no effect on the compressive strength of concrete at 28 days with increasing amount of recycled concrete aggregates content in the mix (Limbachiya *et al.*, 2004).

### ***5.2.3 Optimum laterized quarry dust content***

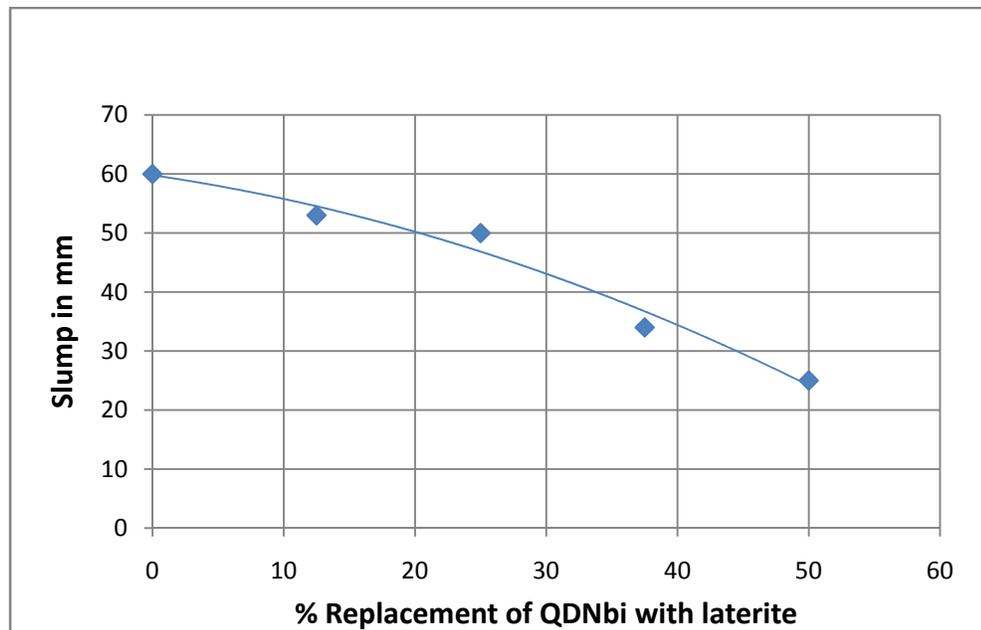
The performance of laterite as fine material replacement in concrete has been studied and found to produce good results. Udoeyo *et al* (2006) demonstrated that

concrete with up to 40% replacement level of sand by laterite attained the designed strength of 20 N/mm<sup>2</sup>. The present study aims at utilizing lateritized quarry dust as river sand replacement in concrete. In concrete of mix ratio 1:1.05:0.45:3 for Portland cement, river sand, quarry dust and natural crushed aggregates respectively, the amount of quarry dust was partially replaced with 0, 12.5, 25, 37.5 and 50% of laterite material. This was aimed at evaluating the behaviour of the combined material when natural crushed aggregates are used. In Figure 5.7, it can be seen that slump of wet concrete was reducing with increasing amount of laterite in the mix. Laterite material generally has a lot of fine material that demand for much water in order to achieve a specified level of workability. This trend in workability was confirmed by the compacting factor test results as shown in Figure 5.8.

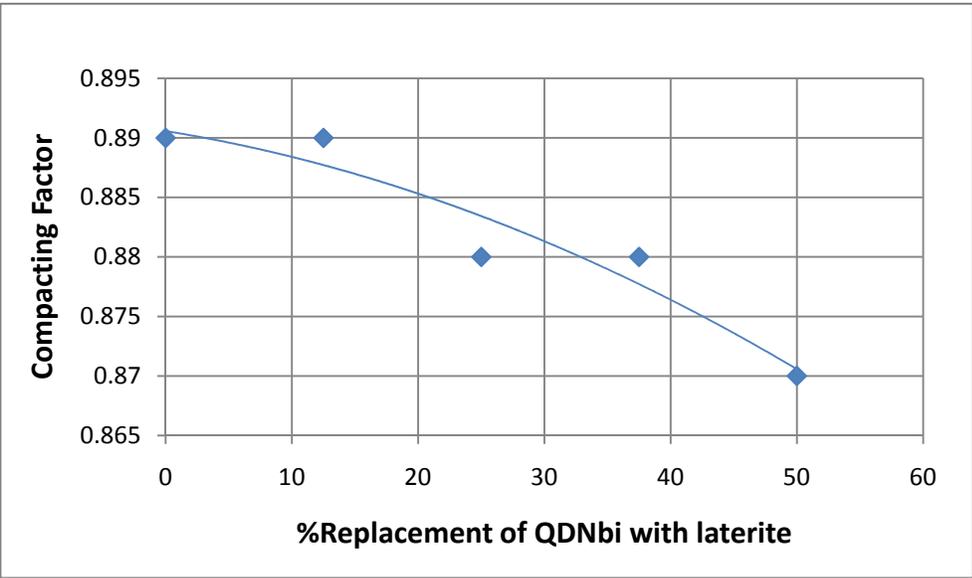
The variation of compressive strength of concrete with the laterite content is shown in Figure 5.9. The 28 days compressive strengths were 20.1 and 17.7 N/mm<sup>2</sup> for 0 and 50% replacement levels, respectively. The results reveal that, compressive strength was reducing with increasing amount of laterite in the concrete mix.

From the results above, it was not clear as to the optimum amount of laterite to replace the quarry dust selected to account for the 30% of the total fine materials in concrete. The above study was repeated with recycled concrete aggregates as coarse aggregates. The quarry dust in the concrete containing 30% of quarry dust replacing river sand was in turn replaced with 0, 20, 40 and 60% of laterite material. Figure 5.10 to 5.12 show the effect of laterite content on slump, compacting factor and compressive strength respectively. Figure 5.10 shows that when recycled concrete

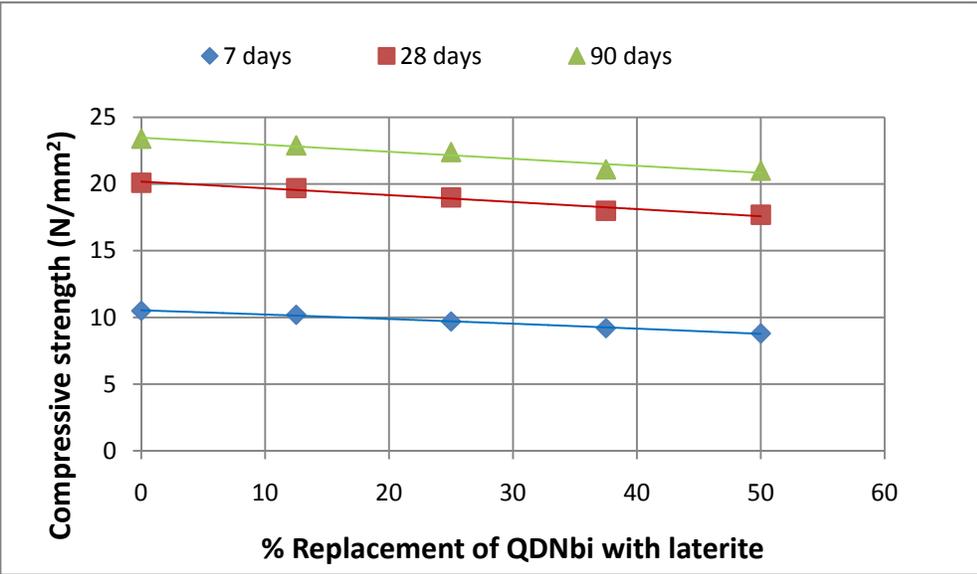
aggregates are used, the slump of wet concrete reduced with increasing laterite content. The compacting factor of the concrete was also reducing with increase in the laterite content. From Figure 5.12, the variation of compressive strength of concrete with laterite content was different. The strengths at 28 days revealed that the values ranged from 20.6 to 20.8 N/mm<sup>2</sup> implying that there was no significant effect of the laterite content on the compressive strength when recycled concrete aggregates are utilized. Strengths at all ages of testing revealed similar behaviour.



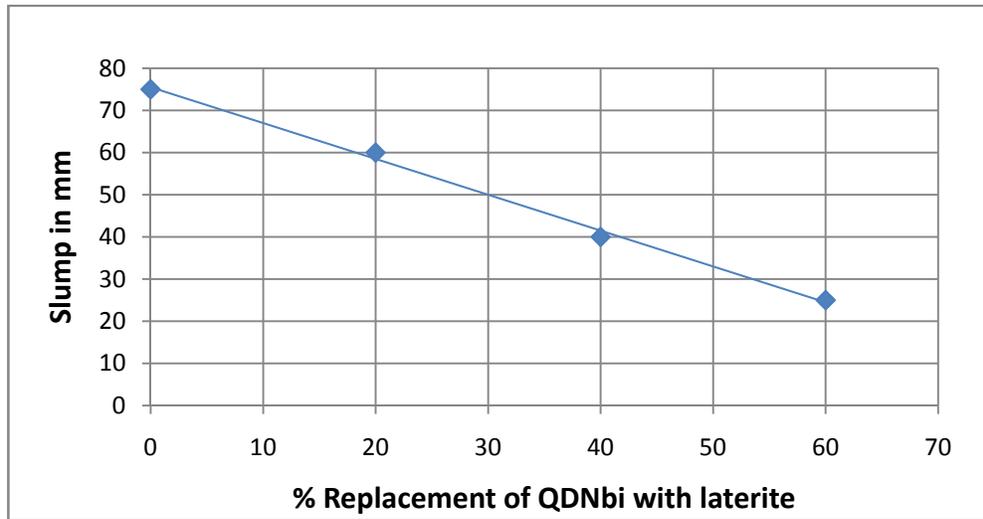
**Figure 5.7** The effect of laterite content on slump and NCA as coarse aggregates



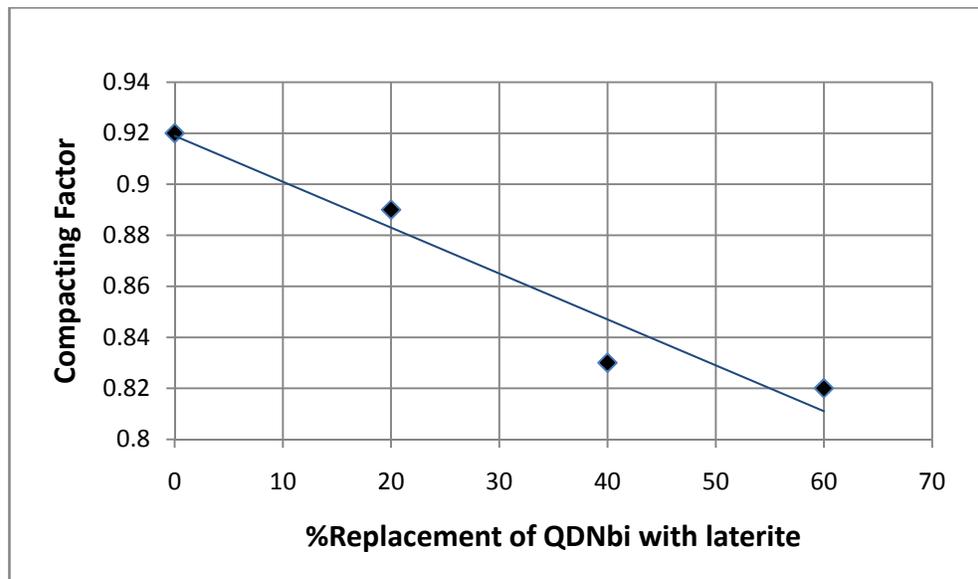
**Figure 5.8** Variation of compacting factors with laterite content and NCA as coarse aggregates



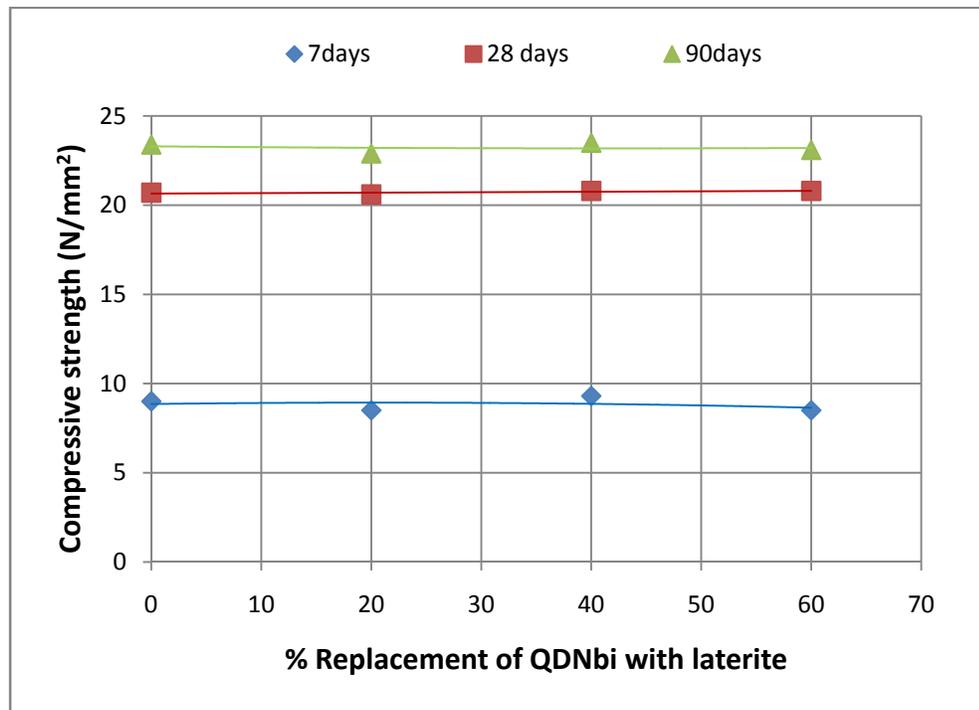
**Figure 5.9** The relationship between compressive strength and laterite content and NCA as coarse aggregates



**Figure 5.10** The effect of laterite content on slump and RCA as coarse aggregates



**Figure 5.11** Variation of compacting factors with laterite content and RCA as coarse aggregates

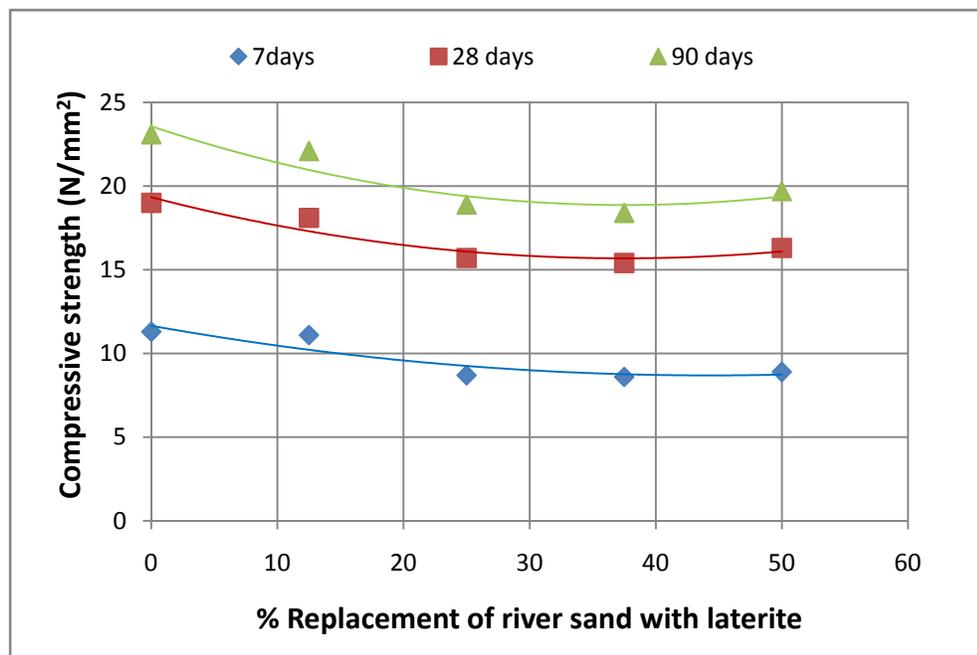


**Figure 5.12** The relationship between compressive strength and laterite content and RCA as coarse aggregates

The results show that the properties of fresh concrete are similar for the concrete utilizing recycled concrete and natural aggregates as coarse materials. The workability of fresh concrete was reducing with increasing amount of laterite content in the concrete mix. Generally the laterite material contains more fine particles than conventional fine aggregates. This leads to increased water absorption for this material. The absorbed water in the laterite is therefore not available for mixing the concrete constituent materials.

When natural aggregates are used, the compressive strength of concrete was found to reduce with increasing replacement level of river sand with laterite. Figure 5.13 shows the variation of compressive strength with the laterite content when natural aggregates are used. The trend is evident at all ages of testing. The compressive

strengths at 50% replacement level of river sand with laterite are about 85% the values for conventional concrete. The aim of this study was to modify the quarry dust with laterite while utilizing recycled concrete aggregates as coarse aggregates. The results show that when recycled concrete aggregates are used, the strength of concrete was not affected by the increase in the replacement level of quarry dust accounting for 30% of fine material with the laterite. The recycled materials act together to form a material that blend together to achieve the required strength. However, the workability of fresh concrete reduces with increasing amount of laterite content in the concrete.



**Figure 5.13** The relationship between compressive strength and laterite content and NCA as coarse aggregates

### **5.3 Summary and conclusion**

The results of studies on the properties of concrete that utilizes various combinations of alternative and conventional material have been presented in this chapter. The main aim was to determine the optimum combination of material that will result in the best alternative concrete for construction of buildings. When natural aggregates are used as coarse aggregates, 30% of river sand can be replaced with quarry dust and still attain a compressive strength of  $24 \text{ N/mm}^2$  at 28 days. The slump of fresh concrete at this replacement level is satisfactory for concrete placed and vibrated with internal vibrator. However, much higher amount of replacement level of river sand with quarry dust significantly affects the workability of the concrete. This has a negative effect on the compressive strength of concrete.

Recycled concrete aggregate content in concrete has no significant effect of the compressive strength. However, the workability of fresh concrete decreases with increase in the amount of recycled aggregates. Hundred per cent (100%) replacement of natural aggregates is therefore considered in subsequent studies. The compressive strength of concrete utilizing river sand and recycled aggregates was  $25 \text{ N/mm}^2$  at 28 days.

The results revealed that when recycled concrete aggregate is used, alternative concrete containing river sand, quarry dust and laterite in the ratio of 70:15:15 respectively has the best performance in terms of strength and workability considerations. The compressive strength of the concrete at 28 days is  $20.8 \text{ N/mm}^2$ .

Although the materials physical properties of alternative aggregates differ from the conventional material, alternative concrete made from such material, have adequate engineering properties that meet the requirements for building construction.

