USE OF WASTE AND LOW ENERGY MATERIALS FOR CONSTRUCTION

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PhD (OCTOBER 2008)

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A thesis submitted to Cardiff University for the Degree of Doctor of Philosophy in Engineering

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October, 2008

DECLARATION

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ACKNOWLEGEMENT

To God be the glory, great things He has done. This work would not have been possible without the support of Takoradi Polytechnic institute, NUFFIC, all the technical, administrative and academic staff of the Cardiff University Engineering Department.

I am especially grateful to Des, Carl and Len, without whom things would not have been connected or tested and to Dr. Alan Kwan for the first class support and supervision.

Lastly, a special mention of thanks to parents, Joseph and Mary Yalley, for support through more than 20 years of education and my wife, Elizabeth Yalley. This thesis is dedicated to the three of them.

ABSTRACT

Considerable work has been done on the mechanical properties of coconut- fibre enhanced concrete. The primary test variables were the fibres weight fraction, and fibres aspect ratio. The addition of coconut-fibres significantly improved many of the engineering properties of concrete, notably torsion, toughness and tensile strength. The ability to resist cracking and spalling were also enhanced. However, the addition of fibres did not improve the compressive strength, as expected, due to difficulties in compaction which consequently lead in increase of voids. When coconut fibre was added to plain concrete, the torsional strength increased (by up to about 25%) as well as the energy-absorbing capacity, but there is an optimum weight fraction (0.5% by weight of cement) beyond which the torsional strength started to decrease again. Similar results were also obtained for different fibre aspect ratios, where again results showed there was an optimum aspect ratio (125). An increase in fibre weight fraction provided a consistent increase in ductility up to the optimum content (0.5%) with corresponding fibre aspect ratio of 125.

The second part of this research, reports on the investigation on cement stabilised soil block. A local soil was stabilised chemically by cement. A better compressive strength at the dry state and after two hours of immersion in water was obtained with chemical stabilisation at cement content of 5%. Blocks stabilised with 5% cement content by weight of soil has a dry and wet compressive strength of 6.64 and 2.27MPa respectively, and dry density of 1910 kg/m^3 at an optimal water content of 12% by weight of cement. The highly decreased compressive strength after two hours of immersion in water, even with higher cement content, indicated that appropriate building design that would prevent stabilised soil blocks from coming into direct contact with rainwater was important.

A newly proposed concept of a plastic carton soil blocks as masonry units for low-cost environmentally friendly construction is proposed in the final part of the thesis. A test system was designed to perform rigorous and comprehensive measurements on seven types of soil block specimens encased in thermoplastic cartons. The cartons were similar to "ice cream tubs" of dimensions 165x60x120mm, thus making a building block/brick of reasonable handling size. Some of the test specimens also had soil mixed with palm or plastic fibres.

Thermoplastic carton soil blocks without the addition of fibres as an enhancement were measured with a minimum compressive strength of 17.5MPa. Even so it should be noted that 17.5MPa is still a very reasonable strength and over half that of a typical concrete block. In the case of the fibre enhanced soil block, the compressive strength increased with increase in fibre content. With fibre addition of 1.5% (by weight), the compressive strength of the thermoplastic cartons increased by 28.5% and 38% respectively for palm and plastic fibres, over the plain thermoplastic carton soil block without fibres. For increase in fibres content from 0.75% to 1.5% (i.e. a doubling of fibre content) the compressive strength increased by only about 20% to 23%. Additionally, stiffness is also greatly improved.

A finite element model was constructed for the thermoplastic carton soil block geometry and input files were generated for non-linear static analyses in MSC Patran. Very good agreement was achieved between the numerical predictions and experimentally measured results in both size and shape of the stress-strain graphs.

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GLOSSARY

Abrasion: The mechanical wearing, grinding, scraping or rubbing away (or down) of a material surface by friction or impact, or both.

Absorption: Weight of water incorporated by a concrete or soli specimen unit during immersion under prescribed conditions, typically expressed as a percentage relating to the dry weight of the unit.

Admixture: Prepared chemicals added to the concrete during the mixing process to improve production efficiencies and/or hardened properties such as density, absorption, efflorescence control, visual appeal, durability and strength.

Aggregate: Sand, gravel, shell, slag, or crushed stone used in base materials, mixed with cement to make concrete, or with asphalt.

Aspect Ratio: The overall length of fibre divided by its thickness (diameter).

Atterberg limit: Comprises the liquid and plastic limits test, which is used as one of the means of classifying soil.

Block: A larger type of brick not necessarily made of fired clay, but stabilised in someway, sometimes with central cores removed to reduce the weight.

Brick: An object (usually of fired clay) used in construction, usually of rectangular shape, whose largest dimension does not exceed 300mm.

Bulk Density: Density calculated including any moisture present in the material.

Cement-Aggregate Ratio: The proportional weight of cement to fine and coarse aggregate in concrete.

Cement, Portland: Hydraulic cement produced by pulverizing clinker consisting essentially of hydraulic calcium silicates, and usually containing one or more forms of calcium sulphate.

Coconut fibre: Mature fibre from coconut husk

Concrete: The finished form of a mixture of cement, sand, aggregate and water.

Clay: Fine-grained soil or the fine-grained portion of soil that can be made to exhibit plasticity (putty-like properties) within a range of water contents, and that exhibits considerable strength when air-dry. The term can designate soil particles finer than 0.002 mm (0.005 mm in some cases).

Coarse Aggregate: Aggregate predominantly retained on the U.S. Standard No. 4 (4.75 mm) sieve; or that portion of an aggregate retained on the No. 4 (4.75 mm) sieve.

Compaction: The process of inducing close packing of solid particles such as soil, sand, or aggregate.

Compressive Strength: The measured maximum resistance of a concrete or mortar or block or any material to loading expressed as force per unit cross-sectional area.

Consolidation: subjection of soil specimen to the state of effective stress required.

Consolidated-undrained test, (CU): The undrained shear strength of a soil specimen subjected to a known initial stress

Degradation Testing: Testing of sands or aggregate to determine resistance to change in particle size or gradation under loading.

Density: The mass per unit volume.

Dry Density: The calculated density at ejection assuming no moisture is present in the formed sample, only solid matter.

Dynamic Compaction: A process that densifies soil by applying a series of impact blows to it.

Fines: General category of silts and clays.

Elastic Deformation: A reaction from applied loads where concrete specimen or soil block returns to its original position after the load is removed. Compare to permanent deformation under loading condition.

Fines: Silt and clay particles in a soil, generally those smaller than the No. 200 or 0.075 test sieves.

First crack deformation: is defined as the value corresponding to the point on the torsion- twist plot, when the matrix has ruptured and whatever the load remaining is taken by the fibre. This is interpreted as the point of maximum torsion.

First crack toughness: is the area under the torsion- twist curve up to the maximum torsion, in other words area under the elastic part of the torsion-twist curve.

Gradation: Soil, sand or aggregate base distributed by mass in specified particle-size ranges. Gradation is typically expressed in percent of mass of sample passing a range of sieve sizes. See ASTM C 136.

Gravel: Rounded or semi-rounded particles of rock that will pass a 3 in. (75 mm) and be retained on a No. 4 (4.75 mm) U.S. standard sieve which naturally occurs in streambeds or riverbanks that have been smoothed by the action of water. A type of soil as defined by the Unified Soil

Classification System having particle sizes ranging from the No. 4 (4. 75 mm) sieve size and larger.

Green: Describing the state of material containing cement and water before it reaches the critical time, after which further plastic deformation hinders the final set strength.

Liquid limit: Moisture content at which a soil changes from the liquid state to the plastic state.

Mix proportion: concrete mixed ratio of cement: sand: stones: water.

Modulus of Elasticity or Elastic Modulus: The ratio of stress to strain for a material under given loading conditions.

Moisture Content: The percentage by weight of water contained in the pore space of soil, sand or base, with respect to the weight of the solid material.

Mortar: A mixture of cement paste and fine aggregate (sand).

Mortar Sand: Sand used in mortar that typically conforms to ASTM C 144 or CSA A179.

Oil palm fibres: fibres from oil palm nut.

Optimum Moisture Content: The water content at which a soil can be compacted to a maximum dry unit weight by a given compactive effort.

Organic Impurities: Peat, roots, topsoil or decomposing materials in soil, sand or aggregate.

Organic Soil: Spongy, compressible soils usually consisting of peat humus or vegetative matter that have undesirable construction characteristics.

Permeability: Describing the degree a material that permits a liquid or gaseous substance to travel through the material.

Plastic fibres: fibres from polythene plastic.

Plastic Limit: (1) The water content corresponding to an arbitrary limit between the plastic and the semisolid states of consistency of a soil. (2) Water content at which a soil will just begin to crumble when rolled into a thread approximately 1/8 in. (3.2 mm) in diameter.

Porosity: A measure of the void volume as a percentage of the total material volume

Sand: Granular material passing the 3 / 8 in. (5 mm) and retained on the No. 200 (0.075 mm) sieve, made from the natural erosion of rocks, and consisting of subangular or rounded particles. Sands made by crushing of coarse aggregates are called manufactured sands.

Saturation: A state when the entire void in the soil was filled with water,

Shrinkage: The reduction in volume in soil when moisture content is reduced. **Silt**: Soil finer than 0.02 mm and coarser than 0.002 mm (0.5 mm and 0.005 mm in some cases).

Slump: A measure of consistency and water content of freshly mixed concrete. Slump is the subsidence measured from a specimen immediately after removal of a cone shaped mold. See ASTM C 143.

Soil Stabilisation: Chemical or mechanical treatment designed to increase or maintain the stability of a mass of soil or otherwise to improve its engineering properties. Lime, fly ash or cement are typical chemical stabilization materials. Geotextiles and geogrids are typical mechanical materials for soil stabilization.

Spall: A fragment, usually in the shape of a flake, detached from the edge or surface of a paver by a blow or sudden force, the action of weather, or pressure from adjacent pavers.

Sustainable Development: Development (including pavement) that meets the needs of the present without compromising the ability of future generations to meet their own needs.

Tensile Strength: Maximum unit stress which a paver is capable of resisting under axial tensile loading, based on the cross-sectional area of the specimen before loading.

Thermoplastic carton soil block: soil block specimens encased in thermoplastic cartons

Top soil: Surface soil, usually containing organic matter.

Toughness of concrete: Energy absorption capacity and is defined here as the area under the torsion-rotation curve calculated up to a specific rotation value.

Void Ratio: The volume of voids around the aggregate in an open-graded base divided by the volume of solids.

Water-Cement Ratio: The weight of water divided by the weight of cement in a concrete mixture.

CHAPTER ONE

INTRODUCTION

Scientists, engineers and technologists are continuously on the lookout for new materials which can be used as substitutes for conventional materials, especially where their properties would enable their use in new designs and innovative applications. There is also an increasingly awareness of the need to re-use or recycle waste. The successful utilisation of waste materials depends on its use being economically competitive with the alternative natural material. These costs are primarily made up of handling, reprocessing, and transportation. Industrialisation and population growth in developing countries and global market economy has resulted in an increase in agricultural output and manufactured waste, and in the context of this project, particularly the accumulation of plastic waste.

1.1 Problem statement

The greatest material need in the world today is the need for housing for sustainable development of civilization. In western countries such as the U.S.A, housing is no longer affordable by large percentage of Americans. Currently in the United Kingdom, price of houses is at its lowest level in ten years, according to HSBC housing survey (Financial Times Friday 3rd October, 2008), not because of appropriate technology, but because people can not secure a mortgage or make savings to buy a house due to the credit crunch the financial sector is facing. In the developing countries, housing is both substandard and expensive. Therefore affordable housing and the ability to sustain civilization without destroying the environment are the critical needs in every country of the world today. Unless we solve the world housing shortage and provide means for people to sustain themselves in a life supporting environment, the world may erupt into competing battles for resources.

1.2 Purpose

The purpose of this project is to investigate the mechanical properties of alternative building materials for sustainable low cost building using available waste and low energy materials, principally for low-rise construction and especially for domestic dwelling.

This study thus investigates the potential of using waste and low energy materials for domestic construction, principally in Africa. Materials under consideration are coconut fibre, plastics containers (thermoplastics-polyethylene and polypropylene) and waste plastics.

1.3 Justification for this work

There is a self-evident need for adequate and durable housing, especially in the urban and peri-urban areas of developing countries. The poor are most adversely affected by this housing shortage. Assuming land availability and planning permission for further development, the need is to increase the number of durable housing at lower cost.

The cost of a typical dwelling can be split into a number of separate areas as follows:

1. Initial land survey
2. Land preparation on paper – division into plots with access, (needs approval)
3. Physical preparation of ground – clearing vegetation, debris, boulders, etc.
4. Installation of services (optional) – water, sewerage, electricity and telephone
5. Purchase of the plot – cost direct to the homebuilder
6. House erection – foundations and walling (entailing materials and labour)
7. Roofing – spanning beams and roof material
8. Openings – windows and doors with fittings

9. Services – connection up to services if available, (optional, may require approval)

Items 6 to 8 constitute the most significant part of the total cost of the dwelling totalling 45% of cost of typical domestic building (Carter, 2008). Furthermore, the walling constitutes the most significant part of the physical structure, typically 60% according to Agevi (1999). From this, there is good sense to concentrate work on low-cost walling, foundation and roofing. The current research studied the use of waste plastic carton for soil block and provides a method of improving the performance of stabilised waste plastic carton soil blocks for walling and at a reduced cost

A further motivation for research into plastic soil blocks is their environmental sustainability. Even if the proposed Thermoplastic Carton Soil Blocks (TCSB) were to be chemically stabilised, it would use only low quantities of cement, and locally available soil and thus have a low energy requirement. Currently popular alternatives such as clamp fired bricks and concrete blocks do not have these advantages.

1.4 Scope

This thesis covers a laboratory investigation on the use of i) natural fibres as enhancement to concrete, ii) cement stabilised soil block and iii) the performance of soil blocks made from waste plastic containers known as Thermoplastic Carton Soil Block (TCSB). The thesis also covers a finite element simulation model on the thermoplastic carton soil block.

After this introduction comes the background on housing situation in Ghana, and a literature review in Chapters Two and Three respectively. Experimentation on natural fibre as enhancement of concrete is described in Chapter Four. Chapter Five concerns the Testing and analysis of cement stabilised soil block. The production, testing and analysis of thermoplastic carton soil block are then reported in Chapter Six. Chapter

Seven details the comparison of experimental results from Chapter Six to that of simulation using finite element modelling of the thermoplastic carton soil block. Finally Chapter Eight summarises the conclusions made throughout the thesis and makes recommendations for further research to be conducted.

CHAPTER TWO

BACKGROUND TO THE HOUSING SITUATION IN WEST AFRICA

2.0 INTRODUCTION

This chapter provide a brief overview of the general housing situation in West Africa in general, as typified by Ghana. Firstly, this chapter briefly assesses the need for alternate building materials in developing countries in general (but focussing on Ghana as a principal example), the housing situation in Ghana and the nature of house building in rural Ghana.

2.1 Housing situation in Ghana

It is a fair reflection to state that most Ghanaians are under a variety of livelihood pressures, especially those on low income and those with uncertain access to secure land. Increasingly, urbanisation is a contributing factor to poor housing conditions. The urban population around the country is expected to double between the years 2005 and 2020 (Yaw Barimah, 2004). The pattern of foreign direct investment, made in partnership with Ghanaian investors, deeply affects urbanisation and infrastructure investment. The impact of urbanisation in Accra, for example, is especially apparent. Study by Osei Tutu et al. (1993) estimates that 61% of metropolitan Accra lives in "informal housing" (i.e. houses owned by individual(s)) An analysis of housing conditions reveals that on a national basis, 48.9% of all Ghanaian households live in accommodation associated with the "compound" (44.5% live in compound rooms). These traditional housing in Ghana takes the form of rooms in a household encircling a shared open compound. Another 25.3% live in detached houses while 15.3% reside in semi-detached houses.

Moreover, 57.4% claim ownership of their dwellings 22% rent their dwellings and while another 19.5% live rent-free. The last group of dwellers could have their accommodation through knowing the head of household or landlord and are exercising certain traditional kinship rights. The rent-free dwellers could also be in a household built by late or living relative. A further 2% of households live in public property set aside as rentals for government civil servants while private employers also provide housing for 4.5% of the population. The situations described above are typical of West African states.

Past and current direct intervention efforts by Ghanaian government have failed to reach the low income target groups or meet their housing requirements. It is estimated that in the last 15 years (i.e. from 1993), at most 50,000 detached and semi-detached houses have been constructed by the private real estate development industry, often with government subsidies. Between 1984 and 1999, about 900,000 additional dwellings were built. The population growth for that period was about 3.4%, while the rate of delivery for houses was about 1.4% which is obviously inadequate to meet the accumulative housing deficit (Frempah, 2000).

Presently, studies have shown that, there is a housing deficit of between 300,000 to 400,000 units in Ghana. The house price to annual income ratio for the average Ghanaian worker is 12 (Akuffo, 2006). It is a known fact that more than 70% of the population cannot afford the cost of the current houses being developed by private investors. Low and medium income families are affected by the shortage of houses and thus have restricted access to basic infrastructure services like water, sanitation and drainage, which in turn seriously hinders personal economic development and directly undermine basic living standards of the majority of the Ghanaian population, most of whom are living in the rural areas (Bochie, 2005).

2.2 Types of Ghanaian Houses

There is a need to briefly outline the type of houses already existing in rural areas, before we go further to suggest the type of houses that are durable and affordable to meet the housing need of the rural population, and the urban poor in the slums of the cities. The reasons for these type of dwellings, and not some other type of dwellings, would be down to availability or abundance of the required raw material, affordability, traditional construction techniques, level of technical knowledge required for successful construction, and social acceptability.

a) Aktagbame

Houses in most Ghanaian villages are built using mud - a mixture of clay and water (or rammed earth), and thatch is used for the roof. This type of house is called "aktagbame". The whole community contributes to the building process and the houses are quite strong in that they normally remain in place for about twenty years if regularly maintained. Rammed earth is more often considered for use in walls, and floors. Rammed earth offer great potential as low-cost material alternatives with low embodied energy. Such buildings are neither fireproof (due to the thatched roof) nor completely waterproof.

b) Sunburnt brick wall building with pitched roof

Sunburnt brick buildings with pith roofs are found both in urban and rural areas throughout Ghana. This construction type is an improvement over the traditional "aktagbame," or adobe, and is gaining popularity at the moment. They constitute an estimated 25% of the Ghana's rural housing stock. The thatched roof is supported by sunburnt mud brick walls set in mud mortar. The walls are built on a stone platform raised above ground for the purpose of protection from floods. These buildings are built without any horizontal or vertical bar reinforcement. However, the strength of the building is relatively low compared with conventional sandcrete block houses. This type of construction is considered to be very vulnerable to earthquake effects, which, though infrequent and mild by world standards, is still a realistic threat in the region (especially in the South). Typical life span for buildings of this construction type is about 50 years.

Figure 2.1 Rehabilitation of sunburnt house using aktagbame method of construction

2.3 Architectural Features of buildings in rural Ghana

In this section, we explore some of the architectural features prevalent in domestic dwellings of rural Ghana. Although new (or "foreign") architectural features can be introduced with new housing technology, it is usually advisable to pay attention to traditional practices and preferences so that any new product has greater likelihood of being accepted and embraced by the local community.

Building shapes are usually rectangular in Southern Ghana but are typically circular in the northern parts of Ghana.

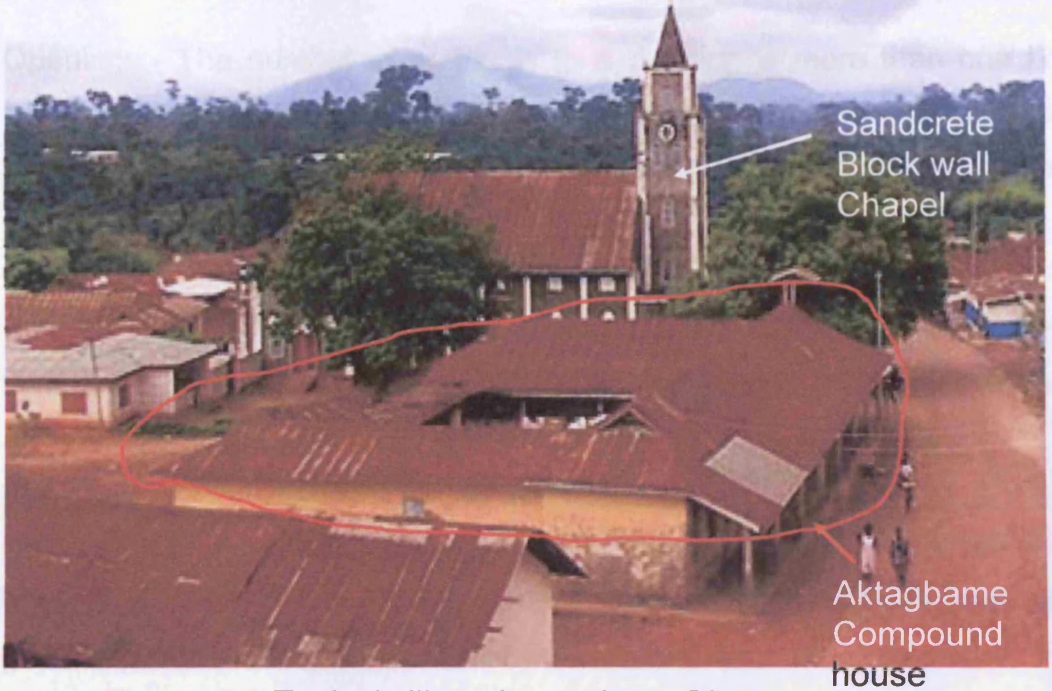


Figure 2.2 Typical village in southern Ghana

Rammed earth with thatched roof typical houses in Northern Ghana

Figure 2.3 Typical village in northern Ghana.

Openings:- The number of openings to a dwelling is more than one i.e. there could be two doors and two to three windows depending on the size of the building. The typical opening area as a fraction of the wall surface area is 8%.

Siting:- The typical separation distance between buildings of different compound is about two meters. In some case, buildings in rural Ghana have common walls with adjacent buildings as indicated in Figure 2.4

Means of Escape: - Most dwellings have no means of alternative fire escapes unless they have two door openings. Domestic windows are generally too small to also serve as fire escapes.

2.4 Building Function and patterns of occupancy

The main function of buildings in rural Ghana is for dwellings purposes. Some rooms of these dwelling units are sometimes used for commercial purposes. There is an average of seven bedrooms in each dwelling unit (mostly "compound" house) and in most cases with shared living rooms. Generally a small family (a family of two) occupies one room. A family with children occupies two or more rooms depending on number of children and the parents' ability to afford more rooms. In general there is an average of four families in one dwelling unit. Bathrooms and toilets are externally provided and are not joined to the housing unit. There is more than one kitchen in a dwelling unit, usually closer to some of the rooms, and the source of fuel is fire wood (see Figure 2.4).

2.5 Economic status of inhabitants

Most of the inhabitants in rural Ghana can be classified as "very poor." They have no regular employment or wage, and most are subsistence farmers in additional to being a casual labourer. The monthly income for such a family is about £20 while the cost of a room is £2.00 per month (both in 2005 terms).

2.6 Typical sources of financing

There is little formal financing (e.g. banks, housing associations, or government agencies) for dwellings in rural Ghana. The typical source of financing for buildings is owner finance from personal savings, or gifts/borrowing from an informal network of relatives and friends. Although some building materials have to be purchased and hence money is required, substantial unaccountable resource in the form of communal building labour should not be underestimated. Any new proposal from this study should minimise requirement for monetary expenditure and maximise the available resource of "free" labour.

2.7 Structural features

Rural domestic dwellings are typically single-storey and are thus relatively simple structural systems. The wall takes the load from the pitched roof and other connected wall elements, and the walls are placed at right-angles to each other (or are circular) to provide lateral load-resistance. The walls are placed on a raised stone platform as a way of keeping them above ground/surface water levels during the rainy season. This platform may be considered as a foundation because it projects outside the wall thickness and is generally constructed of good stone. However, the connection between the wall and the raised platform is not interlocking, so there is no significant transfer of lateral forces other than frictional forces; the wall merely rests on the platform. The connection between the roof and the walls again do not provide lateral transfer of forces. The roof is supported by wooden post (bamboo in the south and wood from the "shea butter" tree in the north), which is embedded in the walls

Gravity load-bearing structure: - The roof loads are supported on the timber members which are supported by wooden post embedded in the walls. Generally gable walls are used both internally as room partitions and at the extreme ends of the building. The walls are made of one of the following: mud walls, with horizontal wood elements serving as lintel, adobe block, brick walls or rammed earth. The building foundation is usually stone raised walls, built to support walls and for rainwater clearance. Thatched roofs supported on wood purlins, and sometimes iron sheets are used as roof cover to the buildings. The buildings have the following typical dimensions per room, length: 3 meters, Width: 3 meters and a height 2.4 meters. These dimensions can vary from one household to another depending on the financial ability of the household. Most houses in typical Ghanaian villages have Kumasi Improved Ventilated Pit toilet (KVIP) (designed by the University of Science and Technology, Kumasi in Ghana) outside the compound, with few households having water closet within the compound.

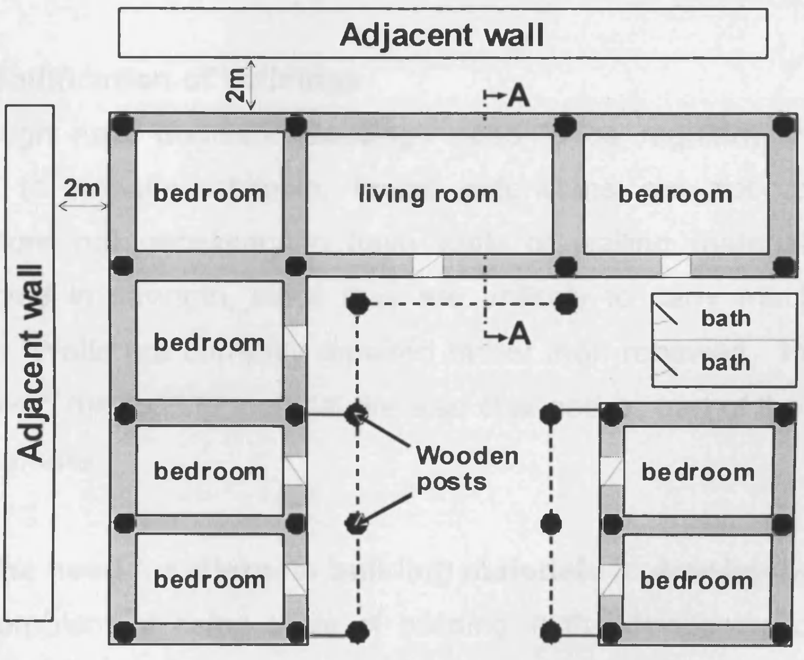
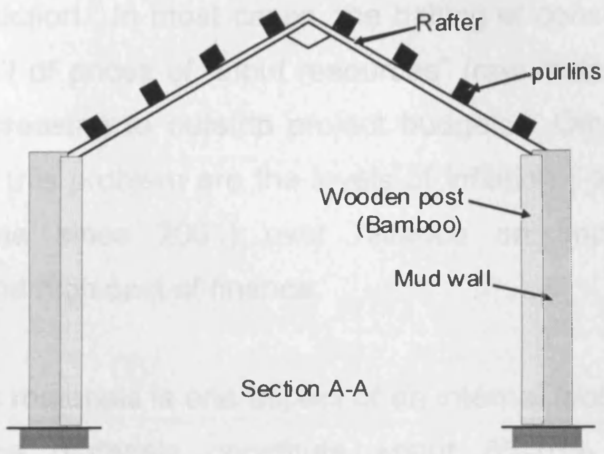


Figure 2.4 Typical building plan for rural dwelling unit in Ghana



Section A - A

2.8 Modification of buildings

Although rural domestic dwellings need to be regularly maintained for them to remain habitable, house extensions are not common. It is therefore not necessary to have walls or walling materials with much reserved in strength, since they are unlikely to carry much load in the future. Walls are currently repaired rather than renewed. In some cases however, the roofing material are also changed as part of the maintenance programme.

2.9 The need for alternate building materials in developing countries

The problem of rising costs of building in the developing countries has existed for sometime now, and especially in rural areas where the local income has often not increased at the same pace as the national average. This has been a source of concern to governments (Frempah 2000). The situation has rather assumed a greater dimension over the last few years. In Ghana for example, rising cost has led to the abandonment of certain construction projects that were already underway. The reason is not just a problem of an inadequate initial estimation of the project cost and finances prior to construction. In most cases, the halting of construction has been the direct result of prices of "input resources" (raw material and services) which kept increasing to outstrip project budgets. Other related factors contributing to this problem are the levels of inflation (an average rate of 15% in Ghana since 2001) over reliance on imported goods for construction and high cost of finance.

Building materials is one aspect of an internal factor needing urgent attention, since materials constitute about 65-70% of the cost of construction in Ghana. A rise therefore in the cost of certain prime materials is very quickly fed into eventual significant increases the building costs. In Ghana, over-dependence on imported materials and the cost of local transportation are some of the major contributing factors to the rising cost of construction. The imported materials have also been identified as having as high a percentage as 65% of their values in foreign currency (typically US Dollar) (BRRI, of CSIR-GHANA 2001).

The local production of construction materials leaves much to be desired. The major raw materials (*e.g.* clinker for cement, Aluminium sheets, steel *etc.*) to feed into this industry are all imported (typically from Norway). Even though construction timber is one material that is "home-grown," its cost is also high due its value as an export commodity to generate income and foreign exchange.

It is against this background that this current study is being carried out to identify alternative building materials that are durable, readily available and cost effective for local consumption, and give appropriate recommendations for improvement.

CHAPTER THREE

LITERATURE REVIEW

3.0 Introduction

The shortage of low cost and affordable housing in Ghana has led to this investigation into low cost construction materials. The searching for alternate construction material to conventional construction materials, out of waste and low energy materials is the main purpose of this research.

Two main factors would be taken into account in the search for new construction materials and techniques. These are ecological impact and production costs. The incorporation of recycle materials originating from renewable sources into a cementitious core is a feasible alternative that this research would investigate. The first part of the experimental studies would investigate the mechanical properties of matrix of plant and mineral waste composite with concrete. In this light, this chapter would review work by other researchers on natural fibres in concrete, and their current applications in the construction industry.

Earth construction is widely spread in the rural areas of Western Africa. This chapter would also review literature on the types of soil block which are used in low cost construction, since the current research intends to conduct experimental studies to investigate the strength of soil blocks stabilised with cement.

More often than not, communities affected by disasters like earth quake, land slides, cyclones, hurricanes, etc. are cut off and this makes it difficult to provide aid including temporary shelter to the affected people. This research would investigate the use of plastic crate recycled from waste plastic containers for soil blocks. The purpose of this investigation is to transport these lockable plastic crates from helicopters into the affected communities, so as to fill them with soil, debris, etc, for construction of temporary houses or repair of partially destroyed houses. This chapter would further review literature on plastics in construction.

In a summary, this chapter would provide the background to this subject and reviews literature on properties of readily available raw materials and their application in housing construction. Two relevant examples of such materials are natural fibres and plastic waste containers. It outlines some of the existing practices and methodology for using the above mentioned raw materials in construction and analyses them for sustainability. Having reviewed the work of other researchers in this chapter, new material selection and characteristics would be outlined for further investigation.

3.1 Natural fibres

This section of the review covers the development in the field of natural fibre reinforced concrete and mortar. Investigations relating to properties of natural fibres in concrete and mortar composite, factors affecting durability and methods of separation of fibres from their bundle and the associated bonding are outlined.

Natural reinforcing materials can be obtained at low cost and low levels of energy using local manpower and technology. Utilisation of natural fibres as a form of concrete enhancement is of particular interest to less developed regions where conventional construction materials are not readily available or are too expensive. Coconut and sisal-fibre reinforced concrete have been used for making roof tiles, corrugated sheets, pipes, silos and tanks (Agopyan, 1988).

Concrete made with portland cement has certain characteristics: it is strong in compression but weak in tension and tends to be brittle. The weakness in tension can be overcome by the use of conventional steel bar reinforcement and to some extent by the inclusion of a sufficient volume of certain fibres. The use of fibres also alters the behaviour of the fibre-matrix composite after it has cracked, thereby improving its toughness.

Generally, natural fibres used in cement-based matrices can be divided into two categories. They are unprocessed natural fibres and processed natural fibres.

3.1.1 Unprocessed natural fibre

Unprocessed natural fibres are available in many different countries and represent a continuously renewable resource. These fibres are obtained at low cost and energy consumption through the use of locally available manpower and technology. Such fibres are used in the manufacturing of low fibre content composites (Mohr et al., 2005). Unprocessed natural fibres are divided into three categories, namely, bast, leaves and seed fibres.

a) Bast fibres (flax, hemp, jute, kenaf)

Bast consists of a wood core surrounded by a stem. Within the stem there are a number of fibre bundles, each containing individual fibre cells or filaments. The filaments are made of cellulose and hemicellulose and are bonded together by a matrix, which can be lignin or pectin. The pectin surrounds the bundle and bonds it to the stem. The pectin can be removed by the retting process (i.e. soaking in water) to enables the separation of the fibre bundles from the rest of the stem.

Cellulose is the chief structural element and a major constituent of the cell wall of trees and plants. Hemicellulose is like cellulose, a polysaccharide, but less complex and easily hydrolysable. Lignin is a complex constituent of plant that cements the cellulose fibres together.

Jute fibre is an example of blast fibre. It is a long, soft and shiny plant fibre that can be spun into coarse strong threads. It is produced from the plant genus corchorus. It is cheap, has a reasonable strength (tensile strength of 400-800MPa) and also resistant to rotting. Jute is mostly used for packaging (sacks and bales). Research by Mejia (1984) and Agopyan (1988) reported that jute-fibre cement is used as roofing sheet for a low-cost building system in Brazil.

Sugarcane refining generates a large volume of residue called bagasse. Disposal of bagasse is critical for both agricultural profitability and environmental protection. The sugarcane stalk consists of two parts: an inner pith containing most of the profitable sucrose and an outer rind with lignocellulosic fibres. During refining, the sugarcane stalk is crushed to extract the sucrose. This procedure produces a large volume of bagasse residue which is made up of both crushed rind and pith fibres.

Previous research on bagasse has suggested many approaches to converting bagasse into value-added industrial products, such as liquid fuels, feed stocks, enzymes and activated carbon. Use of bagasse fibre for manufacturing material products is another prospective solution. Waste bagasse is manually sifted and placed in an alkaline solution for boiling to remove the lignin. After the treatment, bagasse fibre is rinsed with water and dried. This fibre is pressed for the production of panels and sheet in low-cost building (Agopyan, 1988).

b) Leave fibre

In general leaves fibres are coarser than the bast fibres. Common applications for these fibres are ropes, and coarse textiles. Example of leaf fibres used in construction is the sisal fibre. Sisal fibre is obtained from the agave plant. The stiffness is relatively high and it is often applied as binder twines. Sisal fibre has been used for the production of roofing tiles.

c) Seed fibres (cotton, coconut, kapok, etc)

Cotton is the most commonly used of seed fibre and is used for textile all over the world. Other seed fibres are applied in less demanding applications such as stuffing of upholstery.

Coconut fibre has high lignin content and thus have low cellulose content, and is consequently comparatively resilient, strong and highly durable. The remarkable lightness of the fibre is due to the cavities arising from the dried out sieve cells. Coconut fibre contains a high lignin ratio that makes the fibres stiffer and tougher, high air porosity (95%), and heat retardant and biodegradable (Reis, 2004).

Applications of coconut fibre are ropes, matting and brush. Coconut fibre has also been used in construction as roofing sheet and composite doors (Roorkee, 2004). Mechanical properties of some selected unprocessed natural fibres are summarised in Table 3.1.

Table 3.1 Mechanical properties of some selected natural fibres.

Fibre	Density (g/cm^3)	Moisture absorption	Tensile strength (MPa)	Modulus of elasticity (GPa)	Elongation at failure
Cotton	1.51	8-25%	400	8-12	3-10%
Abaca	1.50	11.00%	980	29.00	2-3%
E-glass	2.50	-	2400	75.00	3.0%
Flax	1.40	7%	800-1500	29.00	1.2-1.6%
Hemp	1.48	8%	550-900	60-80	1.6%
Jute	1.46	12%	400-800	26-46	1.8%
Ramie	1.50	12-17%	500	70.00	2.0%
Coconut	1.25	10%	220	10-21	15-25%

3.1.2 Processed natural fibres

Sophisticated manufacturing processes are used to extract the fibres of processed natural fibres, such as kraft pulp fibres. Wood cellulose is the most frequently used processed natural fibre. It is most commonly obtained using the Kraft process, which involves boiling or simmering wood chips in a solution of hydroxide, sodium carbonate and sodium sulphide. Different grades of wood cellulose fibre containing more or less of the three main constituents (cellulose, hemicelluloses and lignin) can be obtained by bleaching.

Wood-cellulose fibre has relatively good mechanical properties compared to many man-made fibres such as polypropylene, polyethylene, polyester and acrylic. Delignified cellulose fibre can be produced with tensile strengths up to approximately 2.0 GPa from selected grades of wood, and by using suitable pulping processes. Fibre tensile strengths of 500 MPa can be routinely obtained by applying a chemical pulping process from common and less expensive grades of wood (Soroushian et al., 1992).

Using conventional mixing techniques, the amount of fibre that can be incorporated into the cement matrix at low water contents is limited by the capacity of the fibres to be mixed uniformly into the matrix. As reported in Campbell et al. (1980), it is thus common to find fabrication techniques that involve mixing fibre with the matrix at initially high water contents followed by some dewatering procedures.

Published information on the performance of wood-cellulose fibre composites is conflicting; evidence from Soroushian et al. (1992) would suggest that wood fibre-reinforced cement provide the highest performance among fibrous cement composite. This evidence in Soroushian et al. (1992) is disputed by Bentur et al. (1989) and Akers et al. (1989). These latter two groups of researchers argue that, the strength and other properties of the cellulose-pulp fibre are inferior to those of many other fibres, such as asbestos. However, both the former and latter group of researchers and others including Kilian et al. (1993), and Campbell et al. (1980) expressed their concern regarding low moisture resistance of wood fibre cement composite. Most authors who have published information on wood fibres agreed that wood fibres are highly cost effective. This combined with their compatibility with processes for producing asbestos cement, makes the cellulose-pulp fibres an attractive alternative to asbestos.

As a result of intensive research and development, cellulose-pulp fibres are now used in some places as partial or full replacement for asbestos in cement composites.

3.1.3 Advantages and disadvantages of Natural fibres

There are several advantages in the use of natural fibres in concrete and mortar. Among others are:

- Low specific weight, which results in a higher specific strength and stiffness than synthetic fibres. This is a benefit especially in design for bending stiffness.
- It is a renewable resource; its production requires little energy, with no CO_2 absorbed while extracting the fibres.

Natural fibres have the following disadvantages:

- Lower strength properties, particularly its impact strength.
- Variable quality, depending on unpredictable influences such as weather.
- Moisture absorption, which causes swelling of the fibres.
- Lower durability, but fibre treatments can improve this considerably.
- Poor fire resistance.
- Price can fluctuate by harvest results or government policies on agriculture.

3.1.4 Durability of natural fibres in concrete and mortar

Many researchers have investigated the durability of some selected vegetable fibres. This section briefly examines the work of Gram (1983), Romildo et al. (2000), Savastano (2000) and Ramkrisha et al. (2004).

Gram (1983) performed a systematic and comprehensive investigation on the durability of natural fibre reinforced Portland cement. The degradation of fibres in an alkaline environment was evaluated by exposing the fibres to alkaline solutions and measuring the variation in tensile strength. The investigation included natural aging of thin sheets in Dar El Salaam in Tanzania and Stockholm and different accelerated aging tests like cycle wetting and drying.

The degradation of natural fibres in alkaline medium was confirmed by Romildo et al. (2000) and Ramkrisna et al. (2004). These reseachers measured the durability of sisal and coconut fibres as strength loss occurred over the time the fibres were subjected to three types of treatments. Fibres were stored in tap water of pH 8.3, solution of calcium hydroxide of pH 12 and solution of sodium hydroxide of pH 11 for 420 days.