## CHAPTER SIX

### 6.0 STUDIES ON LATERIZED QUARRY DUST BLOCK MIXES

#### 6.1 Introduction

This chapter evaluates the performance of laterized quarry dust masonry blocks. The proposed block mix consist of quarry dust from natural crushed aggregates waste (QDNbi), the quarry dust from stone dressing plant (QDNdar), laterite and Portland cement as a stabilizer. The aim of the chapter is to arrive at the optimum combination of the various materials that will result in the best masonry blocks to be utilized in masonry wall panels for further investigation as presented in chapter eight. The physical material properties for QDNbi, QDNdar and laterite are presented in chapter four. The properties used to evaluate the performance of masonry blocks were the water absorption and compressive strength of the blocks at 7, 28 and 90 days age respectively.

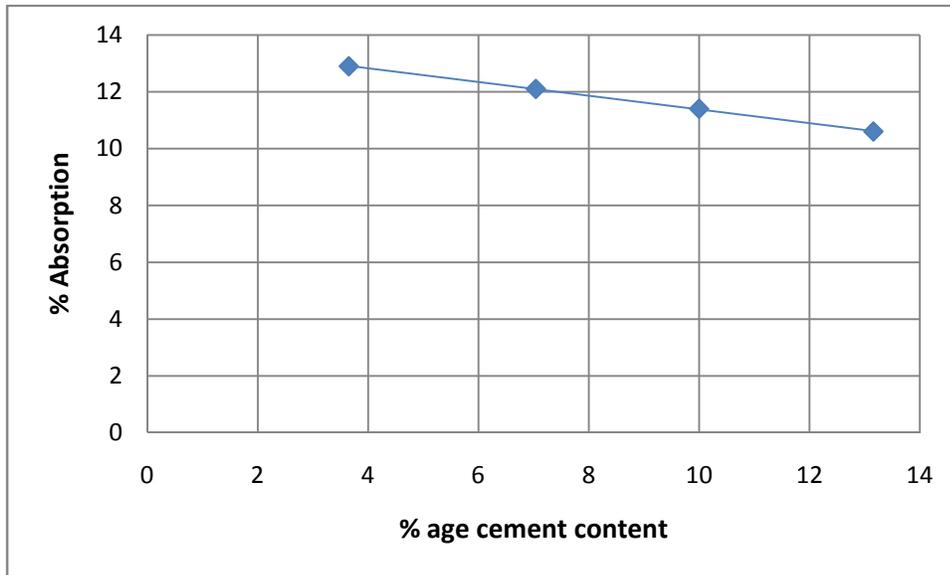
For masonry blocks to function satisfactory, the compressive strengths and water absorption have to be within the acceptable limits of performance. Previous work by Musiomi *et al* (2007) reported that 7% of cement content based on weight batching can be used to stabilize laterized quarry dust and attain compressive strength of at least 2.5 N/mm<sup>2</sup>. The water absorption of the blocks was satisfactory based on the performance criteria specified in British standards.

## **6.2 Results and discussion**

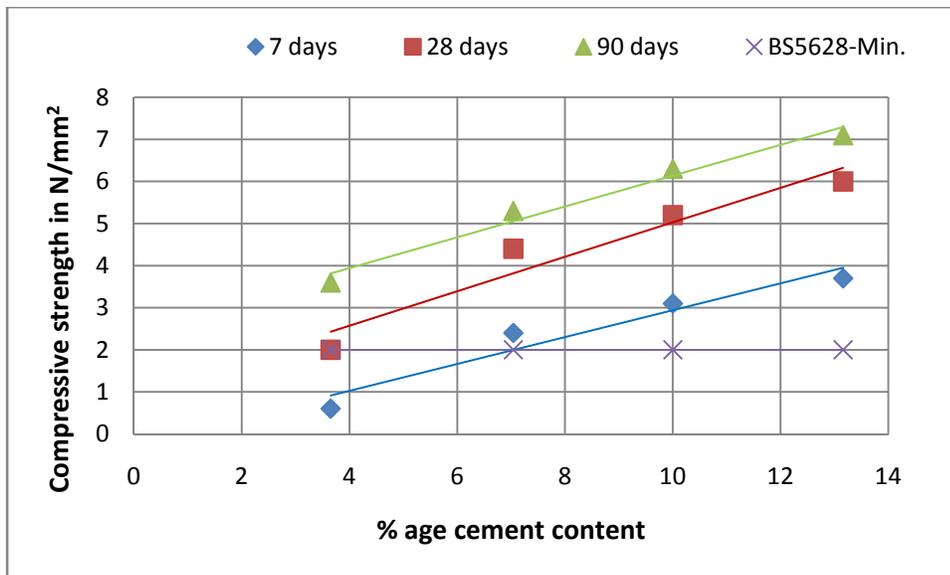
### ***6.2.1 Optimum cement content***

To determine the optimum cement content, laterized quarry dust mix was stabilized with various amounts of cement content. Laterite and quarry dust combined were mixed in the ratio of 1:1 with the ratio of QDNbi to QDNdar being maintained at 1:1. The stabilized quarry dust block mixes were then used to make machine pressed blocks which were cured at room temperature for 7 days and left to dry in a shed. The blocks were then tested for compressive strength on a Universal Testing Machine after 7, 28 and 90 days. Some blocks were investigated for water absorption qualities. Figure 6.1 shows the variation of water absorption with the cement content of the mix. From the figure, the water absorption of blocks utilizing laterized quarry dust reduced with increasing amount of cement content in the mix. The water absorption of the blocks were found to range from 12 to 10.6% for the 4 and 13% cement content, respectively.

The effect of cement content on the compressive strength of alternative block is shown in Figure 6.2. The figure shows that the compressive strength of blocks increased with increase in the cement content in the mix. This was evident at all ages of testing. When at least 3.65% of cement is used to stabilize laterized quarry dust, a minimum compressive strength of 2 N/mm<sup>2</sup> specified in BS5628-1 of 1992 can be attained at 28 days. The cement content of 7% with water absorption of 12% and compressive strength of 3.8 N/mm<sup>2</sup> is proposed to be used in further investigation.



**Figure 6.1** Variation of water absorption with cement content



**Figure 6.2** The effect of cement content on compressive strength of blocks

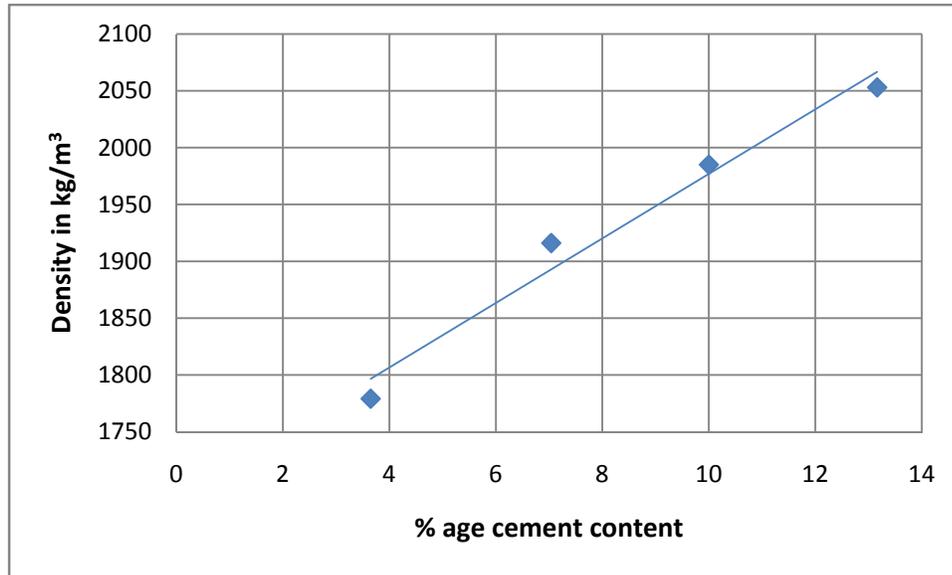
This level of cement stabilization leads to masonry blocks with adequate compressive strength without compromising on the water absorption quality of the material. Study by Musiomi *et al* (2007) revealed that when 7% cement is used to stabilize earth based masonry block, satisfactory performance in terms of water

absorption and compressive strength can be achieved. Higher percentages of cement content would yield better results but tend to be expensive because cement material is expensive compared to the rest of the materials

Masonry blocks containing large amount of Portland cement content generally have lower water absorption as compared to blocks with low amount of cement. The role of cement in the block mix is to bind the materials together thus reducing the air voids in the block fabric. Well bound materials have low water absorption as compared to material that is less bound and can easily disintegrate. Increased cement content leads to increased compressive strength of the blocks. This is due to the binding effect of the cement in the mix. Previous research work has demonstrated that increase in cement content in earth based material leads to higher compressive strength (Alutu *et al.*, 2006). This has been found to take place up to 13% cement content in the mix. Much higher cement content in the mix has little effect on the strength of the blocks. Advantage can therefore be taken of this fact to design economical blocks since cement material is usually the most expensive material in stabilized earth based blocks.

Cement content in block mix has influence on the density of the blocks. Figure 6.3 shows the effects of cement content on density of blocks. From the figure, increased cement content leads to increased density of the block. Studies on earth based blocks have found out that there is a relationship between the block density and the compressive strength of the blocks (Montgomery, 2002). The presence of high

cement content in the mix reduces the air voids in the material leading to high density.



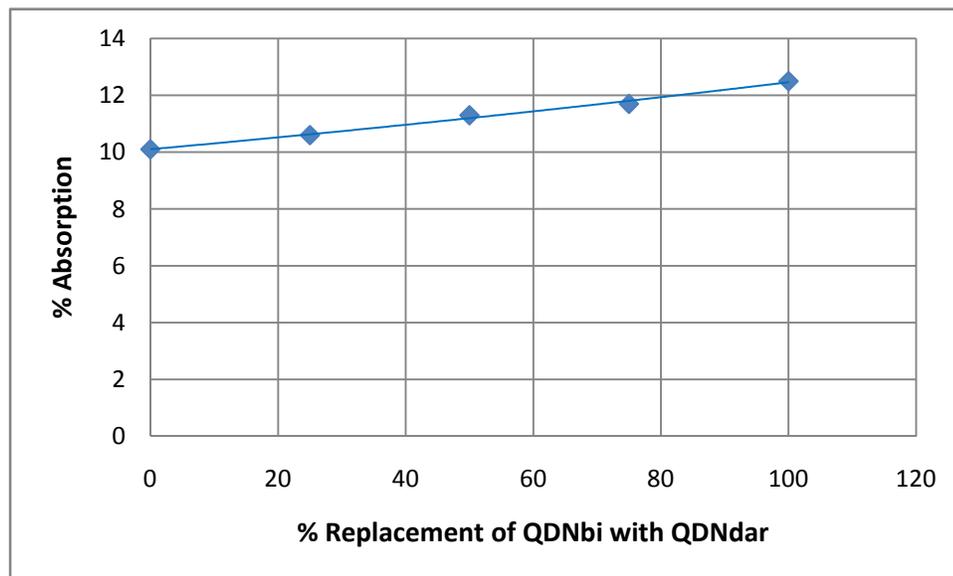
**Figure 6.3** The relationship between cement content and density of blocks

### ***6.2.2 Optimum quarry dust wastes combination***

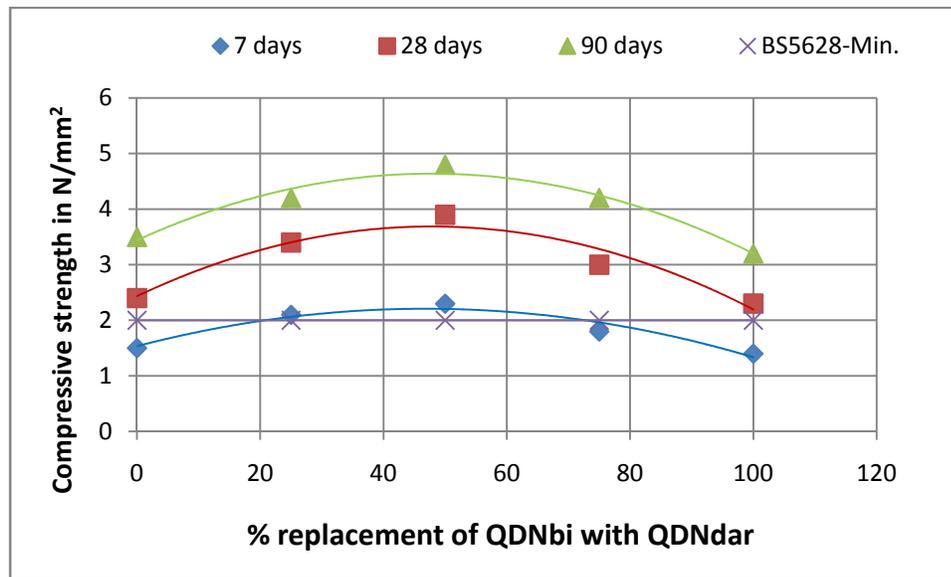
Optimum quarry dust combination was determined by evaluating block mixes containing various proportions of quarry waste from machine dressed blocks. In a block mix containing Portland cement, QDNbi and laterite in the ratio of 1:8:8 respectively, QDNbi was partially replaced with 0, 25, 50, 75 and 100% of QDNdar. Figures 6.4 and 6.5 show the effects of QDNdar content on the water absorption and the compressive strength of blocks, respectively. In Figure 6.4, it is shown that the water absorption of the blocks was increasing with increase in QDNdar content in the mix. The range in water absorption was 10.1 to 12.5% for 0 and 100% replacement level respectively. Figure 6.5 on the other hand shows that compressive

strengths of blocks increased with increase in QDNdar content up to maximum values at 50% replacement level. Further increase in QDNdar resulted to a decrease in the compressive strengths. This was evident at all the three ages of testing. The compressive strengths of blocks at 50% replacement level were 2.3, 3.9 and 4.8 N/mm<sup>2</sup> for 7, 28 and 90 days respectively. From the figure it can also be noted that when 50% of QDNdar is used, a minimum strength of 2.3 N/mm<sup>2</sup> is attained.

Based on the results, 50% replacement level of QDNbi with QDNdar is proposed for use in masonry blocks. At this replacement level, the water absorption and compressive strength of blocks was 11.3% and 4.8 N/mm<sup>2</sup>, respectively. In subsequent study, the two quarry dust components were kept equal in proportions.



**Figure 6.4** Variation of water absorption with QDNdar content

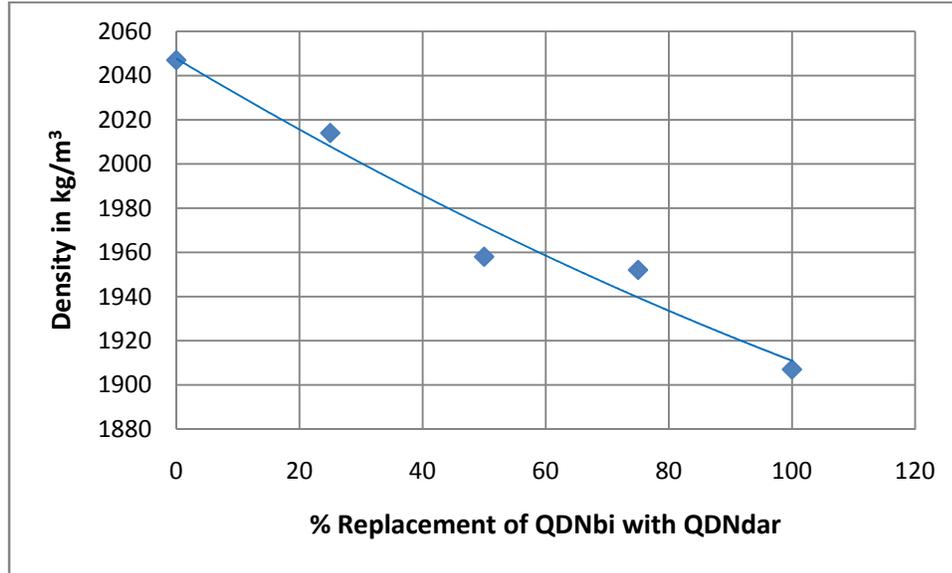


**Figure 6.5** The effect of QDNdar content on compressive strength of blocks

QDNdar generally comes from soft rock as compared to the QDNbi which originate from the processing of the natural crushed coarse aggregates. Increasing the amount of QDNdar in the mix leads to increased water absorption of the blocks. This is due to the high water absorption of the QDNdar as compared to QDNbi. Large amount of QDNdar in the mix reduces the density of the blocks. QDNdar was found to have lower density than QDNbi. This was evident both for the bulk and the specific densities for the materials. The effect of QDNdar content on density of blocks is shown in Figure 6.6.

The variation of the compressive strength of the blocks with QDNdar content revealed that there is increase in strength with increase in QDNdar initially up to 50% replacement level of QDNbi with QDNdar. Further increase in QDNdar content in the mix lead to reduced compressive strength of blocks. Initially,

introduction of QDNdar which contain fine material improves the particle interlock leading to a more bound material. At higher QDNdar content, the softer material dominates in the block mix leading to reduced compressive strength of the blocks.



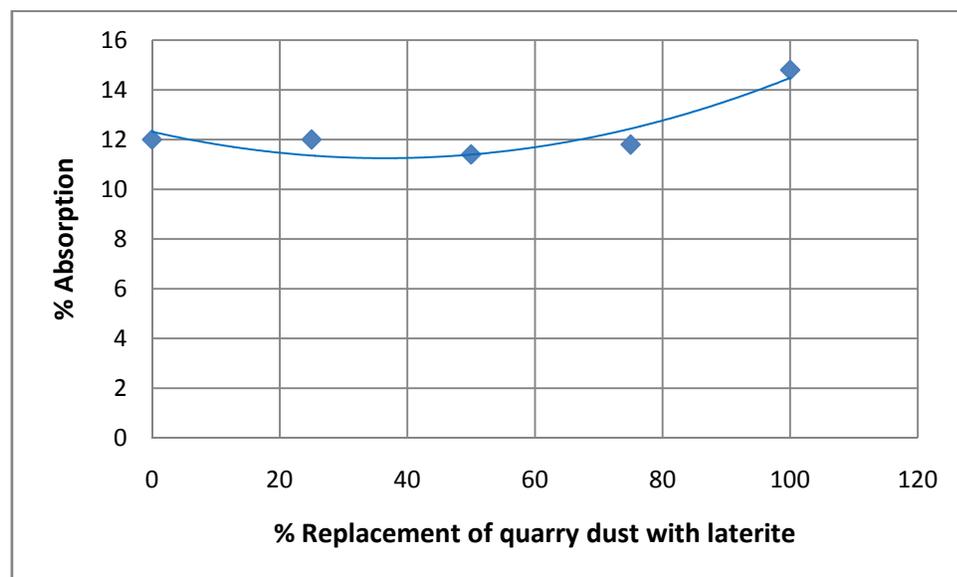
**Figure 6.6** Variation of the density of blocks with QDNdar content

### **6.2.3 Optimum laterite content**

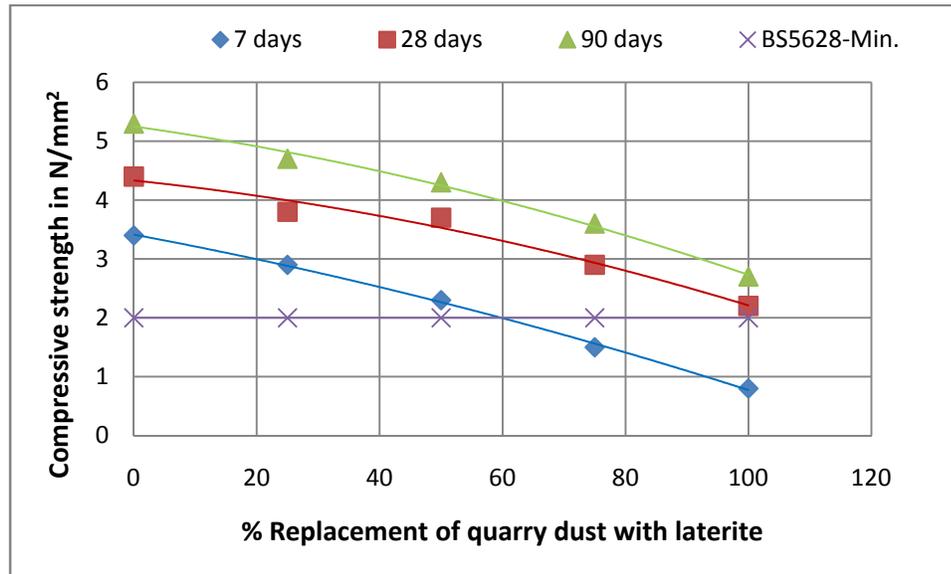
To determine the optimum laterite content, quarry dust material in block mix of cement, QDNbi and QDNdar in the ratio of 1:8:8, respectively, was partially replaced with 0, 25, 50, 75 and 100% of laterite. Figure 6.7 shows the effect of laterite content on water absorption of the blocks. From the figure it can be seen that the water absorption initially reduced with increase in laterite content to a minimum value when 50% quarry dust was replaced with laterite. Further increase in laterite content increased the water absorption of the blocks until a maximum value was attained at 100% replacement level. A minimum absorption of 11.4% was attained at 50% replacement by laterite. The highest water absorption of 14.8% was attained

when laterite is used alone without quarry dust. This value is very high compared to the limits specified in British standards. The results show that when the materials are combined, better absorption performance levels are attained as compared to that of individual materials.

The variation of compressive strength of blocks with laterite content in the block mix is shown in Figure 6.8. From the figure it can be seen that compressive strength of blocks decreases with increase in the amount of laterite content in the mix. The trend was evident at 7, 28 and 90 days age of testing. At 7 days, compressive strength varied from 3.5 N/mm<sup>2</sup> when 0% laterite was used to 0.8 N/mm<sup>2</sup> when 100% laterite was used. The corresponding values at 28 days were 4.5 N/mm<sup>2</sup> and 2.2 N/mm<sup>2</sup>, respectively. The figure also shows that when up to 50% of quarry dust is replaced with laterite, a minimum compressive strength of 2.0 N/mm<sup>2</sup> is attained at the age of 7 days.



**Figure 6.7** Variation of water absorption with laterite content



**Figure 6.8** Variation of compressive strength of blocks with laterite content

Examining Figures 6.7 and 6.8, the compressive strength has a downward trend while the water absorption decreases to a minimum when 40% of quarry dust is used, and then increases with increase in replacement levels. Therefore the optimum combination of the materials is achieved when 40% of quarry dust is replaced with laterite. At this replacement level, the blocks attained compressive strength of 2.4 N/mm<sup>2</sup> is attained at 7 days and also results in the minimum water absorption value of 11.4. The minimum strength of 2 N/mm<sup>2</sup> criteria is based on the requirements of BS5628-1 of 1992.

The water absorption of blocks initially reduces to a minimum then increases with increase in the laterite content in the block mix. This is generally due to interaction between the materials in the mix. Initially, the small amount of laterite present in the mix improves the cohesion between the materials. Further increase in laterite

content has the effect of increasing the water absorption of the blocks as a result of the high water absorption of the laterite material as compared to the quarry dust. Laterite material if present in large amount leads to reduction in compressive strength of blocks. Maximum compressive strength was attained when stabilized quarry dust without laterite is used. Masonry blocks utilizing cement stabilized quarry dust have been used in conventional masonry construction. Other precast items made from cement stabilized quarry dust are paving slabs and hollow blocks among others (ILO, 1990). The grading of quarry dust makes it to have better inter-particle bond than the laterite. Laterite generally contains soft fine material that has the effect of reducing the compressive strength of the blocks.

### **6.3 Summary and conclusion**

The performance of alternative masonry blocks utilizing various combinations of quarry dust and laterite material has been presented in this chapter. The optimum combination of the material was determined in the study. The evaluation criteria for the blocks were the compressive strength and water absorption of the blocks. The results indicated that masonry blocks containing alternative material in the ratio of 1:1:2 for QDNbi, QDNdar and laterite, respectively have better performance in terms of strength and water absorption. Seven per cent (7%) of cement stabilization was found to yield compressive strength of at least  $2 \text{ N/mm}^2$  when tested after 7 days from the day of preparing the blocks. This level of stabilization was therefore used in further studies.

## CHAPTER SEVEN

### 7.0 REINFORCED CONCRETE BEAMS

#### **7.1 Introduction and beam details**

This chapter presents the results of reinforced concrete beams tested to evaluate their flexural behaviour. Central moment versus central deflection for beams with various variables is evaluated. A total of eight reinforced concrete beams were tested. The main variables in the beams were: the type of materials, that is, the alternative concrete and the conventional concrete; the strength of the concrete and the reinforcement steel content.

The main aim of this study was to compare the structural behaviour of reinforced concrete beams made from the alternative concrete with the conventional concrete. The concrete beams tested were measuring 1200x150x200 mm in length, breadth and overall height, respectively. During testing the beams were supported on rollers and loaded at the centre with a point load on the Universal Testing Machine. The clear span of the beams was one metre. The beams were doubly reinforced. Shear reinforcement was also provided in the beams to enhance the shear capacity of the beams. The single point loading of the beams induces shear stresses in all the sections of the beams. The shear reinforcement was proportioned so that the beams fail in flexure before shear failure occurs. The specific details for the beams are shown in Table 7.1.

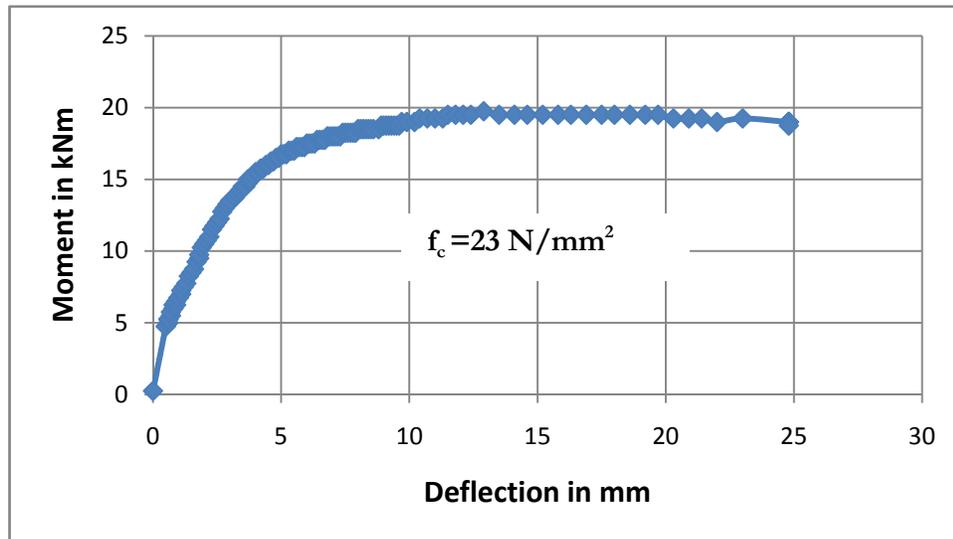
**Table 7.1** Beam details

| Beam                | Material     | Concrete Mix | Compressive strength (N/mm <sup>2</sup> ) | Bottom reinforcement |             |
|---------------------|--------------|--------------|---|----------------------|-------------|
|                     |              |              |   | Rebars               | % age steel |
| ConvConc1-2-4       | Conventional | 1:2:4        | 18  | 2Y10                 | 0.523       |
| ConvConc1-1.5-3(8)  | Conventional | 1:1.5:3      | 23  | 2Y8                  | 0.333       |
| ConvConc1-1.5-3(10) | Conventional | 1:1.5:3      | 23  | 2Y10                 | 0.523       |
| ConvConc1-1-2       | Conventional | 1:1:2        | 28  | 2Y10                 | 0.523       |
| AltConc1-2-4        | Alternative  | 1:2:4        | 17  | 2Y10                 | 0.523       |
| AltConc1-1.5-3(8)   | Alternative  | 1:1.5:3      | 21  | 2Y8                  | 0.333       |
| AltConc1-1.5-3(10)  | Alternative  | 1:1.5:3      | 21  | 2Y10                 | 0.523       |
| AltConc1-1-2        | Alternative  | 1:1:2        | 26  | 2Y10                 | 0.523       |

## 7.2 Results and conclusion

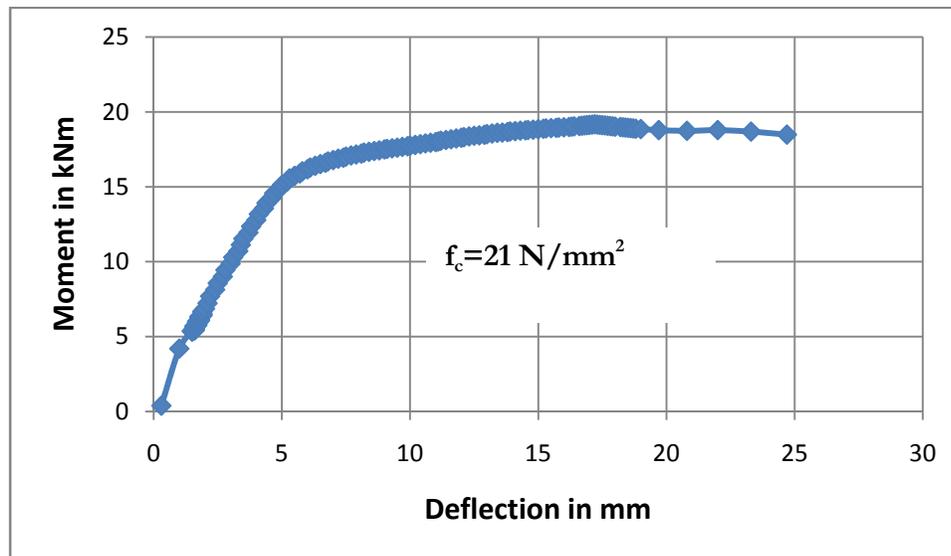
### 7.2.1 Moment versus central deflection

Figure 7.1 shows the moment versus deflection curve for conventional concrete beam reinforced with 2Y10 bars at the bottom. The span moment in the beam was determined by applying statics at every applied load level. The compressive strength of concrete in the beam is 23 N/mm<sup>2</sup>. From the figure, deflection in the beam increased with moment until failure occurred when the applied moment reached 19.5 kNm. The cracking moment was much lower than the ultimate moment for the beam. The figure also shows that the beam sustained the maximum moment from a deflection of 10 to 25 mm.



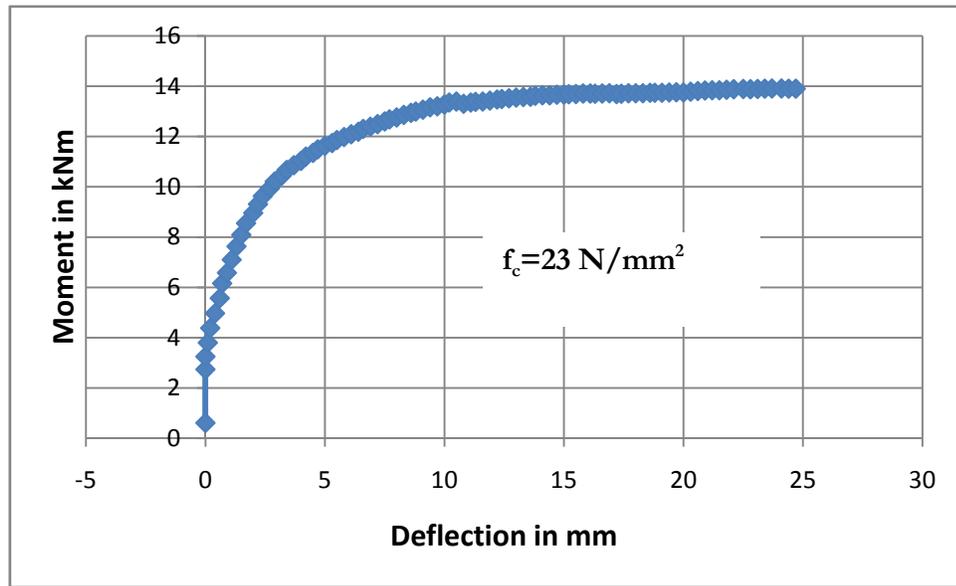
**Figure 7.1** Moment versus deflection of conventional concrete beam with mix ratio 1:1.5:3 reinforced with 2Y10

The variation of deflection with applied moment of alternative concrete beam with mix ratio 1:1.5:3 is shown in Figure 7.2. The beam was reinforced with 2Y10 bars at the bottom. The figure shows that deflection increased with increased moment within the moment application range of 0-15 kNm. The initial primary or main cracks formed at 15 kNm. The formation of cracks lead to a change in the gradient of the curve, larger deflection being experienced for slight increase in applied moment. The beam continued to resist higher moment in spite of the main cracks further increasing in size. At an ultimate moment of 19.6 kNm, the beam deflection increased at constant moment indicating that the steel reinforcement was yielding. After undergoing rapid deflection, the beam failed as the tensile steel reinforcement broke. The failure moment for the alternative concrete beam was comparable to conventional beam containing similar reinforcement even though there was a slight difference in the compressive strength of the concrete.



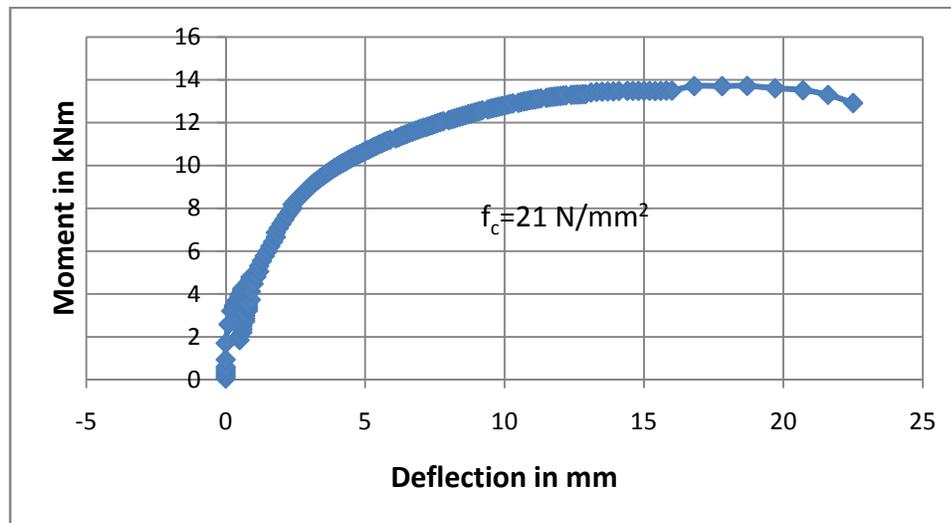
**Figure 7.2** Moment versus deflection of alternative concrete beam with mix ratio 1:1.5:3 reinforced with 2Y10

The relationship between moment and central deflection for conventional concrete beam with mix ratio 1:1.5:3 and reinforced with 2Y8 at the bottom shows two main parts, the parabolic portion and the linear portion (Figure 7.3). The figure indicates that initially deflection increased with increasing moment. The failure moment of 14 kNm for the conventional concrete beam with 2Y8 bars was lower than 19.5 kNm for the beam with 2Y10 bars.



**Figure 7.3** Moment versus deflection of conventional concrete beam with mix ratio 1:1.5:3 reinforced with 2Y8

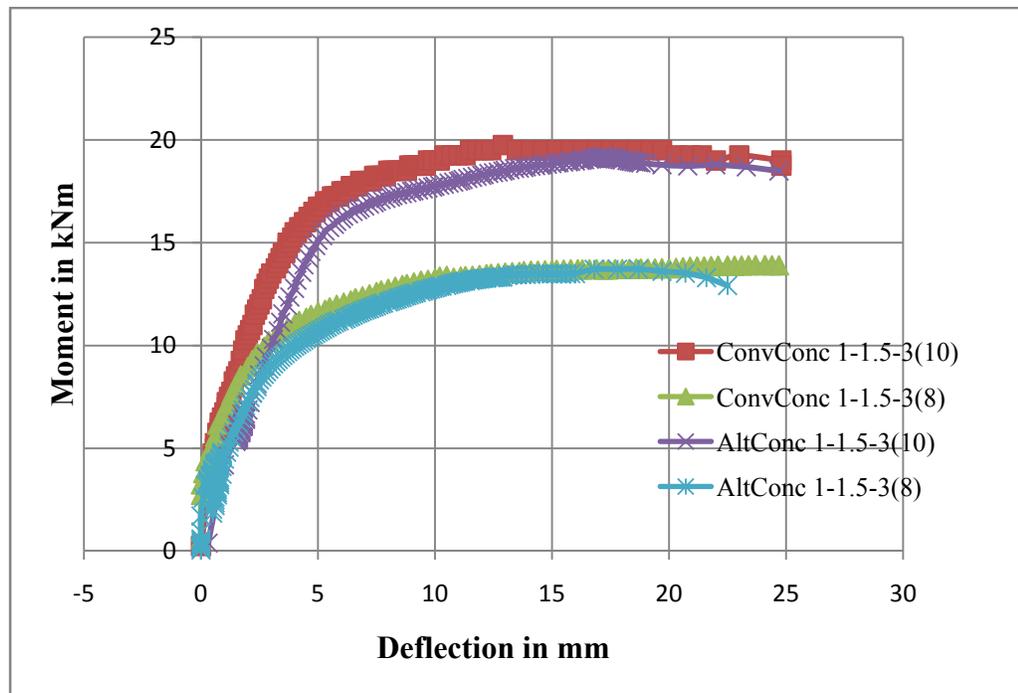
In Figure 7.4, moment versus deflection for alternative concrete beam of mix ratio 1:1.5:3 and reinforced with 2Y8 bars is presented. The figure shows that there is increase in deflection with applied moment at an increasing rate thus resulting to a parabolic shaped curve with a peak moment of 14 kNm. The failure moment of 14 kNm is equal to conventional concrete beam with the same reinforcement. At failure the beam attained a deflection of 15mm which was comparable to the corresponding conventional concrete beam. The compressive strengths for the alternative and conventional concrete were 21 and 23 N/mm<sup>2</sup> respectively.