Similar studies on the durability of natural fibres were conducted by Ramkrisna et al. (2004). The results of the variation in chemical composition and tensile strength of coconut, sisal, jute and Hibiscus cannabinus fibres were presented. These fibres were subjected to alternate wetting and drying and continuous immersion for 60 days in three mediums namely, water, saturated lime and sodium hydroxide. Compressive and flexural strengths of cement mortar (1:3-cement: sand) specimens reinforced with dry and corroded fibres were determined after 28 days of curing under water. The results of the studies are summarised in Table 3.2 below.

Table 3.2 Results on durability test of selected vegetable fibres

Fibre characteristics	Fibre type	Observations
Chemical composition	Coconut, jute, sisal and hibiscus	Reduction of about 20-80% of cellulose
		Reduction of about 30-70% lignin
Tensile strength	Coconut	Retained 40-60% of initial tensile strength
	Sisal, jute and hibiscus	Lost almost all initial tensile strength
Compressive strength	Coconut, sisal, jute and hibiscus	Reduction of 30-60% of compressive strength.

The researchers explained how the fibres were adversely affected by lime and alkaline, but did not explicitly address how the fibres were extracted. Furthermore, putting fibres in sodium hydroxide and calcium hydroxide for 420 days is an exaggerated measure and subject to criticism, since sodium hydroxide and calcium hydroxide is produced during the hydration process of the concrete the bulk of which occurs for only about 30 days.

3.1.5 Pre-treatment of coconut fibre

The purpose of natural fibre reinforcement is to improve the properties of building materials, basically the mechanical properties, which would be otherwise unsuitable for practical applications. Adding fibres to concrete greatly increases the toughness of the material. That is, fibre-reinforced concrete is able to sustain load at deflections or strains much greater than those without fibres at which cracking first appears in the matrix.

If the functions of coconut fibre in a relatively brittle cement matrix are to improve toughness and ductility of the composite, then the durability of such fibres in a highly alkali-cement matrix must be taken into consideration. The problem of durability however, can potentially be overcome by pre-treatment of fibre before using it in mortar or concrete. Some of the methods of pre-treatment have been described by Petten (2000), van Voorn (2000) and Pott (1997). These include retting, moisture, drying, etc.

a) Retting

Retting is soaking of the unprocessed fibres in water for some days or months. This process removes the pectin, which bonds that hemicelluloses and cellulose together. After the retting, hemicellulose and lignin can be removed by alkali reactions. The hemicellulose is responsible for a great deal of the moisture absorption while the lignin is the connecting cement between the individual fibre cells.

b) Moisture

The fibre moisture can affect the chemical reaction. In order to prevent this, the fibres have to be dried before hand, preferably down to 2 to 3% moisture content by weight. In standard room conditions, the moisture content is over 10%. The compatibility of the moisture and the applied resin is important.

c) Drying

Air can be present in the fibres and in the resin. The surface of the natural fibre has geometry and a chemical condition on which air bubble growth would be initiated. In order to prevent many voids and a poor fibre matrix interface it is necessary to dry the fibres and to de-gas the resin (Pott, 1997).

3.1.6 Selection of fibres for fibre-cements

From the properties of natural fibres described in previous sections, it is possible to select some which have properties that make them suitable for use in construction. These properties relate to mechanical behaviour (tensile strength, Young's modulus and elongation at break), physical characteristics (density, water absorption), aspect ratio and durability.

Not all fibres presented above are equally suitable as building material reinforcement in West Africa. Although all have properties which make them suitable for some aspect of building, they are not equal in all aspects and can be replaced by other fibres which have better performance. For example, if costs are taken into account, cotton fibres are not appropriate for building in Ghana as cotton is in high demanding use in the textile industry there. On account of production difficulties, sugar-cane bagasse fibre must be replaced by other fibres more easily produced and mixed with cement. Furthermore, the sucrose in sugar cane retards the dehydration process of concrete and mortar. Bamboo, jute and sisal are typical Latin America plant and thus not abundantly available in West Africa sub-region.

This research would use coconut fibre as an enhancement of concrete for the following reasons:

- Coconut cultivation is concentrated in the tropical belts of Asia, West and East Africa and Latin America and is readily available in West Africa.
- The husks from which the fibres are extracted are waste from the coconut industry and are usually burnt. To make a good use of the waste would help reduce the CO_2 emission from the burning or CH_4 from natural decomposition, both of which are greenhouse gases.
- There has been increase of 1200 hectares of coconut plantation in Ghana from the year 2000 to 2005 due to direct Government incentives (Arthur et al., 2005). Making use of the resultant

increased waste would therefore be a positive and important measure to accompany this intervention.

3.2 Mineral fibre

Trial tests to investigate the suitability of rock wool as enhancement to concrete was also considered at this stage of the current research. Rock wool fibre was considered because it is a natural fibre readily available in the United Kingdom and in most European countries. The main properties of rock wool are tabulated in Table 3.3.

Table 3.3 properties of rock wool

Percentage of rock wool constituent that melt in fire	Maximum 1%
Moisture content	Maximum 5%
Density	60-80kg/m^3
Tensile strength	483 -759MPa
Young modulus	69-117.3GPa

Rock wool fibre has the following uses in the construction industrial:

- The excellent thermal resistance of rock wool is a major factor in its use as residential and commercial insulation, pipe and process insulation, insulation of hot bodies such as industrial boilers and ovens and a wide variety of other applications.

- Rock wool is also an excellent sound absorber. When installed in the walls and ceilings, it reduces the transmission of sound. Rock wool products are used for acoustic insulation of cinema halls, auditoria, commercial buildings, recording studios, car ejector, silencing generators, etc.

- Since the mineral fibres do not melt in fire, they are also used to prevent the spread of fire. Rock wool is used as a primary constituent of fire proof doors, fire proof partitions and ceiling tile.

However, no literature is found on the use of rock wool as an enhancement in/or additive to concrete or mortar.

3.3 Thermoplastics

This section briefly describes the types of the plastic materials commonly used in building construction. This section also describes in some detail the characteristics of polyethylene, since part of the current work is to investigate the uses of waste materials in construction, which includes e.g. Polyethylene (waste plastic bottles) and waste from agro-based industries.

Polyethylene is a semi-crystalline (typically 70-80%) which is whitish and semi-opaque. It is significantly stronger and stiffer and has better chemical resistance than most thermoplastics. Its impact resistance is reasonably high, and this is retained at low temperatures. Applications of polyethylene include a wide range of containers, blown bottles for food, pipe and pipe fittings.

Recycled polyethylene has many uses. Examples include liquid laundry detergent containers, drainage pipe, oil bottles, recycling bins, benches, pens, kennels, vitamin bottles, floor tiles, picnic tables, fencing and industrial containers. In the United Kingdom and the European Union consumption far outstrips collection and recycling rates. Available data on estimated plastics bottles and estimated collected for recycling in the United Kingdom and the European Union are shown in Table 3.4 (Taylor, 2006).

Table 3.4 Polyethylene (HDPE) waste generated and recycling

Area	Estimated plastics bottles market size, tonnes	Estimated collected for recycling ,tonnes	% collected
UK	494,500	11,300	2.3
EU	2097,000	176,000	8.3

3.4 Application of selected raw materials in construction

This section briefly describes the current applications of materials which are intended to be investigated in the present research in construction.

3.4.1 Coconut fibre-cement products

A few examples of the application of coconut fibre as a composite material in the construction industry are given below.

a) Roofing sheet

CBRI-India has developed technology for production of coir-cement roofing sheet having a thickness of 6-8 mm. The manufacturing process involves soaking of coconut fibre in mineralized water and then mixing with dry cement in the ratio of 1:5 by weight. A sheet is made with this wet mix of cement coated fibres and is held under pressure for 4-8 hours. Another type of roofing sheets with a thickness of 3-4mm is fabricated using chopped fibre strand mats, fibrous reinforcing filler, anti-aging agent and unsaturated polyester resin. The properties of coir-cement roofing sheet in tabulated in Table 3.5.

Table 3.5 Properties of coconut fibre–cement roofing sheet

Properties	Coconut fibre cement roofing sheet	Asbestos cement roofing sheet
Density (gm/cm^3)	1.02	2
Water Absorption 24 hrs. (%)	3 – 5.00	25
Thickness (mm)	3.31	6
Weight ($kg./m^2$)	3 – 4.00	13-50
Bending Strength (MPa)	45 – 58.00	25-30

b) Medium density composite doors

CBRI India has developed medium density composite doors containing coconut fibre, cashew nut shell liquid (CNSL as natural resin) and paraformaldehyde as major constituents. Coconut fibre contributes mechanical strength to the composite while the CNSL with paraformaldehyde act as a binder. Coconut is impregnated with CNSL and is compression moulded under high temperature. The pressure required during casting of the board/sheet depends upon the required density of the final product. These boards can be used as wood substitute for panelling, cladding, surfacing and partitioning and other interior applications. The boards have density between 0.5-0.9 gm/cm^3 and can be cut, sawed, nailed & screwed. The boards have very low water absorption and negligible swelling (Roorkee, 2004).

3.4.2 Plastic waste construction product

This section describes application of recycled waste plastic products in construction.

a) Roofs & Floors

Recycled plastics have been used in a variety of ways as components of both roofs and floors, such as in roofing shingles and tiles, and floor tiles. Recycled plastics are also being used as a constituent of cement to form a flooring material (Taylor, 2006).

b) Plastic Lumber

Plastic lumber is a plastic and wood substitutes made from post consumer or industrial scrap plastic. One or more plastic resins are used, together with various additives for colour, strength and density reduction.

Plastic lumber are used for domestic and public purposes, for example for signposts, fences, pilings, piers, bulkheads, sea walls, boardwalk decking, and pier impact protectors and railings.

c) Panels for Cladding & Decoration

Recycled plastics are used as an alternative to chipboard for panels for interior decoration or partition. The product is aesthetically pleasing and is tough and durable.

3.5 Soil blocks

Many different materials are used around the world for walling. Where quarried stone and timber are not readily available, earth is the most common material used. Earthen architecture has been used for centuries in many different parts of the world according to Jones (1985). Archaeological evidence in very dry areas had also shown that earth building was a highly popular material for dwelling construction. Earth is still used today in many parts of the world where access to other forms of building material is restricted by location or cost.

Each building material has its own advantages and disadvantages. Some of the problems with existing building materials are their poor use of environmental resources, poor quality control of the finished product and consequently a significant variation in durability. Alternative building materials that have suitable strength and durability, and also environmental sustainability are being sought after by researchers.

3.5.1 Type of soil blocks

This section describes a few types of soil blocks which are popularly used as walling materials in construction in West Africa sub-region and typically in Ghana.

a) Kiln-fired brick

Parry (1979) describes two methods of brick production in terms of cost and shows quite clearly that where labour costs are low, kiln-fired brick production would be economically unsuitable. Kiln-fired brick production requires a high capital investment and a significant amount of infrastructure to support production. Brick production must be located near to high quality clay deposits and staff need to be more highly skilled. Production output is very high (typically 10,000 - 30,000 bricks per day) and needs to be continuous in order to achieve high efficiency and the greatest return on investment. The characteristics of such kiln-fired bricks are highly desirable as the material has a high wet-compressive strength and does not deteriorate rapidly over time, even in the harshest of climates (Wayne, 2004).

b) Clamp-fired brick:

These can be inexpensive in monetary terms because the raw materials can usually be dug from the ground fairly locally and the energy required firing the brick could come from collected firewood. Clamp fired bricks are of a lower quality than kiln-fired bricks and can tolerate the use of smaller and poorer sources of clay deposits. Forming the blocks requires a wooden or metal mould and after forming they are laid out to dry. After drying they are stacked into a clamp where fires are burnt inside (Keable, 1979). These fires raise the temperature of the blocks to the point where the particles bond together (Montgomery, 2002). Thorough burning is necessary to fire all the blocks properly and this takes several days. The finished blocks can be quite badly mis-shapen and they require a thicker layer of mortar between the blocks, sometimes as thick as 20mm. Furthermore, if the blocks are poorly fired then in order to achieve adequate durability they may need to be rendered as well. Fired blocks are usually considered attractive and so they are not generally rendered unless necessary.

c) Compressed and Stabilised Soil Blocks (CSSB)

These blocks use the same parent material as plain earth blocks but offer a significant advantage in wet compressive strength due to chemical or mechanical stabiliser. Improved strength and stability in wet climates is generally achieved by a combination of two methods of stabilisation. One method is to compact the soil by applying some mechanical effort to reduce the voids in the material. Increasing the density of the material gives it a higher compressive strength and also reduces the potential for ingress of moisture into the block as evidence in Houben (1989) and Montgomery (2002). CSSB are further stabilised with the addition of a chemical stabiliser that helps to bind the particles together. Cement or lime are expensive additives but are generally widely available and although the practice of adding them to soil is reasonably popular the results can be disappointing unless it is done carefully. The greater the level of compaction the greater the compressive strength of the block and the more effective any added stabiliser becomes (Montgomery, 2002). CSSB compacted to higher densities are also usually more dimensionally consistent and therefore can be laid using a thinner mortar layer around (10–15mm). Some CSSB need to be rendered or waterproofed in order to enhance their protection from climatic conditions and the weather as suggested by Yogananda (1999) and Montgomery (2002). The rendering of CSSB can be avoided with higher levels of compaction or higher quantities of stabiliser.

d) Soil block from polypropylene rice bags

Misprinted polypropylene rice bags, had been filled with earth and pumice to create a unique twin dome home, complete with solar/PV and indoor garden, plastering with lime and papercrete, with adobe (earthen) floor (Kelly 1976). This was an experimental earthbag dome. This dome is called "Riceland," after the brand name printed on the polypropylene rice bags that were used to build it. The interior diameter is 14 feet (4.3 meters).

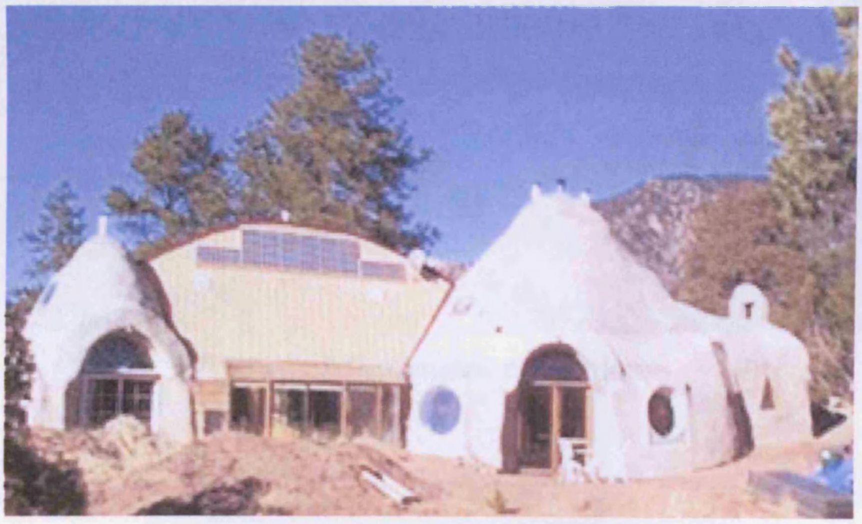


Figure 3.1. A view of the riceland house built from polypropylene rice bag

After the selection of the site, the very centre of where the dome was to be erected was marked with a stake that remained there until the dome was near completion. Using a rope attached to this stake a radius of seven feet plus an extra two feet was marked out to be levelled for the floor as shown in Figure 3.2. This step was then followed by digging out the foundation trench (see Figure 3.3). The third stage of the construction was the filling of the trench with rubbles (see Figure 3.4), and was followed by the fourth step which was about filling the rice bags with the soil and finally the erection the wall with soil bags (see Figures 3.5 and 3.6).

Figure 3.2 Parameter of the dome

Figure 3.3 Step two the perimeter was trench dug out

Figure 3.4 Step three: the trenched was filled with rubble

Fig.3.5 Step four: Filling of bags with soil

Figure 3.6 laying bags in the trench

e) Soil block from rubber tyres

A 'high-tech' solution to recycling rubber tyres was developed by researchers at The Hong Kong University of Science and Technology, which has resulted in double benefit for the environment. The first benefit is the development of a process of recycling used waste vehicle tyres. The second is on turning of the recycled material into rubber soil blocks for use in civil engineering work such as road widening, land reclamation, and the building of embankments, retaining walls and slopes. The rubber soil block has many advantages over compacted soils. They are lighter, stronger, permeable, energy absorbing, flexible for construction, and dust free. The new soil block is also highly porous, eliminating water pressure build-up, and has 5-10 times the compressive strength of compacted granular soils. The resulting material is about half the weight of conventional compacted soil which reduces structural loadings and results in cost savings (Lee, 2005).

The method of extracting steel wires from the used tyres was not mentioned in this particular research work. For the purpose of providing a low cost housing for the rural and urban poor in developing countries at a low-cost technique for extracting steel wires and formulating "soil" blocks from rubber 'crumbs', needs to be examined.

3.5.2 Properties and analysis of soil

Soil is found deposited on the surface of the earth and can consist of many different types. The variation in the soils present at the surface can be attributed to a series of natural effects working on the area over time (Smith, 1981). On the very surface of the soil one typically finds material with a large amount of organic compounds present. This is unsuitable for block manufacture and can usually be distinguished by a musty smell, especially on heating (Wolfs-kill et al., 1965). Material underneath this organic layer is much better as it usually contains a cross section of particle sizes and includes a proportion of small soil particles called "fines".

These are usually defined as particles passing a 63μm sieve and consist of silt and clay. Clay is necessary in block production because it aids the workability of the mixture, increasing levels of consolidation and improving green strength (Smith, 1981). Larger particles "sands" found in soil can generally be assessed as minerals that are silica's, silicates or limestone. As well as the solid rock particles and fragments, soil will have a proportion of water and air that fill the gaps between adjoining particles in the soil. This gives natural soil a non-homogenous and porous nature. Systems for identifying some major characteristics have been developed to define different ranges of soil characteristics. The most common of these is the size distribution of the soil particles. The list of physical characteristics that can define a sample of soil suitable for soil block includes: apparent bulk density, specific bulk density, size or texture, moisture content, porosity or voids ratio, permeability, adhesion, dry strength and linear contraction.

The relevant chemical properties include the composition, mineral content, metallic oxides, pH levels and sulphates in the soil (Craig, 1997). The actual chemical composition of the soil is of little importance once the absence of unstable compounds and organic matter has been established.

The physical properties are of greater interest for making soil blocks as these will help to determine shrinkage, dry strength and apparent bulk density. Controlling or monitoring the clay fraction is important in making soil blocks. Too much clay would result in unacceptably high expansion upon wetting; too little clay causes low adhesion between particles.

3.5.3. Basics of cement usage in stabilising soil for blocks

As a stabilising material, cement is well researched, well understood and its properties clearly defined. Portland cement is readily available in all countries all over the world, as it is one of the major components for any building construction. Earlier studies have shown that cement is a suitable stabiliser for use with soil in the production of CSSB (Montgomery, 2002). Cement is mainly composed of Lime (CaO) and Silica (SiO_2), which react with each other and the other components in the mix when water is added. This reaction forms combinations of Tri-calcium silicate and Di-calcium silicate referred to as C3S and C2S in the cement literature (Akroyd, 1962, Lea, 1970 and Neville, 1995). The chemical reaction eventually generates a matrix of interlocking crystals that surround any inert filler (i.e. aggregates) and provides a high compressive strength and stability.

3.5.4 Soil stabilisation

The addition of a stabiliser to soil in soil block construction is to stabilise the soil, making it more durable, and subsequently also any structure made from it. Soil stabilisation with cement improves the characteristics of the soil so that it can tolerate greater loading and perform better when it is exposed to the weather. The two most common techniques used in block manufacture are binding (with chemical additives such as cement or lime) and increasing the density (through compaction). The next two sub-sections describe cement as a binder to soil and material compaction.

3.5.5 Binding agents

The quantity of cement that is required for adequate stabilisation depends on several criteria, namely: the required compressive strength, soil type, environmental conditions and levels of quality control. Cement can very easily be wasted if it is not utilised in the correct manner and significant cement reduction can be attained through good production management and quality control. Controlling the moisture content, level of compaction and the curing regime play a big role in attaining the desired strength from the added cement.

3.5.6 Compaction of material

Within the civil engineering industry there are several methods of compaction that are used in ground stabilisation. These use methods of static, vibration and dynamic blows to compact soil (Parsons, 1992). Block compaction uses similar methods and similar technology but on a smaller scale and typically compaction takes place in a confined space rather than in unconfined open areas (Houben & Guillaud, 1989 and Norton,1997). Block compaction has predominantly used vibration or slow steady squeezing (quasi-static) compaction to achieve the desired levels of soil consolidation.

3.6 Use of waste materials in the construction industry

This section reviews literature on the use of waste and low energy materials in construction. The most influential factor in determining whether or not a waste material or a by-product is used is the economic cost in comparison with the alternative natural material in a particular application. These costs are primarily made up by handling, processing and transport but the social benefits of using a waste, for example avoiding the dereliction associated with the tipping of a waste material or the quarrying of a natural, should not be forgotten.

3.6.1 Use of waste materials with pozzolan properties as a stabiliser

A pozzolan is a material which, when combined with calcium hydroxide, exhibits cementitious properties. Pozzolans are commonly used as an addition (the technical term is "cement extender") to Portland cement concrete mixtures to increase the long-term strength and other material properties of Portland cement concrete, and in some cases reduce the material cost of concrete. Pozzolans are primarily vitreous siliceous materials which react with calcium hydroxide to form calcium silicates; other cementitious materials may also be formed depending on the constituents of the pozzolan. The most commonly used pozzolan today is fly ash, though silica fume, high-reactivity metakaolin, ground granulated blast furnace slag, and other materials are also used as pozzolans.

i) Use of blast furnace slag

Blast furnace slag and pulverised fuel ash are the two waste materials which are being used to the greatest extent in construction. Blast furnace slag is used to the extent that some 80% of what is available is used and in several countries virtually all that is produced is used. The slag is regarded as a highly satisfactory material.

ii) Pulverised fuel ash in construction

Pulverised fuel ashes are about 20% used overall but up to 70% is used in some countries (Gutt et al., 2006). Much of these two materials is used as binders but many more sophisticated uses are being developed. These materials can make a particular contribution in conserving energy in the manufacture of cementitious materials and of lightweight aggregates. Nonetheless, the overall proportion of mineral wastes which is used is only about 5% of that producted and most is used in relatively low grade applications such as fill in roads and embankments (Gutt et al., 2006).

iii) Paper sludge

The use of paper de-inking sludge in pozzolanic material manufacture permits a disposable residue to be included in the cycle of the materials. A study on the reuse of paper de-inking sludge, undertaken in Spain, showed that, it has the potential as raw material for producing a binding material with pozzolanic properties. Paper de-inking sludge has high organic matter content (cellulose) as well as inorganic compounds, such as clays and calcium carbonate. The inorganic matter comes from the de-inking and whitening processes. The research results showed that calcination paper sludge has higer pozzalnic characteristics as compared to other industrial pozzolanic by-products, such as fly ashes, (Frías et al., 2004 and Vegas et al., 2004) normally used in cements.

The mineral fraction of the dry paper sludge is predominantly calcite (35%) and kaolinite (21%); other minerals are also present in small amounts, such as hyllosilicates type chlorite and micas (11%), talc (2%) and quartz (2%). The calcite and kaolinite permit the possible use of paper sludge as a pozzolanic material.

The results of research conducted by Raquel et al. (2007) indicated that the incorporation of paper sludge as pozzolanic addition in cement manufacturing was feasible. A significant gain of compressive strengths (approximately 10%) was achieved, when 10% calcined paper sludge was blended with ordinary Portland cement.

In the context of soil blocks, sludge from the paper industry, using recovered paper as raw material, could be used as stabiliser to soil in soil blocks production, due to the presence of the clayey materials in the paper sludge (muscovite, talc, and kaolinite) and the high contents of calcium carbonate and cellulose. The clayey materials could act as accelerators in the activation of the pozzolanic reaction in the soil.

iv) Sugar cane wastes as pozzolanic materials

In recent years, the use of solid waste derived from agricultural products as pozzolans in the manufacture of blended mortars and concrete has been the focus of researchers in the construction materials sector. The addition of ashes from combustion of agricultural solid waste to concrete is at present, a frequent practice because of the pozzolanic activity of the ashes toward lime. One of the most interesting materials is the ash obtained from the combustion of sugar cane solid wastes (sugar cane straw and sugar cane bagasse).

In Cuba for instance, significant amounts of sugar cane are processed, generating high volumes of solid waste. These wastes are disposed and burnt in open landfills, negatively impacting the environment.

Some studies that were carried out for characterizing the sugar cane solid waste as pozzolanic material found that bagasse ashes from a furnace operating in the 1000 to 1100 °C range showed very poor pozzolanic reactivity(Villar-Cociña et al., 2008). On the other hand, Payá et al. reported ashes of bagasse and coal (9:1 mass ratio) at 800 °C presented high pozzolanic reactivity. Sing et. al. also stated that good pozzolanicity of bagasse ash occurred when mixed with portland cement but, in this case, the calcining temperature was not specified..

Is worth researching into the use of surgar cane ash as stabiliser to soil for soil blocks production.

v) Fly ash as soil stabiliser

Claivyey soils in road construction may need enhancement for increased durability. One option for roadbed modification/stabilization is to treat subsoil with fly ash, which possesses several beneficial engineering properties (Friends et al., 2004) such as higher binding properties. However some substances such as bituminous coal in fly ash may potentially pose an unacceptable risk to human health. To use fly ash as soil stabiliser, due diligence of these risks need to be considered.

3.6.2 Use of demolishing materials in construction

Construction and demolition debris is produced during new construction, renovation, and demolition of buildings and structures. Construction and demolition materials that can be recovered depends on many factors including the type of project, space on the building site, the existence of markets for materials, the cost-effectiveness of recovery, the time allowed for the project, and the experience of the contractors. Many construction and demolishing materials can be reused or recycled. Construction and demolition debris includes, concrete, masonry, soil, rocks, lumber, shingles, glass, plastics, metals, drywall, insulation, asphalt roofing materials, electrical materials and plumbing fixtures. In 1996 the U.S. produced an estimated 136 million tons of building related construction and demolition debris (EPA, 2000). Furthermore, road, bridge, and land-clearing materials can also be a significant portion of total construction and demolition debris materials discarded. Construction and demolition materials can be recovered through reuse and recycling. In order for materials to be reusable, contractors generally must remove them intact (windows and frames, plumbing fixtures, floor and ceiling tiles) or in large pieces (drywall, lumber). Some materials may require additional labour before they can be reused. For example, lumber may need to be denailed and window frames may need some new panes. In order to be recyclable, materials must be sorted and separated from contaminants (e.g. trash, nails, and broken glass). This can be accomplished if contractors require workers to sort materials as they emove items from buildings or as debris are produced. Many contractors simply use labelled roll-off bins for storage of source-separated materials. For projects where on-site source separation is not possible, contractors often use demolition materials processing firms.

Recovering construction and demolition materials has the following advantages of:

- Reducing the environmental effects of extraction, transportation, and processing of raw materials;
- Reducing project costs through avoided disposal costs, avoided purchases of new materials, revenue earned from materials sales, and tax breaks gained for donations;

- Helping communities, contractors, and/or building owners comply with state and local policies, such as disposal bans and recycling goals;
- Enhancing the public image of companies and organizations that reduce disposal; and
- Conserving space in existing landfills.

Possible disadvantages such as pollution arising during processing or use of a less well proven technical performance should also be taken into account. Adequate knowledge of the properties of the waste materials and products containing them are therefore essential to enable a balanced judgement to be made on the overall advisability of using a waste material in a particular situation.

3.7 Conclusion

This chapter primarily reviewed the work of other researchers, on the use of natural fibres in construction, and on soil blocks for construction.

Literature research on thermoplastic soil blocks has turned up very little. To date, only two pieces of work that cover this topic have been found. These are on the use of polypropylene rice bags as soil blocks and soil blocks from rubber tyres. There are however, other publications that deal with the subject of compacted stabilized soil blocks from metal or wooden moulds using various methods of compaction, both from a theoretical and practical viewpoint. A high-tech solution to recycling plastic containers for civil engineering purposes would be desirable for developing countries, and it would result in double benefit for the environment. Each year, more than one billion plastic containers are scrapped around the world with most ending up as landfill or dumped illegally. This is a substantial environmental issue. in Ghana

3.8 Way forward

The purpose of the current research is to study alternative materials to the conventional building materials which may be suitable as a low-cost construction material/technique, and that have better advantages than the widely used rammed earth (aktagbame) and sunburnt bricks wall houses described in Chapter Two. One such possible material is the soil block, especially if it was encased inside a crate or carton. Thermoplastic Carton Soil Blocks (TCSBs) are an appropriate building material which should be a viable alternative to the more expensive building materials such as blocks, bricks or stone, and be largely dependent on local raw material and labour. Thermoplastic carton would be filled with soil and then compacted into blocks. The blocks as expected would be to be far more aesthetically pleasing, durable, and low maintenance than the aktagbame constructions, usually built by low-income groups, and could support higher and roomier constructions far more cheaply than commercial building materials.

If the results of the experimental studies became successful and the project was implemented as building material for walls of houses, then the material would have the following advantage. Exterior walls should weather well, eliminating the need for constant refinishing and sealing currently required of earth dwellings. Interior use of plastic soil blocks could also provide excellent thermal mass.

The intended thermoplastic carton soil blocks might also have the following qualities over compacted soils block and unburnt bricks; such as:

- Lighter;
- stronger;
- impermeable;
- energy absorbing;

- flexible for construction;
- dust free;
- uniform building component sizes;
- use of locally-available waste and low energy materials;
- reduction of transportation;
- renewable and
- ability for block interlocking.

The uniformly sized building components could result in less waste, faster construction and the possibility of using other pre-made components or modular manufactured building elements. And this is ideal for construction of houses for disaster purposes. Such modular elements as sheet metal roofing can be easily integrated into the newly proposed thermoplastic soil block structure. The possibility of using such components could often improve the overall quality of the structure as well (Wayne, 2004).

Building with local materials could employ local people, and would be more sustainable in times of civil unrest (which is peculiar in Western Africa) or economic difficulties. People in Western Africa could continue to build good shelters for themselves regardless of the political situation of the country.

The reduction of transportation time, cost and attendant pollution could also make the thermoplastic carton soil block more environmentally friendly than other materials. Most of the time, soil for soil block could be found on site or within a short distance. In most of the Western Africa economies, the most cost-effective transportation is often that provided by people or animals. In Ghana, Ivory Coast, Burkina Faso and Togo for instance, 3 x 5 square feet wagons are built with old recycled car wheels and tyres. It is less expensive to have a few people moving 20 cement bags, or two 55-gallon drums of sand in one load with this wagon for two miles up and down hill than to hire any motorised form of transport such as a truck, or tractor with wagon, because all the latter are dependent on fuel and parts purchased from economies outside that of the local community. The earth that would be used would generally be subsoil, leaving topsoil for agriculture and debris from collapsed buildings in the case of an earth quake.

The current research would use existing thermoplastic cartons, for example ice cream tubes, for the experiments. The way forward is to look into acquiring a small inexpensive injection moulding machine capable of moulding plastic crate according to the specification of the designer.

CHAPTER FOUR

THE USE OF NATURAL FIBRES AS AN ENHANCEMENT OF CONCRETE

4.0 Introduction.

With the quest for affordable housing system for both the rural and urban population of Western African countries and other developing countries, various proposals focussing on cutting down conventional building material costs have been put forward. One of the suggestions in the forefront has been the sourcing, development and use of alternative, non-conventional local construction materials including the possibility of using some agricultural wastes as construction materials.

The overall goal for this research is to investigate the potential of using waste and low energy materials for domestic construction, principally in Western Africa.

The specific objective of this chapter is to experiment on the use of natural fibres as an enhancement of concrete. This chapter describes experimental studies on the use of rock wool fibre (mineral fibre) and coconut (vegetable fibre) as enhancement of concrete. Coconut fibre and rock wool fibre are not commonly used in the construction industry but are often discarded as wastes

Rock wool fibres are mainly used as thermal and acoustic insulators (www.rockwool.co.uk). To date the present author has not yet found any literature on the use of rock wool in concrete. The mechanical properties of rock wool fibre (tensile strength of about 700MPa and modulus of elasticity of about 117GPa) are the main motivation for this experimental study of rock wool fibres on cement and concrete matrix.

These mineral fibres are non-flammable, do not melt in normal fire, and an inorganic insulator with large amounts of tiny air pockets, which impart excellent thermal and acoustical properties to the substance.

Coconut fibres obtained from coconut husk belonging to the family of palm fibres are agricultural waste products obtained in the processing of coconut oil, and are available in large quantities in the tropical regions of the world, most especially in Africa, Asia and southern America. In Ghana, they are available in large quantities in the southern part of the country. Coconut fibre has been used to enhance concrete and mortar, and has proven to improve the toughness of the concrete and mortar (Gram, 1983, Romildo et al., 2000). However, the problem of long term durability has not yet been solved. It has also been noticed that the degree of enhancement from using the fibres depended on the type of coconut species and the sub-region that the coconut plant was cultivated. The specific objective of experimenting on coconut fibre as an enhancement of concrete is two fold. Firstly, to assess if the fibres of the species grown in Ghana would improve the mechanical properties of concrete like the species in Latin America and South East Asia. Secondly, once it was proven that vital mechanical properties of concrete and mortar could be enhanced by coconut fibre from species grown in Ghana, then further investigation would be carried out on improving the long term durability of concrete and mortar with coconut fibres as an enhancement.

The coconut fibre used for this experiment is from Ghana and is from the coconut type known as the MYD+PT hybrid (combination of Malayan Yellow Dwarf and Penuate Tall). This is the type of coconut that is currently being cultivated after the devastating attack on the African tall spices by the Cape Saint Paul disease (since 1990).

4.1. Preliminary studies on Rock wool fibre as enhancement to concrete

4.1.1. Materials used

The materials used for casting of test specimens were:

Rock wool fibres: Sample of rock wool fibres were obtained from Rock Wool Manufacturers Ltd, and were soaked in clean tap water for 24 hours before used. The fibres were placed in water so as to clean it from dirt and any undesirable substance that might have an adverse effect on the hydration of the concrete.

Cement: The cementitious materials used as a binding agent in this study were Ordinary Portland Cement (OPC) in accordance with BS EN197-7.

Coarse aggregate**:** The coarse aggregate used was a crushed limestone from a Swansea quarry with nominal size of 10mm.

The fine aggregate: Natural river sand, of size less than 4mm was used as fine aggregate.

4.1.2. Mix proportion

A total of 18 specimens of concrete mixes were prepared in three series with different amount of cementitious constitutions. The series labelled A was a control mix prepared without any fibres. Fibre enhanced concrete mixes prepared with some percentage (by weight of cement) of rock wool fibres were for series B_x. The batches labelled B_x represents concrete with x% rock wool fibre by weight of cement. All the concrete mixes were prepared with a water-to-cement ratio of 0.55 and a mix ratio of 1:1.8:2.8 for cement: sand: limestone (by weight). The concrete mixes and the cubes and the cylinder were in accordance with the provision of BS1881, Part 108, 1983.

Table 4.1 Mix proportion of rock wool enhanced concrete

Weight of materials	A	B_{25}	B_{30}
Weight of fibre (kg)	0.00	1.00	0.84
Weight of cement (kg)	3.34		
Weight of fine aggregate (kg)	6.02		
Weight coarse aggregate (kg)	9.40		
Weight of water (kg)	1.84		

4.1.3. Specimen preparation and test

The concrete mixtures were prepared in a concrete mixer machine. For each mix, a total of 6 specimens, including three 100mm cubes and three 100×200mm cylinders, were cast in steel moulds, compacted on compacted table in two layers with each layer being compacted for about 30 seconds. The specimens, after de-moulding after one day, were cured under water until the age of 28 days. Compression strength tests were carried out on the concrete cubes using the GD10A compression machine (2500kN capacity) at specimen age of 28 days. The cylinders were removed from water on day 27 and prepared with end clamping rings and paste. Cylinder samples under torsion were then tested on day 28 using the Avery-Dension testing machine (ADTM) 600kN with stroke control rate of 0.008mm/s. The load and displacement data were collected using a computer with the "Global Lab" data acquisition and processing system. The data collected are presented in Table A1 in the Appendix A. The results of both the compression and the toughness characteristics are summarised in Tables 4.8 and 4.9.

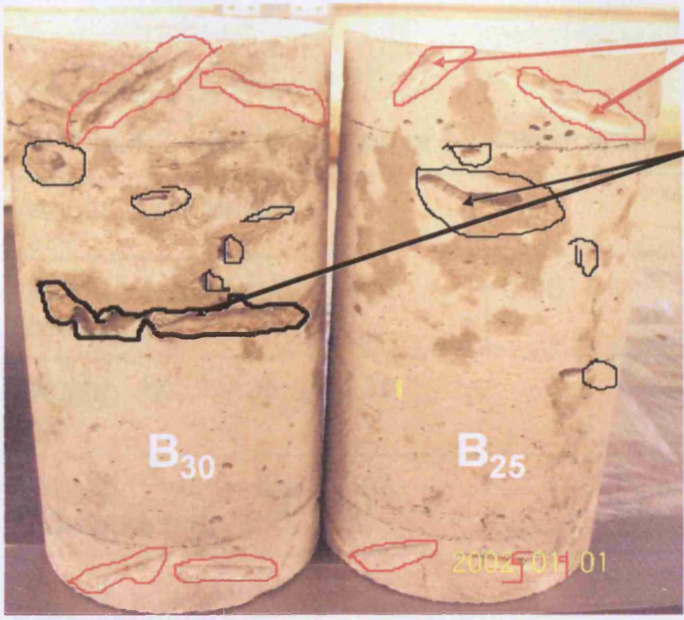


Figure 4.1 Cylinder specimens of Rock wool enhanced concrete.

4.1.4. Rock wool fibre as enhancement to concrete

The decrease in the engineering properties of concrete prepared with the rock wool fibres in the preliminary experimental studies might be due to the insufficient dispersion of the fibres in the concrete during mixing. It is clear that the rock wool fibre formed balls within the concrete matrix and they exhibited larger voids on the concrete specimen as been highlighted in Figure 4.1. A second trial, which then formed the main experimental studies, was thus carried out, with the intention to improve the dispersion of the fibres in the concrete and also to look for the optimal fibre percentage that would best enhance the properties of the concrete. The procedures and testing was as the same as the preliminary test, except that the percentage of fibre was 20% and 10% by weight of cement. Furthermore, the fibre was carefully dispersed into the concrete. This was done by carefully distributing the fibres in smaller quantity into the concrete matrix and further mixing with hands after the concrete had been poured into the barrow.

Table 4.2. Mix proportion of rock wool enhanced concrete

Weight of materials	A	B_{20}	B_{30}
Weight of wool rock fibre (kg)	0.00	0.66	0.33
Weight of cement (kg)	3.34		
Weight of fine aggregate (kg)	5.94		
Weight coarse aggregate (kg)	9.24		
Weight of water (kg)	1.80		

4.2 Preliminary studies on coconut fibre enhanced concrete

4.2.1 Preparation of fibres

Mature fibre from coconut husk from Ghana was used for the current laboratory work. The husk was separated from the nut, and was soaked in fresh water to facilitate the extraction and separation of the fibres. This is the "retting" described in Chapter Three. The soaked husks were removed from the container after one month to extract the fibres by hand. It was difficult to pick the fibres by hand, so the husks were placed in a solution of NaOH for seven days, and then washed with clean water. The NaOH dissolved the vascular bundle (lignin) which surrounded the fibre, and hence made the removal of the fibres easier.

To evaluate structural changes of the surface morphology of the fibre with soaking time and soaking medium, the fibres were examined with an electronic microscope at regular intervals. The observation is described in Section 4.5.2b.

4.2.2 Materials
Ordinary Portland Cement conforming to BS 12, 1971 was used. The fine aggregate was natural sand from Swansea, UK conforming to BS 882 1975, while the coarse aggregate was crushed granite having a maximum size of 10mm (smaller size aggregate as suitable for the mould used for casting), also obtained from Swansea.