**Figure 7.4** Moment versus deflection of alternative concrete beam with mix ratio 1:1.5:3 reinforced with 2Y8

Concrete beams utilizing both the conventional and alternative concrete and containing different percentages of steel were tested in flexure. Figure 7.5 shows the moment-deflection curves for various reinforced concrete beams tested. From the figure, the section moment capacities of alternative concrete beams for 0.333 and 0.523% steel content were 13.7 and 19.6 kNm respectively. The corresponding values for the conventional concrete beams are 13.9 and 19.5 respectively. Reinforced concrete beam bending theory shows that the moment capacity of beam section is dependent on the amount of steel reinforcement present. The moment resistance of a reinforced concrete section is determined by multiplying the axial resultant of various force components by the respective lever arms of the force components. Low reinforcement content results in low reinforced concrete section moment capacity (McGinley *et al.*, 1995). From the results it is apparent that the increase in moment capacity of the concrete section is not in proportion to the

increase in the amount of steel content of the beams. This was observed in both the conventional and alternative concrete beams. The figure also shows that, the modulus elasticity of the alternative concrete beams at both levels of steel content is lower as compared to the conventional concrete. However, the moment-deflection curves for the beams are of the same order throughout the loading history. It was evident that, up to 5 kNm applied moment, the beams have the same flexural stiffness.

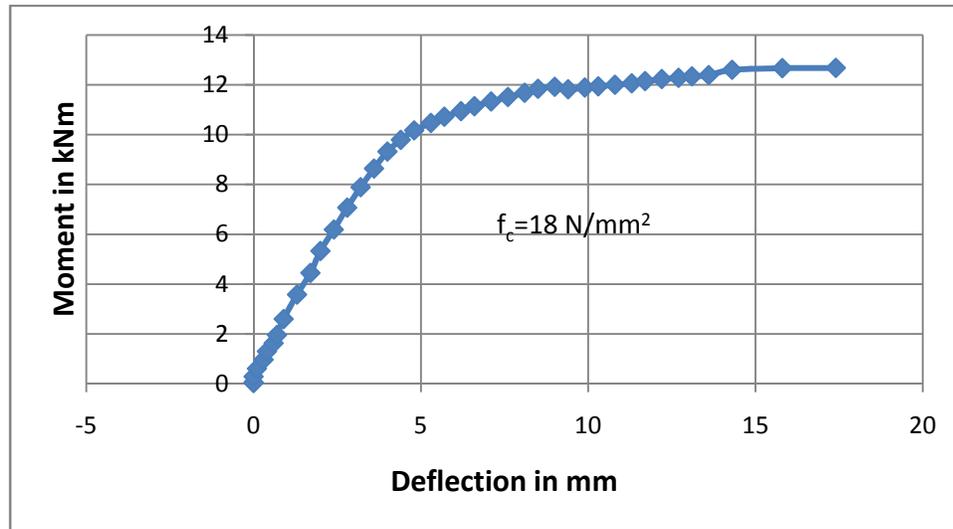


**Figure 7.5** Moment-deflection curves for concrete beams of varying steel content

After the formation of first crack in the beams with low steel content the characteristic curves for the different steel content starts to separate. The low steel concrete beams deteriorating in flexural stiffness. At moments lower than the cracking moment, the whole section of concrete and steel together contribute to the

flexural stiffness of reinforced concrete beams. Immediately the cracking moment is attained, cracks start to form and the contribution of the concrete below the neutral axis to flexural stiffness is reduced with the concrete between the cracks offering some stiffness. The slight difference in the behaviour of beams utilizing alternative and conventional concrete is due to the slight differences in the concrete compressive strength.

The curve for conventional concrete beam with mix ratio 1:2:4 and reinforced with 2Y10 bars is shown in Figure 7.6. The figure shows that at low moment up to 8 kNm, the relationship between the applied moment and deflection is linear. At higher load, the curve becomes parabolic with rapid increase of central deflection with moment. The failure moment of this beam was 13 kNm. In spite of there being the same amount of reinforcement in the beams, the capacity of this beam was lower than that of the conventional concrete beam of mix ratio 1:1.5:3. The lower capacity is due to lower concrete strength which result in reduced liver arm since a large depth of concrete with lower strength is required to balance the tension in the reinforcement steel (McGinley *et al.*, 1995).

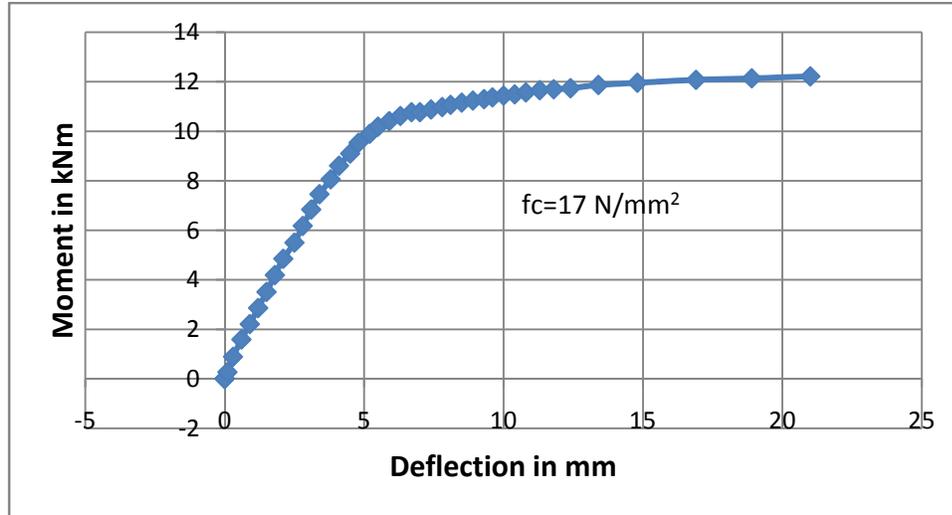


**Figure 7.6** Moment versus deflection of conventional concrete beam with mix ratio 1:2:4 reinforced with 2Y10

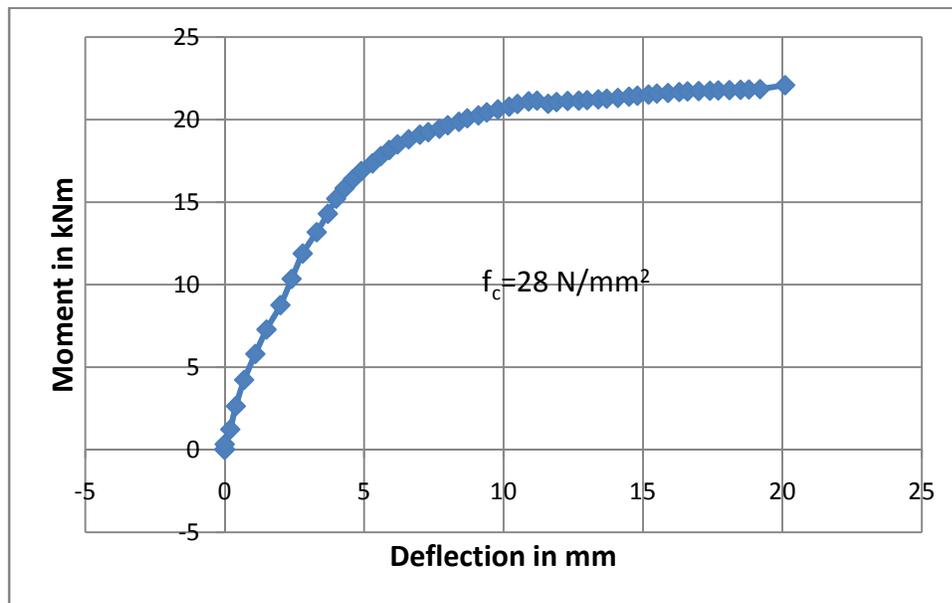
Figure 7.7 shows the moment versus deflection curve for alternative concrete beam of concrete with compressive strength of  $17 \text{ N/mm}^2$  and reinforced with 2Y10 bars. The curve has three distinct sections; the initial linear part running up to 8 kNm, the parabolic section ranging between 8 kNm to the maximum moment equal to 12 kNm and the linear horizontal section indicating the zone of plastic deformation. The failure moment for this beam was 12 kNm, which is lower than the equivalent conventional concrete beam as shown in Figure 7.8.

Conventional concrete beam of mix ratio 1:1:2 and reinforced with 2Y10 bars, on the hand, produced a parabolic characteristic curve. The moment capacity of the beam was 22 kNm which compares with the beam of mix ratio 1:1.5:3 shown in Figure 7.1. The moment-deflection curve for reinforced concrete beam utilizing conventional concrete of mix ratio 1:1:2 is shown in Figure 7.8. Within the range of

strength studied, much stronger concrete has little effect on the capacity of beams since failure of sections is controlled by the yielding of steel.

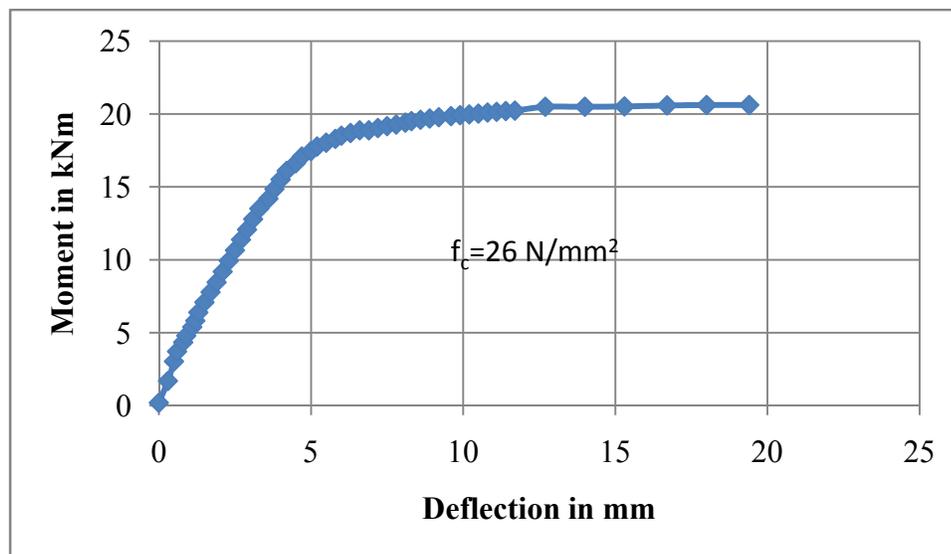


**Figure 7.7** Moment versus deflection of alternative concrete beam with mix ratio 1:2:4 reinforced with 2Y10



**Figure 7.8** Moment versus deflection of conventional concrete beam with mix ratio 1:1:2 reinforced with 2Y10

The characteristic curve for alternative concrete beam with 2Y10 bars shows that at low load levels, there was an increase in central deflection with increasing applied moment up to a moment of 18 kNm (Figure 7.9). At higher load levels, deflection of the beam increased at constant moment of 21 kNm until failure occurred. The ultimate moment for the beam is comparable to the conventional concrete of compressive strength of  $28\text{N/mm}^2$ . Like the conventional concrete, the beams of alternative concrete with compressive strength of  $26\text{ N/mm}^2$  and  $21\text{ N/mm}^2$  attained ultimate moments that compared well. In the design of concrete beams, moment is usually the governing criteria. In practice the beam is designed for bending then, the other modes of failure such as shear and bond are prevented. This implies that moment capacity of reinforced concrete beam is very critical (McGinley *et al.*, 1995).

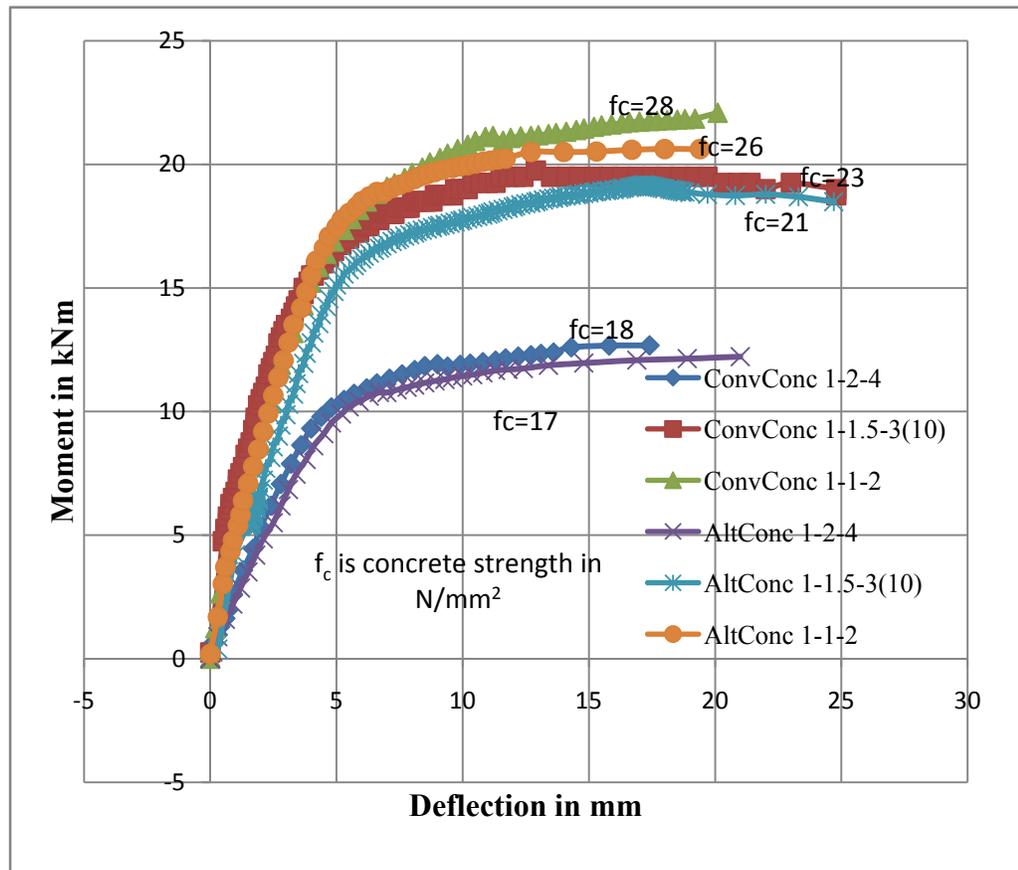


**Figure 7.9** Moment versus deflection of alternative concrete beam with mix ratio 1:1:2 reinforced with 2Y10

Figure 7.10 shows the moment-deflection curve for reinforced concrete beams utilizing conventional and alternative concrete of varying strengths and reinforced with 2Y10 bars. Alternative concrete comprise of Portland cement, river sand and laterized quarry dust as fine aggregates and recycled concrete aggregates as coarse aggregates. In both conventional concrete and alternative concrete, various combinations of the materials were used to arrive at different compressive strength of concrete. In each category, three strength levels were considered. The three strength levels for conventional concrete considered are 18, 23 and 28 N/mm<sup>2</sup>. The corresponding values for alternative concrete are 17, 21 and 26 N/mm<sup>2</sup>.

From Figure 7.10, it is noted that when conventional concrete of compressive strengths 18, 23 and 28 N/mm<sup>2</sup> are used, the beam attains flexural capacities of 13, 19.5 and 22 kNm, respectively. Generally the beam flexural capacity increases with the strength of concrete (McGinley *et al.*, 1995). This is because, when a reinforced concrete beam is loaded in flexure, the concrete in the compression zone takes compressive stresses while the steel reinforcement resist the tension stresses. When high strength concrete is used, less area of concrete is required to balance out the tension in steel reinforcement and hence the depth to the neutral axis of the beam is reduced. The effect of this is that the lever arm of the longitudinal forces increases hence increasing the moment capacity of the beam (McGinley *et al.*, 1995). From the results, however, the increase in beam capacity is not proportional to the increase in concrete compressive strength. The change in strength from 18 to 23 N/mm<sup>2</sup> reflects a 28% increase in strength while the corresponding increase in moment

capacity of the beams is 13 to 19.5 kNm which reflects 50% increase in section capacity. In addition, an increase in strength from 23 to 28 N/mm<sup>2</sup> reflects a 22% increase in strength while the corresponding change in section capacity from 19.5 to 22kNm reflects a 13% increase. The relationship between compressive strength of concrete and the moment capacity of a beam section is therefore non-linear.



**Figure 7.10** Moment-deflection curves for concrete beams of alternative and conventional concrete of varying strength

The non-linearity in relationship between the concrete compressive strength and the moment capacity of the section is brought by the effects of other factors in the structural behaviour of concrete beams under loading. It has been found that the

transverse stresses in the compressive zone of beam have direct influence on the compressive stresses that can be resisted by concrete (Kotsovos, 1982). The presence of shear reinforcement in form of links magnifies these effects; introducing confinement effects, hence increased transverse stresses which cause tri-axial state of stress in the compression zone. The effects of tri-axial stresses in the compression zone of concrete beam introduce non-linearity between the uniaxial compressive strength of concrete and the actual compressive resistance of concrete in the beam.

The results also show that the modulus of elasticity increases with increase in the compressive strength of concrete. Concrete beams are composite elements consisting of concrete and steel reinforcement. The overall behaviour of the beam under load depends on both the properties of concrete and steel reinforcement. Most codes of practice relate the modulus of elasticity of concrete to the compressive strength of concrete. Neville (1981) proposed an expression for modulus of elasticity of concrete  $E_c$  taking the form,

$$E_c = k (f_c)^m \quad (7.1)$$

where  $k$  is a factor which depends on the density of concrete,  $f_c$  is the compressive strength of concrete and  $m$  is constant equal to 0.33. The 1977 edition of the ACI Building Code proposes the expression for  $E_c$  of the form

$$E_c = k (f_c')^{0.5} \quad (7.2)$$

where  $k$  is a factor and  $f_c'$  is the cylinder strength of the concrete.

Utilization of alternative concrete in beams gave the same trend in results as the conventional concrete beams. The three strength cases considered were 17, 21 and 26 N/mm<sup>2</sup>. The respective beam capacities for the strength cases were 12, 19.5 and 21 kNm. The concrete strength values were generally lower than the corresponding conventional concrete. Further, apart from the middle section capacity of 19.6 kNm, the capacities of alternative concrete beams were lower than the corresponding values for the conventional concrete beam. Like the conventional concrete beams, the non-linearity relationship exists between the concrete compressive strength and the section capacity of the beams. This effect is due to tri-axial state of stress that occurs in the compressive zone of the beam enhancing the compressive stresses that can be resisted by the concrete.

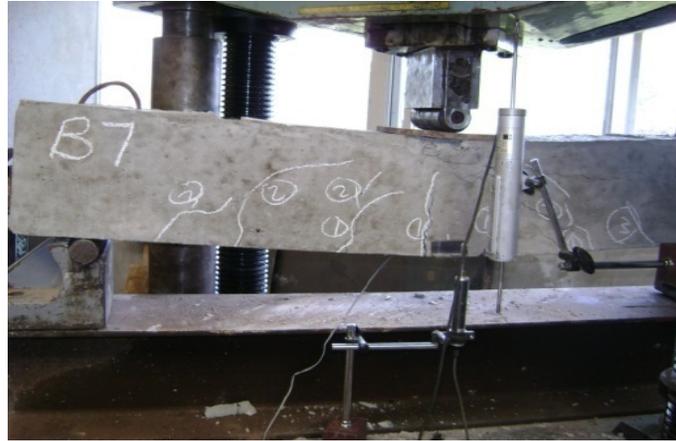
Like conventional concrete beams, the modulus of elasticity as can be shown on the moment-deflection curve increases with concrete strength. The values of modulus of elasticity of various strength of concrete are of the same order for the corresponding alternative and conventional concrete.

### ***7.2.2 Crack patterns***

This section evaluates the cracking behaviour of various reinforced concrete beams subjected to flexural stresses. In all cases flexural cracks formed first followed by combined flexure-shear cracks at high loads close to failure load in few cases.

Cracking patterns for reinforced concrete beams is shown in Plate 7.1 and Appendix C. The results show that, cracks started developing at the mid-span of the beam

during loading (1 in Plate 7.1). As the applied load was increased, more cracks appeared towards the supports of the beams (2 and 3 in Plate 7.1). In some cases, secondary cracks developed from the main cracks which extended towards the neutral axis of the beam as the loading was further increased.



**Plate 7.1** Beam AltConc1-1-2. Crack pattern at ultimate load,

Two levels of steel reinforcement content were investigated in this study. From the results, it was seen that both the alternative and conventional concrete beams reinforced with 2Y8 bars developed few but wide cracks as compared to beams with 2Y10 bars. Beams with lower steel content generally had few wider flexural cracks than the beams with high steel content. In low reinforcement beams, signs of shear cracks hence shear failure was not evident as it was the case with the high reinforcement content beams. The presence of few but wide cracks in low reinforcement content beams is due to the low flexural capacity of the beams. The compression load path had adequate capacity in these beams leading to flexural failure without any signs of crushing of concrete. The high flexural and shear

capacity ratio in low reinforcement content beams eliminated any possibility of shear failure occurring before or at failure. The similarity of failure pattern in beams was due to the fact that the respective concrete compressive strengths of alternative and conventional concrete were within 10%.

Crack patterns for alternative and conventional concrete beams of concrete compressive strengths 17 and 18 N/mm<sup>2</sup> compare well. Both beams failed by development of four main wide inclined flexural cracks which extended towards the neutral axis of the beam. The shear cracks were absent in these beams which implies that there was no possibility of shear failure in the beam. However, after failure there were signs of concrete crushing in the compressive zone due to low strength of concrete. These weaknesses in the compression load path lead to early pure flexural failure. The failure mode of alternative and conventional concrete beams of mix ratio 1:1.5:3 was different from beams of concrete mix 1:2:4. First, flexural cracks formed followed by shear cracks at ultimate load. The development of shear cracks was due to strong compression load path through the strong concrete. The high flexural capacity of these beams lead to the shear capacity being exceeded at failure. Alternative and conventional concrete beams of higher concrete compressive strength failed by yielding of the steel reinforcement. There were no signs of shear failure as no shear cracks appeared. In addition, there was no evidence of crushing of concrete in the compression zone. Shear capacity of high strength concrete is higher than low strength concrete beams.

### 7.3 Cost comparison

The production costs for one cubic metre of conventional and alternative concrete mix of various ratios are shown in Table 7.2. The table also shows the combination of ingredient materials in concrete. From the table, the costs of conventional and alternative concrete of mix ratio 1:2:4 (Class 20) are KShs.7700 and 6314, respectively. When high strength concrete of Class 25 is required, the costs are KShs. 8650 and 7400 for conventional and alternative concrete, respectively.

**Table 7.2** Cost of one cubic metre of conventional and alternative concrete mix

| Materials                                 | Conventional Concrete |          | Alternative Concrete |         |
|---|-----------------------|----------|----------------------|---------|
|   | 1:2:4                 | 1:1.5:3  | 1:2:4                | 1:1.5:3 |
| Mix ratios                                | 1:2:4                 | 1:1.5:3  | 1:2:4                | 1:1.5:3 |
| Concrete grade                            | Class 20              | Class 25 |                      |         |
| Cement (kg/m <sup>3</sup> )               | 314                   | 390      | 314                  | 390     |
| River sand (kg/m <sup>3</sup> )           | 627                   | 585      | 439                  | 410     |
| QDNbi (kg/m <sup>3</sup> )                | -                     | -        | 94                   | 88      |
| Laterite (kg/m <sup>3</sup> )             | -                     | -        | 94                   | 88      |
| NCA (kg/m <sup>3</sup> )                  | 1255                  | 1171     | -                    | -       |
| RCA (kg/m <sup>3</sup> )                  | -                     | -        | 1255                 | 1171    |
| Water/cement ratio                        | 0.65                  | 0.65     | 0.65                 | 0.65    |
| Water content (kg/m <sup>3</sup> )        | 204                   | 254      | 204                  | 254     |
| Compressive strength (N/mm <sup>2</sup> ) | 18                    | 23       | 17                   | 21      |
| Cost (KShs./m <sup>3</sup> )              | 7700                  | 8650     | 6314                 | 7400    |

It was assumed that the cost of labour for producing either conventional or alternative concrete is the same; hence, it was excluded from the analysis. When the costs of conventional and alternative concrete are compared, a saving of 18% is achieved when alternative concrete of Class 20 is used. For higher strength concrete of Class 25, the saving is only 14% because the proportion of cement required in the mix increases substantially. Cement is the most expensive ingredient in concrete mix.

#### **7.4 Summary and conclusion**

The results of flexural testing of concrete beams have been discussed in this chapter. The results have shown that the ultimate capacity of alternative concrete beams compare well with the conventional concrete beams. The ultimate moments of beams made from alternative concrete of compressive strengths of 17, 21 and 26 N/mm<sup>2</sup>, and reinforced with 2Y10 bars, were 12.0, 19.6 and 21.0 kNm, respectively. Generally, the ultimate moment capacities for alternative concrete beams were within 5% of the corresponding conventional concrete beams with similar compressive strength of concrete and containing equal amount of reinforcement.

Cracking pattern for alternative concrete beams was similar to conventional concrete beams containing similar reinforcement content. Generally beams containing low reinforcement content (2Y8) developed wide cracks than heavily reinforced beams. In all the aspects considered, beams utilizing alternative concrete generally have structural behaviour comparable to conventional concrete beams.

## CHAPTER EIGHT

### 8.0 MASONRY WALL PANEL

#### **8.1 Introduction and wall panel description**

In this chapter, the results of the masonry wall panels tested in compression are presented. A total of ten wall panels were tested in this study. Out of the ten walls tested, four wall panels were made from the designed alternative blocks while the other six were made from conventional blocks. In each category, some walls were reinforced with hoop iron while others were not. Several sizes of the walls were also considered to determine the effect of size of the wall panels on the structural behaviour under compressive stress. The main variables investigated were the compressive strength of mortar joint, the effect of reinforcement in bedding joints of the walls, and the size of wall panels. These variables were compared for both alternative and conventional block walls. The stress in the stress-strain curve is given as a fraction of the maximum strength recorded for the wall panel concerned. Table 8.1 shows the details of wall panels tested in this study.

**Table 8.1** Wall panel details

| Wall           | Material     | Size   |        | Mortar ratio | reinforcement |
|----------------|--------------|--------|--------|--------------|---------------|
|                |              | H (mm) | B (mm) |              |               |
| ConvBL 1:3(R)  | Conventional | 520    | 600    | 1:3          | Reinforced    |
| ConvBL 1:3     | Conventional | 520    | 600    | 1:3          | None          |
| ConvBL 1:3L    | Conventional | 520    | 900    | 1:3          | None          |
| ConvBL 1:4     | Conventional | 520    | 600    | 1:4          | None          |
| ConvBL 1:3(R)x | Conventional | 620    | 760    | 1:3          | Reinforced    |
| ConvBL 1:3x    | Conventional | 620    | 760    | 1:3          | None          |
| AltBL 1:3(R)   | Alternative  | 520    | 600    | 1:3          | Reinforced    |
| AltBL 1:3      | Alternative  | 520    | 600    | 1:3          | None          |
| AltBL 1:3L     | Alternative  | 520    | 900    | 1:3          | None          |
| AltBL 1:4      | Alternative  | 520    | 600    | 1:4          | None          |

**H**, Height of wall panel; **B**, Breadth of wall panel

## 8.2 Results and discussion

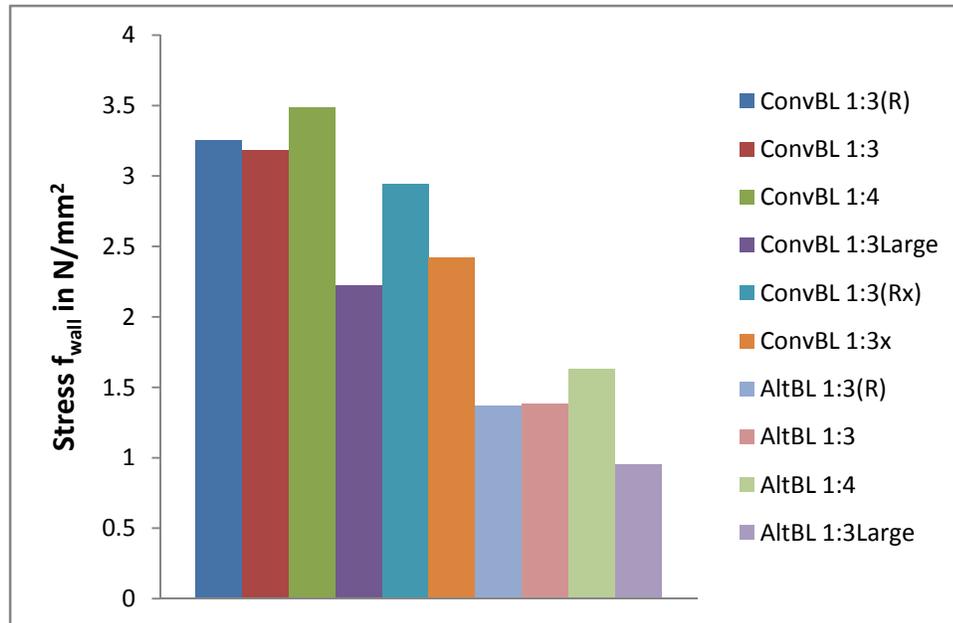
### 8.2.1 Compressive strength of wall panels

The walls were loaded in axial compression on a Universal Testing Machine. The maximum load as obtained from stress-strain curves and confirmed by the values

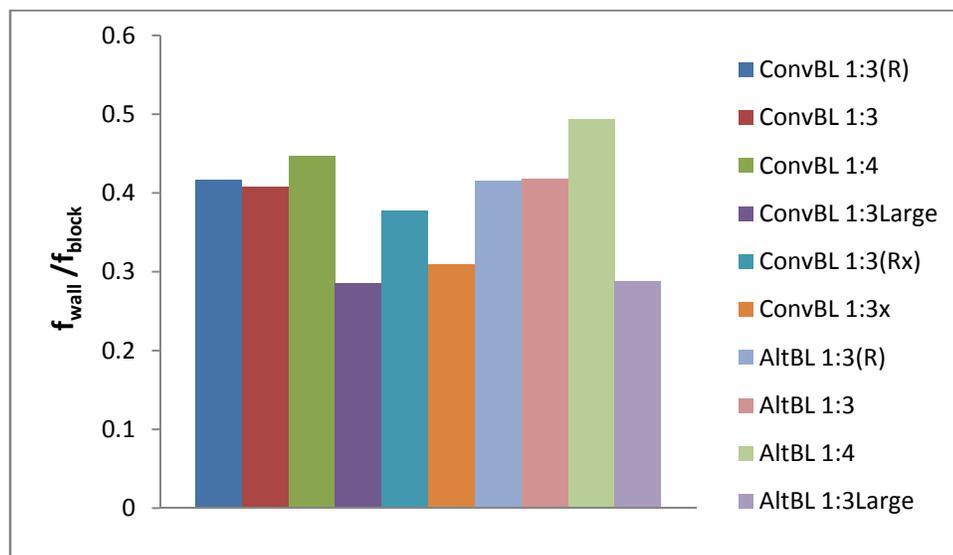
directly noted on the testing machine after failure of the walls are shown in Figure 8.1(a). In Figure 8.1(b), the normalized compressive strengths of walls are presented. The normalized values were obtained by dividing the compressive strength of wall panels with the compressive strength of masonry blocks used in building up the wall panels. The compressive strength of conventional and alternative blocks used in the study is  $7.8 \text{ N/mm}^2$  and  $3.3 \text{ N/mm}^2$ , respectively. From the figure, the highest strength occurred in plain conventional block wall containing 1:4 cement-sand jointing mortar while the lowest occurred in plain alternative block wall with large dimensions and utilizing 1:3 mortar joint. The corresponding compressive strength values for the two above extreme walls are  $3.49$  and  $0.95 \text{ N/mm}^2$ , respectively. Reinforced conventional block wall utilizing 1:3 mortar had a compressive strength of  $3.25 \text{ N/mm}^2$  which is slightly higher than the unreinforced plain wall by  $0.07 \text{ N/mm}^2$ . These values are lower than the wall with weaker mortar joint. Similar trends were noted in alternative block wall panels. Reinforced alternative block wall with 1:3 mortar joint attained a compressive strength of  $1.37 \text{ N/mm}^2$  as compared to  $1.38 \text{ N/mm}^2$  for the unreinforced wall. However, this value is lower than that of unreinforced wall utilizing weaker mortar joint which had a compressive strength of  $1.63 \text{ N/mm}^2$ .

Conventional block walls utilizing standard size blocks as used in the construction industry in Kenya were also evaluated. The walls had compressive strength lower than the laboratory sized block walls of the same material and mortar joint. Reinforced wall had a compressive strength of  $2.94 \text{ N/mm}^2$  as compared to the

unreinforced wall which had 2.42 N/mm<sup>2</sup>. These values are however; higher than the corresponding alternative block walls.



a) Absolute compressive strengths of wall panels



b) Normalized compressive strengths of wall panels

**Figure 8.1** Compressive strengths of wall panels

The normalized compressive strength levels for the alternative and corresponding conventional block wall panels were generally within 5%. The normalized strength reported for conventional wall utilising 1:3 mortar joint was 0.417 as compared to 0.415 for the corresponding alternative wall panels with similar variables. The normalized strength of alternative wall panel with the weaker mortar joint was however higher than the corresponding conventional wall panel. This is because alternative wall panel with weaker mortar joint condition has better load distribution behaviour as compared to the conventional walls (Musiumi *et al.*, 2007).

In this study, it was found that walls with weaker mortar joint were generally stronger in compression than walls utilizing stronger mortar joint. This was evident in both the conventional as well as alternative block walls. When weaker mortar is used, the compressive strength of alternative wall is 18% higher than in stronger mortar wall of the same material. In conventional walls, weaker mortar wall have compressive strength only 10% higher. This indicates that the effects of the compressive strength of mortar are greater in alternative block walls than it is in the conventional walls. This greater mortar strength effect in alternative block walls is due to greater load distribution that takes place in these walls during load application. Previous work by Musiumi *et al.*, (2007) reported that there is greater load distribution in walls with weak mortar joint condition than strong mortar joint when loaded in compression. The strength ratio obtained in this previous work was 1.2 which is comparable to the present study.

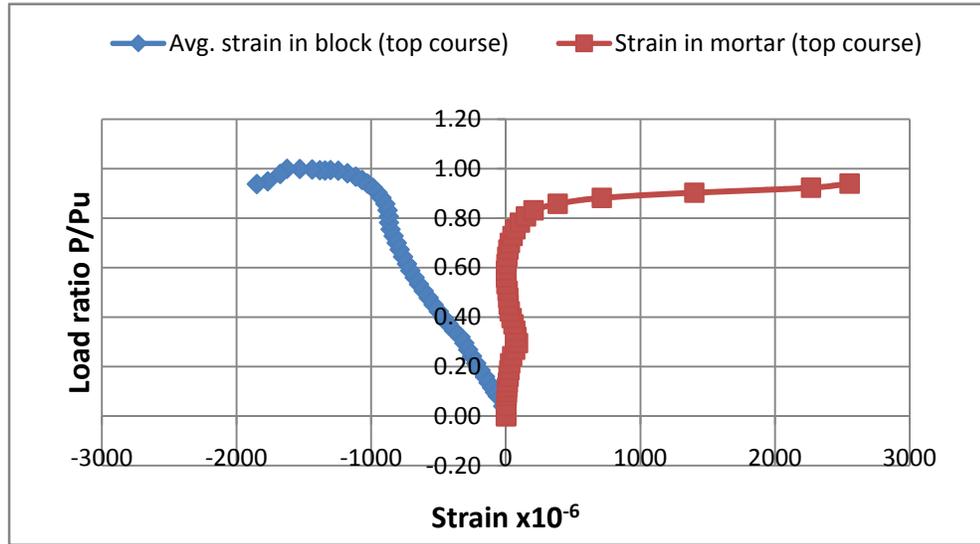
The compressive strengths of reinforced and unreinforced conventional block walls were 3.25 and 3.18 N/mm<sup>2</sup>, respectively. The corresponding values for alternative block walls were 1.37 and 1.38 N/mm<sup>2</sup>. As observed from the failure modes for the walls, the presence of bedding joint reinforcement did not affect the strength of the walls but had the effect of delaying the formation of the first crack in the walls. Assessment of cracking patterns in walls with bedding joint reinforcement showed that the subsequent cracks after the formation of first crack tend to be distributed along the wall pending total collapse. The effect of bond between the reinforcement and the mortar has been found to play a critical role on the structural behaviour of masonry walls under axial compression (BSI, 1992).