The fibres were coconut fibres with diameter ranging between 0.29mm and 0.83mm and length between 6mm and 24mm and approximate mean aspect ratio of 150. Eight fibre specimens were subjected to a tensile test in order to determine the ultimate strength. A graph of tensile strength against aspect ratio is plotted in Figure 4.10.

Sufficient moulds in accordance with BS 1881 were available to enable simultaneous casting of all specimens. This eliminated discrepancies such as variation in mix proportion, water content etc., which might have arisen if more than one mix was required per casting.

4.2.3. Preparation and testing of specimens
Investigations were carried out on test specimens using one basic mix proportion (1:1.8:2.8 of cement: fine aggregate: coarse aggregate) with water cement ratio of 0.55 and three variations of fibre weight fractions. In all, four separate mixes were used in these tests. Six cubes and six cylinders were taken from each mix, giving a total of 48 specimens.

a) Mixing of concrete

All mixing was done using a pan mixer. The constituents of concrete excluding water and fibres were first mixed until a consistent 'grey' colour was obtained. Water was then added gradually while the mixer was in motion. The concrete was mixed for further 120 seconds after final addition of water.

The introduction of fibres to the concrete while the mixer was in motion presented problems. The fibres were forming bundles and also sticking to the sides of the mixer. This was due to the nature of operation, and the mixer cover plates which were provided to prevent spillage. It was therefore necessary to stop the mixer, remove the mixing paddles, sprinkle a layer of fibres onto the concrete surface and reactivate the machine for approximately five revolutions after each addition. This was to ensure complete distribution of fibres throughout the concrete mix, which was achieved.

In an endeavour to separate the fibres and thus prevent balling or interlocking, it was imperative that extreme care was exercised when adding fibres. If too many were added at any one time balling up occurred. In some cases adherence of the fibres to the mixing paddles resulted. When this happened, these fibres, together with some concrete, were removed from the paddles and mixed by hand into the mix before casting. Due to this method of adding fibres, the period between adding water and casting varied between 15 and 30 minutes.

b) Testing of fresh concrete

The slump test was used to test the workability of the concrete. A slump cone mould (diameters 200mm and 100mm, and height 300mm) was filled with concrete in three layers of equal volume. Each layer was compacted with 25 stroke of a tamping rod. The slump cone mould was lifted vertically and the change in height of concrete was measured.

Superplasticizer (Adoflow Extra) was added after the slump test, so as to achieve a comparable workability to that of the plain concrete. A dosage of 0.1% and 0.15% (by weight of cement) according to Ramakrishna et al. (2004) was added to concrete with 1.0% and 1.5% of fibre respectively. The procedure is schematically explained in Figure 4.2, of which "a" represent tools needed for the slump test, while "b" and "c" illustrate lower and higher slumps respectively (Fowler et al., 2004).

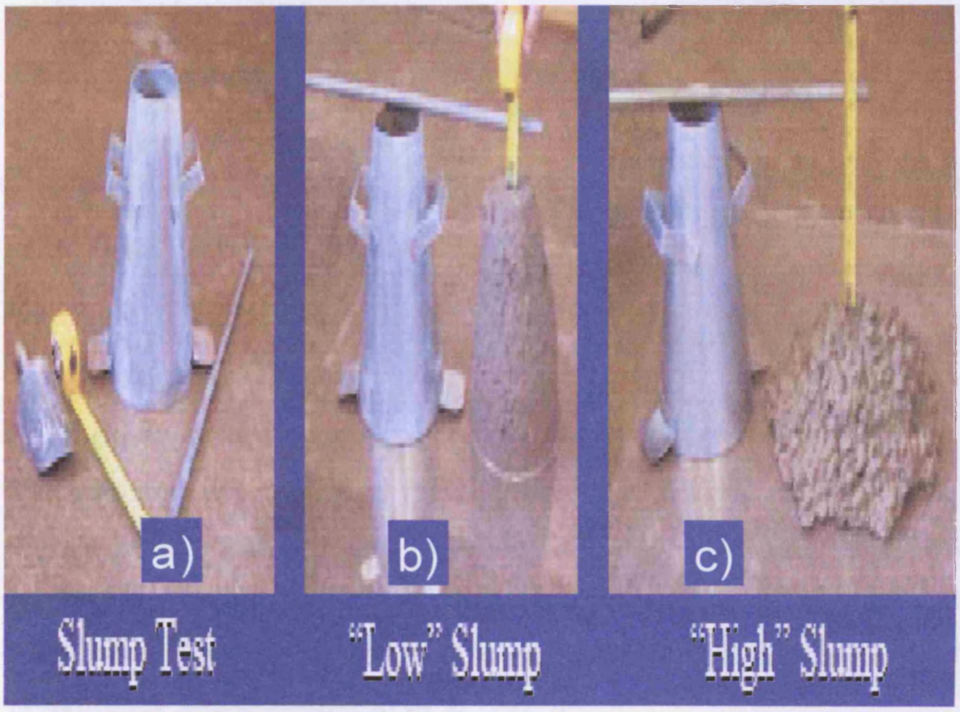


Figure 4.2 slump test of fresh concrete.

c) Test specimen

Six 100mm cubes and six cylinders were cast and tested from each mix. These specimens were cast for each of the mixes A, for the control specimen and $C_{X/i}$, representing concrete specimen with x% of fibre content and i fibre aspect ratio. A total of four separate mixes were cast. The total quantities used per mix per fraction are listed in Table 4.3.

Table 4.3 Mixed proportions in the preliminary test for coconut enhanced concrete

Material	A	$C_{0.5}$	$C_{1.0}$	$C_{1.5}$
Weight of coconut fibre (kg)	0.00	0.22	0.44	0.66
Weight of cement (kg)	4.4			
Weight of fine aggregate (kg)	7.9			
Weight course aggregate (kg)	12.3			
Weight of water (kg)	2.4			

d) Method of compaction

The moulds with half filled fresh concrete were vibrated vertically on the vibrated table while casting for about 30 seconds. The moulds were then fully filled with fresh concrete and vibrated further for about 60 seconds. This method of compaction was to align the fibres normal to the direction of vibration (Parameswaran et al, 1975).

e) Curing

The specimens were stripped from the moulds 24 hours after casting and submerged in water until testing. Some of the specimens were taken out of the water after seven days to test its early (7-day) strength, while the rest were removed from the water after 28 days of submersion in water for testing 28-day strength.

4.2.4. Details of test

Three cubes and three cylinders from each mix were tested after seven days of casting for compressive and tensile strengths, using a GD10A compression testing machine with a maximum capacity of 2500KN.

In order that the cylinders could be tested to obtain the split tensile strength in accordance with BS1881, additional plywood packing strips (10mm wide) were used at point of load contact to prevent stress concentration. Another three cubes and three cylinders from each mix were tested for compression and splitting tensile strength at day 28.

In addition to the testing of compressive and split tensile strengths, three cylinders from each mix were tested for torsional strength. The samples for the torsional test were first roughened at the ends with grinder as highlighted in red ink as shown in Figure 4.1. Steel rings, with Polypaste applied to the inner perimeter, were fixed to each end of the samples. The polypaste then dried and hardened and formed a strong bond between the rings and the concrete. The samples were tested after one day of preparation. The steel rings had protruding radial arms, which could be pushed to induce a torque in the concrete cylinder. The cylinder was set up as shown in Figures 4.4 and 4.5. A twisting load was applied, and the load and respective angle of displacements were record. Details of the recorded values are recorded in Table A3 in the Appendix A.

Figure 4.3 Compression testing machine setup

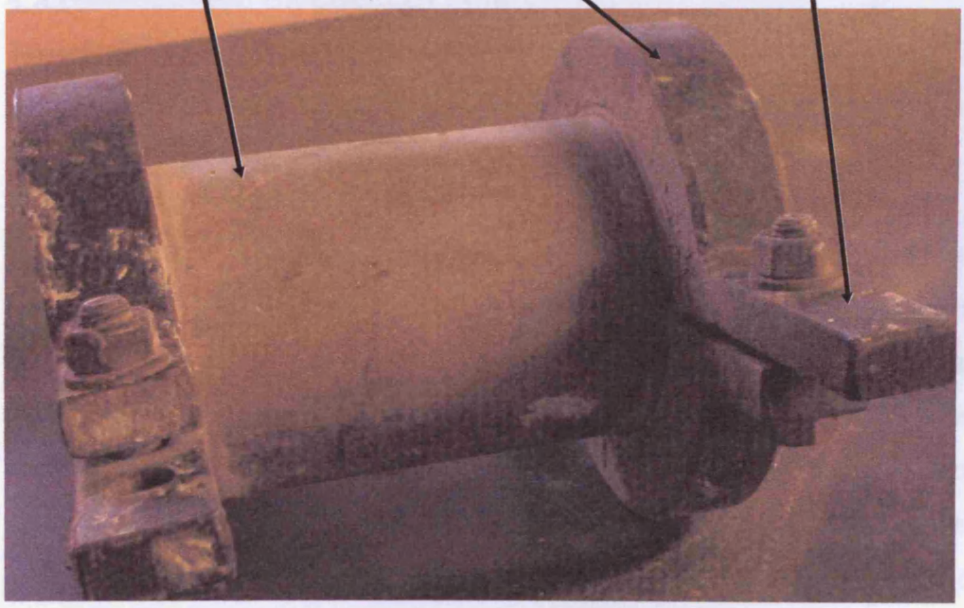


Figure 4.4 Prepared specimen for torsion testing

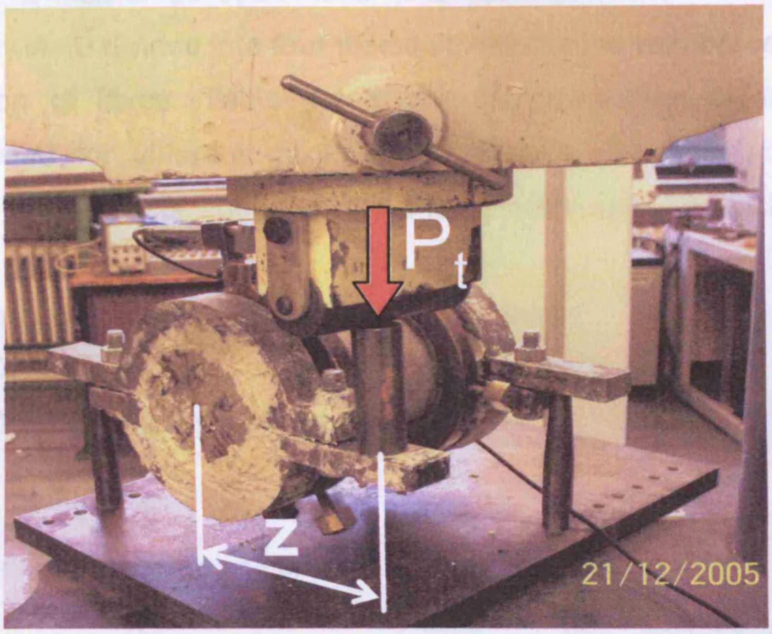


Figure 4.5 Torsion testing machine set up

4.3 Investigation on coconut fibre enhanced concrete

The preliminary studies on coconut fibre as concrete enhancement helped to predetermine the range of fibre content that might give the best possible result with respect to mechanical properties of concrete.

Further experimental investigations on coconut fibres as enhancement of concrete, which then formed the main experimental studies, was thus carried out on test specimens using one basic mix proportion with three variations of aspect ratio of coconut fibres, and different percentage fraction of coconut fibre. This showed the optimum percentage of fibre that would produce the best strength characteristics of concrete, and also the relative importance of fibre aspect ratio on the strength of concrete.

4.3.1 Preparation and testing of specimens

In this study, a total of 60 specimens of a basic concrete mixes were selected. They were divided into four mixes in accordance with percentage weight fraction of fibres. Table 4.4 shows the proportion of various ingredients used for different mixes. The concrete was hand mixed. A number of specimen of cubes and cylinders were cast for each mix:

Three 100mm cubes -subject to compression
Three 100x200mm cylinders -subject to tension splitting
Three 100x200mm cylinders -subject to torsion

Table 4.4 Mix ratio and percentage coconut fibre content

Specimens	A	$C_{0.25}$	$C_{0.5}$	$C_{0.75}$
Design proportion	1 : 1.8 : 2.8 : 0.55 (cement : fine agg.: coarse agg.: water)			
Weight fraction of fibre (%)	0.00	0.25	0.50	0.75

Table 4.5 Detail of quantities of materials used per mix per mass fraction

Materials	Specimens									
	A	$C_{0.25/75}$	$C_{0.25/125}$	$C_{0.25/150}$	$C_{0.5/75}$	$C_{0.5/125}$	$C_{0.5/150}$	$C_{0.75/75}$	$C_{0.75/125}$	$C_{0.75/150}$
Cement (kg)	4.0	2.0								
Sand (kg)	7.0	3.5								
Coarse agg (kg).	11.0	5.5								
Water (kg)	2.0	1.0								
Fibre content (g)	0.0	10			20			30		

These specimens were cast for each of the mixes A, (the control specimen) and $C_{X/i}$, (i.e. specimens with x% fibre content and i fibre aspect ratio). A total of ten separate mixes were thus cast. Details of mixed proportion for each mix are shown in Table 4.4 and the total quantities of materials used per mix per weight fraction are listed in Table 4.5.

4.3.2 Mixing of concrete with coconut fibres

In the previous experiment on concrete with coconut fibres the introduction of fibres to the concrete presented problem due to the way the mixer operated. To ensure complete distribution of fibres throughout the concrete mix, sometimes it became necessary to stop the mixer, remove the mixing paddles, sprinkle a layer of fibres onto the concrete surface and reactivate the machine for approximately five revolutions after each addition. In an endeavour to ensure that the fibres were well distributed and randomly orientated, and thus prevent balling or interlocking, the concrete together with the fibres were mixed by hand in this further investigation.

4.3.3 Mixing procedure

The dry cement and aggregates were mixed for two minutes by hand in a $0.1m^3$ laboratory mixer pan. The mixing continued for further few minutes while about 80% of the water was added. The mixing was continued for another few minutes and the fibres were fed continuously to the concrete for a period of 2–3 min while stirring. Finally, the remaining water along with superplasticizer was added and the mixing was continued for an additional two minutes. This ensured a complete distribution of fibres throughout the concrete mix. For each mix, a total of six cylinders with dimension of 100×200mm and three cubes of 100mm were cast.

4.3.4 Workability tests

The slump test was used to test the workability of the concrete. The method and the procedure for the slump test was the same as described in Section 4.2.3.2. The result of the slump test values are recorded in Table 4.11.

4.3.5 Curing

The specimens were stripped from the moulds 24 hours after casting and then submerged in water until testing. The testing was done after 28 days of casting. The results of the testing are recorded in Tables 4.17 and 4.18.

4.3.6 Types of test

The physical properties (i.e. compressive strength, tensile strength and torsional resistance) were determined following standard laboratory procedures, as described in Section 4.2.4.1. Three cubes from each mix were tested for compression, and three cylinders each from each mix were tested for splitting tensile strength and three more were tested under torsion 28 days after casting. The test setups are as shown in Figures 4.3 4.4 and 4.5.

4.4 Determination of average diameter of coconut fibres

One hundred fibres were randomly selected from the prepared coconut fibres and an average diameter of each of the 100 selected fibres was measured. A detail of this exercise is recorded in Table 4.10.

4.5 Test results and analysis

The analysis of the results from the previous sections is reported here. Raw numerical data is recorded in the Appendix A for cross-referencing where required.

4.5.1 Rock wool fibre for enhancement of concrete

a) State of Rock wool Fibre after soaking in water

The rock wool fibre was soaked in water for 72 hours and was observed under an electronic microscope. Figures 4.6 and 4.7 show that there was no difference in the appearance of the fibres except that the fibre after soaking was clearer and cleaner than before. This means that the rock wool fibre was not affected by water.

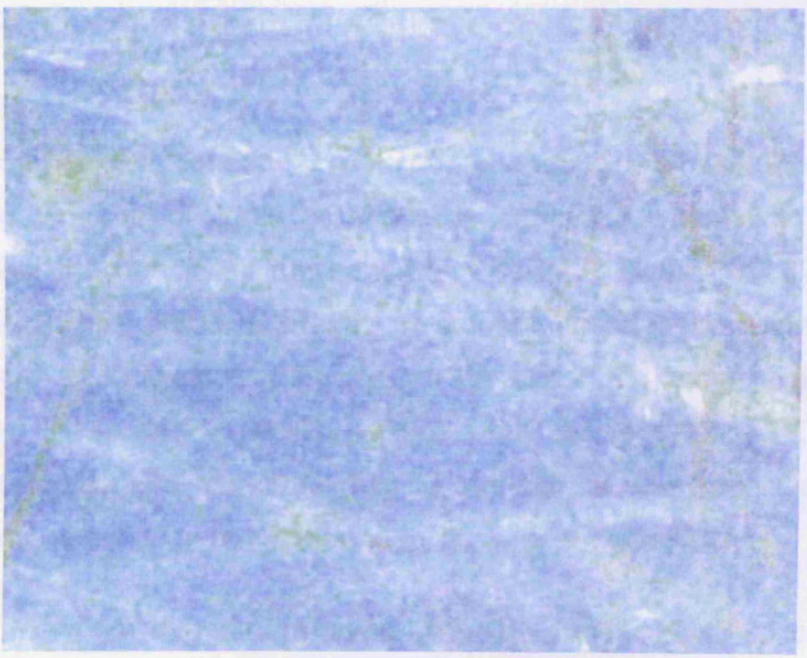


Figure 4.6 Rock wool fibres before soaked in water
(200 times magnification)

Figure 4.7 Rook wool fibres after soaking in water for 72 hours (200 times magnification)

b) Density

The density of the concrete produced decreased with increase in the percentage fraction of fibres as shown in Table 4.6 where with no rock wool fibres, the density of the concrete was 2433.8kg/m^3. At 30% (B_{30}) and 25% (B_{25}) of fibres by weight of cement, the density decreased to 2225.5 and 2285.5kg/m^3 respectively (9% and 6.5% respectively lower then plain concrete). Neville (1995) and Olanipekun et al. (2005) classified lightweight concrete has a density in the range of 300–1850 kg/m^3, which was not achieved by the present specimens.

In the main experiment using 10% (B_{10}) and 20% (B_{20}) rock wool fibre by weight of cement, the density decreased by 3% and 6% respectively.

Table 4.6 Weight of RWFC specimens

Specimen	Weight of cubes (kg)				Weight of cylinder (kg)			
	1st	2nd	3rd	Ave.	1st	2nd	3rd	Ave.
A	2.46	2.43	2.43	2.44	3.82	3.81	3.82	3.82
B_{10}	2.40	2.36	2.37	2.37	3.70	3.69	3.69	3.69
B_{20}	2.32	2.33	2.32	2.32	3.56	3.58	3.59	3.59
B_{25}	2.32	2.29	2.30	2.30	3.57	3.55	3.55	3.55
B_{30}	2.23	2.23	2.22	2.23	3.48	3.51	3.52	3.50

Table 4.7 Density of RWFC specimen

Specimen	Density of cubes (kg/m^3)	Density cylinders (kg/m^3)	Ave. Density, (kg/m^3)
A	2438.9	2428.7	2433.8
B_{10}	2372.0	2350.4	2361.2
B_{20}	2324.0	2266.4	2295.2
B_{25}	2304.0	2256.8	2285.2
B_{30}	2235.0	2230.1	2232.5

The reduction in density with increasing rock wool could be attributed to the fact that concrete, which is a dense material was replaced by rock wool fibre which is less dense, hence an obvious mass loss for concrete enhanced with the fibres. The higher mass loss and lower density of rock wool fibre enhanced concrete are also related to less smooth surface as compared to plain concrete (see Figure 4.1 and Figure 4.2). This shows that the concrete matrix with rock wool fibres was more porous than the plain concrete, which might be due to the fact that the rock wool fibre might have been destroyed by the cement hydration product hence creation of voids. The densities of the cube specimens are consistently higher than their counterpart cylindrical specimens. This might be due to the fact that the cube specimens were well compacted then the cylindrical specimens, because of their relatively smaller size.

c) Compressive strength

The test results of the cubes are summarised in Table 4.8. It could be seen that the compressive strength obtained from the concrete without the rock wool fibres was about 1.6 and 1.8 times higher than that obtained from the concrete with 30% and 25% fibre addition respectively. Concrete with 10% and 20% rock wool fibre enhancement have compressive strength values of 35.7 and 34.2MPa respectively (1.2 and 1.3 times respectively lower than plain concrete). Concrete with 10% rock wool fibre had the highest compressive strength of the samples with rock wool, though sample with 20% had only slightly smaller strength. This then implies that, the optimum percentage fraction for compressive strength criterion according to these studies would be between 10% and 20% by weight of cement.

The lower compressive strength of the concrete prepared with the rock wool fibre might be due to the following reasons.

!) The presence of a weaker fibrous matrix in the concrete might also disrupt the continuity of the concrete matrix and hence loads paths. A longitudinal compressive loading would set up a lateral tensile stresses, and hence lower ultimate compressive strength inversely proportional to the fibre content was observed.

2) Insufficient dispersing of the rock wool fibres occurred in the concrete during mixing. These micro fibres that failed to disperse fully in the concrete tend to form a multifilament structure (Bentur, 1991) in concrete during mixing and therefore increase the local porosity of the concrete.

Table 4.8 Result of compressive strength of rock wool fibre enhancement concrete

Specimen	Maximum load (kN)				Compressive strength (MPa)	Decrease in compressive strength
	1^{st} test	2^{nd} test	3^{rd} test	Average		
A	461.2	450.7	442.2	451.3	45.1	-
B_{10}	359.0	352.2	343.4	352.2	35.7	21.0%
B_{20}	347.6	371.2	352.3	357.0	34.2	21.3%
B_{25}	280.2	276.9	277.1	278.1	27.8	38.4%
B_{30}	240.1	248.8	205.0	244.2	24.4	45.9%

d) Toughness of concrete enhanced with rock wool

Energy absorption capacity or toughness of concrete is defined as the area under the torsion-rotation curve calculated up to a specific rotation value. In this test, the area under the torsion-rotation curve was calculated up to a rotation of 9×10^{-3} radians. The applied torque T is given by:

$$T=P_TZ \tag{4.1}$$

where,

P_T = applied load causing torsion (N), see Figure 4.5.

Z = Distance between the point of application of the load and the centre of the cylinder (mm). (Z= 0.13m)

and the twist rotation (radian) of the cylinder due to the torque.

$$\Phi = \frac{Y}{Z} \tag{4.2}$$

Y= angular displacement

The raw data of load and displacements are recorded in Tables A1 and their corresponding torsion and twist values are recorded in Table A2 in the Appendix A. The energy dissipation was obtained by considering the area enclosed by the torsion-twist curve (see Figure 4.8).

Table 4.9 Results of toughness rock wool fibre enhanced concrete

Specimen	Max applied Load (kN)	Torsion (Nm)	Twist at peak torsion radian	(Toughness) (Nm)
A	2.69	348.40	1.00	0.88
B_{10}	2.49	332.10	1.00	0.66
B_{20}	2.48	293.40	0.76	0.57
B_{25}	2.61	285.00	0.68	0.33
B_{30}	1.80	228.80	0.63	0.23

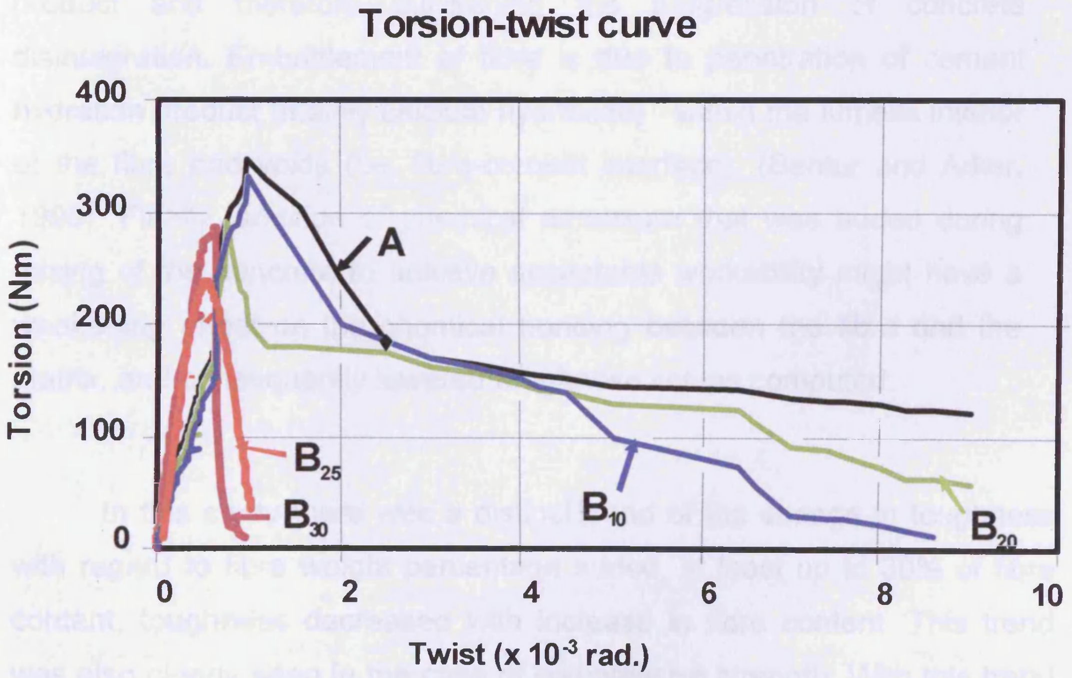


Figure 4.8. Torsion-twist curve of concrete enhanced with rock wool fibre

It could be seen from Figure 4.8 that, after the ultimate torsion, the decrease in measured torsion with increase in deformations was much less in plain concrete than in samples with rock wool fibre. As a result the total energy absorbed as measured by the area under torsion-twisting curve up to complete failure of the cylinder was between 12.5% and 99% higher for plain concrete than that of rock wool fibre enhanced concrete. There was a considerable decrease in ductility or toughness as the percentage weight of fibre increases.

It was expected that much energy would be needed in debonding and stretching the fibre in the concrete and hence greater toughness was to be seen in fibre enhanced concrete than that for plain concrete. In this investigation the above expectation was not achieved. The possible explanation for the reduced toughness could be that the rock wool fibres disintegrated due to pore alkaline cement solution, and/or rock wool fibre might have been embrittled by the cement hydration product and therefore quickening the progression of concrete disintegration. Embrittlement of fibre is due to penetration of cement hydration product (mainly calcium hydroxide) within the lumella interior of the fibre and voids (i.e. fibre-cement interface). (Bentur and Arker, 1998). Finally, addition of chemical admixture that was added during mixing of the concrete to achieve acceptable workability might have a weakening effect on the chemical bonding between the fibre and the matrix, and consequently lowered toughness values computed.

In this study there was a distinct trend of the change in toughness with regard to fibre weight percentage added, at least up to 30% of fibre content, toughness decreased with increase in fibre content. This trend was also clearly seen in the case of compressive strength. With this trend one might suggest that the optimum fibre percentage by weight of cement should be less 10 %. Further investigation is needed within this percentage range to ascertain more precisely at percentage that there would be the most improvement in the toughness and other mechanical properties of rock wool fibre enhancement of concrete.

Although it is universally recognized that the importance of fibre in concrete is to enhance the toughness of the composite (Surendra, 2004), in this investigation it could be said that rock wool fibre did not satisfy this accepted understanding.

e) Conclusion on rock wool fibre enhancement concrete

In the experimental studies of concrete matrix with the addition of rock wool fibre as an enhancement, there was an unexpected depreciation of toughness, and tensile strength. The main factors affecting fibre-cement strength and toughness are fibre-concrete contact and fibre-concrete bonding (Mohr et al., 2003a). The possible decease in rock wool fibre-concrete contact might have allowed cement hydration product formation within the lumella of the rock wool fibre that resulted in an decreased in fibre-concrete bonding and subsequent embrittlement of the concrete.

4.5.2 Coconut fibre as enhancement to concrete

This section describes work on coconut fibres in enhancing the performance of concrete.

a) Preparation of fibres

To facilitate the extraction of fibres, coconut husks were soaked in water for one month and later placed in 10% concentration of sodium hydroxide (NaOH) for seven days, before physical extraction of the fibres by hand. Fibres were separated while minimising structural damages during the extraction process. The fresh water was meant to remove the pith particles and the lignin from the surface of the fibres (Nanayakkrar et al., 2005). Studies conducted by Ramakrisma el al. (2004) on the durability of natural fibres, indicated that NaOH is also a good solvent for both lignin and hemicelluloses, and also coconut fibre retained about 73% of its initial tensile strength when placed in NaOH for up to 60 days. Based of this knowledge, the coconut husks were further placed in NaOH for seven days to dissolve the lignin and hemicelluloses to facilitate extraction of fibres.

b) Surface morphology

Husks soaked in fresh water and then in NaOH were observed to ascertain the effect of the soaking mediums. An electronic microscope was used to observe the surface structure of the fibre soaked in water and then in NaOH. Figures 4.9a-4.9e show the changes of the fibre surface with soaking medium and soaking time.

Figure 4.9a Coconut husk before being soaked in water

Before soaking the husk in water pith cells remained tightly attached to the fibre surface, (Figure 4.9a). The pith (or dust) is non fibrous tissues surrounded the fibre and are bonded by lignin.

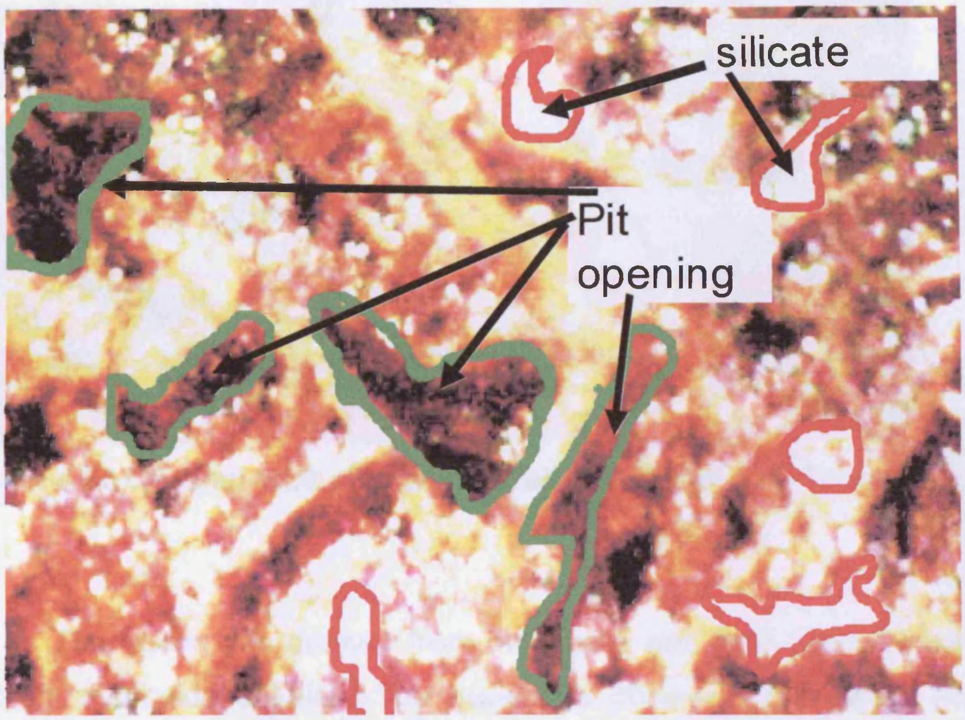


Figure 4.9b Coconut fibre after being soaked in water for 30 days

After soaking the husk for thirty days, the surface became clearer due to the removal of pith cells adhering to the fibres. The fibres display many "pin-hole" like structures on the surface of the husk, known as pits. The pits are variously arranged in between the fibres on the husk surface (see Figure 4.9b), and are not uniformly spaced. The pit shape is somewhat irregular in its facial view. The soaking process thus facilitates the removal of pit particles from the husk surface. In addition white crystal-like structure could be seen on the pits, which according to Bowlike and Debnath (1984) are reported to be silicate crystals. Silicate usually occurs in plant in the form of its oxide, silicon dioxide (Lanning et al., 1958). These structures give roughness to the fibre surface.

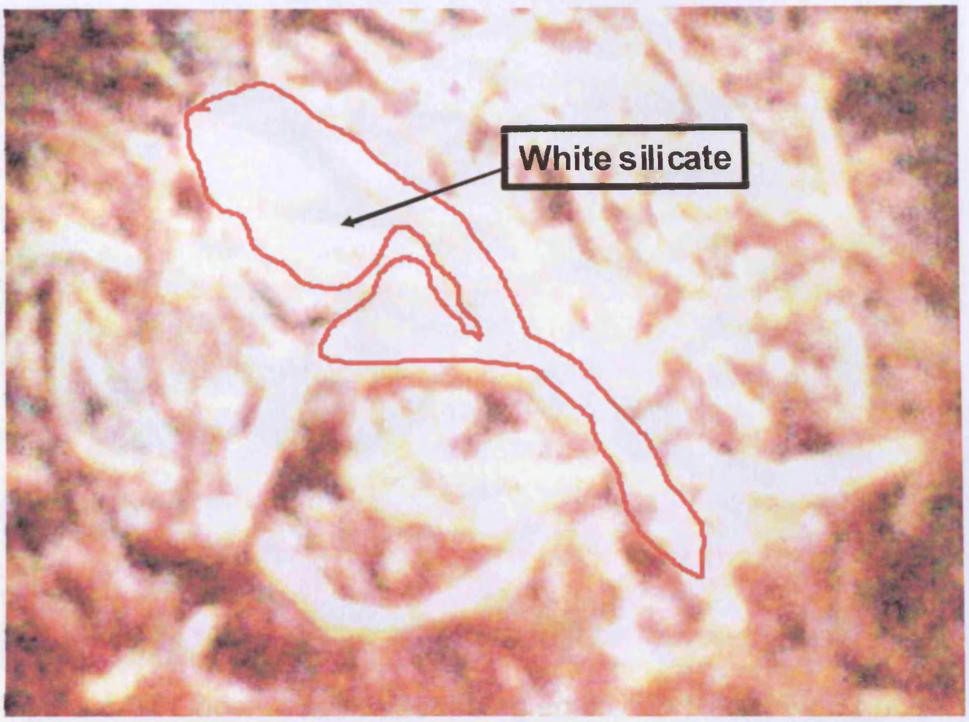


Figure 4.9c Coconut husk after being soaked in NaOH for 3 hours

The water was drained from the husks and then replaced with a sodium hydroxide solution. The husk was observed after three hours under electronic microscope. It could be seen from Figure 4.9c that the silicate crystals had almost disappeared. The silicate crystal could not be completely dissolved by the sodium hydroxide as some of the crystals could be seen under the microscope even after seven days of soaking in sodium hydroxide solution. With soaking time in sodium hydroxide solution (Figures 4.9 c-e), the fibre became clearer and thinner. This indicated that the vascular bundle surrounded the fibre had dissolved. This vascular bundle is believed to be lignin. Lignin is a complex constituent of plant that cements the cellulose fibres together. In all cases the fibres themselves were not affected with soaking mediums and soaking time.

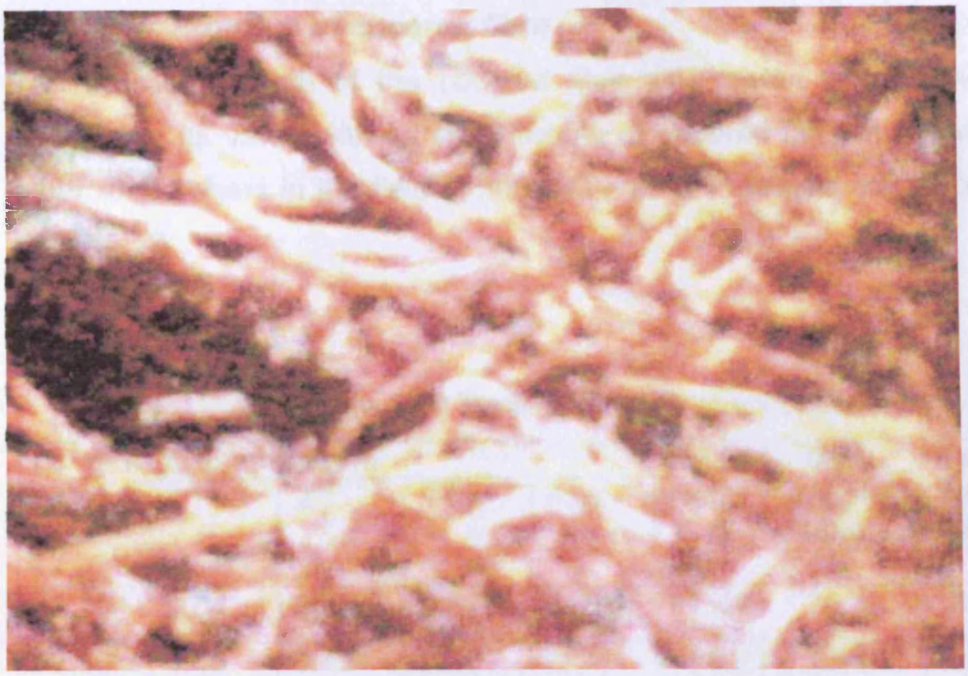


Figure 4.9d Coconut husk after being soaked in NaOH for 6 hours

Figure 4.9e Coconut husk after being soaked in NaOH for 7 days

c) *Tensile strength of coconut fibre*

The tensile strength of the coconut fibre was tested with the view of comparing its value with tensile strength of coconut fibre in literature by other researchers in the field of natural fibres. The results are recorded in Table 4.10.

Table 4.10 Tensile strength of coconut fibre

Length (mm)	Diameter (mm)				Aspect ratio	Max. Applied load (N)	Tensile strength (MPa)
	1st	2nd	3rd	Ave.			
45.15	0.64	0.73	0.82	0.73	61.85	25.3	60.45
52.16	0.46	0.43	0.38	0.42	168.67	22.87	165.07
65.06	0.29	0.42	0.49	0.39	176.80	22.70	190.02
67.25	0.61	0.53	0.54	0.57	118.00	17.30	67.80
70.46	0.54	0.58	0.61	0.58	121.50	18.20	68.60
84.22	0.56	0.48	0.46	0.49	161.88	17.10	90.68
84.85	0.51	0.48	0.47	0.48	166.80	18.40	101.68

Average tensile strength =88.9MPa

On the basis of these tests it could be said that increase in the aspect ratio also increased the tensile strength, but the increase was not linear as could be seen in Figure 4.10. This is because there was no consistent numerical pattern in the selected aspect ratio, since the coconut fibres were randomly picked, and hence non-linear shape of the graph of tensile strength verses aspect ratio.

As reported in the literature (Agopyan, 1988 and Resi, 2004), the tensile strength of coconut fibre ranges from 15MPa to 220MPa. The coconut fibre in this study which has an average tensile strength of 88.5MPa which is within the expected range.

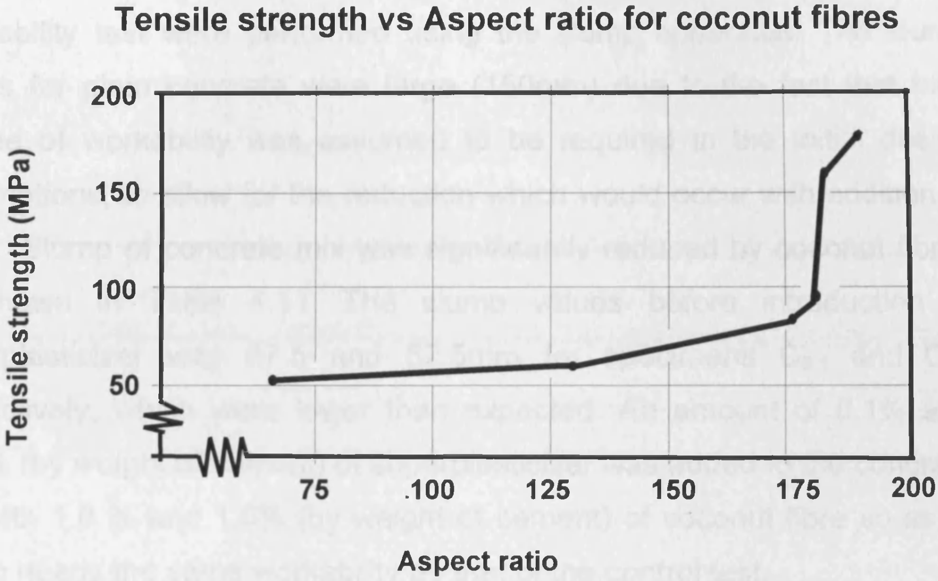


Figure 4.10. Tensile strength against aspect ratio

d) Density of fibre

One hundred different fibres of average length seven centimetres and average diameter 0.48cm were weighed and the density calculation performed. The average mass of 100 fibres was 3.65g and a volume of $100(\pi\, 0.48^2 x7/4)$ was used to give a density of $0.029 g/cm^3$

The density of the MYD+PT species ($0.03\ g/cm^3$) used in this studies is much less than the species from Latin America and South East Asia origin which is recorded in literature as $1.25\ g/cm^3$.

e) Concrete strength and characteristics

Standard quality control test methods (slump, density and strength test) were used for the coconut fibre enhanced concrete

i) Workability test.

Workability test were performed using the slump apparatus. The slump values for plain concrete were large (150mm) due to the fact that high degree of workability was assumed to be required in the initial design assumptions, to allow for the reduction which would occur with addition of fibres. Slump of concrete mix was significantly reduced by coconut fibres as shown in Table 4.11. The slump values before introduction of superplasticizer was 87.5 and 62.5mm for specimens $C_{0.1}$ and $C_{1.5}$ respectively, which were lower than expected. An amount of 0.1% and 0.15% (by weight of cement) of superplasticizer was added to the concrete mix with 1.0 % and 1.5% (by weight of cement) of coconut fibre so as to obtain nearly the same workability as that of the control test.

Table 4.11 Fresh concrete test results (CFC)

Mix	Initial slump height (mm)	Adjusted slump height (mm)	Remarks
A	150.0	Nil	150mm was the initial assumed slump height
$C_{0.5}$	137.5	Nil	No supperplasticizer was added since the slump height is close to the assumed height
$C_{0.1}$	87.5	140	0.1% (by weight of cement) of superplasticizer was added.
$C_{1.5}$	62.5	150	0.15% (by weight of cement) of superplasticizer was added.

ii) Density of composite

As can be expected the density of the concrete produced decreased with increasing percentage of added coconut fibre, as shown in Table 4.12. The density of plain concrete at 28-day curing age was 2442.8 kg/m^3. With 0.5%, 1.0% and 1.5% (by weight of cement) of coconut fibre addition, the density decreased respectively, to 2378.2, 2340.5 and 2296 kg/m^3. The decrease in density is about 2.6%, 4.2% and 6.0% respectively. This can be expected since the coconut fibre is less dense than the concrete.

Table 4.12 Average densities of coconut fibre concrete (Preliminary study)

Mix	Weight (kg)							Density (kg/m^3)		
	Cube				Cylinder			Cube	Cylinder	Ave.
	1st	2nd	3rd	Ave	1st	2nd	Ave			
A	2.5	2.4	2.5	2.5	3.8	3.8	3.80	2460	2425.5	2442.8
$C_{0.5}$	2.3	2.4	2.4	2.4	3.8	3.8	3.80	2350	2406.4	2378.2
$C_{0.1}$	2.3	2.3	2.3	2.3	3.7	3.8	3.75	2300	2381.0	2340.5
$C_{1.5}$	2.2	2.3	2.3	2.3	3.6	3.6	3.60	2250	2287.0	2268.5

iii) Compressive strength of concrete

It could be seen from Table 4.13 that the compressive strength obtained from mix with the addition of 0.5% of coconut fibre by weight of cement at 28-day curing age was about 1.1 times higher than that obtained from the plain concrete. However, the mixes with 1.0% and 1.5% (by weight of cement) of coconut fibre resulted in a lowering of the compressive strength at 28-day curing age, at about 0.9 and 0.7 times respectively less than that of the plain concrete. This suggests that the optimum percentage of coconut fibre from a compressive strength enhancement point of view is around 0.5%. The coconut fibre was added to particularly enhance the toughness so an enhancement in compressive strength is a bonus, which has not yet been found in literature.

In the case of early strength, i.e. compressive strength at 7 days there was no improvement for mixes with fibre addition as compared with the control sample. Similar results had been reported by other researchers including Poon et al (2004), Olanipekun (2006) and Kriker et al (2004). There was however an increase in compressive strength for all samples with further curing beyond 7-days as shown in Table 4.13, which was due to the effect of concrete maturity.

Table 4.13 Results of compressive strength of concrete of all mixes at 7 days and 28 days

Specimen	Maximum applied load (kN)				Compressive strength (MPa)	
	7 Days		28 Days		7 Days	28 Days
A	362.9 317.7 342.7	341.1	429.3 422.2 415.3	422.2	34.1	42.2
$C_{0.5}$	317.1 314.2 315.0	315.4	441.0 445.0 442.0	442.7	31.5	44.3
$C_{0.1}$	343.0 318.8 327.0	329.6	382.8 376.0 361.7	373.5	33.0	37.4
$C_{1.5}$	227.0 215.0 220.8	220.9	288.8 306.0 301.6	298.3	22.1	29.8

iv) Splitting tensile strength

The splitting tensile strength test was used to determine the tensile strengths of the plain concrete and concrete with the various amounts of fibre inclusions at 7-day curing age and 28-day curing age. For the plain concrete, the cylinders split down the centre as expected. Unlike the plain concrete cylinders, the two halves for mixes with various amounts of fibres were held together by the fibre after the splitting as shown in Figure 4.11. The method for the determination of tensile strength according to BS1881, Part 117, 1983 is given by

$$\sigma_t = \frac{2F}{\pi Ld} \qquad (4.4)$$

where,

F = Applied force

L =Height of cylinder

D =Diameter of cylinder

The summarised results of the splitting tensile strength in the Table 4.14 indicate that there was an increase in tensile strength from 7 days to 28 days for all the mixes of about 6%-11% for concrete with coconut fibre weight, and 17% for plain concrete, which was due to the effect of concrete maturity.

Table 4.14 Splitting tensile strength of coir fibre reinforced concrete for 7-day and 28-day curing age

Specimen	Maximum applied load (kN)				Splitting tensile strength (MPa)	
	7 Days		28 Days			
	Values	Ave.	Values	Average	7 Days	28 Days
A	96.1 92.1 94.1	94.1	115.0 113.8 114.4	114.4	3.0	3.6
$C_{0.5}$	107.8 115.8 117.0	111.7	116.0 121.0 120.0	119.0	3.6	3.8
$C_{0.1}$	98.9 96.0 97.0	97.3	104.0 109.0 107.0	106.5	3.1	3.4
$C_{1.5}$	71.4 67.8 69.0	69.4	87.0 70.4 78.0	78.7	2.2	2.5

The results at 28-day curing age for the concrete mixed with 0.5% weight fraction of fibre indicated that there was an improvement of tensile strength by 4% over the plain concrete. The increase was more pronounced for the same mix at 7-day curing age (about 19%). However, there was decrease in tensile strength at 28 days for the mixes with fibre weight fraction of 1.0% and 1.5% as compared with the plain concrete.

In conclusion, a low 0.5% of coconut fibre did improve the tensile strength of fibres but larger amount of fibres then weakened the concrete. Evidence from Shah (1991) concluded that the major contribution of fibres is in enhancing toughness, rather than the tensile strength, but as in the case of compressive strength, a small 0.5% addition of coconut fibre actually had a positive contribution to strength both tensile and compressive.

Figure 4.11 Condition of concrete cylinder after splitting test

v) Toughness

For the purpose of this study, the area under the torsion-angle of twist curve produced from the cylinder twisting test was used to calculate the toughness, and twist was limited to 0.012 radians.

Table 4.15 Toughness characteristics of coconut fibre Enhanced concrete at 28 days

Specimens	First crack		Max. Torsion (N.m)	Twist at max torsion ($x10^{-3}$rad.)	Toughness (Nm)
	Torsion (Nm)	Toughness (Nm)			
A	3.10	0.45	405.6	5.00	0.83
$C_{0.5}$	3.40	0.48	417.0	3.00	0.95
$C_{0.1}$	2.60	0.36	341.0	1.00	0.60
$C_{1.5}$	2.20	0.31	308.9	0.50	0.43

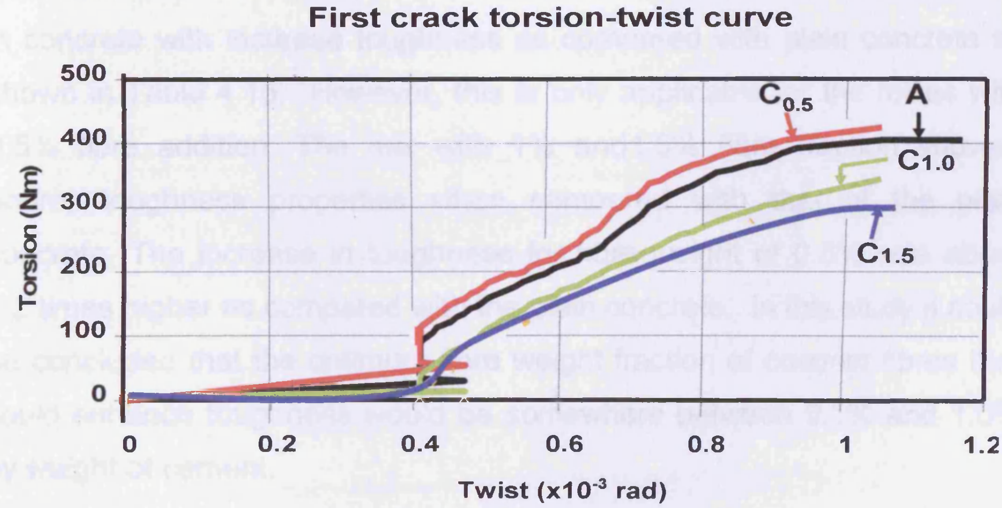


Figure 4.12a Graph of first crack toughness of coconut fibre enhanced concrete

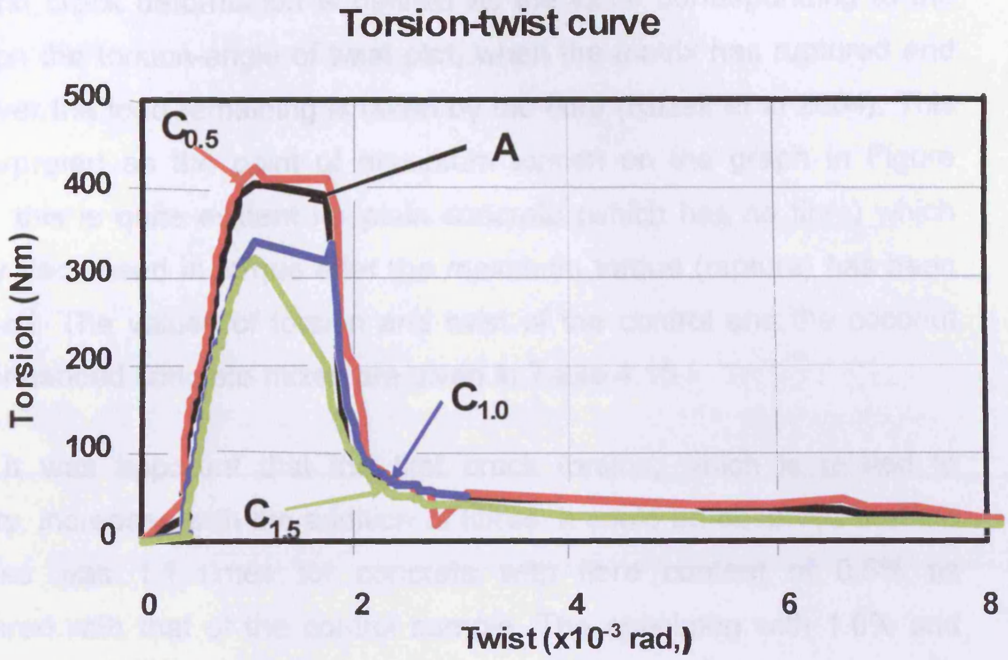


Figure 4.12b Torsion-twist curve of coconut fibre enhanced concrete

The addition of coconut fibre as enhancement of concrete resulted in concrete with increase toughness as compared with plain concrete as shown in Table 4.15. However, this is only applicable for the mixes with 0.5% fibre addition. The mix with 1% and1.5% fibre fraction showed poorer toughness properties when compared with that of the plain concrete. The increase in toughness for fibre weight of 0.5% was about 1.2 times higher as compared with the plain concrete. In this study it could be concluded that the optimum fibre weight fraction of coconut fibres that could enhance toughness would be somewhere between 0.1% and 1.0% by weight of cement.

vi) *First crack deformation (twist)*

The first crack deformation is defined as the value corresponding to the point on the torsion-angle of twist plot, when the matrix has ruptured and whatever the load remaining is taken by the fibre (Razak et al 2004). This is interpreted as the point of maximum torsion on the graph in Figure 4.12a. this is quite evident for plain concrete (which has no fibre) which rapidly decreased in torque after the maximum torque (rapture) has been released. The values of torsion and twist of the control and the coconut fibre enhanced concrete mixes are given in Table 4.15

It was apparent that the first crack torsion, which is related to ductility, increases with the addition of fibres. It could be observed that the increase was 1.1 times for concrete with fibre content of 0.5% as compared with that of the control sample. The specimen with 1.0% and 1.5% weight fraction of fibre have their first crack torsion lower than the plain concrete which is about 1.2 and 1.4 times respectively lower.

vii) *First crack toughness*

The first crack toughness is the area under the torsion- twist curve up to the maximum torsion, in other words area under the elastic part of the torsion-twist curve. The first crack toughness at 28 days for all mixes is presented in Table 4.15 and Figure 4.12a (areas under the curves not marked by gridlines). The results indicated that there was no increase in first crack toughness due to fibre addition for 0.1% and 1.5% weight fraction of fibre. The increase of first crack toughness was pronounced with 0.5% fibre weight fraction of cement, which was about 1.1 times that of the control. There was a distinct trend of the first crack toughness with regard to fibre weight content. At least, up to a fibre content of 0.5% first crack toughness deceases with increase in fibre content.

Generally, it was found that the addition of coconut fibres up to certain fibre content would enhanced the toughness of the concrete, as is now generally reported in literature (Shah, 1991). Further laboratory investigation is needed to ascertain the optimum fibre content and also to find out if the fibre aspect ratio would have effect on the strength and toughness of concrete.

f) Summary

From the results of the investigation on coconut fibre enhanced concrete, it could be deduced that the addition of coconut fibres to concrete up to certain fibre content would result in increase in torsion, energy absorption capacity and to a certain extent splitting tensile strength. However, the relative magnitude of increase in these properties in the 0.5% fibre range is much greater than any such increases at higher fibre content. Therefore the optimum weight fraction of coconut fibre that might enhance the engineering properties of mortar and concrete, notably torsion, toughness and tensile strength would be between 0 and 1% (by weight of cement) of coconut fibre. Further studies on coconut fibre enhancement of concrete should thus be focused on weight percentages between say 0.25% and 1% by weight of cement.

4.5.3 Further experiments of coconut fibre enhanced concrete

Generally, it has been observed that the addition of up to one percent of coconut fibre could improve the strength performance of concrete. The extent of improvement of the properties of concrete could be expected to depend upon the fibre content and fibre geometry (diameter and length of fibre). Various combinations of these parameters would give rise to different strength characteristics. However, there is limited information regarding the quantitative influence and relative importance of fibre aspect ratio on the strength of concrete. Therefore the introduction of fibre aspect ratio and fibre weight percentage between 0.25% and 1% in the subsequent experiment would add a further dimension to this study of coconut fibre in concrete.

The shape of the torsion-twist curve is strongly affected by the testing conditions such as size and shape of the specimen, loading rate and concrete characteristics such as water/cement ratio, aggregate type, percentage weight fraction of fibre, etc (Lee et al.,2004). However, careful attention was given at all stages to avoid variations in the casting and testing procedure so that any variation observed in the current test could clearly be attributed to the differences in the concrete mix.

a) Slump test

Slump test values showed concrete workabilities ranging from medium (25-50mm) to high (51mm and over). Using water and cement ratio of 0.5 (for normal concrete) the slump value obtained was 125mm which indicates high workability. This value deceased progressively as percentage of fibre added to the concrete increased. Adjustments were made in the water cement ratio and superplasticizer was added to the mix with higher fibre content so as to obtain a slump test value of 125 ± 25 mm. The result is recorded in Table 4.16.

Table 4.16 Fresh concrete test results of coconut fibre enhanced concrete

Mix	Initial slump height (mm)	Adjusted slump height (mm)	Remarks
A	125	Nil	Initially assumed a high workability value
$C_{0.25}$	105	Nil	A high workability range was achieved.
$C_{0.5}$	75	115	Superplasticizer was added (0.5% by wt of cement.
$C_{0.75}$	48	115	Superplasticizer was added (0.75% by wt of cement)

b) Behaviour under Compression

The results of compressive and tensile strengths are presented in Tables 4.17 and 4.18 respectively. Discussions are based on the average results of three samples.

Figures 4.13a and 4.13b show variation of compressive strength for plain concrete and concretes with different aspect ratios containing three different coconut fibre contents. The results of compressive strengths are presented in Tables 4.18. The addition of fibres did not increase the compressive strength. Figure 4.13a shows that there is a loss in compressive strength for higher aspect ratio for all percentage of added coconut fibre content.

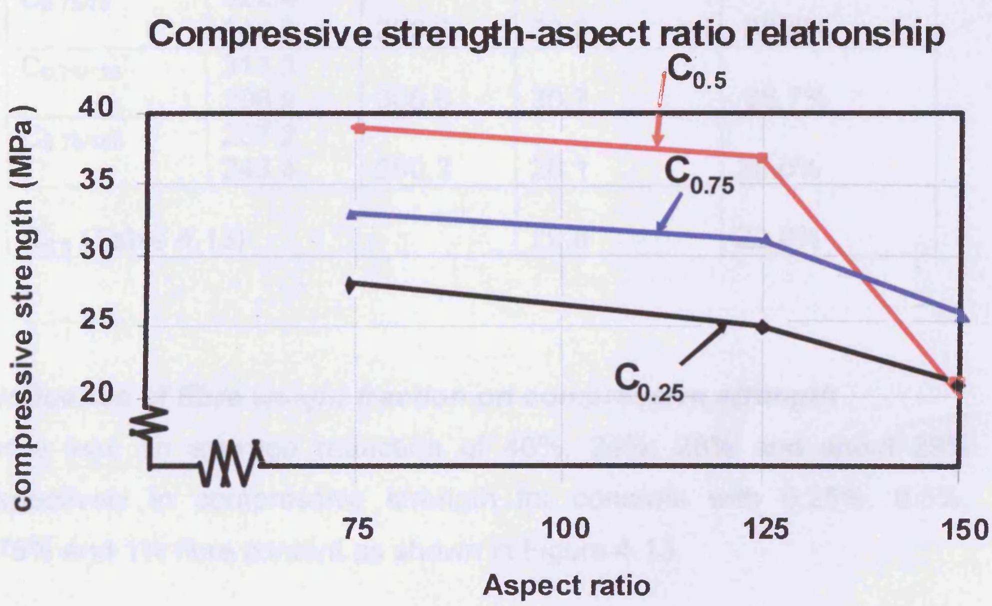


Figure 4.13a Compressive strength verse aspect ratios for coconut fibre enhanced concrete

Table 4.17 Results of compressive strength of coconut fibre enhanced concrete

Specimen	Maximum Applied force		Compressive Strength MPa	Decrease in compressive strength
	Values	Average		
A	418.0 420.0	419.0	41.9	-
$C_{0.25/75}$	281.0 277.0	279.0	27.9	33.0%
$C_{0.25/125}$	250.0 253.0	251.5	25.2	39.9%
$C_{0.25/150}$	212.8 210.7	211.8	21.2	49.0%
$C_{0.5/75}$	383.6 360.4	372.0	37.2	11.0%
$C_{0.5/125}$	375.3 354.2	369.8	37.0	11.6%
$C_{0.5/150}$	210.8 197.4	204.0	20.4	51.0%
$C_{0.75/75}$	322.4 344.0	333.2	33.3	20.5%
$C_{0.75/125}$	313.3 296.9	306.6	30.7	26.7%
$C_{0.75/150}$	267.2 243.4	260.3	26.1	38.0%
$C_{0.5}$ (Table 4.13)			29.8	29.0%

i) Influence of fibre weight fraction on compressive strength

There was an average reduction of 40%, 24%, 28% and about 29% respectively in compressive strength for concrete with 0.25%, 0.5%, 0.75% and 1% fibre content as shown in Figure 4.13.

ii) Influence of fibre length on compressive strength

For all weight fractions, the compressive strength decreased as the aspect ratio of fibre increased. The fibres with lower aspect ratios provided better performance in compressive strength. For this investigation the best aspect ratio was about 75 for all fibre contents as shown in Figures 4.13a compressive strength (closer to plain concrete) might yet have been obtained)

In general, the addition of fibres adversely affected the compressive strength, as expected; this might be due to the difficulties in compaction which consequently created voids. This is reflected in the increase in the air content with increase in the fibre length; hence at the same fibre content specimens with highest aspect ratio have the lowest compressive strength as shown Table 4.17.

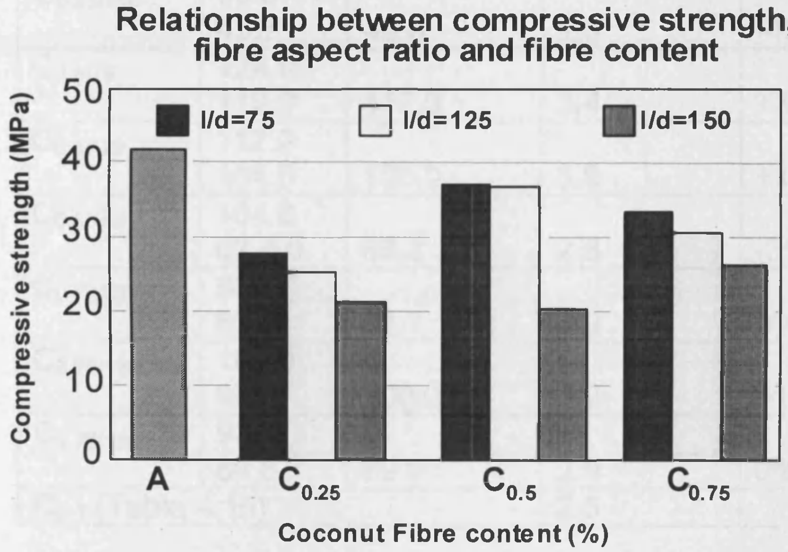


Figure 4.13b Compressive strength verse aspect ratios for coconut fibre enhanced concrete

c) Behaviour under tension

In the splitting tensile tests cylindrical specimen are subjected to splitting tension along their axis. Figures 4.14a and 4.14b compare tensile strength values for three fibre aspect ratio and three fibre weight fractions. The results of tensile strengths are presented in Table 4.18.

Table 4.18 Results of tensile strength of coconut fibre enhanced concrete

Specimen	Maximum applied load(kN) at 28-day curing age		Tensile strength (MPa) at 28-day curing age	Increase in tensile strength
	Test values	Average Value		
A	92.0 87.0	91.0	2.9	-
$C_{0.25/75}$	68.3 60.7	64.5	2.1	-27%
$C_{0.25/125}$	75.4 69.0	72.2	2.3	-21%
$C_{0.25/150}$	50.3 45.7	48.0	1.5	-48%
$C_{0.5/75}$	124.0 110.0	117.0	3.4	+30%
$C_{0.5/125}$	112.0 104.0	108.0	3.9	+17%
$C_{0.5/150}$	104.0 82.4.0	88.2	2.8	-3%
$C_{0.75/75}$	96.4 87.0	91.7	2.9	0%
$C_{0.75/125}$	104.0 96.0	100.0	3.2	+10%
$C_{0.75/150}$	93.0 86.8	89.9	2.9	0%
$C_{0.5}$ (Table 4.15)			2.5	-13%

i.) Influence of fibre weight fraction on tensile strength

For a higher percentage (0.5% and 0.75%) of coconut fibre the cylindrical specimens had only cracked in the specimen, but did not split into two halves like those of the plain concrete and those with 0.25% coconut fibre enhanced concrete as shown in Figure 4.15.

In this investigation the fibre weight fraction of 0.25% did not improve the splitting tensile strength, there was a reduction of an average about 34% as shown in Table 4.18 and Figures 4.14a and 4.14b. The better bond in the specimens with higher weight fraction resulted in higher splitting tensile strength. On average there was an increase of about 15% and 3.2% for concrete with fibre content of 0.5% and 0.75% respectively. However, beyond 0.75%, the tensile strength again decreased

ii) Influence of fibre length on tensile strength

It was established that the optimum fibre aspect ratio of 125 (length/diameter) provided the best performance in splitting tensile strength in all weight percentage, except for 0.5% although the value for 0.75% was only marginally higher than that for 0.5% in the case of aspect ratio of 150. It is also observed in this experiment that, at the same fibre content, composites with the fibre aspect ratio of 125 have higher tensile strength than those of fibre aspect ratio of 75 (Figure 4.14b).

It is expected that, at the same fibre content, composites with the highest fibre aspect ratio of 150 would have higher tensile strength than those with smaller fibre aspect ratios (Romildo et al., 1999), but this is not the case in this study. It appears that the critical fibre aspect ratio is 125, and any increase of the critical fibre length leads to a corresponding decrease in tensile strength. This fact suggests that the shorter fibres became mineralised, in other words embrittled, earlier than the critical fibres length (aspect ratio of 125). One of the reasons could be attributed to the fact that in the short fibre-enhanced composite (e.g. fibre aspect ratio of 75) there are more end points which allowed faster penetration of cement hydration products into the fibre lumen walls and voids and therefore, accelerating the loss of flexibility of the fibres. Coconut fibre, like many natural fibres, is originally flexible and become stiffer when in contact with hydration products of concrete (Mohr et al., 2004).

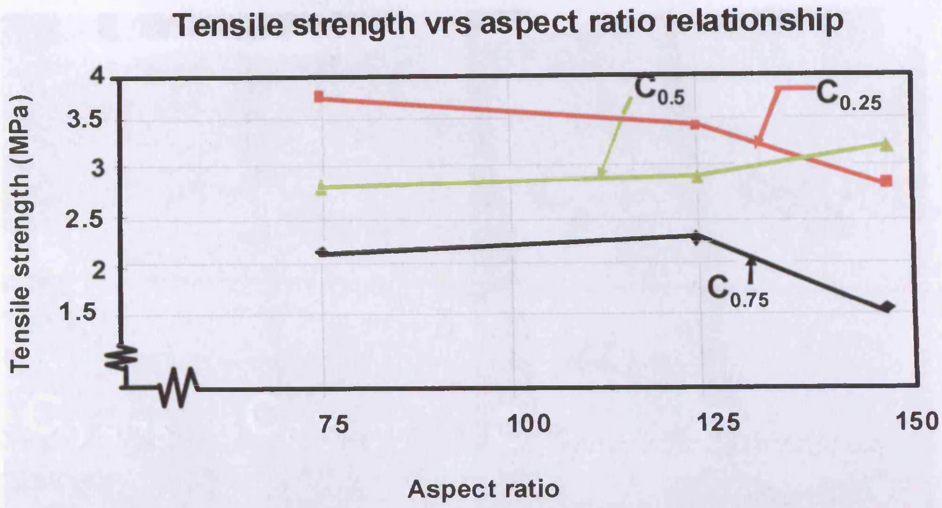


Figure 4.14a Tensile strength verse aspect ratios for coconut fibre enhanced concrete

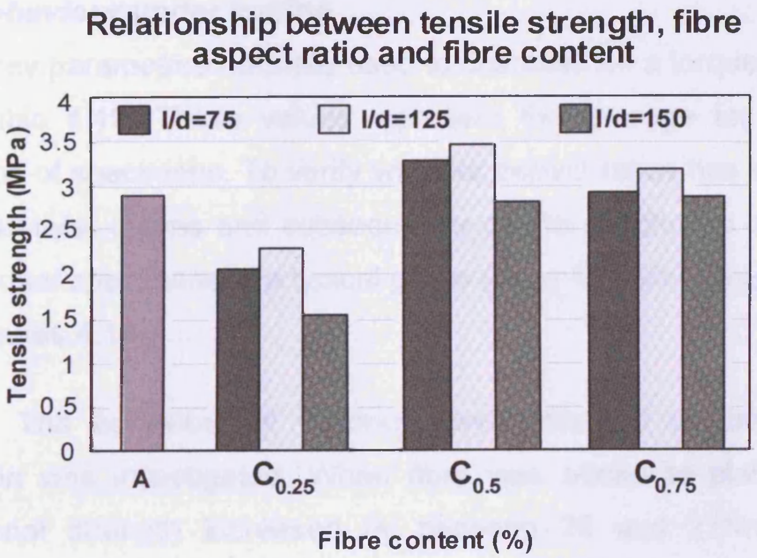


Figure 4.14b Tensile strength with respect to fibre aspect ratio for different fibre content

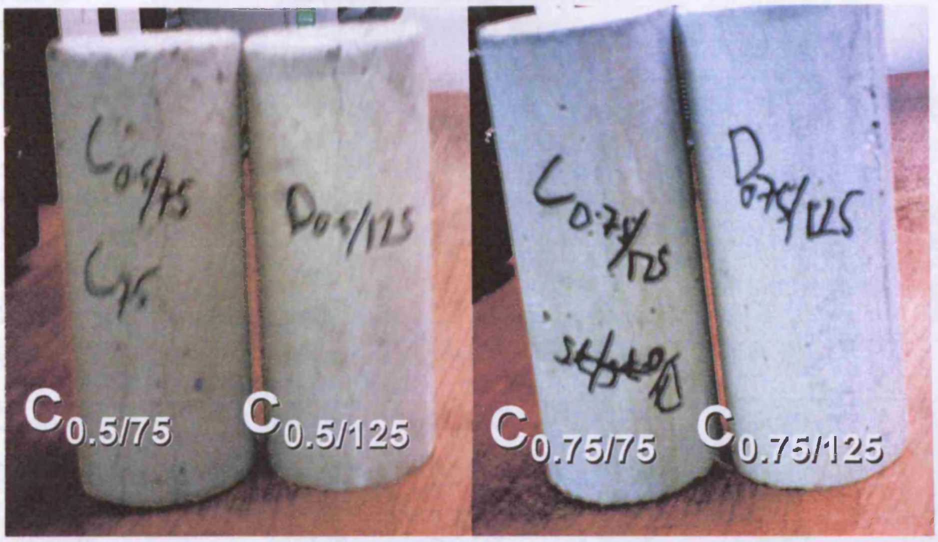


Figure 4.15.Specimen after splitting test for concrete fibre concrete

d) Behaviour under torsion

The key parameters normally used to characterise a torque-twist curve are in Table 4.19. These values represent the average for the respective groups of specimens. To verify whether aspect ratios has influence on the torsion-twist curves and subsequently on the toughness of the concrete, individual specimens of a typical group ($C_{0.25}$, $C_{0.5}$ and $C_{0.75}$) are presented in Figures 4.16.

The behaviour of coconut fibre–enhanced concrete under pure torsion was investigated. When fibre was added to plain concrete the torsional strength increased by between 26 and 77Nm which is an increase between 10% and 20% (Table 4.19). There is an optimum weight fraction beyond which the torsional strength started to decrease again. Similar pattern were also obtained for different fibre aspect ratios for the same fibre content, where again results showed there was an optimum aspect ratio as seen in Figure 4.16.

Table 4.19 Torsion characteristics of coconut fibre enhanced concrete at 28-day curing age

Specimen	Applied torsional Force (kN)	Angular Displacement, (mm)	Ultimate torsion (Nm)	Angle of twist at max. torque ($x10^{-3}$ rad)	Modulus of resilience (Nm)	Modulus of toughness (Nm)
A	2.2	71.0	285.0	0.57	0.13	0.66
$C_{0.25/75}$	2.5	113.0	322.0	0.87	0.66	0.87
$C_{0.25/125}$	2.6	413.0	337.5	3. 18	0.87	1.07
$C_{0.25/150}$	2.4	43.0	311.5	0.33	0.64	1.06
$C_{0.5/75}$	2.5	65.0	325.0	0.50	0.90	1.09
$C_{0.5/125}$	2.8	418.0	362.5	3.35	0.98	1.27
$C_{0.5/150}$	2.8	395.0	362.2	3.16	0.98	1.18
$C_{0.75/75}$	2.5	61.7	308.8	0.49	0.84	1.07
$C_{0.75/125}$	2.4	32.1	328.8	0.53	0.81	1.09
$C_{0.75/150}$	2.1	89.6	273.0	0.72	0.63	1.08

e) Toughness

The addition of fibres increased the angle of twist corresponding to the peak torsion. Increase in peak angle of twist at peak torsion is maximum for concrete having 0.5% fibre weight fraction with aspect ratios of 125 and 150, and resulted in the highest value of energy absorption without creating a permanent distortion (modulus of resilience) (see Figure 4.17b).

The twist capacity and the elastic deformation capability of the concrete matrix just before failure were increased considerably with the inclusion of coconut-fibres. However the relative magnitude of energy increased up to the elastic limit (modulus of resilience) in 0.5% fibre weight fraction is on average about 30% and 24% greater than energy increase in

0.25% and 0.75% fibre contents respectively. Overall coconut fibre enhanced concrete could absorb much more energy before failure compared with the plain concrete counterpart. The optimum weight percentage of fibre and aspect ratio for this investigation are 0.5% and 125 respectively. The increase in toughness for fibre weight 1.0% was about 1.8 times higher respectively as compared with the plain concrete at 28 days curing age (section 4.5.2.5)

The total area under torsion-twist curve, which measures toughness, increased substantially with the addition of coconut fibre, by between 1.3 and 1.9 times, resulting in a more ductile behaviour. The increase in toughness of the concrete could be attributed to the probable increase of fibre-cement contact of the coconut fibres due to higher lignin content of the coconut fibres (about 30%) (Nanayakkra et al., 2005) which stiffened the cell-wall of the fibre preventing embrittlement of the fibres.

The improvement in ductility is more pronounced in specimen with fibre weight fraction of 0.5% and an aspect ratio of 125 (Table 4.19 and Figures 4.17-4.18). It was further established from this experiment that with a constant weight fraction, the toughness is higher with specimens having an aspect ratio of 125 (Table 4.19) and (Figures 4.17 a-c). Again it is clear from the investigation that, at a constant aspect ratio, 0.5% fibre had the highest modulus of toughness and 0.25% fibre had the least toughness.

The above phenomenon could be explained by suggesting that there is a better alignment of fibres with a certain critical fibre length. Beyond the critical length, any increase in fibre length, or fibre aspect ratio, would worsen fibre-fibre interactions thus reducing toughness, strength and modulus. On the other hand, fibres with fibre aspect ratio of 75 become mineralised earlier as explained in page 91.

In short, the addition of coconut fibre to concrete enhanced the toughness, torsion and to some extent the tensile strength of the concrete. The increase in toughness, torsion and the tensile strength could be attributed to the fact that, the fibre presence in the concrete contributed greatly in offering restrain to early twist or strain in the concrete unlike the rock wool the coconut fibres suffered no harm in the alkaline pore water in the concrete, hence, much energy was needed to debond and stretch the fibres, and hence, higher concrete toughness.