### ***8.2.2 Stress-strain characteristics of wall panels***

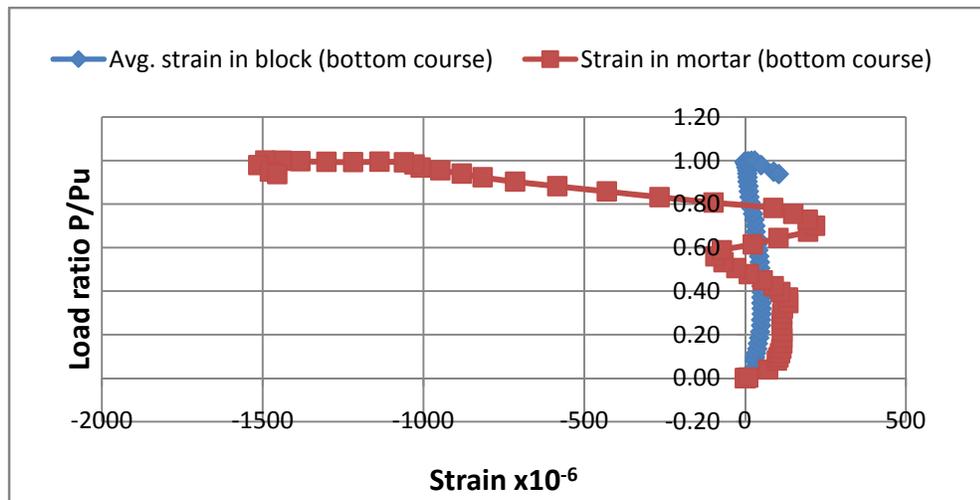
The main aim of studying stress-strain characteristics of the wall panels was to determine the behaviour of the walls to failure throughout the loading period. The stresses were obtained by dividing the total load applied on the wall by the plan area of the wall.

Figure 8.2 shows the stress-strain curve for reinforced conventional block wall utilizing 1:3 mortar joint conditions. In Figures 8.4a and 8.4b, the stress-strain curves for blocks and mortar joint in top and bottom course, respectively are shown. In the top course, the strain in the blocks was compressive and it increased from 0 to 0.002. The strain in the mortar remained constant at 0.0 up to load ratio of 0.8. Beyond this point, the tensile strain in the mortar started increasing to 0.0028 at total failure. In the bottom course, the behaviour was different. The small strain in the

blocks and mortar in the bottom course as compared to the top course is an indication of deformation being confined on the top zone of the wall panels. This was evident from the failure mode as seen in the cracking pattern in the walls.



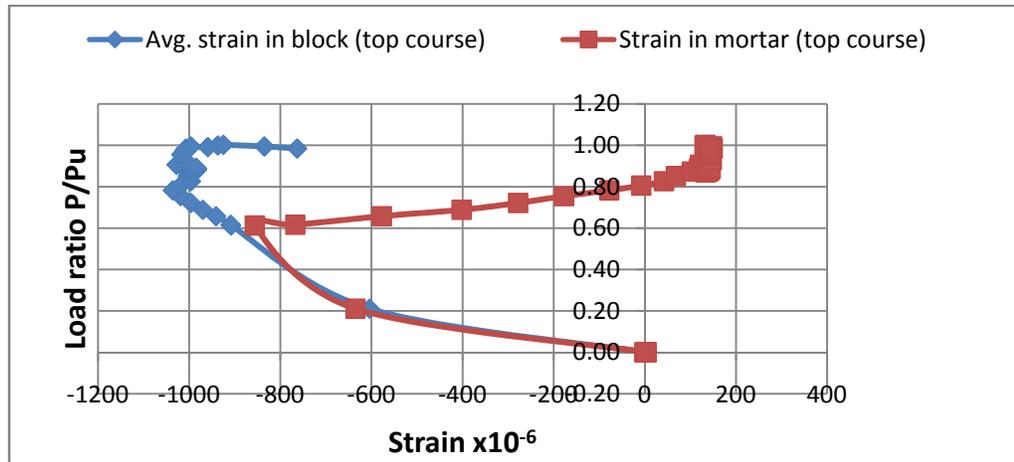
(a) Stress versus strain in top course



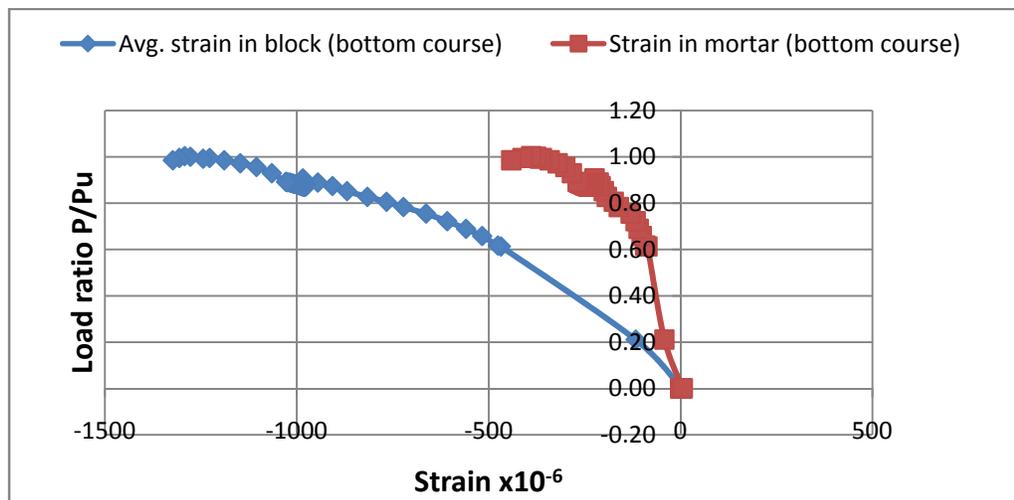
(b) Stress versus strain in bottom course

**Figure 8.2** Stress versus strain for reinforced conventional block wall jointed in 1:3 mortar

The stress-strain characteristic curve for unreinforced conventional block wall panel is shown in Figure 8.3. From Figure 8.6(a), the strains in block and mortar in the top course were compressive and of the same order up to load level of 60%. At higher stress levels, the strain in the mortar reduced gradually up to a value of 0.0 at stress levels of 80%. Further increase in stress levels lead to development of tensile strains in the mortar which continued to increase until failure. The compressive strains in blocks on the other hand increased in magnitude until failure. Figure 8.3(b) shows the stress-strain curve for the blocks and mortar for the bottom course in the wall. The strain condition in both the blocks and mortar are compressive with the magnitude in blocks being greater than mortar. At failure the strain in blocks and mortar were 0.0013 and 0.0005, respectively. When Figures 8.4(b) and 8.5(b) are compared, it is noted that the strain in the block in bottom course of the unreinforced is generally greater than that of the reinforced wall. In the reinforced wall the presence of reinforcement in the bedding joint lead to crushing of the blocks that are confined in the upper courses of the wall. The strains in the bottom course of the unreinforced conventional block wall were of the same order as those in the top course. This was not the case for the reinforced wall which had strains in the top course that were generally higher in magnitude than those in the bottom course.



(a) Stress versus strain in top course



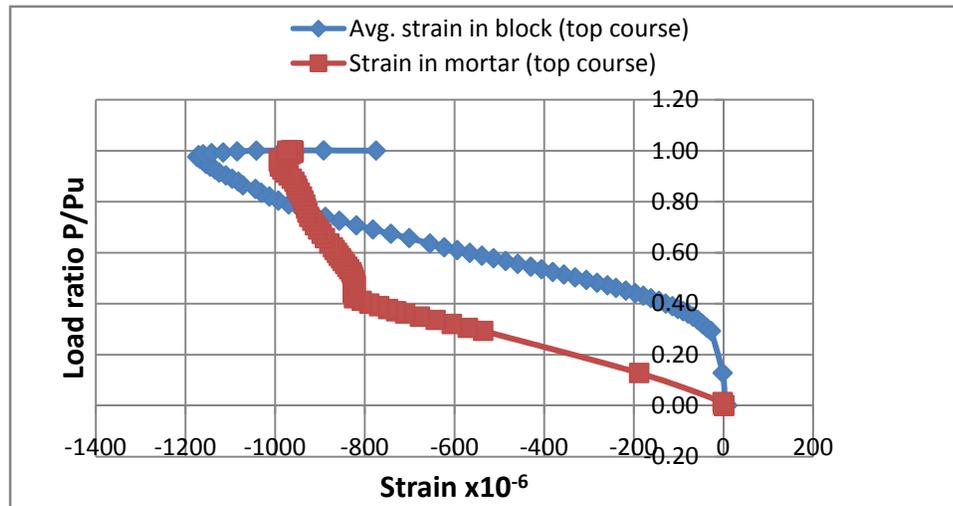
(b) Stress versus strain in bottom course

**Figure 8.3** Stress versus strain for unreinforced conventional block wall jointed in 1:3 mortar

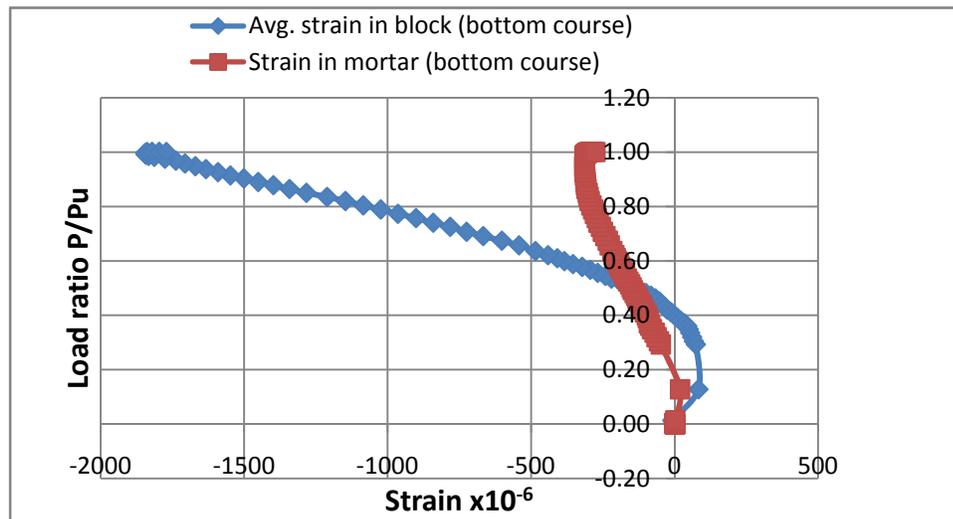
The stress-strain curves for the wall utilizing alternative blocks and 1:3 mortar joint with bedding joint reinforcement are shown in Figure 8.4. Figure 8.4(a) shows that the strains in the top course were compressive throughout the loading period. However, the strain in the mortar was higher than that of the blocks up to load level

of 79%. Beyond this point, the strain in blocks increased until failure occurred at strain of 0.0012. The failure strain in the mortar was 0.001 which is lower than the strain in the blocks. Figure 8.4(b) shows the stress-strain curves for the block and mortar in the bottom course. Below 40% load levels, the strain in the blocks was tensile. At higher load, the strain was compressive and it increased in magnitude until failure. On the other hand, strain in the mortar strain was compressive throughout the loading period. At failure, the strains in the block and mortar were 0.0017 and 0.0004, respectively. This is an indication of the high load resisted by the blocks at the lower parts of the wall. Failure strain in the blocks was generally greater in the bottom course than the top course, which implies that there was better load distribution in this type of construction. On the other hand, failure strain in the mortar was lower in the bottom course than the top course. It has been proved that the role of mortar in masonry is to distribute the stresses in the blocks (BSI, 1992).

Comparison of the conventional and alternative block walls shows that the strains in the blocks for reinforced conventional block wall were 0.0019 and 0.0 for top and bottom courses, respectively, while the corresponding values for reinforced alternative block wall were 0.0012 and 0.0015. In conventional block wall, the deformation was generally confined at the top level while alternative block wall exhibited better load distribution and hence load sharing between the blocks as indicated by the strain distribution at the top and bottom course.



(a) Stress versus strain in top course



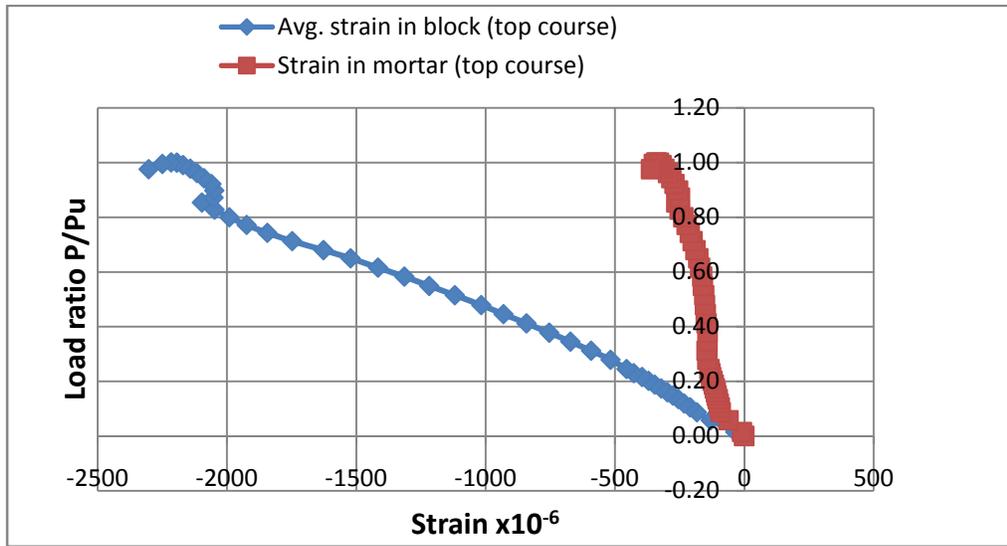
(a) Stress versus strain in bottom course

**Figure 8.4** Stress versus strain for reinforced alternative block wall jointed in 1:3 mortar

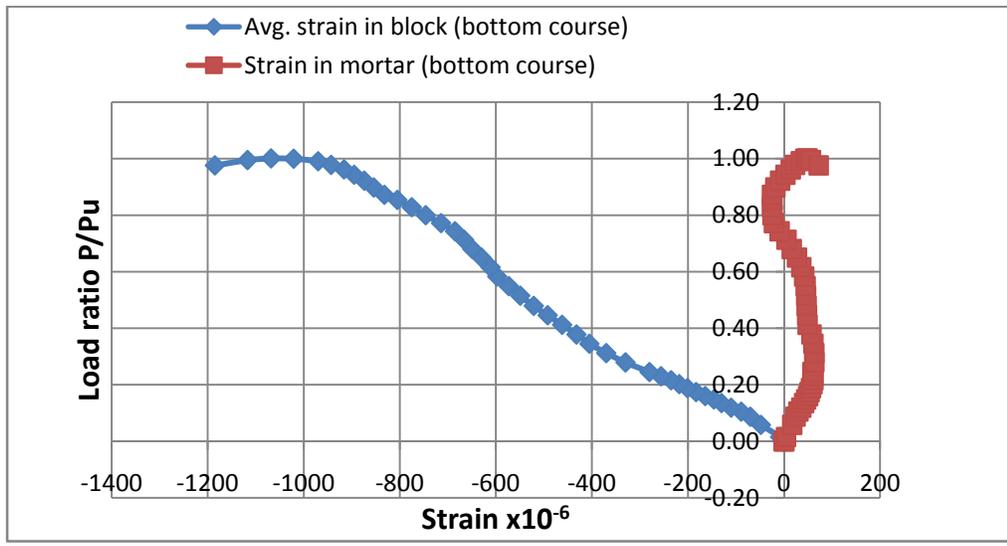
Figure 8.5 shows the stress-strain characteristic curve for unreinforced alternative block wall. The block and mortar strain in the top course (Figure 8.5(a)) was compressive and it increased in magnitude until failure. At failure, the strain in the block was 0.0023 while that of the mortar was 0.00025 and this was almost a tenth the strain in the blocks. The strain in the top course block for the unreinforced

alternative block wall was higher than the corresponding strain in reinforced wall. However, the strain in the mortar for the unreinforced wall was much lower than the corresponding strain in reinforced alternative wall. Figure 8.5(b) also shows the strain-stress curve for the bottom course in unreinforced alternative block wall. The vertical strain in the block remained compressive increasing continually until failure. The strain at failure was 0.0012 which is almost a half in magnitude for the strain in the block for the top course of the same wall panel. On the other hand, the failure strain in the bottom course for the blocks was lower than the corresponding value for the reinforced alternative block wall.

Mortar strain in the unreinforced alternative block wall remained tensile increasing from zero and then reduced to zero at 80% load level. At failure the strain was tensile and its magnitude was 0.0001. The average strain in the top course blocks of the unreinforced alternative block wall and unreinforced conventional block wall were 0.0012 and 0.0013, respectively. The corresponding values for the bottom course were 0.0011 and 0.0023 for conventional and alternative block walls, respectively.



(a) Stress versus strain in top course



(b) Stress versus strain in bottom course

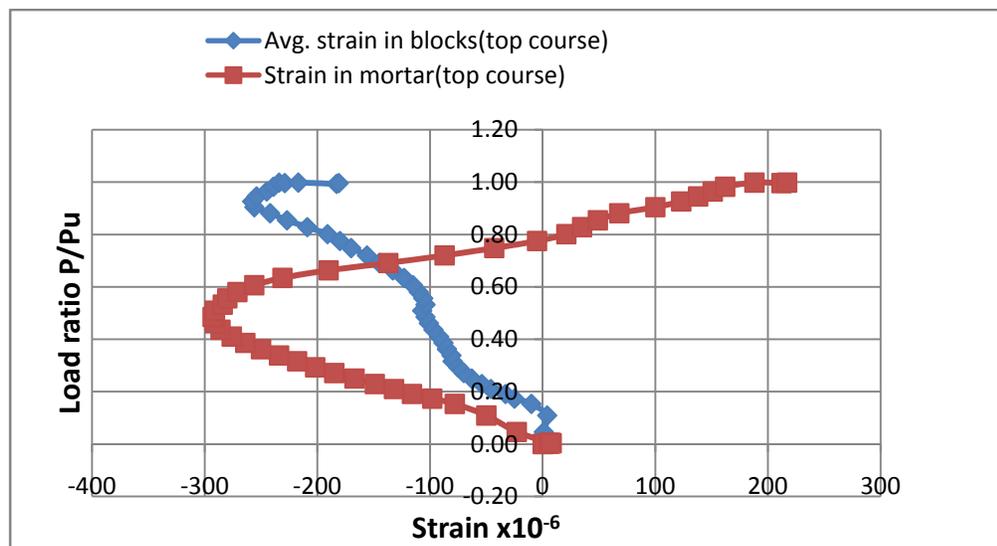
**Figure 8.5** Stress versus strain for unreinforced alternative block wall jointed in 1:3 mortar joint

The section capacities for reinforced and unreinforced conventional block wall panels were 3.25 and 3.18 N/mm<sup>2</sup>, respectively. The corresponding values for the alternative block walls were 1.37 and 1.38 N/mm<sup>2</sup>, respectively. These values are

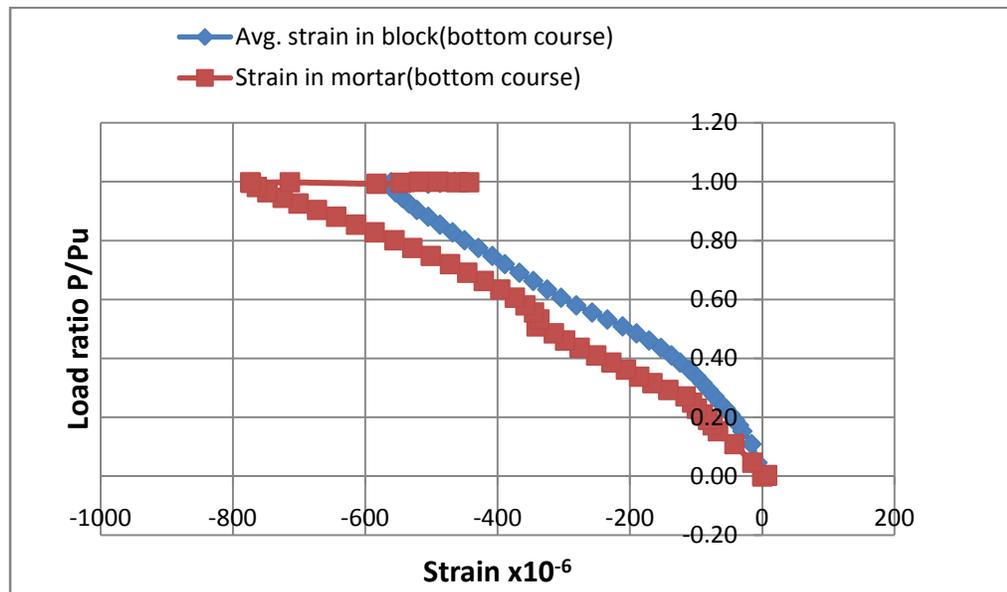
close and they indicate that there is no effect of bedding joint reinforcement on the strength of the masonry subjected to axial compression forces. This is because the reinforcement is placed in the direction perpendicular to the direction of loading. However, the presence of bedding joint reinforcement in the conventional block wall panel, lead to high ultimate strain and fewer cracks as compared to the unreinforced wall panel. In reinforced alternative block wall panel, the ultimate strains were lower than those in the unreinforced wall. However, just like in conventional block wall, few cracks formed in the wall.