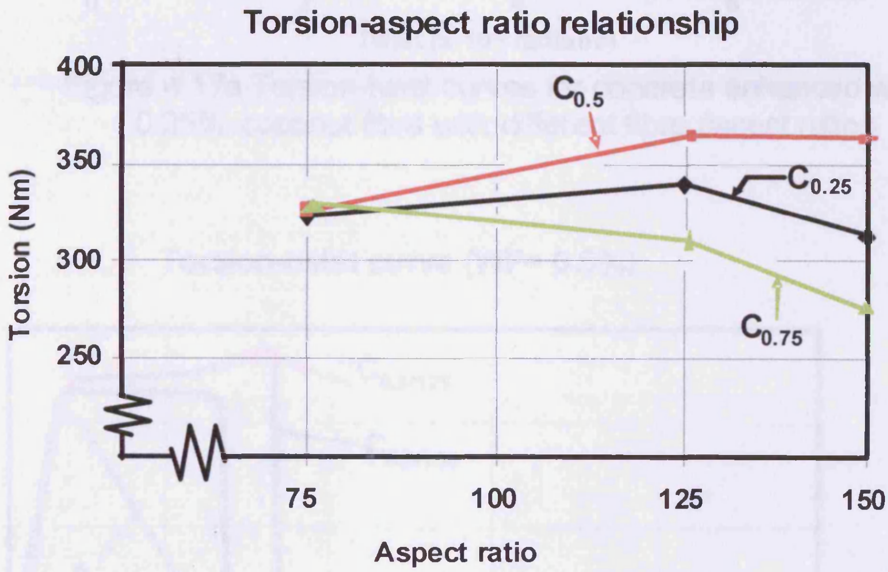


Figure 4.16 Torsion with respect to aspect ratio for different coconut fibre content

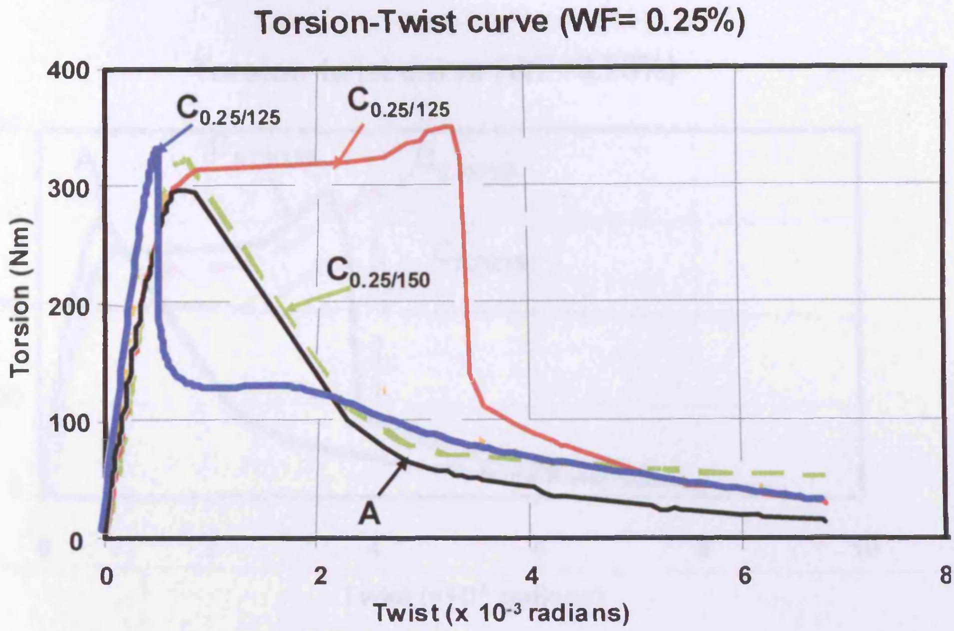


Figure 4.17a Torsion-twist curves for concrete enhanced with 0.25% coconut fibre with different fibre aspect ratios

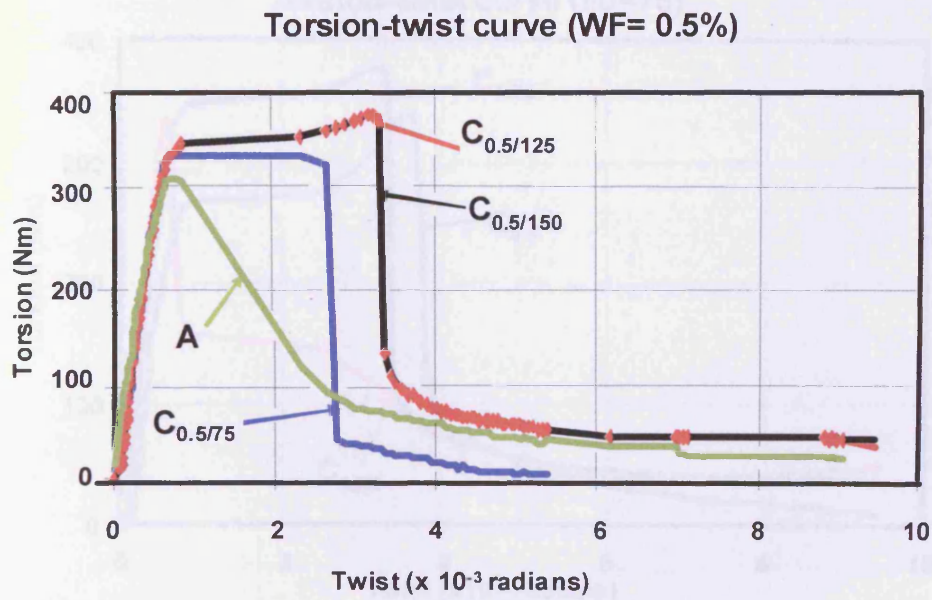


Figure 4.17b Torsion-twist curves for concrete enhanced with 0.5% coconut fibre with fibre different aspect ratios

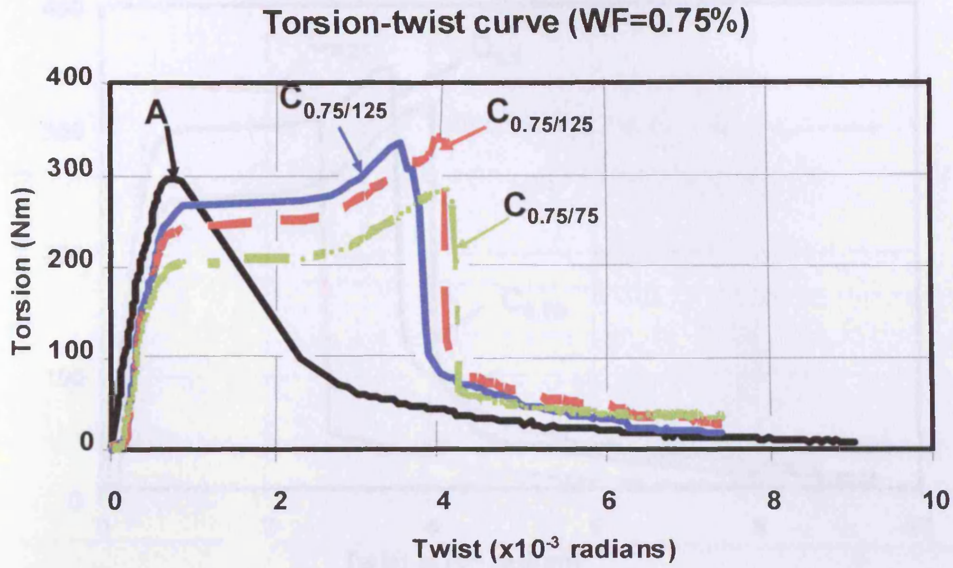


Figure 4.17c Torsion-twist curves for concrete enhanced with 0.75% coconut fibre with different aspect ratios

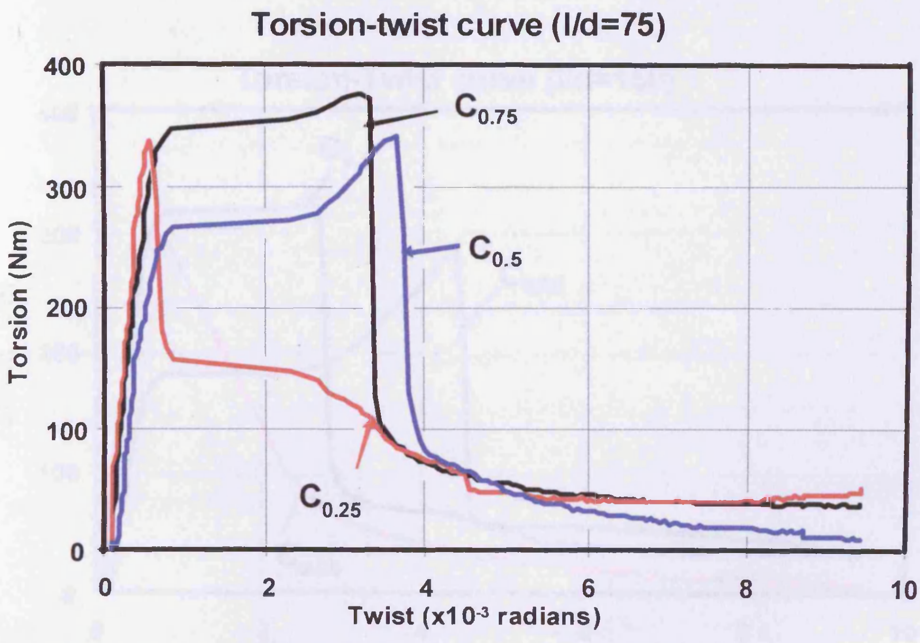


Figure 4.18a.Torsion-twist curve for concrete with different coconut fibre content with aspect ratio of 75

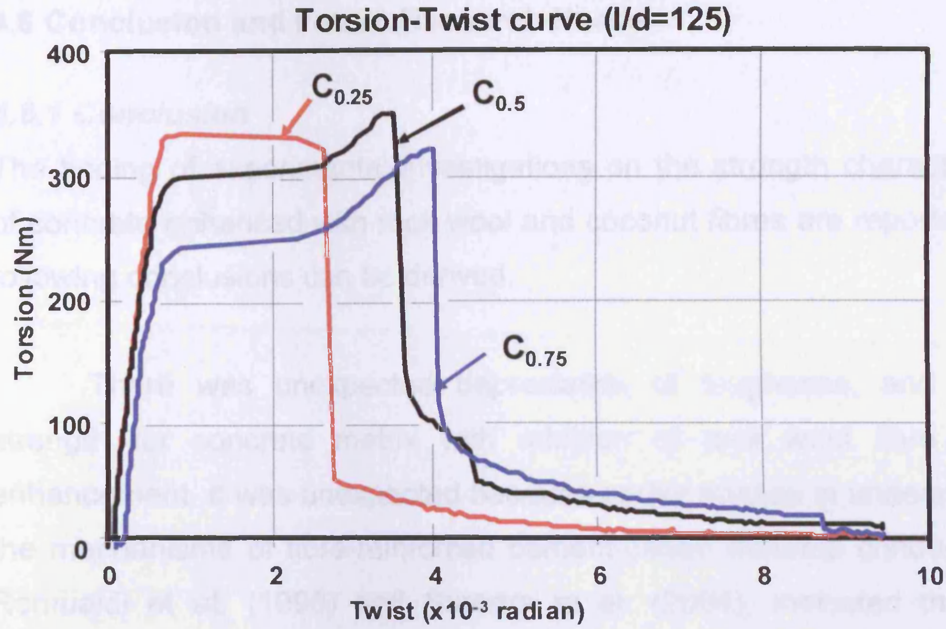


Figure 4.18b Torque-twist curve for specimens with different fibre content aspect ratio of 125

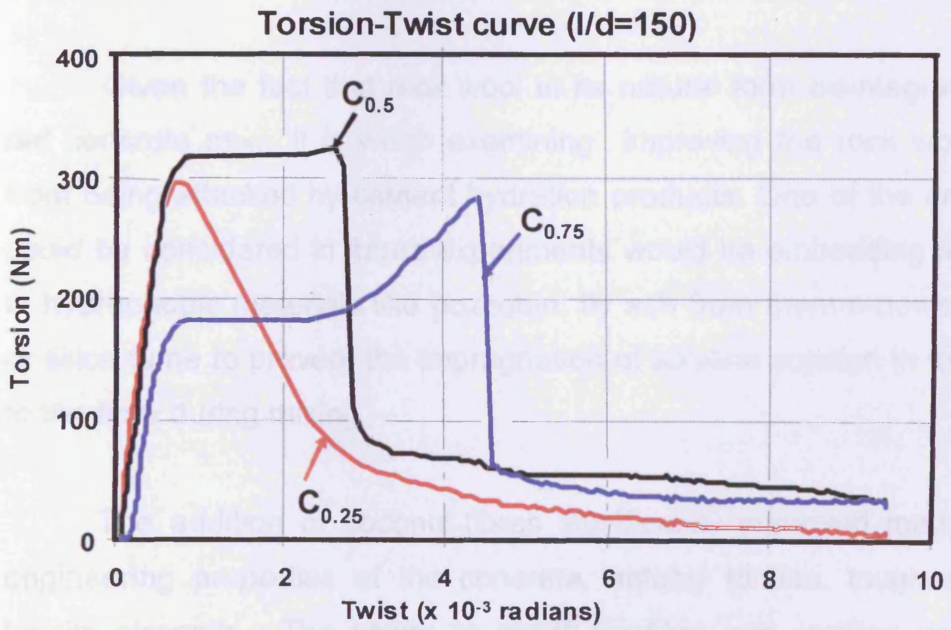


Figure 4.18c Torque-twist curve for specimens with different weight fraction witch aspect ratio of 150

4.6 Conclusion and Future Research Needs

4.6.1 Conclusion

The finding of experimental investigations on the strength characteristics of concrete enhanced with rock wool and coconut fibres are reported. The following conclusions can be derived.

There was unexpected depreciation of toughness, and tensile strength for concrete matrix with addition of rock wool fibre as an enhancement. It was unexpected because earlier studies in understanding the mechanisms of fibre-reinforced cement-based material conducted by Romualdi et at. (1996) and Suredra et al. (2004), indicated that fibre addition improves the tensile strength and toughness of the concrete. Although they used steel fibres which are clearly stronger and stiffer than rock wool fibres, it was nonetheless thought that use of rock wool fibres would not actually lead to deterioration of the properties.

Given the fact that rock wool in its natural form disintegrate in the net concrete mix, it is worth examining improving the rock wool fibres from being attacked by cement hydration products. One of the areas that could be considered in future experiments would be embedding the fibres in hydrophobic materials like pozzolan, fly ash from thermo-power plants, or silica fume to prevent the impregnation of alkaline solution in the matrix to the fibre during mixing.

The addition of coconut-fibres significantly improved many of the engineering properties of the concrete, notably torsion, toughness and tensile strength. The ability to resist cracking and spalling were also enhanced. However, the addition of fibres adversely affected the compressive strength, as expected, due to difficulties in compaction which consequently led to increase of voids.

When coconut fibre was added to plain concrete, the torsional strength increased (by up to about 25%) as well as the energy-absorbing capacity, but there is an optimum weight fraction (0.5% by weight of cement) beyond which the torsional strength started to decrease again.

Similar results were also obtained for different fibre aspect ratios, where again results showed there was an optimum aspect ratio (125). An increase in fibre weight fraction provided a consistent increase in ductility up to the optimum content (0.5%) with corresponding fibre aspect ratio of 125.

Despite its excellent properties, coconut fibre as an enhancement of concrete is unlikely to replace steel for the vast majority of structures. Experiments and demonstration projects around the world have shown that natural fibre enhancement is a viable and cost effective alternative to conventional building materials. However, the construction industry is extremely conservative, and so the most likely development route is the use of the new materials in non-structural applications or in ones where the consequences of failure are not too severe.

Previous researchers like Gram (1983), Le Huu Do et al. (1995) Romildo et al. (2000) Savastano (2000) and Ramkrisha et al. (2004) have identified the following disadvantages in using natural fibres in cement based composite:

a) high water absorption of natural fibre causes unstable volume and low cohesion between fibre and matrix; and
b) natural fibre decomposes rapidly in the alkaline environment of cement and concrete.

Based on the above disadvantages future work on coconut fibre-enhanced concrete and mortar should concentrate on minimising the impact of these disadvantages.

Given the variety of fibre materials, the number of mix constituent and method of production, it is evident that product development should be the prime future research objective. Economic methods of natural fibre extraction, handling, and economical and automated methods of dispersing fibres at a batching plant is needed if large quantities of fibres are going to be used in construction.

Applications for coconut fibre enhanced concrete and mortar composite for housing need to be expanded. Since cement-based materials are well known insulators, another avenue for future research and product development would be the use of coconut fibre-cement composites for sound and heat insulation. Such products might be composed wholly of fibre-cement or use the fibre-cement as one component in an insulating member. It must be acknowledged that aerated concrete would be better, cheaper and easier than the proposed coconut fibre composite insulator, however it could be used as replacement where aerated concrete might not be available or comparatively expensive to produce.

CHAPTER FIVE

STABILISATION OF SOIL BLOCK BY CEMENT

5.0 Introduction

Various natural building materials (e.g. wood, straw, bamboo, soil) are used in developing countries for the building of what is often considered as sub-standard housing of a temporary nature. The same view applies to low-cost urban housing built with a large variety of waste materials, such as scrap metal, cardboard, and so on. To produce materials of permanent nature, cement stabilised soil blocks should be of good-quality blocks, durable and as protective (against hot or cold weather, rain, wind, etc.) as building materials such as concrete blocks, fired bricks or building stones jointed with cement-based mortars. It is necessary, however, to choose carefully the raw materials for the manufacturing of stabilised soil blocks, to apply standard and appropriate soil preparation and to ensure that stabilised soil blocks produced are properly cured. The production of good-quality blocks also requires careful testing of the raw materials, especially soil, as well as testing of the output in order to ensure that blocks of the quality standard would be marketed.

This chapter focuses on experiments conducted on the chemical stabilisation of soil block by cement. Another purpose of this chapter was to identify input variables, like optimum moisture content, optimum cement content and to use the results to predict the input variables for the production of thermoplastic carton soil block stabilised with oil palm and plastics fibres.

5.1 Classification of material and test methods

Typical top soil from the region of Cardiff was used. The soil was firstly passed through a 20 mm sieve before being characterised for its grading curve and consistency limits.

5.1.1 *Determination* of particle size

Sieve analysis test was used to determine the mass proportion of various particle size and hence the soil type. The jar test was used as a confirmation test of the soil type.

a) Sieve analysis

Particle size analysis expresses quantitatively the proportions by mass of the various sizes of particle present in the soil. Particle size is usually given in terms of equivalent particle diameter:

Coarse gravel	: particles from 60mm to 20mm
Medium gravel	: particles from 20mm to 6mm
Fine gravel	: particles from 6mm to 2mm
Coarse sand	: particles from 2mm to 0.6mm
Medium sand	: particles from 0.6mm to 0.2mm
Fine sand	: particles from 0.2mm to 0.06mm
Silt	: particles from 0.06mm to 0.002mm
Clay	: particles smaller than 0.002mm
Fine	: particle which passes a 63µm sieve

(Head, 1992, Kezdi, 1980 and BS1377-1:1990 clause 2.2.22).

A three kilogram of soil sample was dried in an oven, at a constant temperature of 110°C for 24 hours (clause 7.3.4 of BS1377-1: 1990). Soil particles were crumbled between the palms of the hands so that when the particles were sieved on the specific test sieve only individual particles were retained. The soil was sieved using a standard stack of test sieves with aperture sizes of 37.5, 20, 10, 5, 4, 2.36, 1.18, 0.6, 0.3 and 0.15 mm. The results of the particle distribution test are presented graphically in Section 5.4.1 and the experimental raw data are recorded in Table B1 in Appendix B.

b) Jar test

The jar test was used to determine the soil type (clause 8.2 of BS 1377-2:1990). An empty clean glass bottle was filled with the sample of soil to about two-thirds full as shown in Figure 5.1. The jar with soil was then filled with clean water leaving about 50mm of air space. The lid was then screwed tightly and the jar shaken vigorously for two minutes until all the soil particles were broken into suspension. The jar was left for another one minute whiles the suspended soil begun to settle. A mark was made on the side of the jar at the top of the layer that had settled out. The jar with the contents was left for one hour and another mark was made on the side of the jar at the top of the next layer that had settled out. After 24 hours the top of the final layer that had settled out was marked. The results of the jar test are recorded in Table 5.2 showing the composition of soil (soil type and their quantity in percentage) and the analysis is discussed in Section 5.4.1.

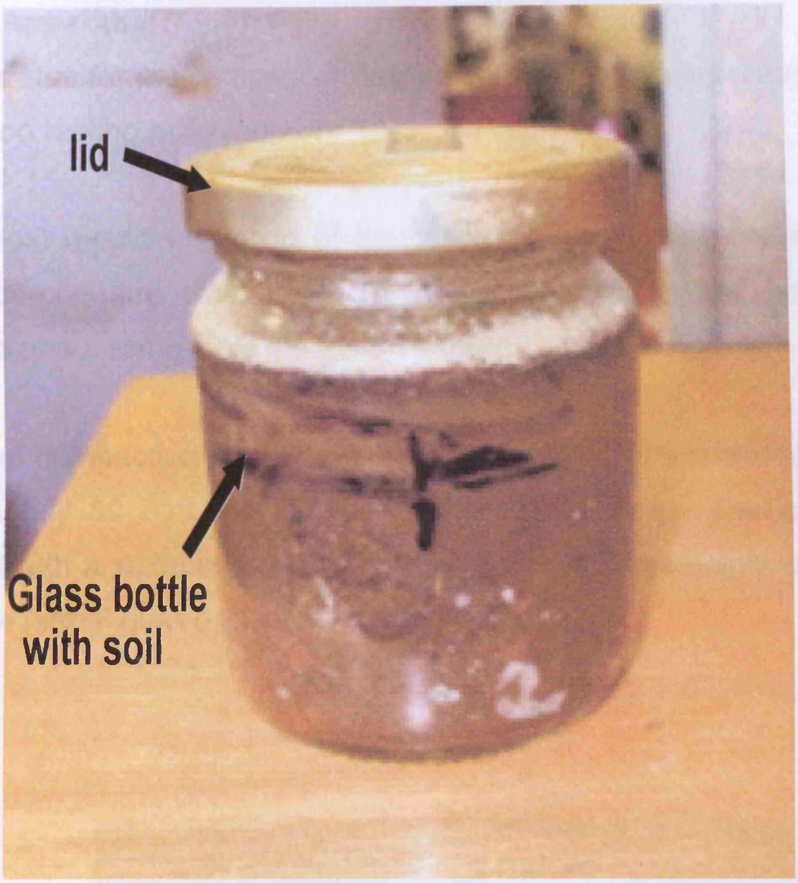


Figure 5.1 Jar test set up

5.1.2 Compaction test

The objective of this test was to obtain relationship between compacted dry density and soil moisture content. Compaction of soil is the process by which the soil particles are packed more closely together, usually by mechanical means, thereby increasing the dry density of the soil.

This test covered the determination of dry density of soil passing through 20 mm test sieve when compacted over a range of moisture contents. The range included optimum moisture content at which the maximum dry density for this degree of compaction was obtained. In this test a 2.5 kg rammer falling through a height of 300 mm was used to compact the soil in three layers into a one-litre compaction mould.

a) Main Apparatus

The apparatus for the compaction test consist of a cylindrical mould and compaction testing machine.

A cylindrical mould made up of corrosion-resistance metal with internal volume of one litre was used. The mould was fitted with detachable baseplate and a removable extension (Figure 5.2).

The compaction testing machine is made up of a metal rammer of approximately 50 mm in diameter and weighs 2.5 kg. The rammer is equipped with a suitable arrangement for controlling the height of drop up to 300mm (see Figure 5.3).

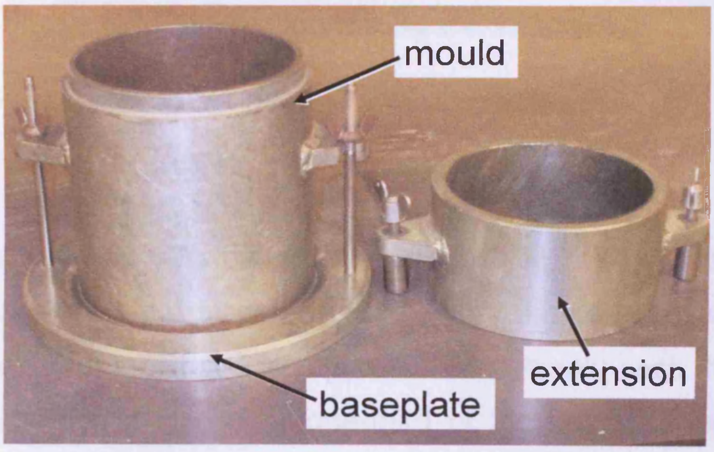


Figure 5.2 Cylindrical mould with baseplate and extension

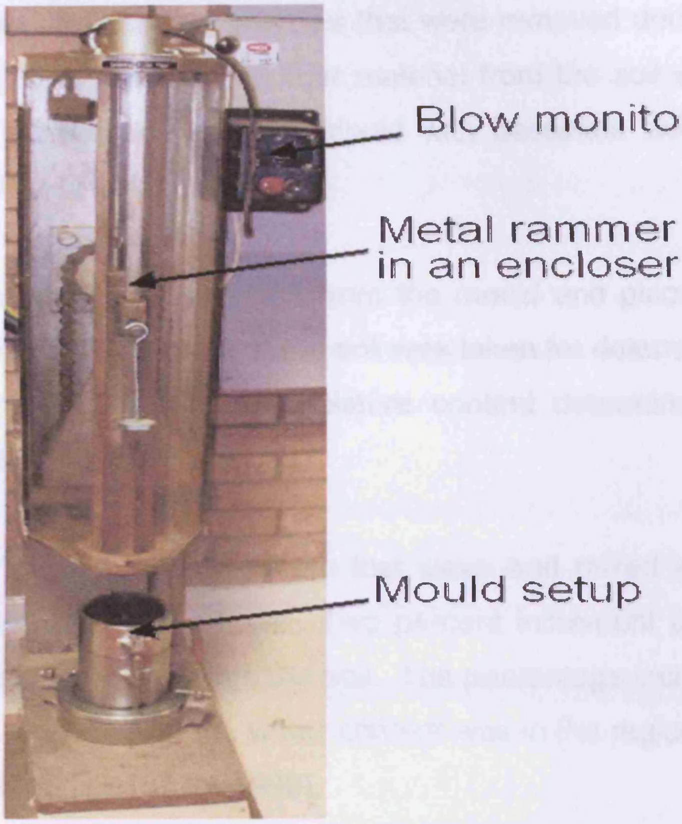


Figure 5.3 Compaction testing machine setup

b) Procedure

The mould was weighed with baseplate attached (M_1). The extension of the mould was attached and placed on the base of the compaction testing machine. A quantity of moist soil was placed in the mould such that when compacted it occupied a little over one-third of the height of the mould body. Twenty seven blows from the rammer (as recommended by BS 1377-4:1990) were dropped from the height of 300 mm above the soil by a motorised means.

The process above was repeated twice more with additional soil, so that the amount of soil used was sufficient to fill the mould body with the surface not more than 6 mm proud of the upper edge of the mould. The extension of the mould was removed and the excess soil was stroke off and the surface of the compacted soil was levelled to the top of the mould using a straight edge. Any coarse particles that were removed during the process of levelling was replaced by finer material from the soil sample and well pressed in. The soil and the mould with baseplate was then weighed (M_2) (see Figure 5.4).

The compacted soil was removed from the mould and placed in a metal tray. A representative sample of the soil was taken for determination of moisture content. The method of moisture content determination is provided in Appendix B, Table B2.

The soil was rubbed through 20mm test sieve and mixed with the remainder of the prepared test sample. Two percent increment of water was added and mixed thoroughly into the soil. The percentage increase of water was reduced to one when the water content was in the region of the optimum water content (BS 1377-4:1990).

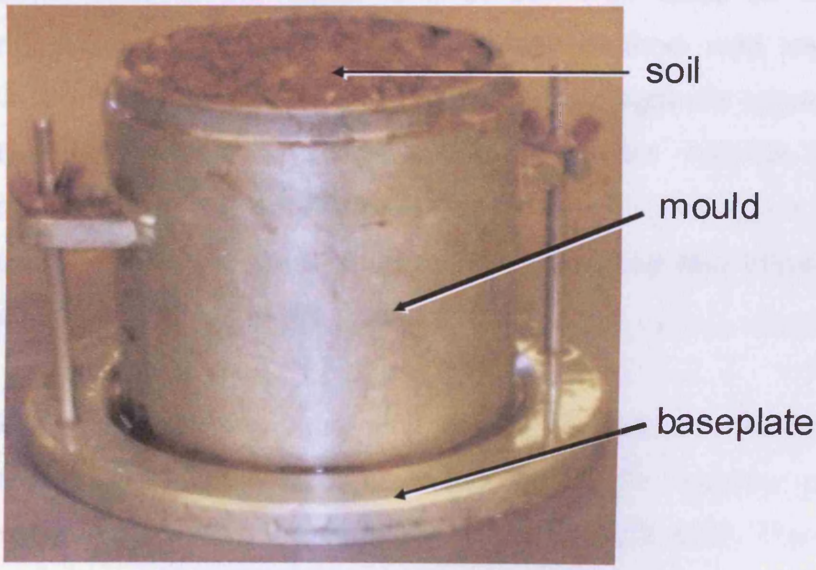


Figure 5.4 Mould with soil after compaction

The steps above were repeated to determine the moisture content of soil with 2%, 3%, 4% and 5% cement content by weight of soil. Past research on cement stabilised soil indicated that cement content less than 2% does not enhance compressive strength of block or improve soil stabilisation (Rigassi, 1995). Cement content above 5% is not deemed (according to the present researcher's opinion) economically attractive, especially when the purpose of this research is to produce blocks for low cost housing. Subsequently cement contents between 2% and 5% were selected for this experimental study.

Dry density and optimum moisture content were determined using samples of the five type of soil, i.e. soil with 0% 2%, 3%, 4% and 5% cement (by weight of soil) as stabiliser. Summary of the experimental results is recorded in Table 5.4.

5.1.3 Determination of liquid limit

The determination of the liquid limit of soil was used as means of classifying the soil. The cone penetrometer method was used. This method is preferred to that of employing the Casagrande apparatus, as this test is both easier to carry out and is more capable of giving reproducible results. The penetrometer apparatus (Figure 5.5) is easier to maintain in correct adjustment and the test procedure less dependent on the subjective judgement of the operator.

Determination of the liquid limit was carried out on soil according to procedures in BS 1377-2: 1990. The liquid limit is the moisture content at which a soil changes from the liquid state to the plastic state. The purpose of determining plastic limit was to know the type and the properties of the soil to be used for the blocks. It is also the moisture content corresponding to a penetration of 20mm by a standard cone in the cone penetrometer test.

Approximately 500g of soil in the natural state was taken from soil sample that had passed through the 425 μm test sieve. The soil was placed on a flat glass plate and was mixed thoroughly with distilled water using two palette knives until the mass became a thick and homogeneous paste. The paste was placed in an air tight container and allowed to stand for 24 hours to enable the water to permeate through the soil.

A portion of the mix was forced into a cup with a palette knife to make sure air was not trapped. Excess soil was stroke off with the straight edge to give a smooth level surface. The soil sample was then placed on the cone penetrometer apparatus for testing.

The supporting assembly of the penetration cone in the raised position was lowered so that the tip of the cone just touched the surface of the soil. The stem of the dial gauge was lowered to contact the cone shaft and the initial reading of the dial gauge was recorded to the nearest 0.1 mm. The cone was then released for five seconds and the reading on the dial gauge was recorded in Table B10 of Appendix B.

The cone was then lifted and thoroughly clean to avoid scratching. A little more wet soil was added to the cup, taking care that air was not trapped and the surface was smoothened and the experiment was repeated. The average of the two readings of cone penetration was recorded. About 10g of the soil within the area penetrated by the cone was taken and the moisture content determined according to Section 3.2 of BS1377-2:1990.

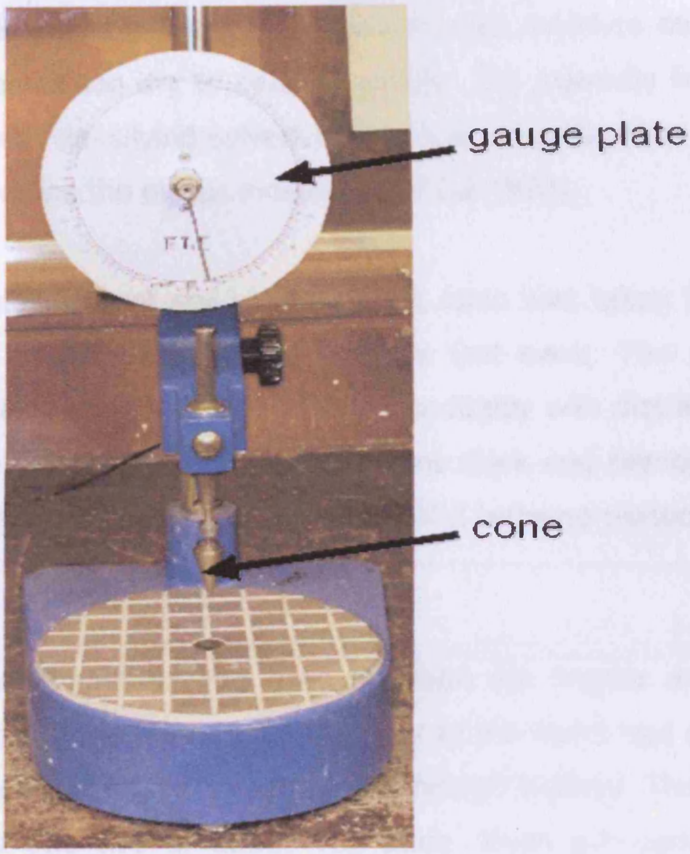


Figure 5.5 Penetration cone for liquid limit test

The test was repeated three more times using the same sample of soil to which further increment of distilled water was added (clause 4.3.3.10 of BS1377-2:1990). The amount of distilled water added was such that the range of the penetration of the cone was between 15 mm to 25 mm for the four tests conducted.

The glass plate was always covered with damp cloth at any time the soil sample had to be left for a while during test, to prevent the soil drying out. Summary of the values of the liquid and plastic limits are recorded in Table 5.4 and the details of the result are recorded in Table B10 in the Appendix B. Figure 5.5 shows the relationship between the penetration of the cone into the soil sample and the moisture content.

5.1.4 Determination of plastic limit and plasticity index

The plastic limit is the empirically established lowest moisture content at which a soil becomes too dry to become plastic. The plasticity limit (W_p) provides a means of classifying cohesive soils. It is used, together with the liquid limit to determine the plastic index (BS1377-2:1990).

Approximately 20g of soil in the natural state was taken from soil sample that had passed through the 425 μm test sieve. The soil was placed in a flat glass plate and was mixed thoroughly with distilled water using two palette knives until the mass became thick and homogeneous paste. The soil was allowed to dry partially until it became plastic enough to be shaped into a ball.

The soil was moulded into balls between the fingers and rolled between the palms of the hands until the heat of the hand had dried the soil sufficiently for slight cracks to appear on the soil surface. The sample was then divided into two of about 10g each. Each sub-sample was divided into four more equal parts and each part was moulded in the finger to ensure even distribution of moisture in the soil sample, (clause 5.3.3.4 to 5.3.3.8 of BS 1377-2:1990). They were then formed into threads of about 6 mm long. Using sufficient pressure, the samples were rolled along a glass plate until they were approximately 3 mm in diameter. The moisture contents of each of the sample was then determined and the average was recorded. Details of the experimental records are found in Table B11 in the Appendix B.

5.1.5 Determination of linear shrinkage

Shrinkage due to drying is significant in clays but less so in silt and sand. This test enables the shrinkage limit, W_s, of clay to be determined i.e. the moisture content below which clay ceases to shrink.

The determination of linear shrinkage was carried out on 500g of soil that has passed through the 425µm test sieve in accordance with Section 6.5 of BS1377 Part 2. The soil was mixed thoroughly with distilled water using palette knives till the mass became a smooth homogeneous paste with the moisture content at about the liquid limit of the soil. Four brass moulds were smeared with silicon grease and soil paste was added to each, ensuring that there was no air bubble present. The surfaces were smoothened using a straight edge. The moulds were then placed on a table where the wet soil could air dry slowly until the soil had shrunk away from the walls of the moulds. The drying was completed by placing the sample in an oven, first at a temperature 60°C until shrinkage had largely ceased, and then at a temperature of 105°C for 24 hours to complete the drying. Shrinkage was considered complete when three successive measurements showed no change in length. Details of the measurements are recorded in Table 5.5.

5.1.6 Organic content

The organic content of soil greatly influenced the strength characteristics of the soil block. The amount of organic material can be determined by ignition. Organic materials are carbon based. In the ignition process, a dry solid sample was heated to a high temperature (of about 140°C) until the organic materials in the soil sample had been given off as gases. This resulted in a weight change which allowed for calculation of the organic content of the sample.

In this study, a sample of the soil was oven-dried to remove any water. An evaporating dish and cover was weighed. A sample of the oven-dried soil of approximately 10 gram was placed in the container and covered. The sample, container and cover were weighed together. The container with the sample was heated on a gas stove which resulted in fume generation. The heating continued until there was no visible fume.

The container with lid and soil sample was weighed again and the percentage of organic sample was thus calculated.

5.2 Shear strength test (effective stress) of soil used

In order to determine other mechanical characteristics of the soil used, a triaxial test was conducted. The following mechanical properties were determined from the triaxial test: maximum shear stress, modulus of elasticity, Poisson's ratio, coefficient of cohesion, angle of shear resistance and shear modulus. These mechanical properties are of particular importance for numerical modelling which would be treated in the next chapter.

The triaxial test was used to measure the shear strength of a soil used under controlled drainage conditions. In this triaxial test, a cylindrical specimen of soil encased in a rubber membrane (the impermeable membrane provided protection to the sample against leakage from the cell fluid) was placed in a triaxial compression chamber, subjected to a confining fluid with two pressure cells, and then loaded axially to failure. Connections at the ends of the specimen permitted controlled drainage of pore water from the specimen. The three principal stresses were measured and recorded in Tables B12, B13, and B14 in Appendix B.

5.2.1 Type of test performed

The consolidated-undrained (CU) test was conducted. This test gives undrained shear strength of a soil specimen subjected to a known initial stress. In this test, drainage or consolidation was not allowed to take place during the application of the confining cell pressure σ_3. Loading did not commence until the sample ceased to drain (i.e. consolidated). The axial load was then applied to the specimen, with no attempt made to control the building of excess pore pressure. For this test, the draining valve was closed during axial loading, and the excess pore pressures were measured.

The consolidated-undrained test was used due to the following advantages over the consolidated-drained test. In the CU application of the confining pressure caused changes the pore pressure with no change in the volume and the void ratio and a shorter time period is needed for consolidation and dissipation of excess pore pressure developed during compression (Head, 1986). In the consolidated-drained test, volume change is observed with no change in pore pressure.

With the exception of shorter time period needed for consolidation and dissipation of excess pore pressure developed during compression in the consolidated-undrained test, there is no other particular advantages in consolidated-undrained over consolidated-drained test. However, consolidated-undrained test was selected because the constant volume and void ratio are of particular interest to this research, (in the next chapter), there would be experimental studies on soils encased in a plastic carton. The plastic carton would allow a comparatively small volume change and negligible void ratio change in the soil. Having knowledge of the stress of the soil with constant volume and void ratio would thus facilitate better computation of the performance of the plastic carton soil block under stress.

5.2.2 Preparation of sample

Compacted samples with moisture content of 10% were forced into a metal tube of diameter 38 mm using a compression test machine (see Figure 5.6), and three test specimens were taken for the test.

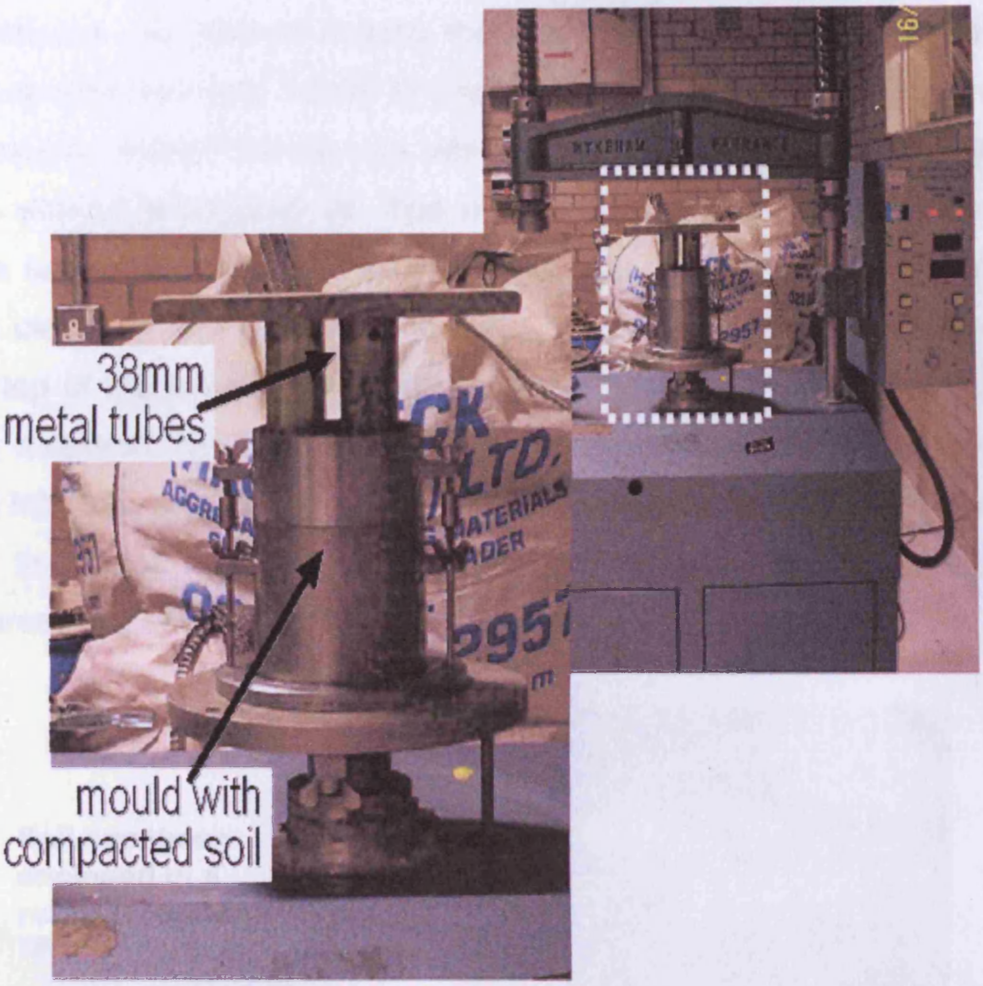


Figure 5.6 Compression test machine used to compressed soil sample into mould

5.2.3 Mounting of the test specimen

The procedure for mounting of triaxial test specimen was according to BS 1377-8: 1990 clauses 4.2.2 to 4.2.13. A saturated porous disc was placed on a layer of water on the triaxial base pedestal thus making sure air was not trapped and excess water removed. The specimen was placed on the disc without delay and without trapped air. An identical disc was placed on top of the specimen. After surplus water was allowed to drain from the soaked membrane, the membrane was placed around the specimen. The base pedestal was sealed with two rubber O-rings. The back pressure valve was opened to moisten the top cap, which was then fixed onto the porous disc without entrapping air. The membrane on top of the cap was then sealed with another two O-rings, using a split-ring stretcher. The cell body and loading piston was then assembled well clear of the top of the centrally aligned specimen. The air in the triaxial cell was displaced by filling the cell with de-aerated water. Castor oil was introduced on top of the water to act as a lubricant to the piston and to reduce leakage around the piston. The setup is shown in Figures 5.7a and b.

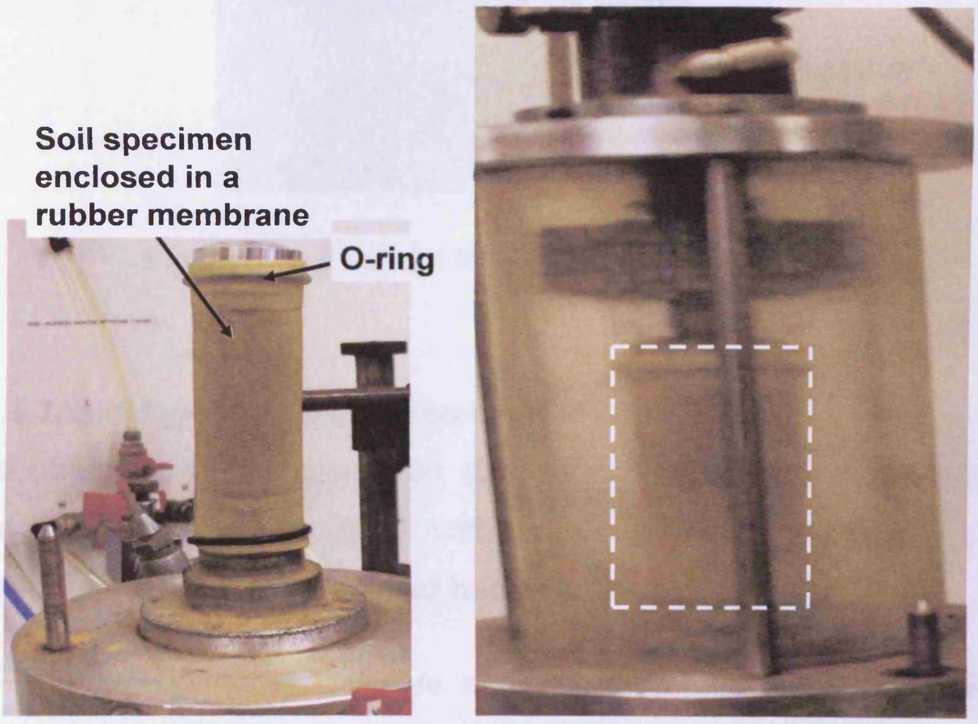


Figure 5.7a Mounting of soil specimen for triaxial test

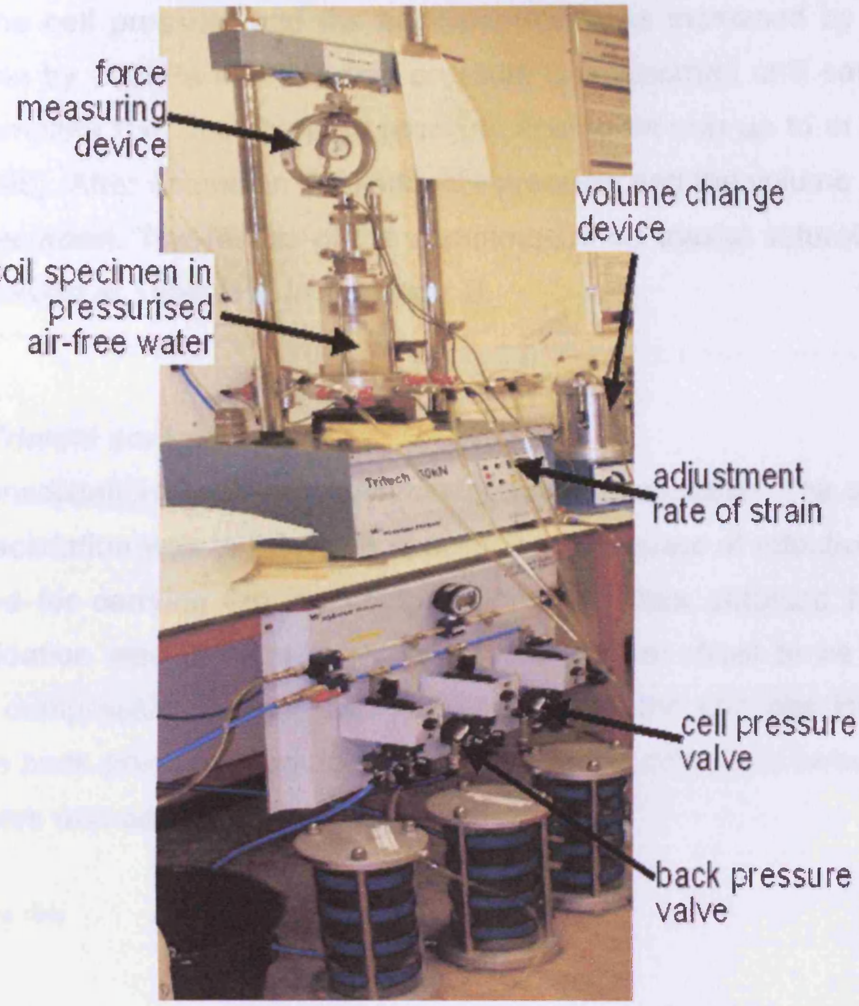


Figure 5.7b Essential features of the triaxial test.

5.2.4 Triaxial under saturation condition

The objective of the saturation state was to ensure that the entire void in the soil was filled with water. This was achieved by increments of cell pressure and back pressure at the same time.

Increments of cell pressure and the back pressure were applied alternatively such that small positive effective stress of 10 kPa was maintained. A differential pressure of 10 kPa has been found to be suitable for many soils (clause 5.2 of BS1377-8:1990). The pore pressure was observed until an equilibrium value was achieved.

The cell pressure and the back pressure was increased by 50kPa and then by 100kPa and the pore pressure was recorded until saturation was complete (i.e. until the pore pressure coefficient was up to or greater than 0.95). After saturation the final pore pressure and the volume change were recorded. The results of the experimental on triaxial saturation test are recorded in Table B12 in Appendix B.

5.2.5 Triaxial consolidation

The consolidation stage follows immediately after saturation. The objective of consolidation was to bring the specimen to the state of effective stress required for carrying out the compression test. Data obtained from the consolidation was used to estimate suitable rate of strain to be applied during compression triaxial test. The pressure in the cell was increased and the back pressure was adjusted such that the difference between the pressures was equal to the required.

$$\sigma'_3 = \sigma_3 - u_b \tag{5.1}$$

where,

σ'_3 = effective consolidation pressure

σ_3 = cell pressure (total minor principal stresses)

u_b = back pressure (pressure applied directly to the pore fluid in the specimen void).

The pore pressures and volume changes were recorded at a suitable time interval as recommended by BS 1377-8:1990, until the final pore pressure, volume change and consolidation were attained. Consolidation was considered to be complete when the coefficient of consolidation was equal to or greater than 95% of the excess pore pressure that had been dissipated. The results of the experimental on triaxial consolidation test are recorded in Table B13 in Appendix B

5.2.6 Consolidated-undrained triaxial compression test with measurement of pore pressure

In this test, the cell pressure was maintained constant during the compression stage, while the soil specimen was sheared at a constant rate of axial deformation until failure occurred. No drainage was permitted and the moisture content remained constant during compression. The rate of axial deformation was low enough to ensure adequate equalisation of excess pore pressure.

In the triaxial compression test on a saturated soil, an excess pore pressure was induced by the increasing deviator stress in accordance with Equation 5.1. The procedure was conducted according to the requirements of BS1377-8:1990. The machine platen was adjusted until the cell loading piston was brought to within a short distance of the specimen top cap. This load was recorded as initial reading. The machine was adjusted to give the rate of displacement equal to the rate of displacement that was calculated when the specimen was subjected to consolidation.

Further adjustment of the force measuring device of the triaxial test apparatus was made to bring the loading piston just into contact with the seating on the top cap of the specimen, and centrally aligned. The axial deformation gauge was set to read zero. The back pressure valve was then closed while the cell and pore pressures valves were opened. All initial readings were directly recorded on a computer (i.e. date and time, deformation gauge reading, force device reading and cell pressure). Compression was applied to the specimen and the test continued until constant shear stress and pore pressure were attained. The results of the experimental on triaxial compression test are recorded in Table B14 in Appendix B.

5.3 Experimentation on cement stabilised soil block

This section describes experimental studies on soil stabilised with cement. While all the data from both preliminary and actual studies are recorded in Section 5.4 and the experimental raw data are recorded in Appendix B.

A few small-sized blocks without cement as stabiliser were produced as a preliminary test to assess the optimum compression pressure that might be required to produce a block of maximum strength. The optimum moisture content of soil in its natural state (not dried in oven) and also the mass of soil required to produce a batch of 15 blocks were also required. A BREPAK earth block press (see figure 5.8) that could deliver pressures of up to 35 MPa for block production was available in the laboratory.

Tests such as dry density and moisture content of the soil in the natural state were first conducted on the soil blocks without stabilisation. Afterwards, chemical stabilisation was investigated by adding 2%, 3%, 4%, and 5% of cement by weight of soil and its effect on the dry density, compressive strengths, abrasive and water absorption coefficients at different moistures contents were analysed after 28-days of air curing.

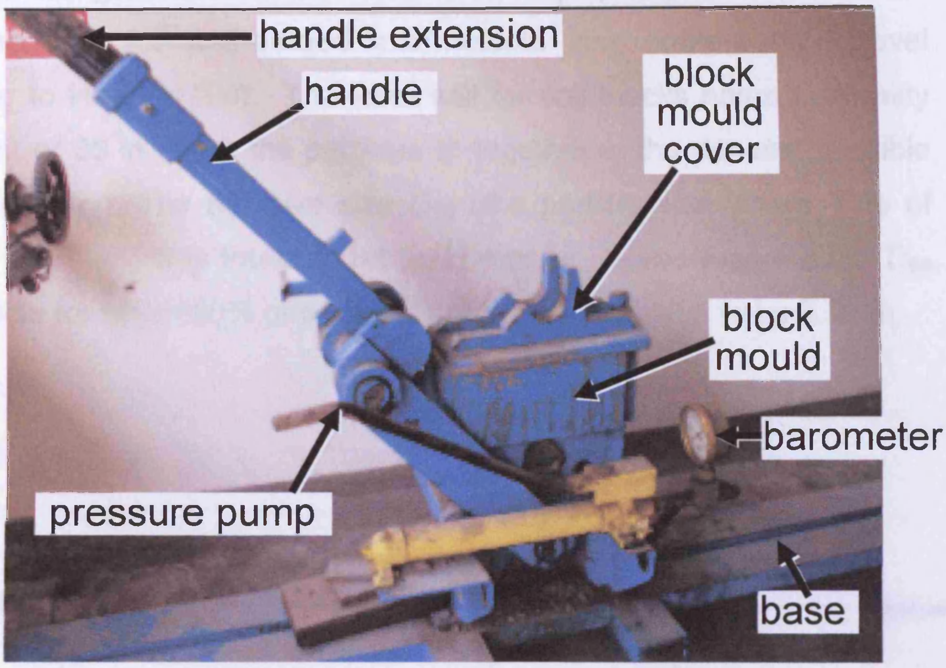


Figure 5.8 BREPAK block making machine

5.4 Experimental results

This section provides the results of the analysis of experimental studies in the previous sections.

5.4.1 Classification of soil

Sieve test analysis, jar test and atterberg limit was used to classify the soil used in this these experiments.

a) Sieve analysis

Table B1 in Appendix B summarises the characteristics of the soil used. It could be seen from Figure 5.9 that the grading curve of the soil used was within the limit for well graded soil but with a small excess of 0.1mm particles and small amount of fine particles. The soil had a coefficient of uniformity and gradation were 20 and 0.56 respectively. This soil is thus within the limit for well-graded intermediate clay content and gravel according to Head (1990). The ideal soil for soil blocks has a uniformity coefficient of 36 in which the particles fit together in the densest possible state of packing. The effective size D_{10} (the particle size where 10% of particles are finer) was found to be 0.125mm (see Figure 5.9). D_{60} (the particle for which 60% of particles are finer) was found to be 2.5mm.

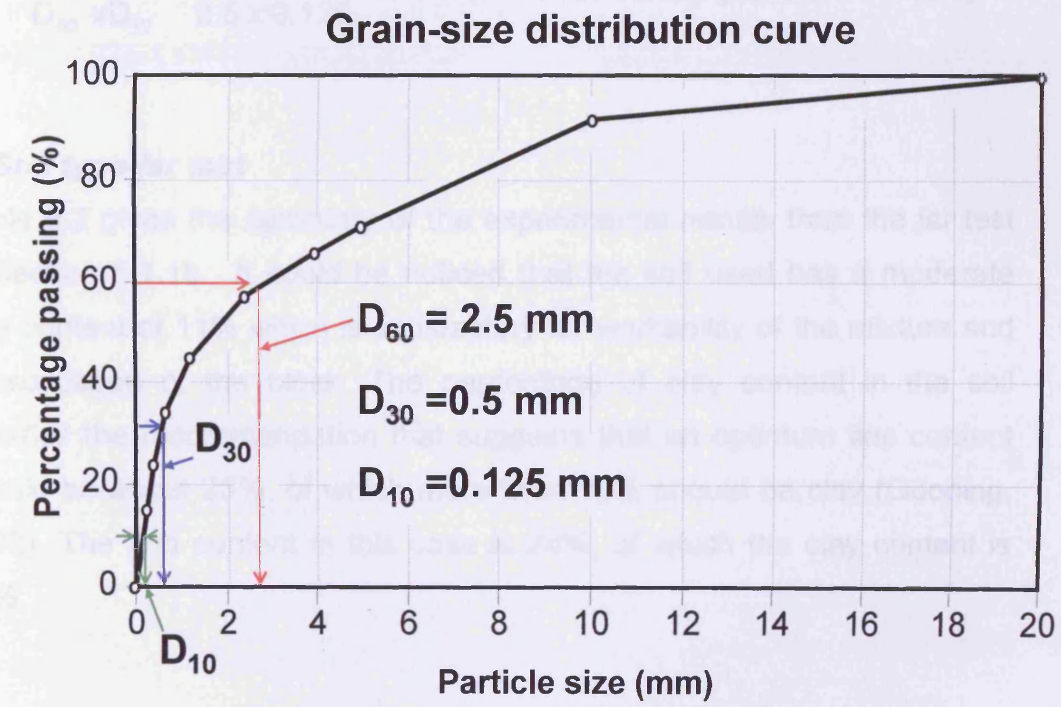


Figure 5.9 Particle size distribution graph

Table 5.1 Soil characteristics

Grading curve	D_{10}	D_{30}	D_{60}	Uniformity coefficient (U)	Description
Figure 5.6	0.125	0.5	2.5	20	Well-graded clay and gravel

Uniformity coefficient, U

$$U = \frac{D_{60}}{D_{10}} = \frac{2.5}{0.125} = 20 \text{ i.e. well graded} \quad (5.2)$$

Coefficient of Gradation or curvature C_g

$$C_g = \frac{(D_{30})^2}{D_{60}\, xD_{10}} = \frac{0.5^2}{2.5 \text{ x } 0.125} = 0.8 < 3 \quad \text{i.e. Uniformly graded} \quad (5.3)$$

b) Soil type-jar test

Table 5.2 gives the summary of the experimental results from the jar test in Section 5.1.1b. It could be noticed that the soil used has a moderate clay content of 11% which is satisfactory for workability of the mixture and consolidation of the block. The percentage of clay content in the soil satisfies the recommendation that suggests that an optimum fine content should be about 25%, of which more than 10% should be clay (Gooding, 1993). The fine content in this case is 26%, of which the clay content is 11%.

Table 5.2 Results from jar test (soil composition)

Soil type	Amount %
Clay	11
Silt	15
Sand/ fine gravel	74

c) Atterberg limits

The Atterberg limit comprises the liquid and plastic limits test, which is used as one of the means of classifying soil.

The liquid limit expresses the moisture content corresponding to a cone penetration of 20mm (BS1377-2:1999). In this investigation the soil used has a liquid limit of 35% (Figure 5.10), plastic limit of 24 (Table B4 in Appendix B) and plasticity index of 11% (Equation 5.4) and hence the soil could be classified as moderately plastic clay according to plasticity chart of BS 5930:1999 (Figure 5.11). Line A is obtained from measurements made on materials having passed through a 425μm test sieve, according to clause 41.4.4.5 of Bs 5930:1999.

Plasticity Index: $I_p = w_L - w_p = 35 - 24 = 11\%$ (5.4)

Table 5.3 Classification for fine soil and finer part coarse soil

Soil sample	Values (%)
Liquid limit	35
Plastic limit	24
Plasticity index	11

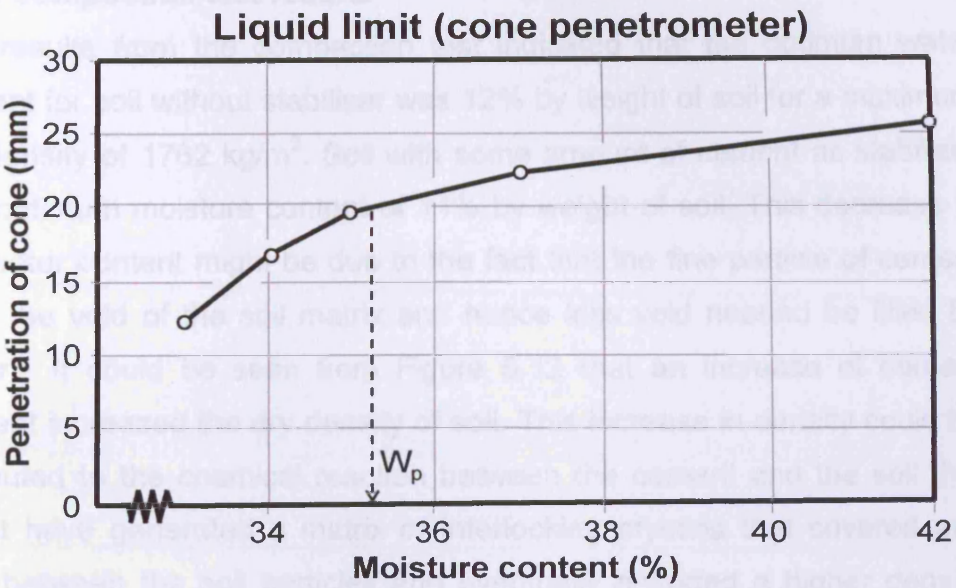


Figure 5.10 Liquid limit for the soil used

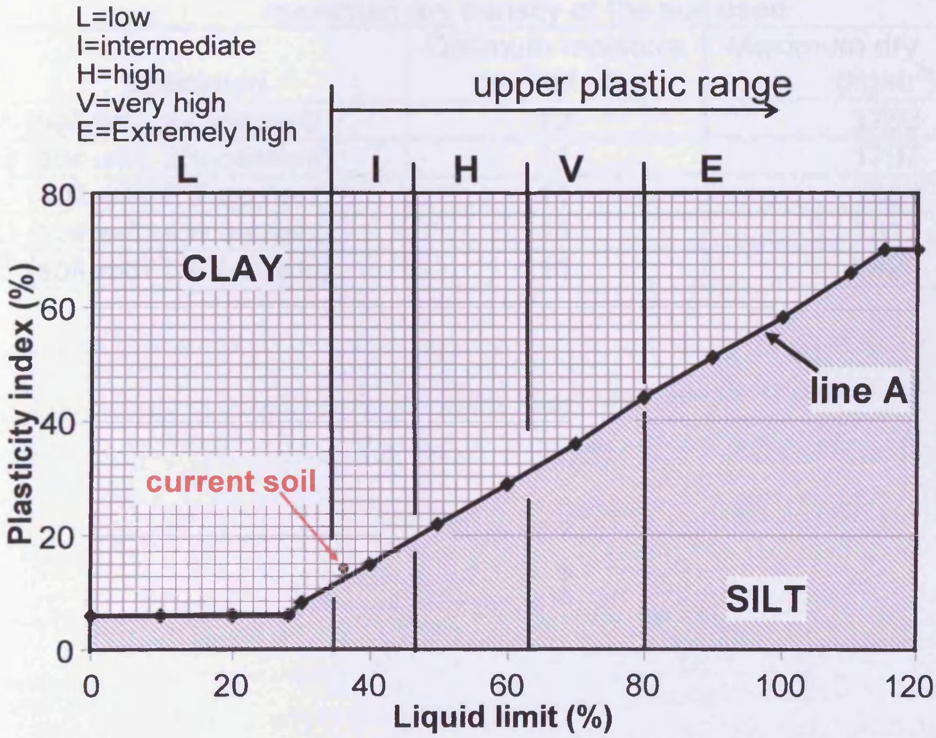


Figure 5.11 Plasticity chart for soil classification

5.4.2 Compaction test results

The results from the compaction test indicated that the optimum water content for soil without stabiliser was 12% by weight of soil for a maximum dry density of 1762 kg/m^3. Soil with some amount of cement as stabiliser had optimum moisture content of 11% by weight of soil. This decrease in the water content might be due to the fact that the fine particle of cement filled the void of the soil matrix and hence less void needed be filled by water. It could be seen from Figure 5.12 that an increase of cement content increased the dry density of soil. This increase in density could be attributed to the chemical reaction between the cement and the soil that might have generated a matrix of interlocking crystals that covered any void between the soil particles and eventually provided a higher density and better stability.

Table 5.4 Results of optimum moisture content and maximum dry density of the soil used

Specimen	Optimum moisture content (%)	Maximum dry density (kg/m^3)
A (soil without cement)	12	1762
B (soil with 2% cement)	11	1797
C (soil with 3% cement)	11	1804
D (soil with 4% cement)	11	1813
E (soil with 5% cement)	11	1820

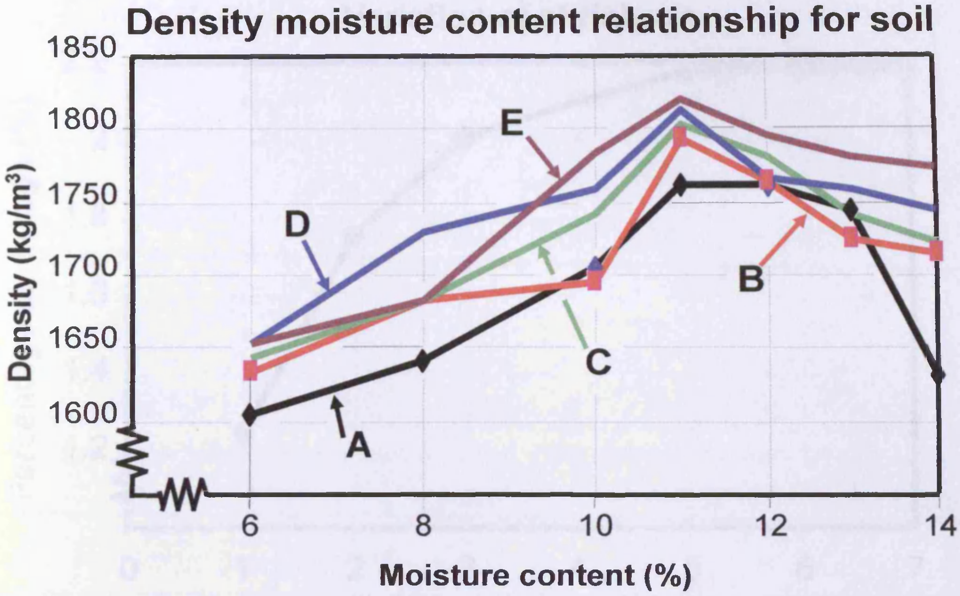


Figure 5.12 Dry density-moisture content relationships for soil used

5.4.3 Linear shrinkage

Shrinkage due to drying is very important in clay and this test enabled the shrinkage limit of clay to be determined. The shrinkage is defined here as the moisture content below which clay ceases to shrink. Figure 5.13 shows the results of variation of shrinkage with time for un-stabilised soil in the natural state from experiment described in Section 5.1.5. From the results, shrinkage increased rapidly during the first three days and then later the increase slowed down. Shrinkage was considered complete when three successive readings showed no change in length of the specimen. The maximum percentage linear shrinkage at age seven days was 2.18mm.

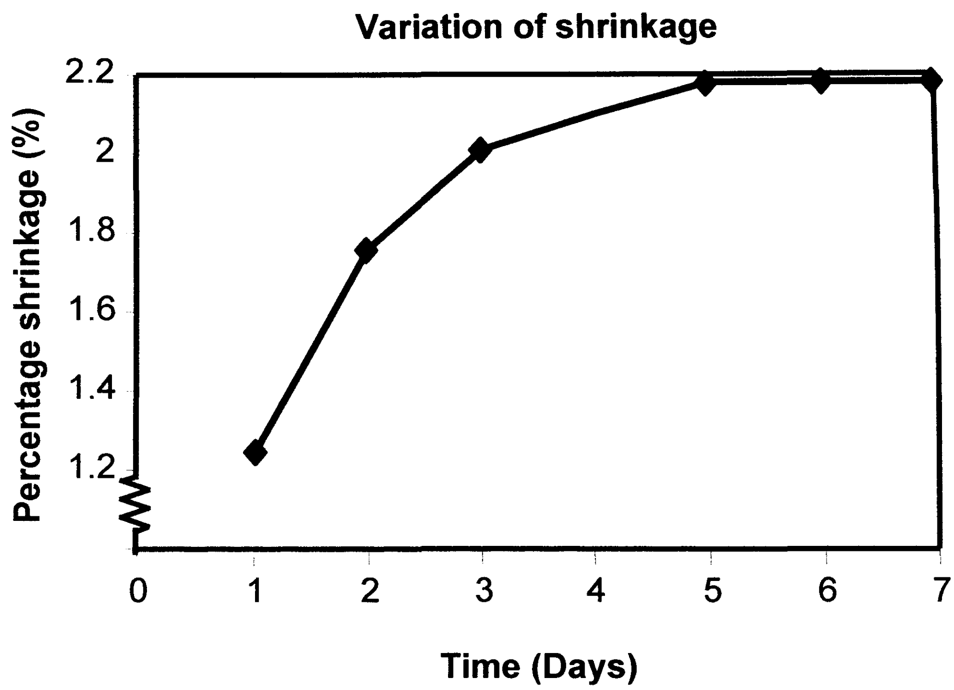


Figure 5.13 Effect of time on the development of shrinkage

The strength of soil block does not depend only on the level of stabilisation but also among other factors on the curing regime. The rapid shrinkage in the first three days draws the attention that particular care should be taken for curing at least the first three day of the moulded block. Hence the blocks were covered with a polythene mat for the first three days of curing and this would be beneficial in reducing drying shrinkage and cracking.

5.4.4 Loss on ignition

The purpose of conducting this test was to find out if the soil would be suitable for the production of soil block irrespective of its excellent clay content. If there is a large amount of organic compound present, then the soil might need special treatment or large amount of chemical stabiliser (cement) to produce soil blocks of satisfactory strength and stability. The organic compound is usually distinguished by a musty smell especially on heating.

A sample of oven-dried soil was heated on a gas stove which resulted in fume generation. The heating continued until there was no visible fume. The weight lost was determined and the percentage of organic sample was thus calculated.

The soil was found to have organic content of 1.9% (Table 5.5). Houben and Guillard (1994) express the view that up to 2% of organic matter does not have a significant influence on the mechanical performance of the soil block.

Table 5.5 Percentage of organic content determination by ignition

Weight readings	Values
Mass of container + lid (m_1)	61.5 g
Mass of container + lid + soil sample (m_2)	71.7 g
Mass of soil sample before ignition $m_3 = m_1 - m_2$	10.2 g
Mass of container + soil sample after ignition (m_4)	71.3 g
Mass of organic materials $m_5 = m_3 - m_4$	0.4 g
Percentage of organic content $\frac{m_5}{m_3} = \frac{0.2}{10.2} x 100$	1.9 %

5.4.5 Results of shear strength test

The optimum aim of conduction the triaxial test was to obtain the maximum stresses from three sets of test samples and use the result to further calculate Modulus of Elasticity E, Cohesion coefficient C′ and Angle of shear resistance Φ from the Mohr's circle diagram. The Poisson's ratio ν, was calculated from dimensions of the soil test specimen before and after the test was completed (see Figure 5.14). The shear modulus was calculated from the modulus of elasticity and Poisson's ratio. The modulus of elasticity, shear modulus and Poisson's ratio would be used in numerical modelling in Chapter Seven.

The calculation necessary for the analysis of data from consolidated-undrained compression triaxial test and the associated graphical plotting are described in this section. Raw data from the test are recorded in Tables B12–B14 in Appendix B.

Table 5.6 Data of consolidated-undrained (CU) triaxial test

Soil description : dark brown medium clay soil				
Type of specimen : remoulded and compacted				
Specimen dimensions : 38 mm diameter ; 77 mm long				
Specimen		A	B	C
Saturation stage	Initial pore pressure (kPa)	6.10	0.00	1.80
	Saturated pore pressure (kPa)	184.30	285.17	91.12
	Final cell pressure (kPa)	400.00	400.00	200.00
	B- coefficient of saturation	0.99	0.98	0.98
Consolidation Stage	Cell pressure (kPa)	400.00	510.00	620.00
	Back pressure (kPa)	290.00	290.00	290.00
	Initial pore pressure (kPa)	277.00	169.00	58.5.00
	Final pore pressure (kPa)	264.00	167.00	57.00
Compression Stage	Cell pressure (kPa)	400.00	510.00	620.00
	Initial pore pressure (kPa)	250.00	168.00	52.00
	Initial effective stress (kPa)	110.00	220.00	330.00
	Rate of strain (mm/min)	0.10	0.10	0.10
Failure conditions	Strain failure condition %	6.70	5.20	6.80
	Deviator stress (σ_1-σ_3) (kPa)	272.00	270.00	668.00
	σ_1 (kPa)	672.00	780.00	1278.00
	σ'_1 (kPa)	325.00	470.00	1199.00
	σ'_3 (kPa)	83.00	155.00	580.00
Shear stress Parameter	Cohesion coefficient C'			40 kPa
	Angle of shear resistance Φ			21°
	Elastic modulus E (MPa)			10.6
	Poisson ratio ν			0.33

Modulus of elasticity, $E = \dfrac{\Delta\sigma}{\Delta\varepsilon} = \dfrac{700 - 30}{6.4 - 0.1} - \dfrac{670}{6.3} x1/10 = 10.6\,\text{MPa}$ (5.5)

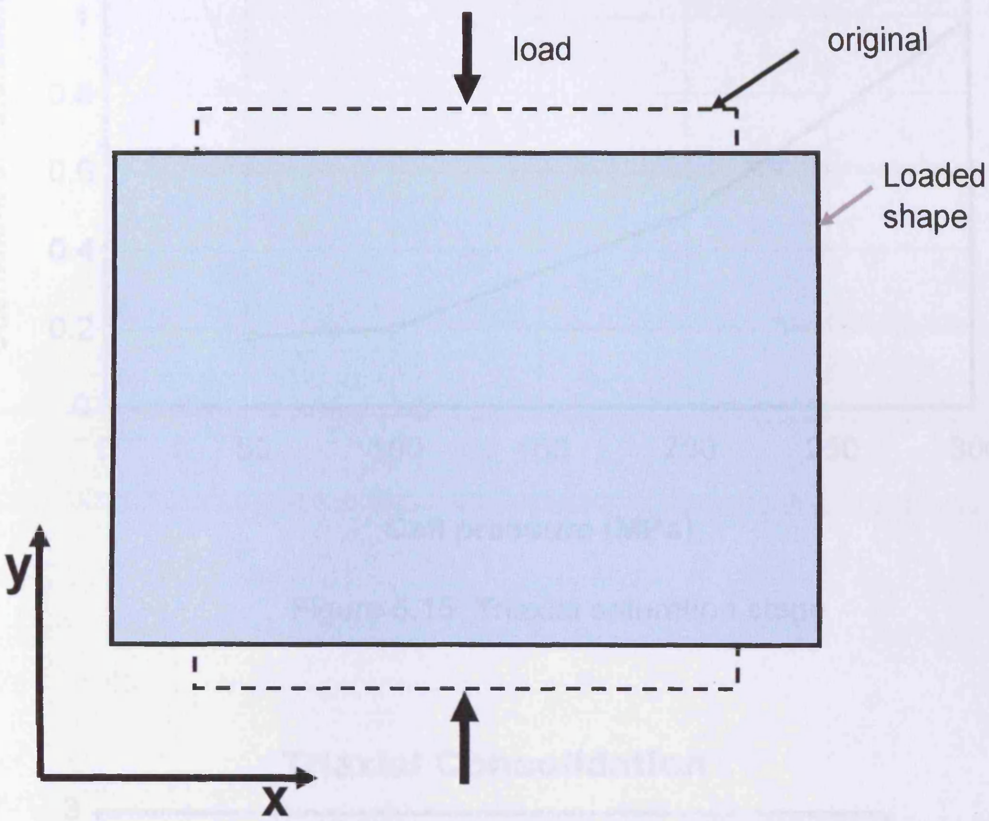


Figure 5.14 Sketch of specimen for calculation of Poisson ratio

Poisson ratio $v = -\dfrac{\varepsilon_x}{\varepsilon_y} = \dfrac{0.026}{0.061} = 0.33$ (5.6)

Shear modulus $G = \dfrac{E}{2(1+\upsilon)} = \dfrac{10.6}{2(1+0.33)} = 3.98 \approx 4\,\text{MPa}$ (5.7)

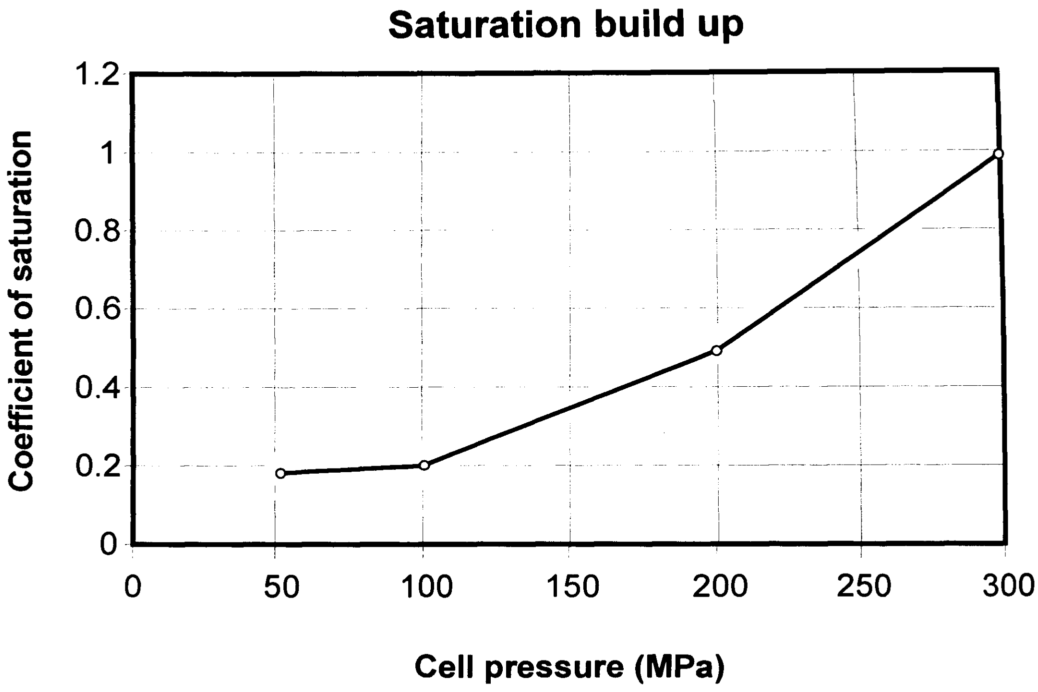


Figure 5.15 Triaxial saturation stage

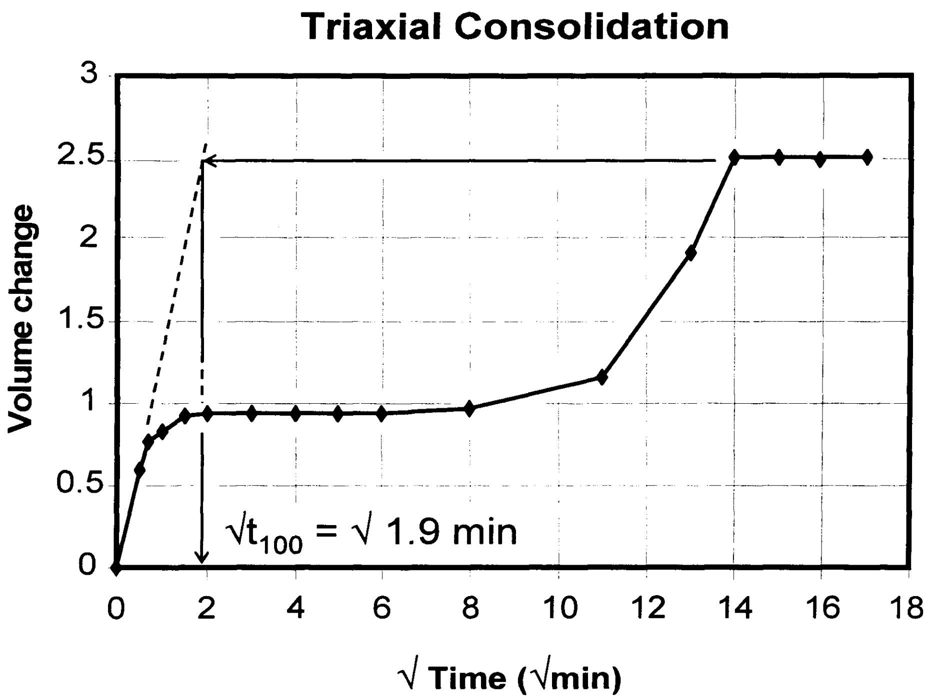


Figure 5.16 Triaxial consolidation stages

The data from the triaxial consolidation test is used to calculate the rate of axial displacement for the triaxial compression test and this is shown below.

$$\sqrt{t_{100}} = \sqrt{1.9\,\text{min}}\ ;\ \text{so}\ t_{100} = 3.61\,\text{min}$$

$$t_f = 0.5 \times 3.61 \quad = 1.88 \tag{5.9}$$

$$d_r = \frac{0.5}{100} \times \frac{l}{t_f} = \frac{5}{100} \times \frac{77.23}{120} = 0.032\,\text{min/mm}$$

where,

t_f = minimum time of failure

d_r = rate of axial displacement

Strain to failure was assumed as 5%, and time of failure was set at 2hrs as suggested by Head (1986). The time of failure should not be less than two hours according to Clause 6.3.6 of BS1377-8:1990.