The stress-strain characteristic curve for unreinforced conventional block wall with 1:4 mortar joint is shown in Figure 8.6. Figure 8.6(a) shows the characteristic curve for the average strain in block and mortar for the top course. From the figure, the average compressive strain in the block increased to a maximum of 0.00026 at load level of 90% of the ultimate capacity of the wall panel. Further increase in stress level lead to reduction in average strains in the blocks. At failure the final strain in the block was compressive with a magnitude of 0.0002. This value is very low compared to the corresponding value for unreinforced conventional block wall utilizing 1:3 mortar joint conditions. The figure also shows that the vertical compressive strain in mortar joint was increased to a maximum of 0.0003 at load level of 50% of the ultimate load. At higher loads the strains in the mortar reduced with increase in load. This continued until the strain changed to tensile. At failure of the wall panel, the tensile strain in the mortar was 0.00022.

The stress versus strain for the bottom course is shown in Figure 8.6(b). From the figure, both the average strain in the block and strain in the mortar were compressive. At all load levels, the strains in the mortar were greater than the average strains in the blocks. The ultimate strain for the block and the mortar were 0.0008 and 0.0006, respectively. The failure strains in the unreinforced conventional block wall with 1:3 mortar joint were generally lower than the corresponding values for the unreinforced conventional block wall with 1:3 mortar joint. In the top course, the strains in the blocks for strong mortar joint wall were twice the value for the weak mortar joint wall. In the bottom course, the strains for the stronger mortar joint wall were three times greater than those of the weak mortar wall. However, the wall with weaker mortar joints had higher axial compression capacity than the walls with stronger mortar joint.



(a) Stress versus strain in top course

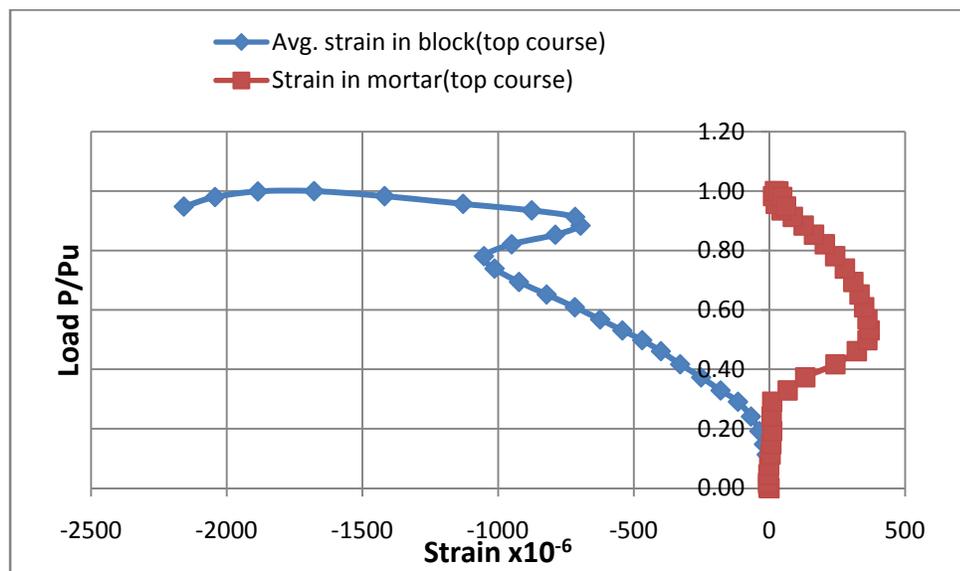


(b) Stress versus strain in bottom course

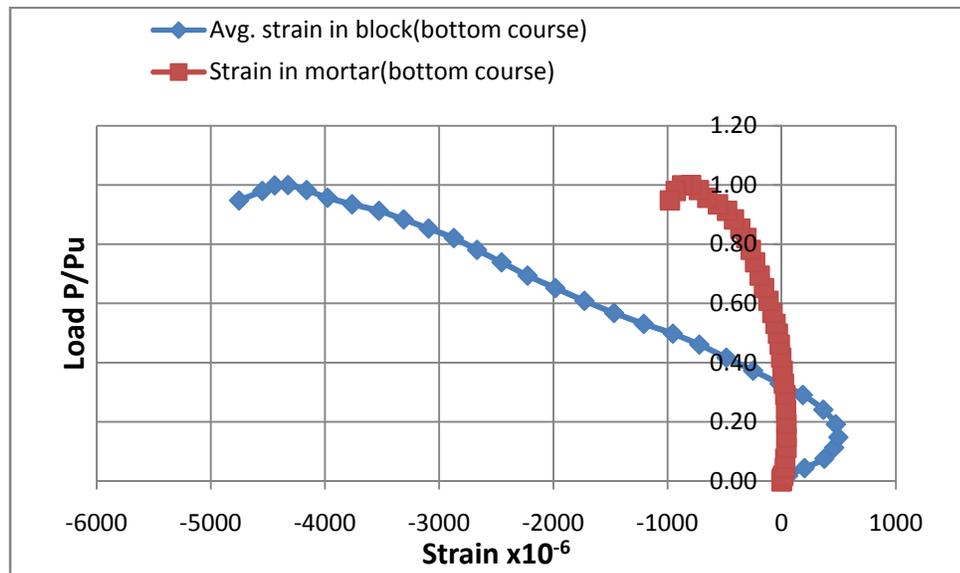
**Figure 8.6** Stress versus strain for unreinforced conventional block wall jointed in 1:4 mortar joint

In Figure 8.7 the stress versus strain for unreinforced alternative block wall with 1:4 mortar joint is shown. The average strain in the blocks for the top course was compressive while that of mortar joint was tensile. Below 20% load level the average strain in the block was very low, and then it increased rapidly with increase in load application until 80% load level. Generally, the average strain in the block remained compressive. At failure, the ultimate compressive strain in the block was 0.0022. Below 30% load level, the mortar joint in the top course did not develop any strains. Further increase in the load led to increase in strain until a maximum value of 0.0004 tensile strains was attained. In the bottom course, average strains were generally compressive apart from the block strains that were tensile for the low load levels. The ultimate strains in the blocks and mortar joint were 0.0048 and 0.001, respectively. The ultimate strains in both the top and bottom courses were generally

greater in unreinforced alternative block wall than the conventional block wall counterpart. The strain values were also higher than the values for the unreinforced alternative block wall utilizing 1:3 mortar joint. Weaker mortar effect was more pronounced in the alternative block wall than the conventional block wall. The strength of the wall with weaker mortar joint was greater than the wall with stronger mortar joint.



(a) Stress versus strain in top course



(b) Stress versus strain in bottom course

**Figure 8.7** Stress versus strain for unreinforced alternative block wall jointed in 1:4 mortar joint

The strength of mortar in masonry had a great influence on the ultimate capacity of the wall panels. In both the conventional and alternative block masonry wall panels, the low strength mortar joint resulted in high ultimate capacity of the walls. Previous work by Musiomi *et al* (2007) reported similar conclusion. In the previous study, a total of three large scale wall panels were tested on a loading frame. The walls were made from cement stabilized laterized quarry dust blocks jointed with mortars of various compressive strengths. The strength of mortar in the study ranged between 1.5 and 5 N/mm<sup>2</sup>. The average compressive strength of blocks was 4.0 N/mm<sup>2</sup>. The large scale wall panels utilizing weaker mortar joint yielded higher compressive strength as compared to stronger mortar jointed wall panels. This was attributed to the fact that weaker mortar joint condition has better load distribution between the main structural elements (i.e., the masonry blocks). British code of practice,

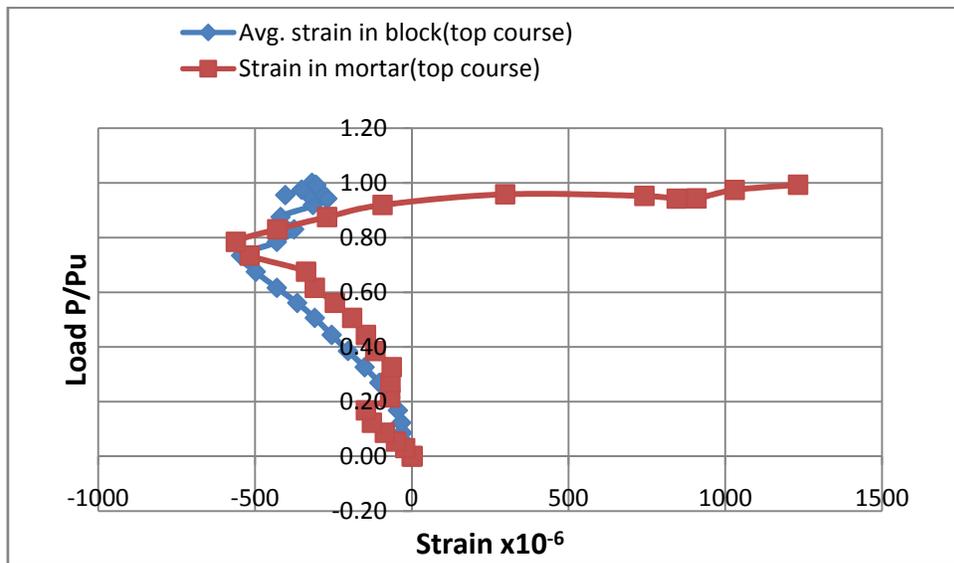
however, relates the capacity of masonry walls to block and mortar joint strength empirically where the higher the mortar or block strength, the higher the capacity of wall section (BS 5628, 1992). The same code of practice recognizes the ability of weaker mortar jointed masonry wall to accommodate movement such as due to settlement and moisture changes.

In conventional block walls, the ultimate average vertical strains on the wall surface were higher for stronger mortar jointed wall panel. The compressive strength of conventional block was  $5.7 \text{ N/mm}^2$  and strengths of mortar used were 4.1 and  $6.8 \text{ N/mm}^2$ , respectively. Ultimate strains in alternative block wall panel were different from the conventional block walls. In the alternative block masonry wall panels, the ultimate average vertical strains on the wall surface were higher for weaker mortar jointed wall panels. The block had a compressive strength of  $4.4 \text{ N/mm}^2$ . In conventional block walls, high strength mortar has compressive strength similar to the strength of the block used. The ultimate strain of this wall was high, since the compressive strengths of the structural elements (i.e. the block and the mortar) have similar magnitudes. This fact was also evident in the alternative block masonry wall panels. In the alternative block walls, the weaker mortar had compressive strength of  $4.1 \text{ N/mm}^2$ . This value is close to the compressive strength of the blocks used in the walls. The weaker mortar jointed wall had better performance in terms of ultimate strains or ductility which is a very important desired property of modern structures.

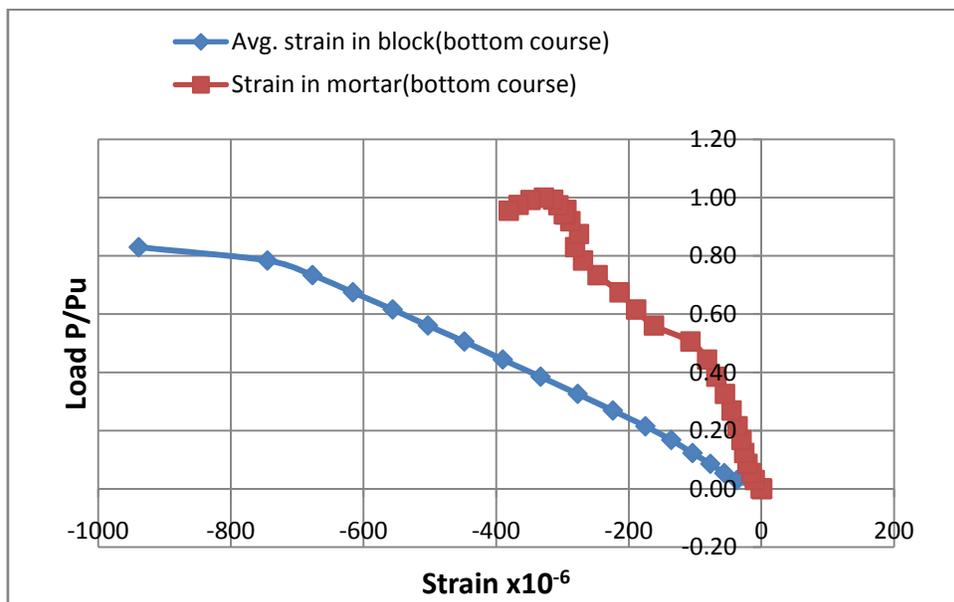
The study has research revealed that when alternative block walls are used; it is possible to obtain better performance in both strength and ductility. In conventional

walls, it is only possible to achieve good performance in either strength or ductility of the wall at the expense of the other. This has led to the conclusion that masonry construction is a brittle construction because the conventional material used allows one to achieve high strengths at the expense of limited ductility.

The stress versus strain curve for the large unreinforced conventional block wall is shown in Figure 8.8. Up to 80% load level, the compressive strain increases with stress in the top course. At 80% load level, there is strain reversal both in the block and the mortar. The strain reversal took place after attaining a value of 0.0005 compressive strains as compared to 0.001 and 0.0008 for block and mortar respectively in small wall of the same material. The ultimate strains in the blocks and mortar were 0.0004 and 0.0013, respectively. In the bottom course, the average strain in the block and mortar were compressive and increased with increase in load level until failure. The strain gauges in the blocks failed before the wall attained its capacity and hence, readings up to 80% load level were available. The failure strain for the mortar joint was 0.0003. The failure strains in the large walls were generally greater than the corresponding values in the laboratory standard unreinforced conventional block wall. The capacity of the large wall panel was also lower than the standard wall panel used even though the material was the same in terms of blocks and mortar joint. The large sized wall panels had compressive strength lower than the small wall panel with similar material.



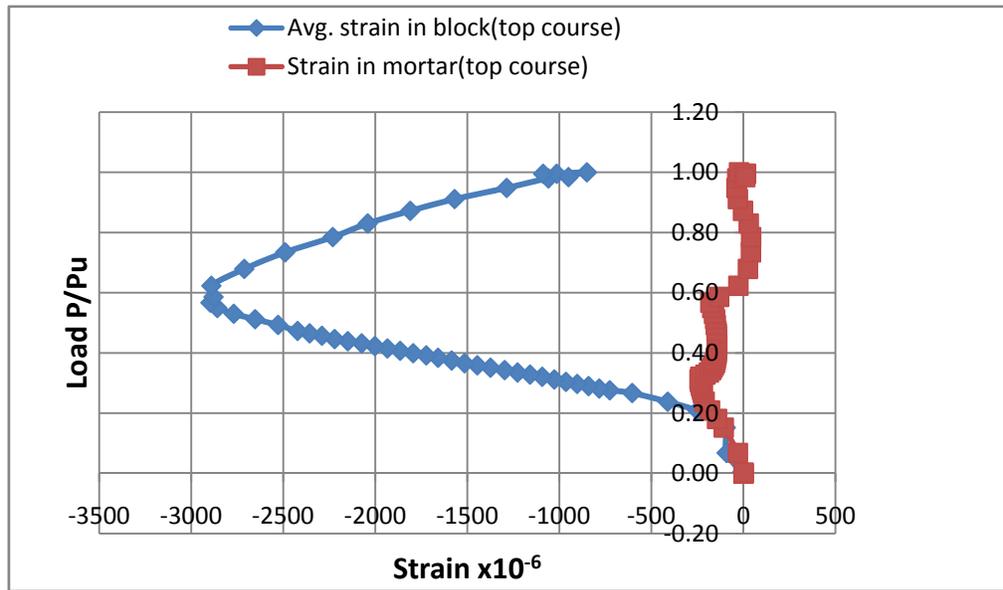
(a) Stress versus strain in top course



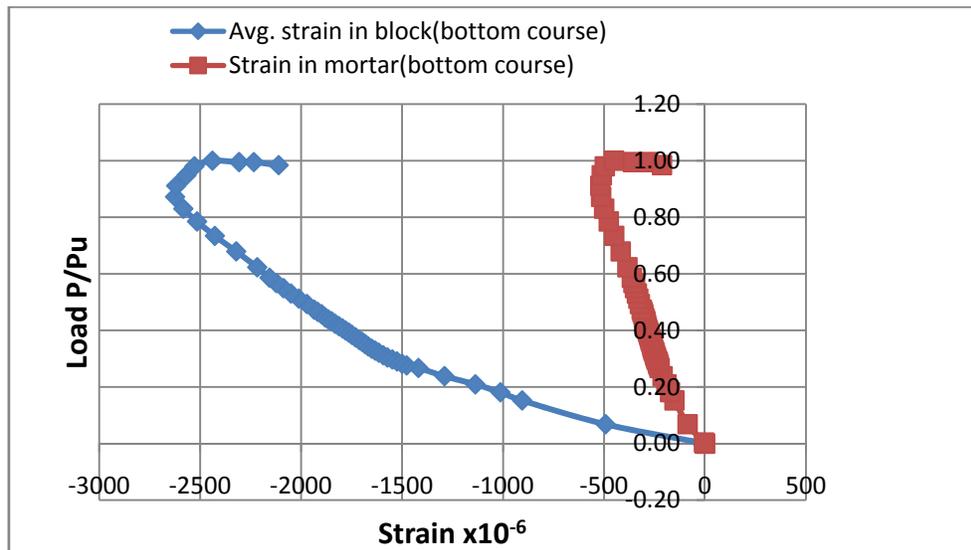
(b) Stress versus strain in bottom course

**Figure 8.8** Stress versus strain for large unreinforced conventional block wall jointed in 1:3 mortar joint

Figure 8.9 shows the stress versus strain curve for large unreinforced alternative block wall panel tested in axial compression. In this wall, the average strains at the top and bottom courses were compressive. In the top course, the compressive strain in the blocks increased with load until maximum strains at 60% load level were attained. Further increase in the load led to reduction in the average block strains. The failure strain in block at this level was 0.0008 compressive. The strains in mortar at this level were low as compared to the block strains. The mortar joint underwent strain reversal throughout the loading period with no strains at failure. In the bottom course, the curves for the block and mortar assumed the similar shape. The ultimate strains in the blocks and vertical mortar joint were 0.0026 and 0.0006, respectively. However, the strain levels in the blocks were generally greater than the mortar. As opposed to the conventional block walls, the strains in large unreinforced alternative block wall panel were generally higher than the standard laboratory wall panel size. The strains in blocks were 1.5 and 2.5 times higher for the top and bottom courses, respectively, while the strains in the mortar joints were 2 times higher. Like the conventional block walls, the capacity of the large wall in terms of stress was lower than that of the standard laboratory wall panel size used in the study. The capacity ratio for the two alternative block walls was 0.7.