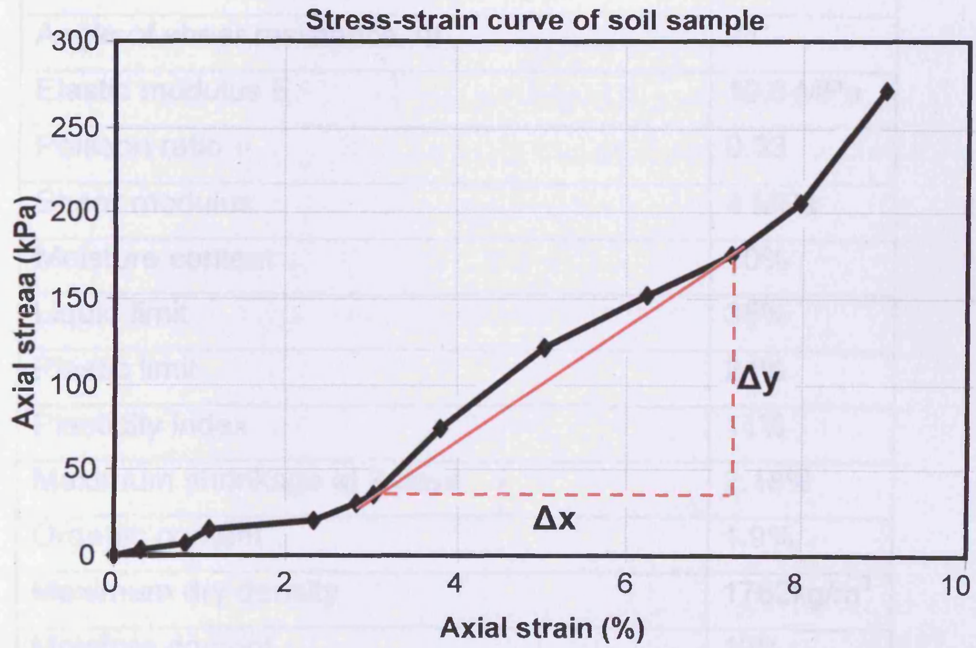


Figure 5.17 Deviator stress against strain

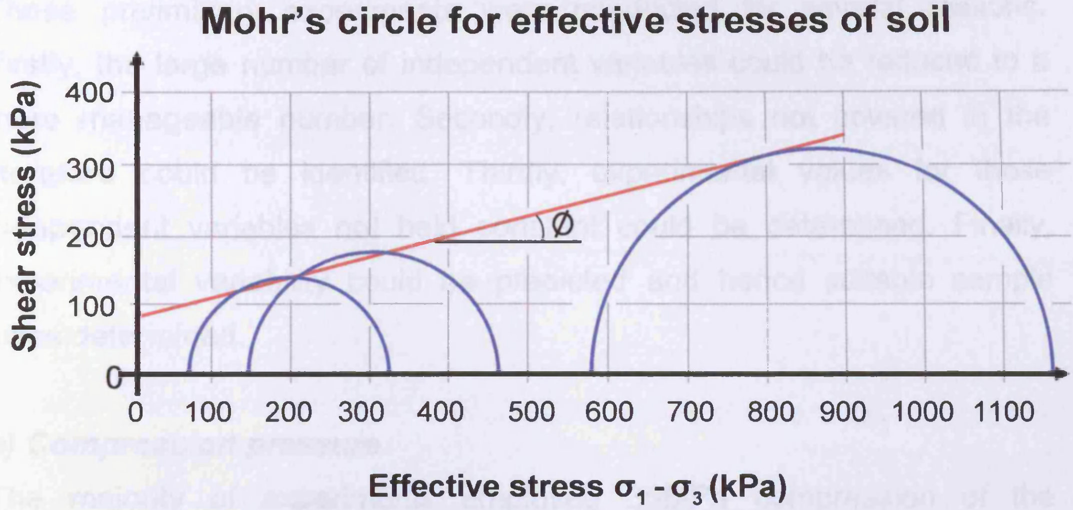


Figure 5.18 Mohr circle of effective stress

Table 5.7Summary of characteristic of soil used

Cohesion coefficient. C′	80 kPa
Angle of shear resistance Φ	21°
Elastic modulus E	10.6 MPa
Poisson ratio ν	0.33
Shear modulus	4 MPa
Moisture content	10%
Liquid limit	35%
Plastic limit	24%
Plasticity index	11%
Maximum shrinkage at 7 days	2.18%
Organic content	1.9%
Maximum dry density	1762kg/m^3
Moisture content	12%
Clay content-intermediate	11%

5.4.6 Results of the Preliminary test of soil block production

These preliminary experiments were conducted for several reasons. Firstly, the large number of independent variables could be reduced to a more manageable number. Secondly, relationships not covered in the literature could be identified. Thirdly, experimental values for those independent variables not held constant could be determined. Finally, experimental variability could be predicted and hence suitable sample sizes determined.

a) Compression pressure

The majority of experiments employed 35MPa compression of the BREPAK block making machine but other pressures were also checked to discover trends within the material. Selected values for pressure trials were 25, 30, 35 and 40MPa. The preliminary investigation involved the compression of soil into blocks at three different moisture contents of 10, 11 and 12%, and the pressure within the mould during the compression cycle of up to 40MPa were monitored. The purpose was to establish the optimum compression pressure that would be used in the production of blocks for the experiments. It was established that a compression pressure of 35MPa produced apparently the strongest block. This compressive pressure was thus used in the production of blocks for the main investigation on cement stabilised soil block. The results of the trials are presented in Table 5.8.

Table 5.8 Compression pressures of BREPAK machine

Compression pressure (MPa)	Average densities (kg/m^3)		
	Block specimens		
	A10	A11	A12
25	1729	1725	1716
30	1742	1729	1721
35	1775	1769	1747
40	1775	1769	1747

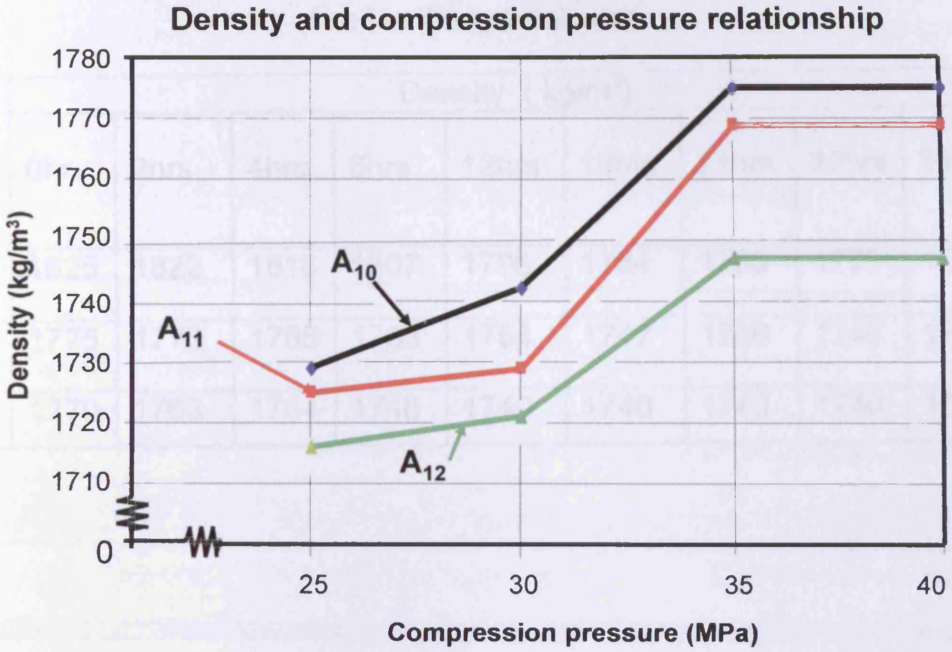


Figure 5.19 Compression pressure and density relationship

b) Moisture content

The moisture contents, which were determined using compaction test described in Section 5.1.4, used an oven dried soil sample. It was not practically possible to dry the soil block to be used in the remaining experiments in this way (since in field construction, soil would be used in the natural state), so it was deemed necessary to determine the optimum moisture content of the soil in the natural state. The results for the experiment are presented in Table 5.9 and Figure 5.20.

Table 5.9 Results of dry density and moisture content of soil in the natural state

Specimen	Density (kg/m³)								
	0hr	2hrs	4hrs	6hrs	12hrs	18hrs	24hrs	30hrs	36hrs
A_{10}	1825	1822	1815	1807	1796	1784	1780	1775	1 775
A_{11}	1775	1773	1765	1763	1754	1747	1746	1746	1746
A_{12}	1770	1763	1754	1746	1740	1740	1740	1740	1740

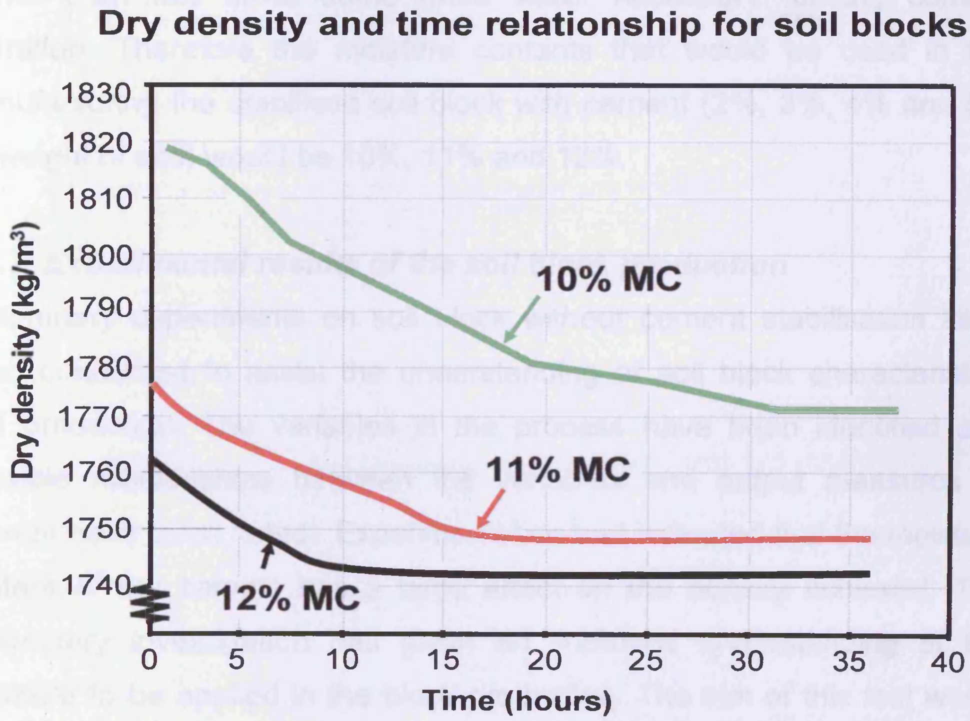


Figure 5.20 Dry density-time relationship of soil at different moisture content

Figure 5.20 shows the typical variation of dry density with time for un-stabilised soil. Dry density decreases rapidly during the first six hours for all moisture contents in question but rate of decrease was less after that until a constant mass was attained. This observation cautions that carefully curing of the blocks should be carried out for at least the first six hours. This was done by covering the blocks with polythene sheets

In this experiment the maximum dry density of the soil at a constant mass was 1775 kg/m^3 and this corresponded to a moisture content of 10% by weight soil. A moisture content of 12% by weight of soil had the least dry density of 1740 kg/m^3. From the results of the trials it could be concluded that the optimum moisture content for the soil used in the natural condition (without any cement) was 10% by weight of soil. Clearly, the moisture content for the same soil with some added cement could be higher than this since some more water necessary for the cement hydration. Therefore the moisture contents that would be used in the manufacturing the stabilised soil block with cement (2%, 3%, 4% and 5% by weight of soil) would be 10%, 11% and 12%.

5.4.7. Experimental results of the soil block production

Preliminary experiments on soil block without cement stabilisation have been conducted to assist the understanding of soil block characteristics and production. The variables in the process have been identified and possible relationships between the variables and output measures of interest have been noted. Experimentation had indicated that the moisture content of the sample has a large effect on the density achieved. The preliminary investigation has given an improved understanding of the pressure to be applied in the block production. The aim of this test would be to achieve a projected density of at least 1775 kg/m^3 as this represent a BREPAK compression of 35 MPa.

a) The effects of moisture on Dry Density

The objective of this test was to determine how the moisture content influences the density of the blocks and the role of cement in density.

Three blocks from each batch were selected after four weeks of curing. These blocks were gently wiped with non-absorbent cloth in order to remove any dust or loose matter stuck to them. Each dimension of these blocks in the middle of each face was measured and the average calculated (see Figure 5.21). Their volumes were then calculated. These blocks were oven-dried at a temperature of 105°C until constant masses of the blocks were obtained. The mass of the blocks were considered to be constant when the difference between two weighings at 24 hour intervals was less than 0.1% of the initial masses. On removal from the oven the blocks were left open to ambient air to cool (typically for two hours). After cooling, the blocks were weighed and then the densities were calculated and the average was then taken from each batch.

Length, $L = \frac{L_1 + L_2}{2}$ Height, $h = \frac{h_1 + h_2}{2}$ Width, $w = \frac{w_1 + w_2}{2}$

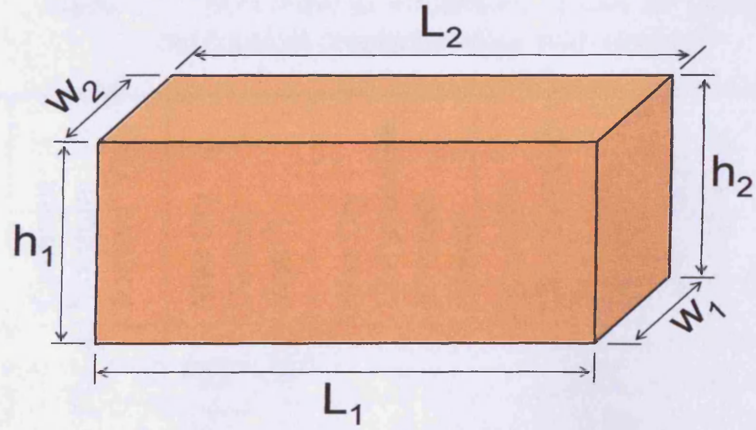


Figure 5.21 Illustration of block dimension

Table 5.10 Mechanical characteristics and strength of CSSB

Batch	Dry density (kg/m^3)	Dry compressive strength (MPa)	Wet compressive strength (MPa)	Coefficient of abrasion, C_a (%)	Water absorption coefficient, C_b (%)
A_{10}	1748	4.62	0.00	1.45	16.80
A_{11}	1739	4.26	0.00	0.52	21.30
A_{12}	1713	4.13	0.00	0.34	17.10
B_{10}	1841	4.91	0.90	1.47	14.20
B_{11}	1797	4.60	0.85	0.57	19.90
B_{12}	1766	4.20	0.86	0.41	15.60
C_{10}	1797	5.20	1.90	0.96	11.30
C_{11}	1847	5.77	1.94	2.25	10.40
C_{12}	1807	4.24	1.70	0.63	14.20
D_{10}	1817	4.69	2.14	1.61	14.20
D_{11}	1861	6.14	2.25	3.40	7.20
D_{12}	1763	4.40	1.74	1.54	15.60
E_{10}	1813	5.75	2.27	1.47	10.40
E_{11}	1910	6.46	2.76	3.40	7.20
E_{12}	1837	5.80	2.10	0.60	11.36

The batches are labelled such that X_i represents batch X with i% of water content by weight of soil. Furthermore, batches with 0%, 2%, 3%, 4% and 5% of cement content were assigned letters A, B, C, D and E respectively.

Table 5.11 Summary of maximum values for CSSB mechanical characteristics and strength

Specimen	Optimum Moisture content (%)	Max Dry Density (kgm^3)	Max Dry Compressive. Strength (MPa)	Max Wet Compressive. Strength (MPa)	Absorption Coefficient (%)	Abrasive Coefficient (%)
A	10	1748.00	4.62	0.00	16.80	1.45
B	10	1841.00	4.91	0.90	14.20	1.47
C	11	1847.00	5.20	1.97	10.40	2.25
D	11	1861.00	6.14	2.25	7.20	3.40
E	11	1910.00	6.46	2.27	7.20	3.40

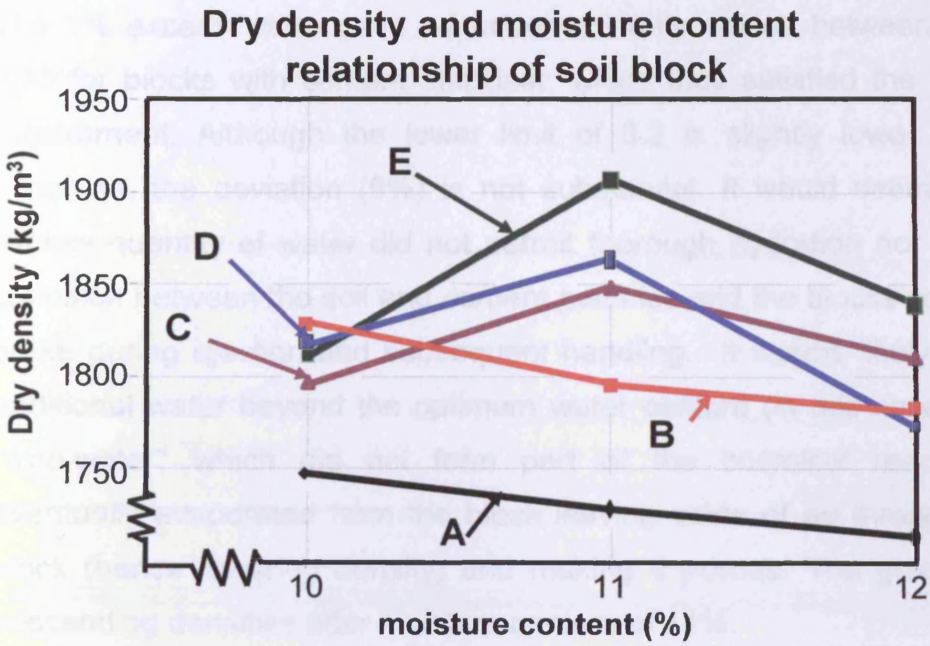


Figure 5.22 Moisture content and dry density of soil block at different cement content

Figure 5.22 shows that there were different optimum moisture contents for soil blocks with cement as stabiliser compared to those without any stabiliser. The optimum water content for the soil without stabilisation and for blocks with 2% cement content was 10% by weight of soil. The corresponding maximum densities were 1748 kg/m^3 and 1841 kg/m^3 respectively. The optimum moisture content for soil block stabilised with 3%, 4% and 5% of cement at 28-day curing age was found to be 11% and had maximum densities of 1847 kg/m^3, 1861 kg/m^3 and 1910 kg/m^3 respectively.

Although 1% addition of water seemed only a small amount (about 450g), it was possible that 1% additional water added permitted more complete hydration of the cement. The 1% addition of water gave water to cement ratio of 0.2, 0.25 and 0.33 for soil blocks with cement content of 5%, 4%, and 3% respectively. The minimum water/cement ratio for adequate hydration is between 0.22 and 0.5 (Akroyd, 1962, Lea, 1970). The 1% excess water gave water/cement ratio values between 0.2 and 0.33 for blocks with cement stabiliser, which thus satisfied the minimum requirement. Although the lower limit of 0.2 is slightly lower than the minimum, the deviation (9%) is not substantial. It would seem that the excess quantity of water did not permit thorough hydration nor sufficient cohesion between the soil and cement particles and the blocks sometimes broke during ejection and subsequent handling. It seems likely that, the additional water beyond the optimum water content (in this case 11%) is "free-water" which did not form part of the chemical reaction and eventually evaporated from the block leaving voids of air throughout the block (hence lowering density) and making it porous. The graphs show descending densities after moisture content of 11%.

Comparing cement content and density on the graphs shown in Figures 5.23 shows that there was a possible relationship between the two variables. The increase in cement content was in some way connected to the increase in the density. The density stabilisation increased from 1748 kg/m^3 for the un-stabilised block to 1910 kg/m^3 for 5% cement stabilised soil block, which was about a 9% increase.

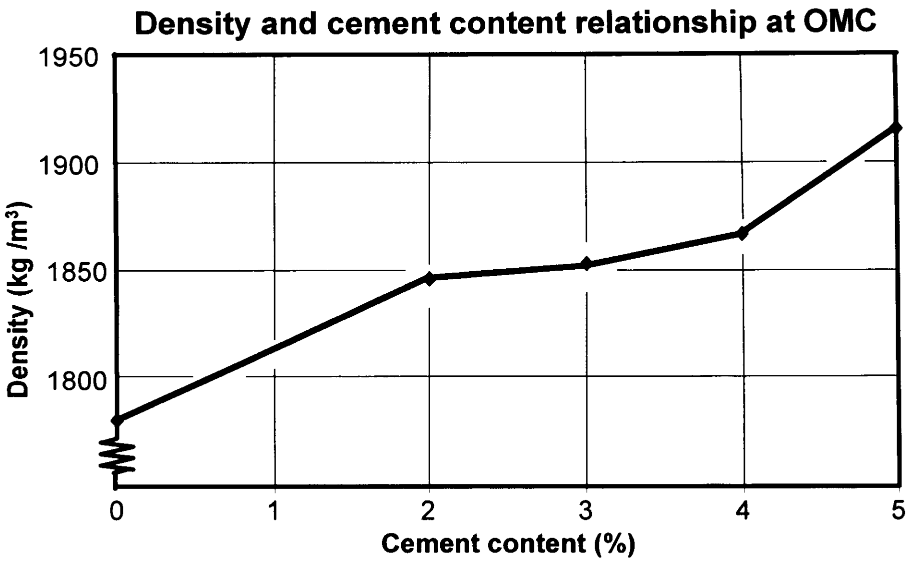


Figure 5.23 Density of soil blocks with variable cement content at the OMC

b) Dry Compressive strength

Three blocks which had no surface cracks visible to the naked eye were selected from each batch of moulded blocks. The blocks were oven-dried to a temperature of 40°C until constant masses were obtained. The blocks were then removed from the oven and left to cool in open air, and gently wiped of any dust or loose dirt stuck to them. The blocks were then tested for their dry compressive strength using a compression test machine.

The compressive strength at the dry state is given in Table 5.10 and shown in Figure 5.24. It could be seen that increase in the cement content increased the dry compressive strength. This might be attributed to the fact that the hydration products of the cement filled in the pores of the matrix and enhanced the rigidity of its structure by forming a large number of rigid bonds connecting soil particles. Up to about 4% of cement content there was an increased of about 33% over un-stabilised soil block but beyond that, the increase in dry compressive strength was slight (about 5.2% increase between 4% and 5% cement content).

b) Wet Compressive strength

Buildings are often exposed to the effect of water, particularly as a result of capillarity and of spraying from rain water. The mechanical strength (tensile and compressive strength) of wet blocks was found to be weaker than those of dry blocks.

The main purpose of this test was to find the minimum strength of the blocks and also to improve on the strength of the wet blocks if they were found to be unsatisfactory. As in Section (a) above, three blocks were selected from each batch and oven-dried at 40°C until constant masses were obtained. The blocks were then air cooled. The blocks were dusted and then fully immersed in water of temperature of about 20°C in the laboratory for two hours. The blocks were then remove from water and dried with a tissue. The blocks were then test for their compressive strength using compression test machine.

The compressive strength after immersion in water for 2 hours at the age of 28 days is given in Figure 5.22. The immersion in water for 2 hours reduced the compressive strength by up to 82% for cement-stabilised samples compared to the compressive strength in their dry state. Furthermore, complete disintegration of un-stabilised specimens was observed in a few minutes after immersion in water. The reduction in strength was lower with higher cement content up to 4% cement, which gave the lowest reduction in strength of about 63%. Specimens with 5% cement content did not give any significant improvement of strength of the wet samples. The lower strength of the wet samples could be prevented by treating the surface with cement render, with polymers or cement–lime renders, especially when the construction is to be exposed to water.

In summary, Figure 5.25 shows that there might be optimum cement content for strength, since the graph almost levelled off above 4% of cement content. It could be seen that the increase in strength was also connected to increase in density; the denser the block the greater the compressive strength. While there is a definite trend between dry compressive strength and density. The ratio of compressive strength to density is constant. The constant of proportionality is 3.1 x 10^4 m for blocks stabilised with 3%, 4% and 5% cement and 2.6 x10^4 m for 2% and blocks without any stabiliser. More tests at higher cement content would have to be conducted to confirm this finding.

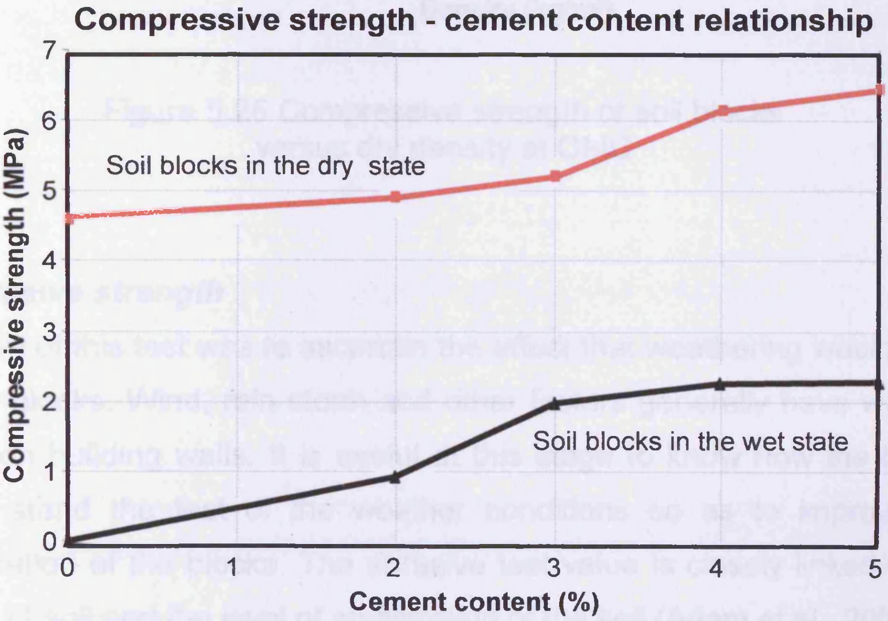


Figure 5.24 Compressive strengths soil blocks versus cement contents at the OMC

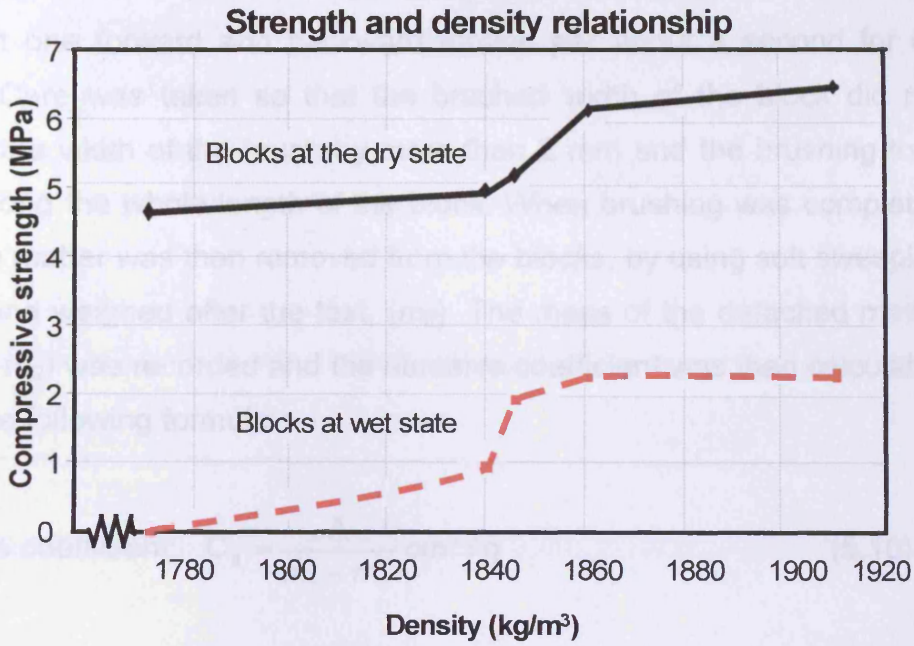


Figure 5.25 Compressive strength of soil blocks versus dry density at OMC

c) Abrasive strength

The aim of this test was to ascertain the effect that weathering would have on the blocks. Wind, rain storm and other factors generally have wearing effect on building walls. It is useful at this stage to know how the blocks would stand the test of the weather conditions so as to improve the stabilisation of the blocks. The abrasive test value is closely linked to the nature of soil and the level of stabilisation of the soil (Adam et al., 2001).

In these tests the blocks were subjected to mechanical erosion applied by brushing, with a metal brush at a constant pressure over a number of cycles on the face of the blocks, which would be used as facing.

Three blocks for the test were weighed (m_1) and each was placed on a table and the surface of the block was brushed in turns with wire brush at one forward and backward motion per about a second for 60 cycles. Care was taken so that the brushed width of the block did not exceed the width of the brush by more than 2 mm and the brushing took place along the whole length of the block. When brushing was completed all loose matter was then removed from the blocks, by using soft sweeping brush, and weighed after the test, (m_2). The mass of the detached matter (i.e. m_1–m_2) was recorded and the abrasive coefficient was then calculated using the following formula:

Abrasion coefficient: $$C_a = \frac{A}{m_1 - m_2}\ \mathrm{cm^2/g} \qquad (5.10)$$

where,
A =Area of brushed surface
m_1=mass of block before brushing
m_2 = mass of block after brushing.

The results of the abrasion coefficient are recorded in Table 5.10 and the relationship between the abrasive strength and the cement content is expressed graphically in Figure 5.26.

It could be seen from Figure 5.26 that the abrasive resistance increased with increase in the cement content. As defined, a high abrasive coefficient shows a large brushing area is required to yield a certain amount of discarded material. This then implied that the cement in the block helped to reduce wear of the blocks from external factors. However, there was no increase in abrasive resistance when the cement content was beyond 4% although more tests at higher cement content would have to be conducted to confirm whether the 4% is indeed the optimum cement content.

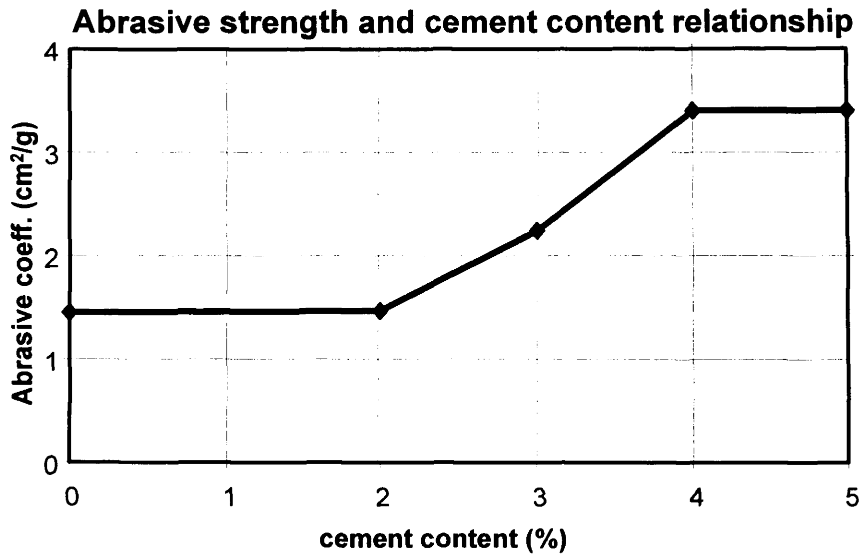


Figure 5.26 Abrasive strength versus cement content at OMC

It was also observed that at the same cement content (see Table 5.10) blocks moulded with optimum moisture content for each batch (i.e. A_{10}, B_{10}, C_{11}, D_{11} and E_{11}) had the highest abrasive resistance. This is expected because blocks moulded with optimum moisture content have the highest density, and thus resulted the best resistance to abrasion from rain water and from any form of rubbing against the blocks

e) Water absorption by capillary

Water absorption was measured by the increase in weight for a specimen stored for 28 days in a laboratory environment and then immersed in 5mm depth of water for 10 minutes as shown in Figure 5.27. The increase in weight is summarised in Table B8 in Appendix B. Cement stabilisation reduced substantially the absorptivity from 16.8% for 0% cement content to 14.2%, 10.4% and 7.2% when cement contents were 2%, 3% and 4% respectively (Table 5.10 and Figure 5.28). The higher cement content thus resulted in lower migration of water into the block (i e. lower permeability).

This could be explained that the higher cement content eventually led to higher hydrated cement and higher mortar content. The higher mortar content makes the block with some amount of cement less porous and more impermeable than the soil matrix probably by infilling the voids and displacing some of the soil with far less permeable cement hydration products, thereby reducing paths for water ingression. Again increasing cement content above 4% did not improve the impermeability of the block.

It was also observed that at the same cement content (see Table 5.10) blocks with moisture content 1% higher than the optimum water content (i.e. A_{11}, B_{11}, C_{12}, D_{12} and E_{12}) had the highest water absorption coefficient. This might be due to the fact that, the additional 1% water content (free-water) evaporated from the blocks leaving voids of air, and hence the ability of the blocks to absorb more water to infill the voids was higher, which consequently increased the water absorption coefficient.

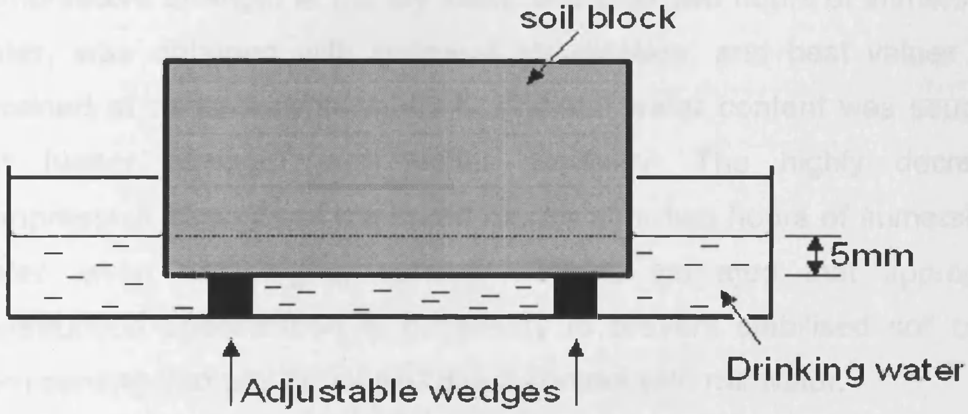


Figure 5.27 Sketch of set up for measuring capillary rise

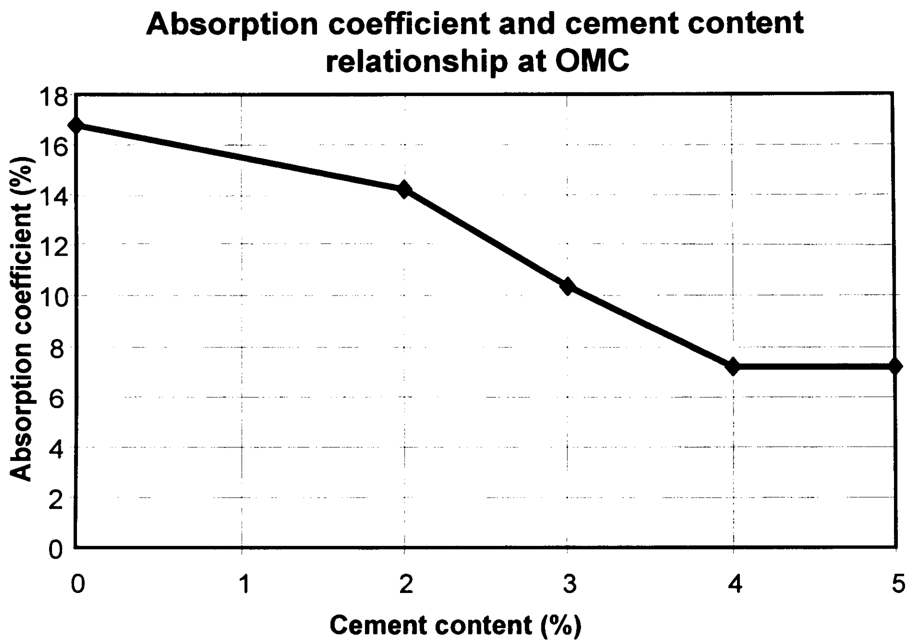


Figure 5.28 Absorption coefficient-cement content graph

5.5 Conclusion

A local Cardiff soil was chemically stabilised by cement. A better compressive strength at the dry state, and after two hours of immersion in water, was obtained with chemical stabilisation, and best values were obtained at cement content of 4%. Optimal water content was sought to get higher strength and higher durability. The highly decreased compressive strength of the cured blocks after two hours of immersion in water, even with higher cement content, indicated that appropriate construction specification is necessary to prevent stabilised soil blocks from coming into any prolonged direct contact with rainwater.

To produce low cost and environmentally friendly stabilised soil block for construction, it is worth examining alternative stabilising materials (like lime and fly ash) which have lower energy requirement, low cost and are waste products. Future research to be conducted in West Africa should consider the use of, for example, lime and rice husk ash (Pichai, 1991).

The degree of stabilisation depends largely on the soil texture. The soil used in the experiments was from Cardiff. This soil texture can be different form the soil texture in Ghana or in Africa in general. Once the percentages of sand, silt and clay are determined from basic soil identification tests, the soil texture could be determined from the USDA soil texture triangle (Friend, 2004). This then implies that the technology acquired in the United Kingdom can be transferred to Africa and to other parts of the world.

CHAPTER SIX

PERFORMANCE OF THERMOPLASTIC CARTON SOIL BLOCK (TCSB)

6.0 Introduction

There has been the problem of rising costs of building construction in the developing countries for sometime now, and especially in rural areas where the local income has often not increased at the same pace as the national average, has been a source of concern to governments (Ghosh, 1984).

Building materials are one aspect of internal factors needing urgent attention, since materials constitute about 65-70% of the cost of construction in Ghana and in Western African countries as a whole. Therefore, a rise in the cost of certain prime materials is very quickly fed into eventual significant increases in the building cost. In Ghana, over-dependence on imported materials and the cost of local transportation are some of the major contributing factors to the rising cost of construction.

The current production of construction materials in Ghana and most Western Africa countries leaves much to be desired. The major raw materials (*e.g.* clinker for cement, Aluminium sheets, steel *etc.*) to feed into this industry are all imported. Even though construction timber is one material that is "home-grown," its cost is also high in most developing countries due to its value as an export commodity to generate income and foreign exchange. It is against this background that this current research is being carried out to identify alternative building materials that are durable, readily available and cost effective for local consumption.

The purpose of this current investigation is to study the performance of compacted thermoplastic carton soil blocks (TCSBs). They are an appropriate building material which should be a viable alternative to the more expensive building materials such as concrete blocks, bricks or stone, and be largely dependent on local readily available raw material and labour.

6.1 Materials used and testing methods.

i) *Plastic container*

Most of the samples were produced using open cuboid shaped polyethylene containers with dimensions 165x120x60 mm.

ii) Soil

Typical top soil from the region of Cardiff was used. Soil was first passed through a 20 mm sieve before being characterised for its grading curve and consistency limits as described in Chapter six. Table 7.1 shows the summary of the characteristics of soil used.

iii) Oil palm and plastic Fibres

The oil palm fibres (about 38mm diameter and 55mm long) came from oil palm nut and were brought in from Ghana, while the plastic fibres (0.7mm diameter and 34mm long) were polythene fibres obtained in the United Kingdom.

Table 6.1 Summary of characteristic of soil used

Cohesion coefficient. C′	40 kPa
Angle of shear resistance Φ	21°
Elastic modulus E	10.6 MPa
Poisson ratio ν	0.33
Moisture content	10%
Liquid limit	35%
Plastic limit	24%
Plasticity index	11%
Maximum shrinkage at 6 days	2.18%
Organic content	1.9%
Maximum dry density	1762kg/m^3
Moisture content	12%
Clay content (intermediate)	11%

6.2 Testing method and testing program

A test system was designed to perform rigorous and comprehensive measurements on seven types of plastic carton soil block specimens in this study. The materials consist of the plastic containers, soil, oil palm fibres and plastic fibres (see Figure. 6.1). The plastic carton was used as an external container and as well as a tensile hoop stress provider. The soil was the principal core fill material, while the fibres were used as enhancement. The soil, and in some cases the soil-fibre mix, were placed in the plastic cartons and compacted on a vibration compaction table and then with a compression test machine before testing.

Figure 6.1 Plastic cartons soil block ready for testing

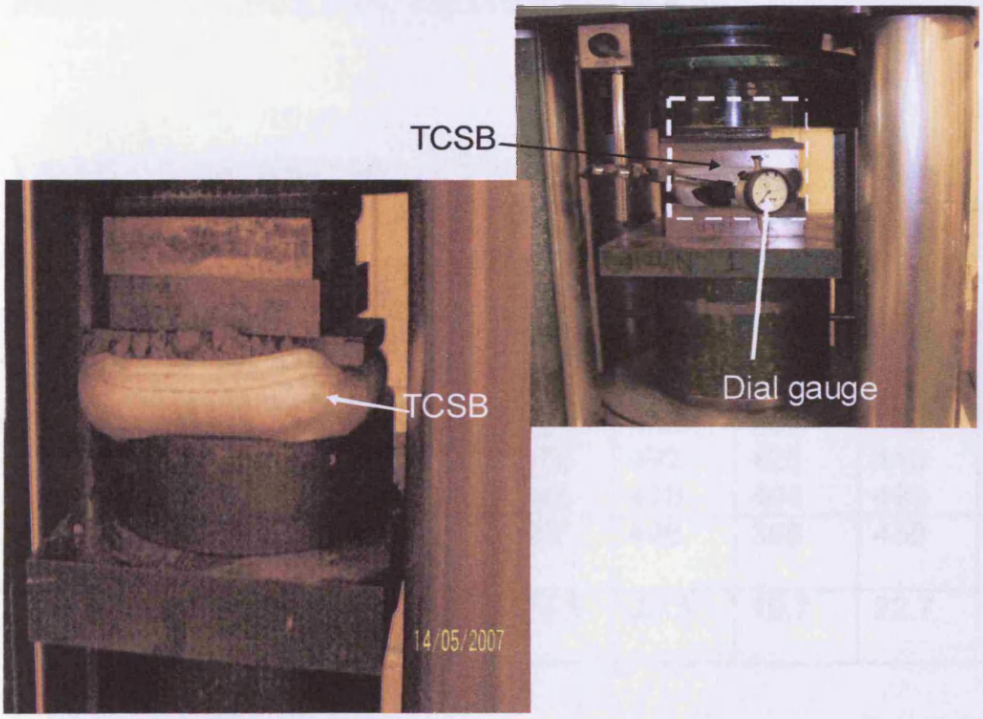


Figure 6.2 compression test set up

6.3 Experimental results for TCSB

This section discusses and analyses the results from the compression test on the thermoplastic carton soil blocks.