(a) Stress versus strain in top course



(b) Stress versus strain in bottom course

**Figure 8.9** Stress versus strain for large unreinforced alternative block wall jointed in 1:3 mortar joint

In this study, the sizes of the wall panels investigated were varied by increasing the length of the walls. It was not possible to alter the height because the practical height possible was determined by the loading machine used. The small walls used in most of the investigation were 600 mm long while the large ones were 900 mm long, the height of the walls being constant. In both cases, the strengths of large masonry walls were lower than the small walls. In conventional walls the ratio of strengths was 0.700 while in the alternative block walls, the ratio was 0.688.

In conventional block walls, the ultimate surface strains on small wall panels were high as compared to the large walls. However there was strain reversal in the large wall panel during the loading period. Small alternative block wall had low surface vertical strains as compared to the large wall panel. Strain behaviour in alternative block wall was generally the reverse of the conventional block walls. In large alternative block wall panel, there was also strain reversal during the loading period.

### ***8.2.3 Cracking pattern***

Plate 8.1 shows the cracking pattern for reinforced conventional block wall after failure. The wall was build with conventional blocks jointed in 1:3 mortar. During loading cracks appeared in blocks and mortar in the top course. As loading continued the cracks widened while others appeared in the lower parts of the wall. The cracks in the lower part were few in number and were oriented in different directions. The cracks were also isolated as opposed to long continuous cracks. There were no signs of spalling of either block material of the jointing mortar.



**Plate 8.1:** Cracking pattern for reinforced conventional block wall utilizing 1:3 jointing mortar

The cracking behaviour of plain conventional block walls with 1:3 mortar joint is shown in Plate 8.2. At a lower load, cracks were formed in the top course of the wall panel. The cracks became wide and extended downwards the wall panel as loading was increased. At every load level, the cracks were generally wider than the reinforced wall of the same material. Compared to reinforced wall panel, the cracks were also continuous in nature as opposed to isolated short cracks experienced in the reinforced wall panel. Spalling of block surfaces and mortar joint was also noted in this wall panel. This occurred especially in the top course of the wall panel.



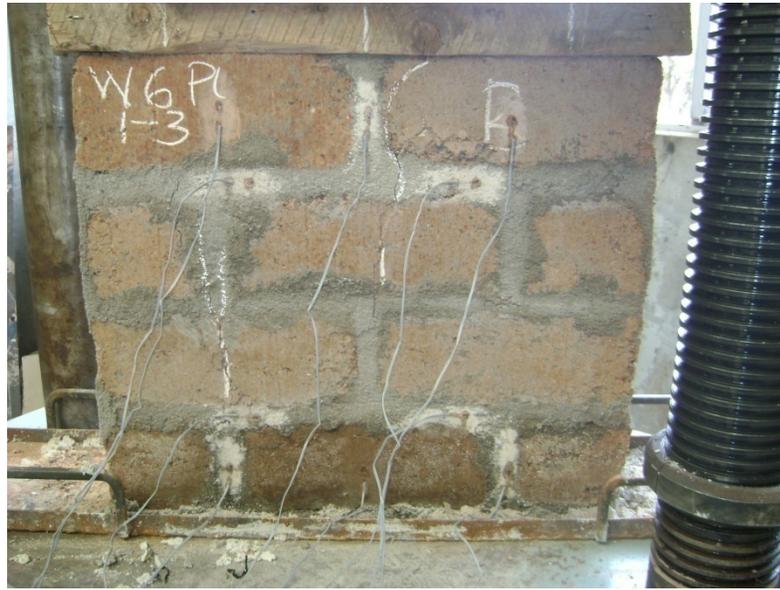
**Plate 8.2:** Cracking pattern for plain conventional block wall utilizing 1:3 jointing mortar

The cracking behaviour of alternative block wall generally differed from the conventional block walls because more spalling on blocks and mortar was experienced in the walls. Plate 8.3 shows the cracking pattern for reinforced alternative block wall with 1:3 mortar joint. During loading, cracks were formed in the top zone of the wall and they were generally isolated with their orientation in different direction. The crack pattern in the reinforced alternative block wall was generally similar to the reinforced conventional block wall. However, the alternative block wall had wider cracks as compared to the conventional block wall panel. As the load was increased, the blocks and mortar in the wall panel started to spall. The spalling of alternative wall surface was generally more than the spalling in the corresponding conventional block wall.



**Plate 8.3:** Cracking pattern for reinforced alternative block wall utilizing 1:3 jointing mortar

Plate 8.4 shows cracks formed in the unreinforced alternative block wall panel subjected to axial compression. Examination of the wall panel revealed that the cracking pattern of the wall was generally similar to the plain conventional block wall. The cracks in the alternative block wall were, however, wide and vertical in orientation. Continuous cracks formed in blocks and mortar running from the top to the bottom of the wall panel. The cracks were generally wider than those formed in reinforced alternative block wall.



**Plate 8.4:** Cracking pattern for plain alternative block wall utilizing 1:3 jointing mortar

The cracking pattern in masonry structures is very important because the ultimate behaviour of the structures depends on the size and orientation of the cracks. In conventional block walls, vertical cracks through the blocks and mortar were formed. The cracks in stronger mortar jointed wall were concentrated at the top zone which indicated lack of distributed deformation in the wall panel. Weaker mortar jointed wall on the other hand, formed cracks that were distributed along the height of the wall panel. Generally, the cracks that were formed in the weaker mortar jointed walls were wider than the strong mortar jointed walls.

Compared to conventional wall panels, the cracking patterns in alternative block wall panels were different. The stronger mortar jointed wall panel failed by developing vertical cracks running through the blocks and mortar joints. There were also signs of spalling of the blocks and mortar at ultimate load. The weaker mortar

jointed wall on the other hand failed by developing vertical cracks in the top zone of the wall. In the middle and the bottom zones, inclined and discrete cracks formed in different directions.

### 8.3 Cost comparison

In this study, the cost of the alternative block with optimum mix was compared with that of the conventional natural stone blocks. The results of the cost comparison studies are presented in Table 8.2. The cost of the conventional blocks is based on the market price of the material obtained from Ndarugu River in Thika District. The results show that the costs per block of alternative and conventional blocks are 23.50 and 40.00, respectively. Therefore, when used in construction, alternative masonry blocks will result in a saving of 41%. The saving is substantial since masonry form the main structural load bearing system in low rise residential houses.

**Table 8.2** Cost of conventional and alternative blocks

| <b>Materials</b>                             | <b>Alternative Block</b> | <b>Conventional Block</b> |
|--|--------------------------|---------------------------|
| Cement (kg/1000 blocks)                      | 480                      | -                         |
| QDNbi (kg/1000 blocks)                       | 2000                     | -                         |
| QDNdar (kg/1000 blocks)                      | 2900                     | -                         |
| Laterite (kg/1000 blocks)                    | 4700                     | -                         |
| Compressive strength<br>(N/mm <sup>2</sup> ) | 4.07                     | 4.30                      |
| Cost (KShs./ 1000 blocks)                    | 23,500                   | 40,000                    |
| Cost (KShs./ block)                          | 23.50                    | 40.00                     |

#### **8.4 Summary and conclusion**

In this chapter, the results of studies on wall panels utilizing conventional as well as alternative blocks have been presented. The structural behaviour of alternative block walls has also been compared with the conventional block walls. The effect of mortar joint strength on the structural behaviour has also been evaluated. Other variables such as the effect of bedding joint reinforcement, the size of walls have also been discussed. From the results, it is noted that the effect of strength of the mortar joint on the behaviour of walls is similar for both the alternative and conventional block walls. Weaker mortar jointed walls generally had higher compressive strength than the stronger mortar joint wall panels. In addition, the effect of bedding joint reinforcement on behaviour of walls was found to be similar for alternative and conventional wall panels. In both wall types, the effect of bedding joint reinforcement delayed the formation of the first crack in the wall panels and reduced final crack width at failure load of the walls. However, the compressive strengths of conventional block walls were generally higher than the alternative block walls.

Structural design seeks to achieve both strength and ductility in any system for satisfactory performance. The results in this study revealed that when alternative blocks are used, it is possible to achieve strength and ductility in wall panels at the same time. In conventional masonry construction this is not possible and the designer can only achieve one property at the expense of the other.

## CHAPTER NINE

### 9.0 CONCLUSION

This study has examined the properties of alternative aggregates and filler materials for production of alternative concrete and eco-blocks, respectively. The viability of the alternative concrete and blocks in structural elements has also been evaluated. The findings of this study clearly demonstrate the great potential of laterized quarry dust concrete and cement stabilised laterized quarry dust blocks in construction. Reinforced concrete beams and eco-masonry walls produced from the alternative concrete and blocks, respectively, show great viability in sustainable building construction. More specifically, the study determined that;

1. The main material physical properties of quarry dust, laterite and recycled concrete aggregates are different from the conventional materials. All the alternative aggregates and filler materials investigated fall outside the standard gradation curves specified by both the British and American Standards. Their silt content as well as water absorption was generally at least twice as great as the conventional material.
2. Alternative concrete made from laterized quarry dust and a recycled concrete aggregate has performance comparable to conventional concrete. The compressive strength of the alternative as well as conventional concrete of similar mix ratios were within 10%. The workability of fresh alternative concrete was generally within the standard limits for concrete placed and compacted by internal vibrator.

3. Alternative blocks made of blended laterite soil and quarry dust can attain compressive strengths comparable to the conventional machine dressed stones. Any strength is achievable with alternative materials by varying the cement content. When stabilized with 13% of Portland cement, lateritized quarry dust blocks can attain compressive strength of  $6 \text{ N/mm}^2$  at 28 days. The compressive strength and water absorption of the blocks when 7% of cement is used satisfy the requirements of the Kenya Standards.
4. Alternative concrete beams have performance behavior similar to the conventional concrete beams. The flexural capacity of reinforced concrete beams utilizing alternative and conventional concrete beams with similar reinforcement content and concrete compressive strength were within 5%.
5. Alternative block wall panels can achieve both strength and ductility while conventional block wall panels can only achieve either strength or ductility at the expense of the other. However, the normalized compressive strengths of the alternative and conventional wall panels were within 5%.

## CHAPTER TEN

### 10.0 RECOMMENDATIONS

This study attempted to ensure that important information on alternative building materials was obtained. However, there are other aspects which were not evaluated although they may affect the overall performance of the materials. Therefore, the following recommendations are proposed.

1. Further studied should be carried out on concrete beams made from the alternative concrete to determine the effects of shear, compression, bond and deflection on performance of the structural elements. The present study focused on the flexural behavior of beams. Despite the fact that flexural behavior dominate in beams, other aspects such as shear, bond and deflection affect the design and overall performance of the element in service.
2. There is need for further research on the performance of concrete slabs and columns made from alternative concrete. Slabs and columns form key structural elements in housing projects and hence, knowledge of their structural behavior is required. In the present study, only concrete beams were investigated.
3. The findings on masonry walls made from eco-blocks are based on uniaxial compressive stresses in the walls. In real structures, a generalized tri-axial stresses are likely to be induced especially where lateral forces are significant. Further studies are therefore recommended to ascertain the behavior of wall panel made from eco-blocks when subjected to multi-directional stresses.

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## APPENDICES

### Appendix A. Concrete mixes

**Table A1.** Concrete mixes for conventional and alternative concrete

| MATERIALS                                    | CONVENTIONAL<br>CONCRETE |         | ALTERNATIVE<br>CONCRETE |         |
|--|--------------------------|---------|-------------------------|---------|
|  |                          |         |                         |         |
| Mix ratios                                   | 1:2:4                    | 1:1.5:3 | 1:2:4                   | 1:1.5:3 |
| Cement (kg/m <sup>3</sup> )                  | 314                      | 390     | 314                     | 390     |
| River sand (kg/m <sup>3</sup> )              | 627                      | 585     | 439                     | 410     |
| QDNbi (kg/m <sup>3</sup> )                   | -                        | -       | 94                      | 88      |
| Laterite (kg/m <sup>3</sup> )                | -                        | -       | 94                      | 88      |
| NCA (kg/m <sup>3</sup> )                     | 1255                     | 1171    | -                       | -       |
| RCA (kg/m <sup>3</sup> )                     | -                        | -       | 1255                    | 1171    |
| Water/cement ratio                           | 0.65                     | 0.65    | 0.65                    | 0.65    |
| Water content<br>(kg/m <sup>3</sup> )        | 204                      | 254     | 204                     | 254     |
| Compressive strength<br>(N/mm <sup>2</sup> ) | 18                       | 23      | 17                      | 21      |
| Cost (KShs./m <sup>3</sup> )                 | 7700                     | 8650    | 6314                    | 7400    |

In the table, NCA is Natural crushed aggregates; RCA is recycled concrete aggregates and in the mix ratio for example 1:2:4, the ratios are given for Portland cement, fine aggregates and coarse aggregates respectively.

## Appendix B. Masonry block mixes

**Table B1.** Block mixes for conventional and alternative blocks

| <b>MATERIALS</b>                             | <b>ALTERNATIVE<br/>BLOCK</b> | <b>CONVENTIONAL<br/>BLOCK</b> |
|--|------------------------------|-------------------------------|
| Cement (kg/1000 blocks)                      | 480                          | -                             |
| QDNbi (kg/1000 blocks)                       | 2000                         | -                             |
| QDNdar (kg/1000 blocks)                      | 2900                         | -                             |
| Laterite (kg/1000 blocks)                    | 4700                         | -                             |
| Compressive strength<br>(N/mm <sup>2</sup> ) | 4.07                         | 4.30                          |
| Cost (KShs./ 1000 blocks)                    | 23,500                       | 40,000                        |
| Cost (KShs./ block)                          | 23.50                        | 40.00                         |

## Appendix C. Concrete beams crack patterns



(a) Cracks pattern during testing



(b) Final cracks pattern at ultimate load

**Plate C1** Beam AltConc1-2-4. Beam utilising alternative concrete of mix ratio 1:2:4 for cement, fine aggregates and course aggregates, respectively and reinforced with 2Y10 bars.



(a) Cracks pattern during testing



(b) Final cracks pattern at ultimate load

**Plate C2** Beam ConvConc1-2-4. Beam utilising conventional concrete of mix ratio 1:2:4 for cement, fine aggregates and coarse aggregates, respectively and reinforced with 2Y10 bars.



**(a)** Cracks pattern during testing



**(b)** Final cracks pattern at ultimate load

**Plate C3** Beam AltConc1-1.5-3(10). Beam utilising alternative concrete of mix ratio 1:1.5:3 for cement, fine aggregates and course aggregates, respectively and reinforced with 2Y10 bars.



**(a)** Cracks pattern during testing



**(b)** Final cracks pattern at ultimate load

**Plate C4** Beam ConvConc1-1.5-3(10). Beam utilising conventional concrete of mix ratio 1:1.5:3 for cement, fine aggregates and course aggregates, respectively and reinforced with 2Y10 bars.

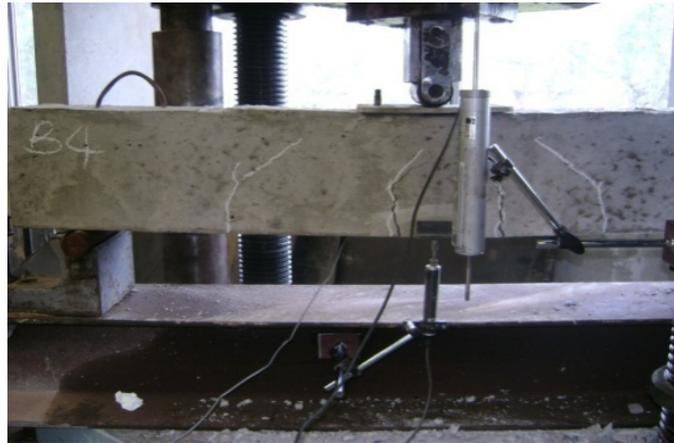


**(a)** Cracks pattern during testing



**(b)** Final cracks pattern at ultimate load

**Plate C5 Beam AltConc1-1.5-3(8).** Beam utilising alternative concrete of mix ratio 1:1.5:3 for cement, fine aggregates and coarse aggregates, respectively and reinforced with 2Y8 bars.



(a) Cracks pattern during testing



(b) Final cracks pattern at ultimate load

**Plate C6** Beam ConvConc1-1.5-3(8). Beam utilising conventional concrete of mix ratio 1:1.5:3 for cement, fine aggregates and course aggregates, respectively and reinforced with 2Y8 bars.



**(a)** Cracks pattern during testing



**(b)** Final cracks pattern at ultimate load

**Plate C7** Beam AltConc1-1-2. Beam utilising alternative concrete of mix ratio 1:1:2 for cement, fine aggregates and course aggregates, respectively and reinforced with 2Y10 bars.



**(a)** Cracks pattern during testing



**(b)** Final cracks pattern at ultimate load

**Plate C8** Beam ConvConc1-1-2. Beam utilising conventional concrete of mix ratio 1:1:2 for cement, fine aggregates and course aggregates, respectively and reinforced with 2Y10 bars.