The compressive strength of plastic carton containing soil in the dry state was tested using a compression testing machine. Table 6.2 summarises the strength characteristics of the thermoplastic carton soil without any cement enhancement but with fibre enhancement.

Thermoplastic carton soil blocks without addition of fibres have the lowest compressive strength of 17.5MPa as compared with those with fibre addition. Even so it should be noted that 17.5MPa is still a very reasonable strength and over half that of typical concrete block. In the case of fibre enhanced soil block, the compressive strength increased with increase in weight fraction of fibre content, for both types of fibre, as shown in Table 6.2 and Figures 6.3a and 6.3b.

Table 6.2 Experimental strength results

specimen	A	$B_{0.75}$	$B_{1.0}$	$B_{1.5}$	$C_{0.75}$	$C_{1.0}$	$C_{1.5}$
Displacement at failure (mm)	24						
Strain at failure (%)	40						
Max applied force (kN)	328 393 347	385 363 343	421 378 348	448 492 470	365 426 404	452 410 496	498 463 480
Average force (kN)	347	372	406	446	386	450	479
Compressive strength (MPa)	17.5	18.7	20.5	22.5	19.7	22.7	24.2

A - soil block without any fibre

B_x - soil block with x% oil palm fibre

C_x - soil block with x% plastic fibre

At the lowest level of 0.75% oil palm fibre and plastic fibre addition, compressive strength was 6.8% and 12.5% respectively higher than blocks without fibre. At 1.0% fibre addition, the compressive strength increased, by 17% and 29.7%, for the palm and plastic fibre respectively. At 1.5%, the corresponding values are even higher at 28.6% and 38.3% increase. It should be noted that these strength values attained (i.e. around 20MPa) are very respectable strengths for building blocks.

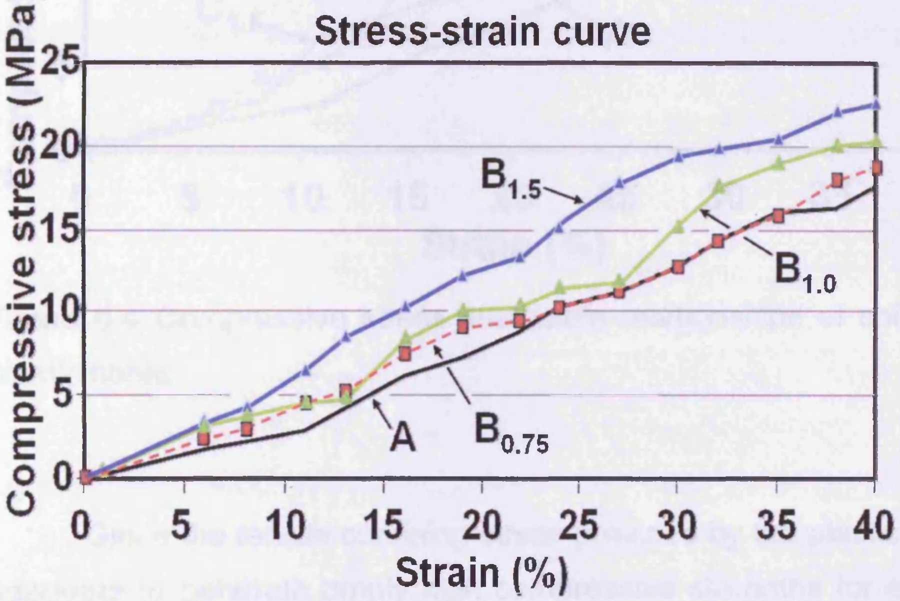


Figure 6.3 Compressive stress and strain relationships of soil blocks with oil palm fibre

Figures 6.3 and 6.4 show some experimental stress-strain curves for the soil blocks under compression. It can be seen that within the range of straining applied, the soil blocks had a fairly linear response even for strains up to 40%. Although these soil blocks could have achieved higher strength values before true ultimate failure (usually upon a splitting of the plastic carton), a strain of 40% was deemed a practical limit and thus the stress at 40% strain was designated the maximum stress.

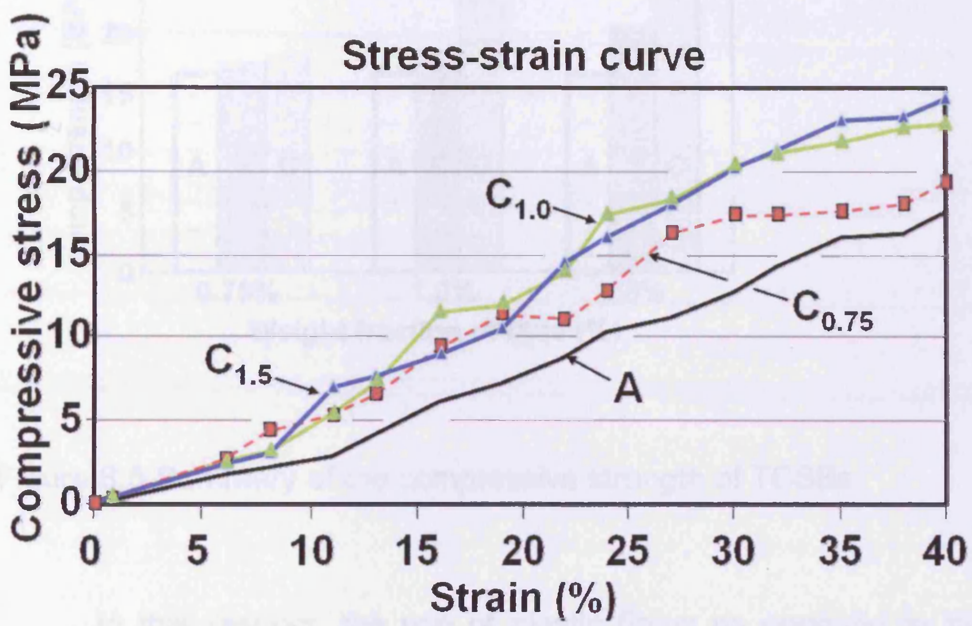


Figure 6.4 Compressive stress and strain relationships of soil block with plastic fibres

Since the tensile confining stress provided by the plastic carton was adequate to generate amply high compressive strengths for even a plain soil block (e.g. 17.5MPa at 40% strain), the advantage of fibre additions is actually not so much in the higher strengths achievable – true though that is – but in the ability to achieve high stresses at lower strains. The advantage is thus the higher stiffness generated by the fibre enhancement.

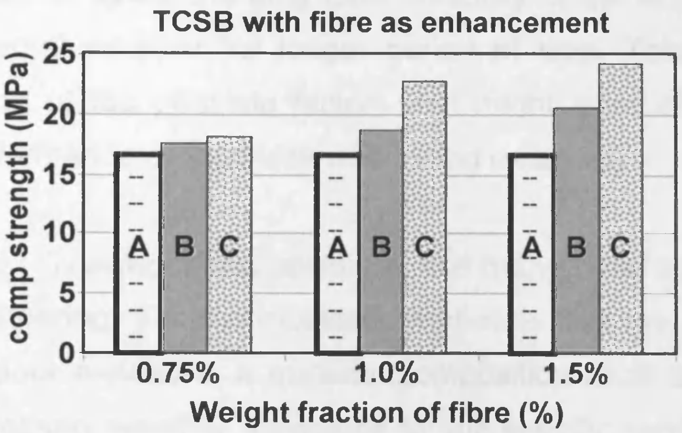


Figure 6.5 Summary of the compressive strength of TCSBs

In this respect, the use of plastic fibres as opposed to the palm fibres consistently produced the higher stiffness, as indicated in Figure 6.5. For the same fibre content of 0.75%, 1.0% and 1.5% by weight of soil, the strength at 40% strain of blocks C_x is about 5.4%, 11% and 6% respectively higher than blocks B_x. This could be expected since the plastic fibres are both stiffer and stronger than the natural palm fibres.

For increase in fibres content from 0.75% to 1.5% (i.e. a doubling of fibre content) the compressive strength increased by only about 20% to 23%. However, the stiffness of the block is much improved. For example, a stress of 15MPa is achieved at about 10.4% and 29.5% lower strain for 1% and 1.5% palm fibre, when compared with the plain soil block. The corresponding lower strain value for plastic fibres is about 33%. There is thus a clear advantage in adding fibres to the current newly proposed plastic carton soil block.

6.4 Durability of thermoplastic crate soil block (TCSB)

Though promising structural performance results have been reported, little is known about the long term durability of the block when exposed to the external weather for longer period of time. This section would discuss some of the possible factors that might have an adverse effect on the performance of the block when used externally.

Thermoplastic polymers, like many other materials, are affected by weathering. For thermoplastic materials that are intended for continuous outdoor exposure, a material composition must be selected that has the necessary weather resistance for the specific conditions involved. Much of the discussion would focus on the effect of ultravoilet radiation exposure, as this is generally the weathering factor which would have the greatest impact on the performance of the thermoplastics.

i) Factors influencing weathering

The environment surrounding the thermoplastic material must be considered, when making a determination for the suitability of a particular plastic material for either outside storage or long term above ground service. A brief description of the more important environmental parameters is given below.

a) Sunlight

Sunlight emits a significant amount of ultraviolet radiation in the wavelength range of 290-400nm on the earth (National Institutes of Health report, 1989). The ultraviolet radiation that is absorbed by a thermoplastic material may result in actinic degradation (i.e., a radiation promoted chemical reaction) and the formation of heat. The energy may be sufficient to cause the breakdown of the unstabilised polymer and, after a period of time, there would be changes in the compounding ingredients.

b) Temperature

The daily range of temperature varies considerably both with season and location and can be quite large (e.g. 20°C difference between the coldest season and warmest period). Furthermore, heat from solar radiation can raise the temperature of directly expose thermoplastic carton soil block by as much as 20°C higher than ambient. Such extremes of temperature over an extended period can cause physical damage to the thermoplastic carton soil block. Therefore, it is important that heat stabilizers (e.g. 2% of carbon black) be incorporated into the compounding ingredients in order to offset the deleterious effects of high temperature.

c) Moisture

Rain and humidity are the two main contributors of moisture with humidity having the greater overall effect. Rain produces a washing and impacting action.

d) Location

Location is also a factor that could affect the durability of the new building material. Less impact is found where there are fewer sunlight hours per year and where the radiation is less intense. For example, a six month period of exposure in Ghana is more detrimental than the same period in United Kingdom due to the obvious extra hours of ultraviolet exposure in Ghana.

ii) Improvement of durability

Thermoplastic polymers, like many other materials, are affected by weathering. A material composition that has the necessary weather resistance for specific conditions must be selected for thermoplastic materials that are intended for continuous outdoor exposure.

The long term durability of all thermoplastics can be improved by the incorporation of carbon black which is the combination of the basic thermoplastic polymer with carbon black (about 2% by volume of basic polymer) would result in the finished thermoplastic compound that could stand the ultra violet rays and weather conditions.

6.5 Conclusions, recommendation and way forward

This section summarises the findings from the results of the laboratory experiments on the thermoplastic carton soil blocks with and without fibres, and gives recommendation for future work.

6.5.1 Conclusion

The compressive strength obtained from the laboratory experiments on thermoplastic carton soil block was very promising. Such strength values of around 20MPa is about four times higher than chemically stabilised soil blocks stabilised without plastic cartons (from earlier work of this thesis). This is true even when the soil blocks had as high as 5% of cement content (by weight) which is approaching the economic limit. There is thus clearly a case for a larger scale study, and taking mitigating steps to ensure the durability of the TCSB as an alternative building material in construction of low cost housing, especially for disaster purposes and for low income earners in developing world.

It should also be noted that the proposed thermoplastic cartons and plastic fibres are actually environmentally friendly in that they are to be made from recycled plastic, which would otherwise be a waste material. Furthermore, there is no stringent specification for these cartons or fibres, since the current tests were conducted with disposable food cartons and recycled waste plastic fibres. The current newly proposed scheme of using plastic cartons with soil block thus achieves considerable improvements over the plain soil blocks (without plastic cartons) and at the same time provides a use for plastic waste which is abundant worldwide.

6.5.2 Recommendation

For the practical implementation of this research, ultraviolet stabilised carbon black systems are recommended for the manufacturing of the waste plastic containers which are intended to be used as the thermoplastic crate. Furthermore, it is clear that production of interlocking rectangular plastic crates from waste plastic containers, using an injection moulding machine capable of moulding according to a designed specification would result in a better building block product where the individual blocks would have some additional mechanical connections between themselves. Such simple mechanical interlocking would also produce a wall more resistant to dynamic loading, as in the case of an earthquake. The interlocking shapes of these plastic soil blocks could also help to reduce the skill level needed for homeowners to build their own homes. In addition, several layers of blocks could be placed in the wall at a time, reducing construction time. This could be helpful in cases of shelter provision post a natural disaster.

Use of filling materials other than soil needs to be considered in the next experimental studies on thermoplastic interlocking crate blocks. The filling materials that would be possibly studied would be demolistion debris from buildings, which is perhaps augmented with non-biodegradable agricultural waste such as oil palm kernel aggregate. Successful usage of the demolition debris (which mostly ends up at the landfill) as a filler material for the thermoplastic crate block would be useful for practical applications in the case of housing for earthquake disaster relief. Application of palm kernel (which is normally burnt) as a filler, would have benefits to the oil processing industry. There would be provision of cheap, affordable and durable houses for the rural community, the problem of disposal of the waste would be solved, and greenhouse gases from either the burning (CO_2) or decomposition (CH_4) of the kernel would also be avoided

Another area that should be studied in the next experimental studies would be the formation of entire wall from thermoplastic crate soil blocks and the testing wall strength both in the vertical compression and the horizontal impact.

Formulation of plastics with a minimum of two percent finely dispersed carbon black fibres, would greatly increase the weather resistance of the compound and give sufficient protection for continuous outdoor service.

6.5.3 The way forward

The results of experiments conducted in this research have provided some knowledge on the mechanical characteristics of the proposed thermoplastic plastic soil block. A numerically based analysis would be performed, to further establish the internal stresses and a fuller understanding of the interaction of forces between the constituent components of the plastic soil block with fibres.

The motivation for a theoretical analysis is threefold. Firstly a better understanding of the performance of thermoplastic crate soil blocks is obtained. Secondly, it would be possible to assess whether the magnitude of the theoretical stresses within the soil block are within the permissible stress, and thus unnecessary local "failure" could be avoided. And finally, when the confidence in the numerical model has been gained, this model could be used to more rapidly investigate effects of changes to the soil carton block geometry, material and configuration.

CHAPTER SEVEN

NUMERICAL TESTING OF THERMOPLASTIC CARTON SOIL BLOCK

7.0 Introduction

A three-dimensional finite element (FE) simulation of deformation and compression load of thermoplastic carton soil block with different degree of fibre enhancement was carried out.

Although the literature review has not revealed any previous studies on the thermoplastic carton soil block, there are not too dissimilar things. An example is gabion baskets, which are widely used in civil engineering projects like construction of motorway embankment and retaining walls. The principle thus is similar to the thermoplastic carton soil block, since the gabion basket consists of rectangular sided wire baskets that constrain the loose rock content. Built in a variety of sizes, the baskets are then positioned where they are needed, much in the same way as the plastic carton soil block are placed on top of each other in a wall construction.

The results of experiments conducted in the previous chapters provide us some knowledge about the mechanical characteristics of thermoplastic carton soil block and cement stabilised soil blocks. A computational analysis was performed to establish and verify the relationship between the experimental and theoretical results using finite element analysis model.

The motivation for conducting computational analysis is threefold; firstly to improve the understanding of the performance of thermoplastic carton soil blocks, secondly to assess whether the magnitude of the stresses are within the permissible limits (after comparing the computated deformation with the experimental values) and thirdly, when confidence in the finite element result has been established, the finite element model can be used to more rapidly investigate effects of changes to the soil carton block geometry, material and configuration.

The objectives of this work are to develop numerical models of various types of thermoplastic soil blocks and to correlate measured performance with finite element model predictions. Thermoplastic carton soil block with fibres as an enhancement, and without were implemented using commercial code MSC Nastran. Details of the computational models are presented in Appendix D, while a correlation of the experimental results and the numerical results are presented in this chapter. Numerically generated values of displacement, various stresses of the blocks present an important tool for the prediction of material behaviour, and the optimization of mechanical properties of materials. This study serves as the first validation of the thermoplastic carton soil block from experimental model, so the parallel numerical and measured results would be described and compared.

Seven finite elements models of thermoplastic carton soil blocks with various degree of enhancement were developed. The types and the annotation of the created blocks are indicated in Table 7.1.

Table 7.1 Specimen's names and symbols

Thermoplastic carton soil block						
Without fibres	With oil palm fibres			With plastic fibre		
	0.75%	1.00%	1.50%	0.75%	1.00%	1.50%
A	$B_{0.75}$	$B_{1.0}$	$B_{1.5}$	$C_{0.75}$	$C_{1.0}$	$C_{1.5}$

7.1 Finite element modelling

i) Geometry

Finite element simulations giving the displacements and stresses of soil blocks were carried out for different degrees of fibre enhancement of the soil. In all cases, the numerical simulations were carried out three dimensionally. It has been shown in several works, e.g. Mishnaevsky (2004) that results of 2D approximations are generally not good approximation for 3D case. This is partially relevant in the current case since the 3D block does not have symmetry all round in the plan view. A 3D solid of dimension 165x60x120 mm^3 was created for the thermoplastic carton soil block. 3D FE models were chosen for their advantages over axisymmetric and two-dimensional model in that they consider a more realistic fibre geometry at fibre ends and do not ignore any portion of the matrix material (Andrade-Compos et al., 2001).

ii) Element

Finite element models of thermoplastic carton soil block were developed in MSC Patran for pre-/post-processing. For the thermoplastics carton, the MSC/Nastran model used solid element of 80 hexahedron solid mesh, with type "hex 20" for the core material (plastic container) and "80 tetrahedron" for the soil with/without fibre contained in the carton. The tetrahedral mesh element is used for any closed solid, e.g. regular solid region like soil block. The hexahedral mesh element is a proximity-based meshing that produces high quality meshes through the thickness of a thin walled. The soil/plastic interface was modelled using spot welded elements to join the plastic carton representing the core to the face of the soil composite with/without fibre. The interface surface of the carton was off set to coincide with the interface of the soil.

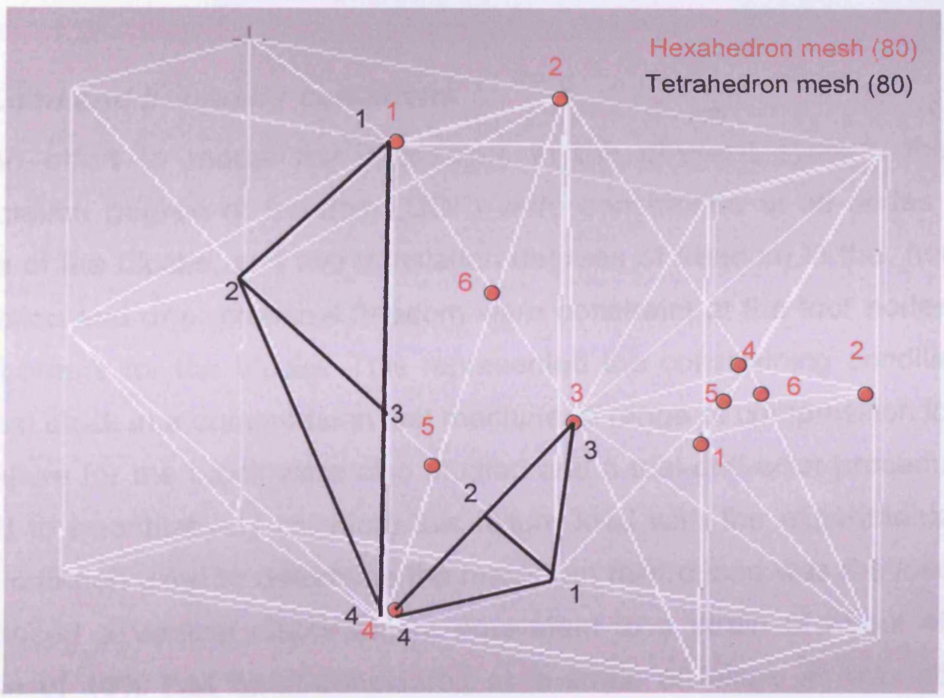


Figure 7.1 Soil block geometry and FE mesh

iii) Material and properties

Mechanical characterisation tests had been performed in the laboratory at Cardiff. Triaxial tests on the soil were performed and the results were used to determine the modulus of elasticity, Poisson ratio and shear modulus of the soil used. However, properties like modulus of elasticity and Poisson ratio for thermoplastic carton, plastic fibre and oil palm fibre used in this study were not available in direct experimentation. Instead typical values from literature were used in the numerical prediction of the thermoplastic soil blocks performance.

iv) Load and boundary conditions

In an effort to model the constraints in the physical system, the three translation degree of freedom (DOF) were constrained at all nodes on the base of the blocks, and two translation degrees of freedom in the horizontal direction and one rotational freedom were constraint at the four nodes at the top corners for the blocks. This represented the constraining condition of a typical block in a compression test machine. A range of compression loads up to failure for the block were also studied and a trial-and-error procedure was used to quantitatively correlate this failure load with the experimental data. The criterion used to determine the maximum failure load was the load which produced a vertical displacement equivalent to a strain of about 40%. A strain of 40% had been considered as a strain at failure as was observed during the experimental studies. An initial compression load of 250 kN was applied to the top surface of the block. Incremental loads of 50 kN each were subsequently added till a failure strain of 40% was achieved.

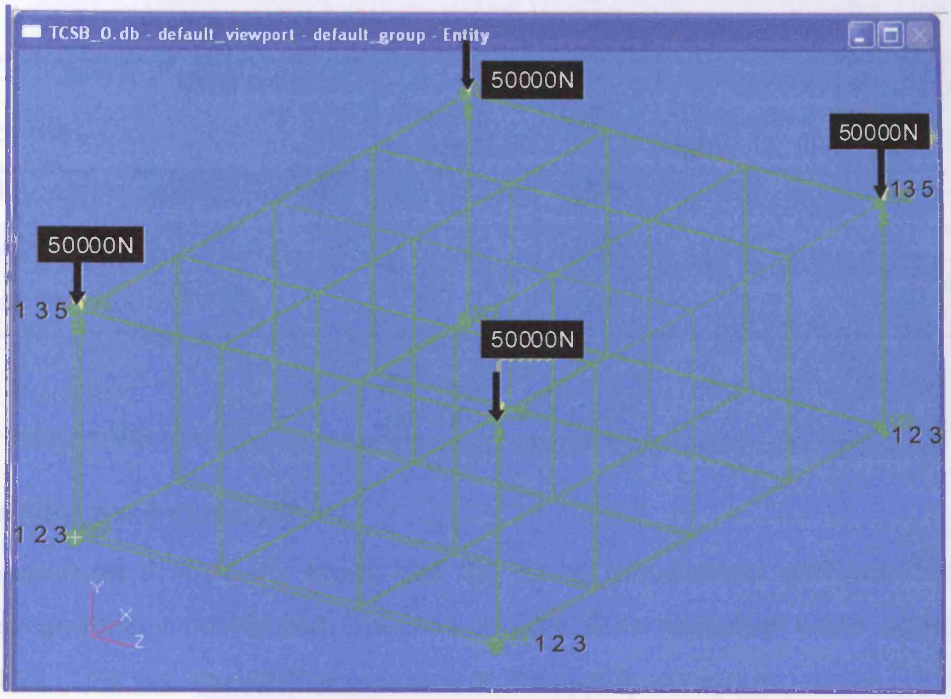


Figure 7.2 Loading and boundary conditions

v) Analysis

The analysis input file from Nastran was generated from the finite element model in MSC Patran. Non-Linear static analyses were performed to determine an approximate stress and displacement at each surface and node. Stresses in all direction and Von Mise stresses and displacement fringe were generated as shown in Figures 7.3a-n, and a graph of stress verse strain for soil blocks are shown in figures 7.4a and 7.4b for comparison with experimental results. The load dependent composite materials properties used in this study to model the blocks are shown in Table 7.2. The process of modelling are explicitly explained in Appendix C.

Table 7.2 Material properties for the model

Material / Properties	Plastic container	Soil	Oil palm fibre	Plastic fibre
Modulus of elasticity (MPa)	170	10.6	15	170
Poisson ratio (MPa)	0.3	0.33	0.3	0.3
Shear modulus (MPa)	77.3	4.4	5.7	77.3

vi) Finite element simulations

It was assumed that all the fibres had the same dimensions and orientation and were uniformly distributed. The soil and the fibre materials were isotropic in stiffness. The representative element of the block, shown in Figure 7.1 has a tetrahedron mesh for the soil and this includes the cylindrical enhancement fibre. An orthogonal cartesian coordinate system was used as reference with 0x, 0y and 0z axis aligned with the main dimensions of the soil block. The longitudinal axis of the enhancement fibre was placed perpendicular to the uniaxial loading direction (Teixeira-Dias et al., 2001).

Finite element meshes of a block composite consisted of plastic cartons 160 x 120 x60 mm in size filled with soil matrix with and without fibres, generated with the use of the program commercial code MSC/PATRAN. The soil together with fibre was modelled as "Halpin-Tsai" discontinuous fibre composite material. This is a two phase composite in which the matrix phase is isotropic and the fibres are uniform, discontinuous, cylindrical, and transversely isotropic. The resultant composite is therefore considered as transversely isotropic. In addition to the values of the elastic modulus and Poisson ratio, the fraction of each material constituents and fibre aspect ratio were required. The characteristics of the modelled materials in the material data input are shown in Table 7.2. A predicted 3D displacement and stress diagram at a fully deployed state is exhibited below (see Figures 7.3a-n).

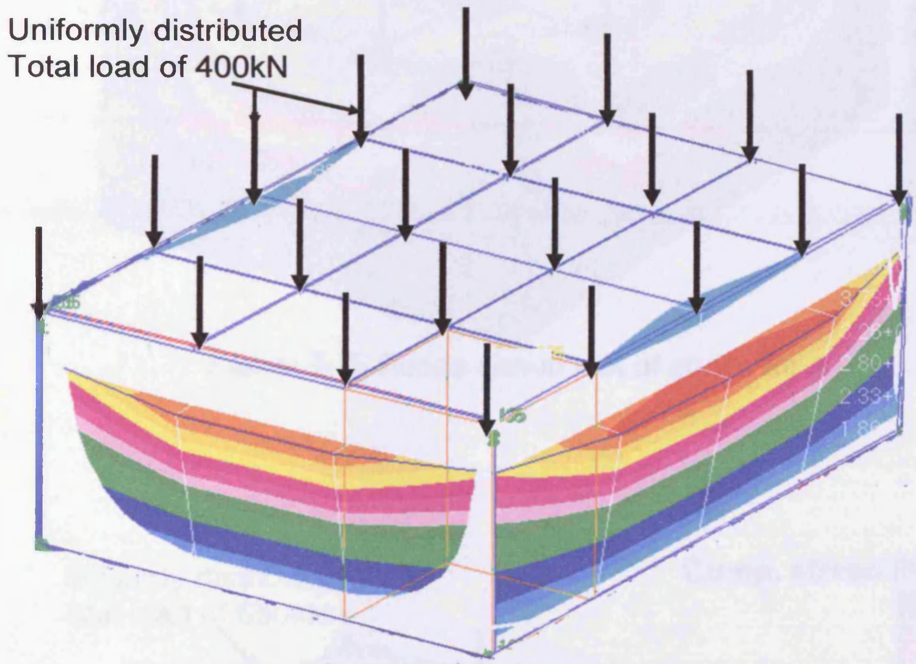


Figure 7.3a Fringe carton plot of deformation for A

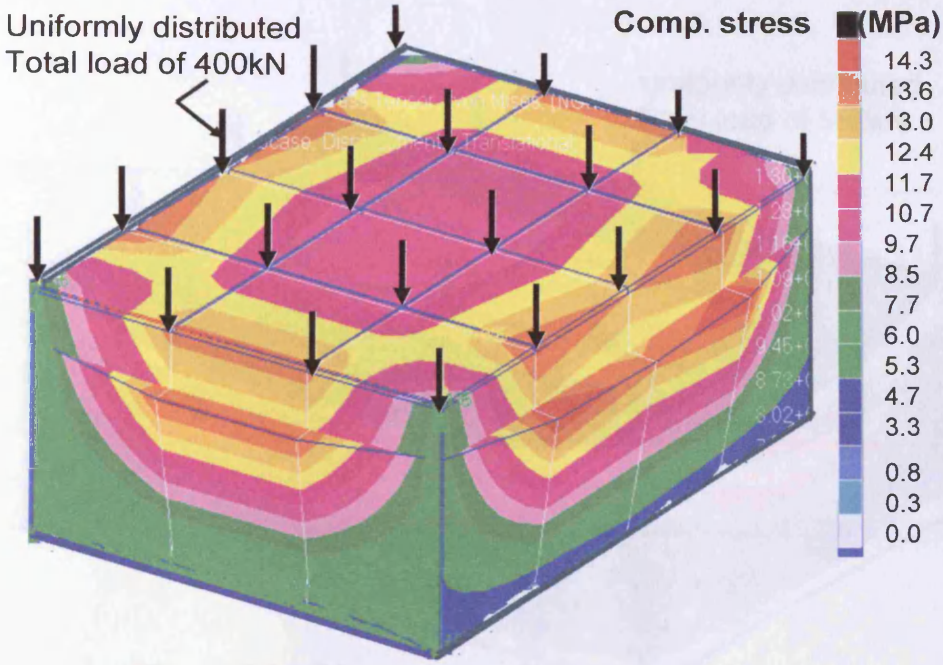


Figure 7.3b Fringe carton plot of stress for A

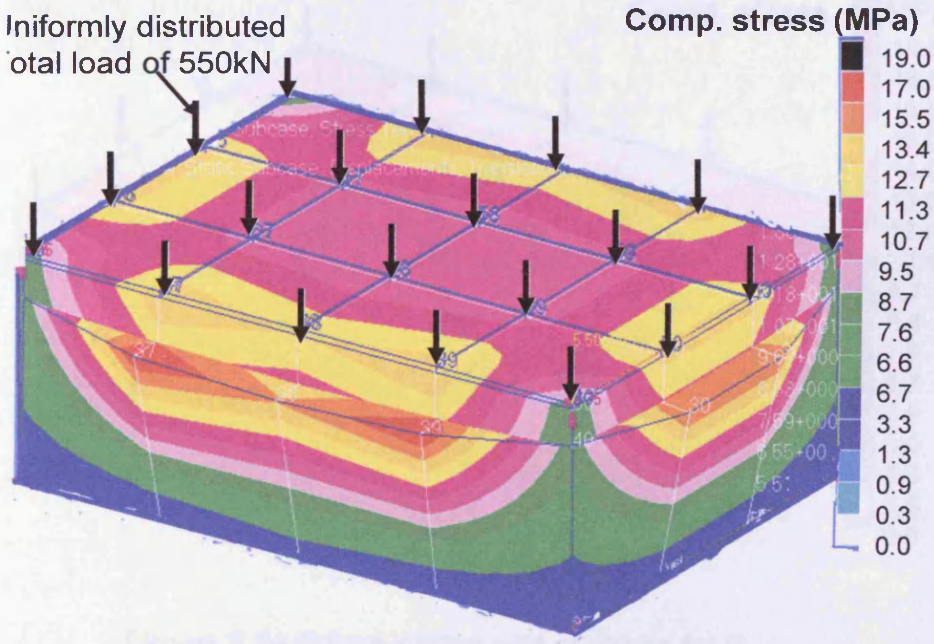


Figure 7.3c Fringe carton plot of stress for $B_{0.75}$

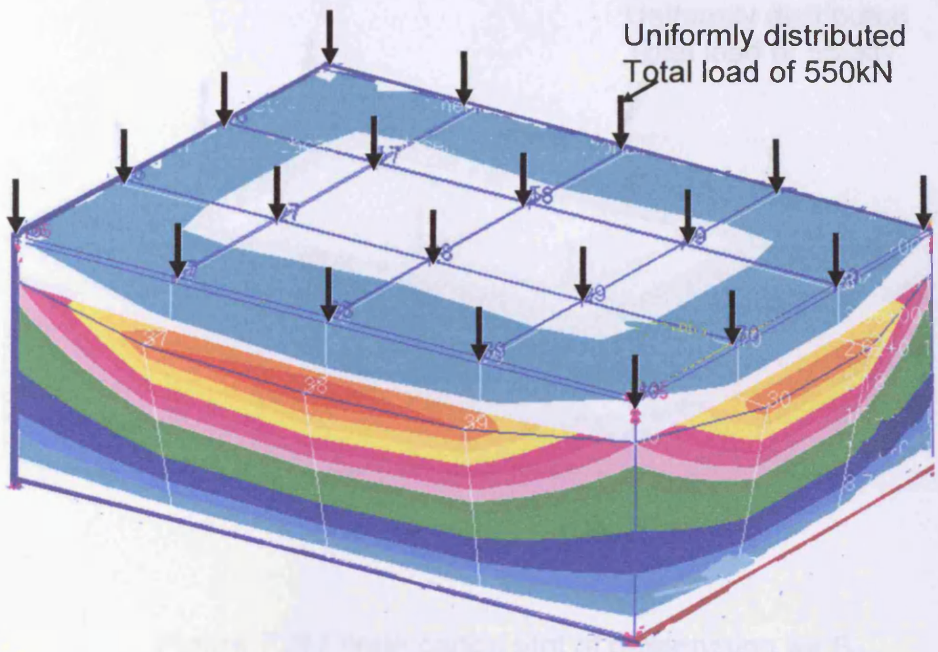


Figure 7.3d Fringe carton plot of deformation for $B_{0.75}$

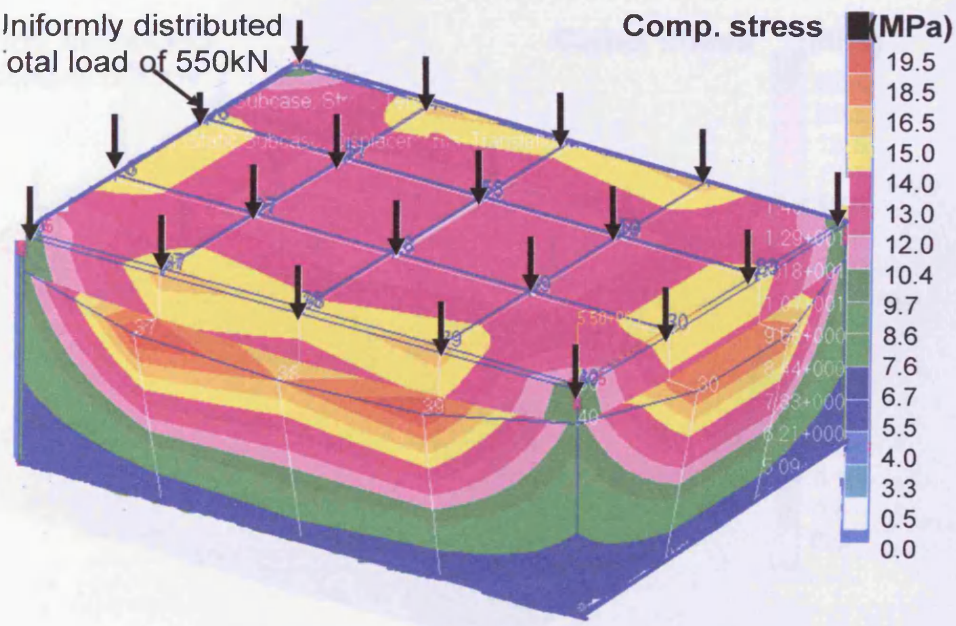


Figure 7.3e Fringe carton plot of stress for $B_{0.1}$

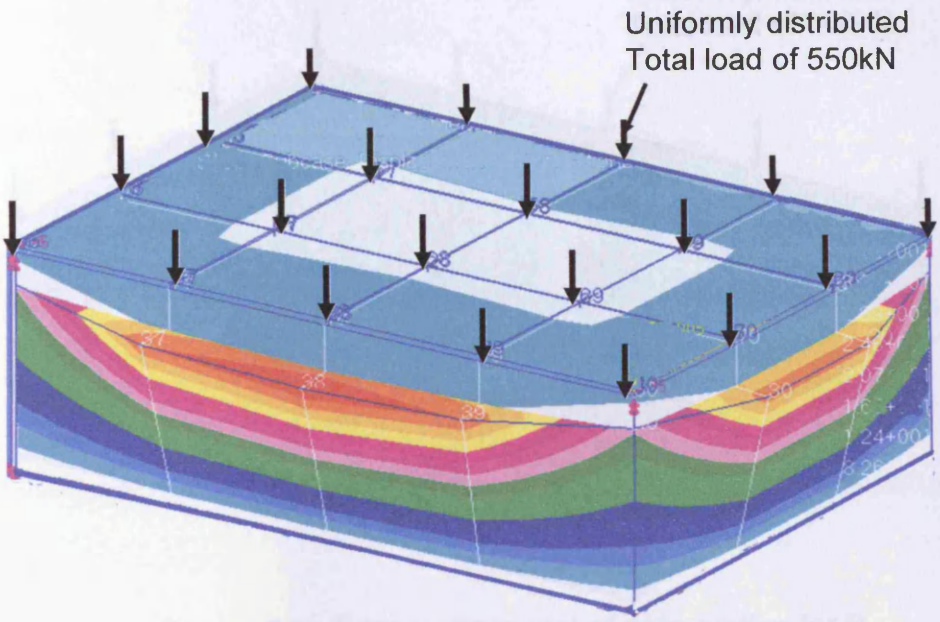


Figure 7.3f Fringe carton plot of deformation for $B_{0.1}$

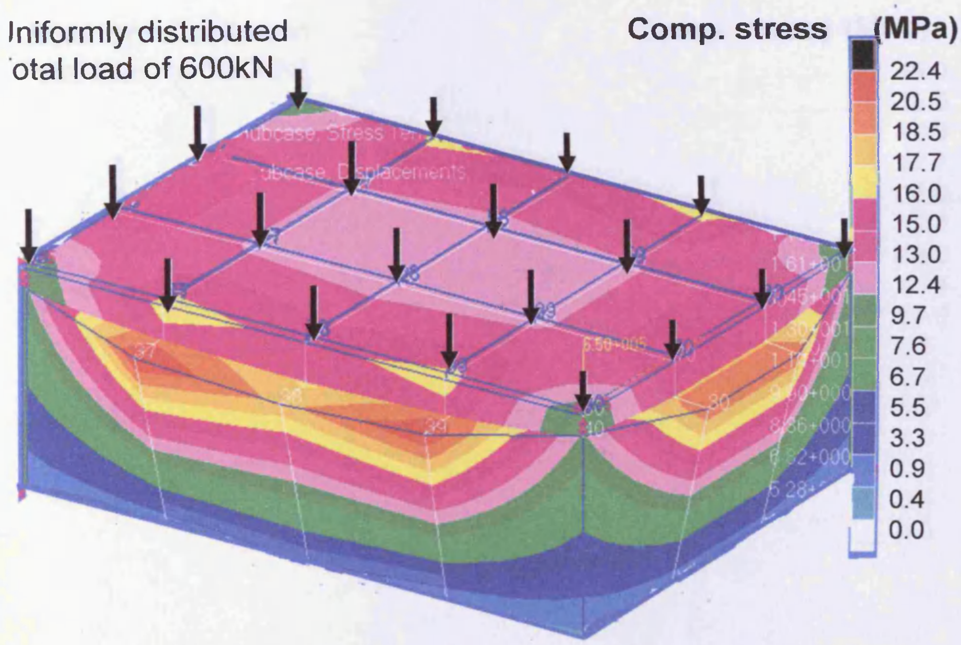


Figure 7.3g Fringe carton plot of stress for $B_{1.5}$

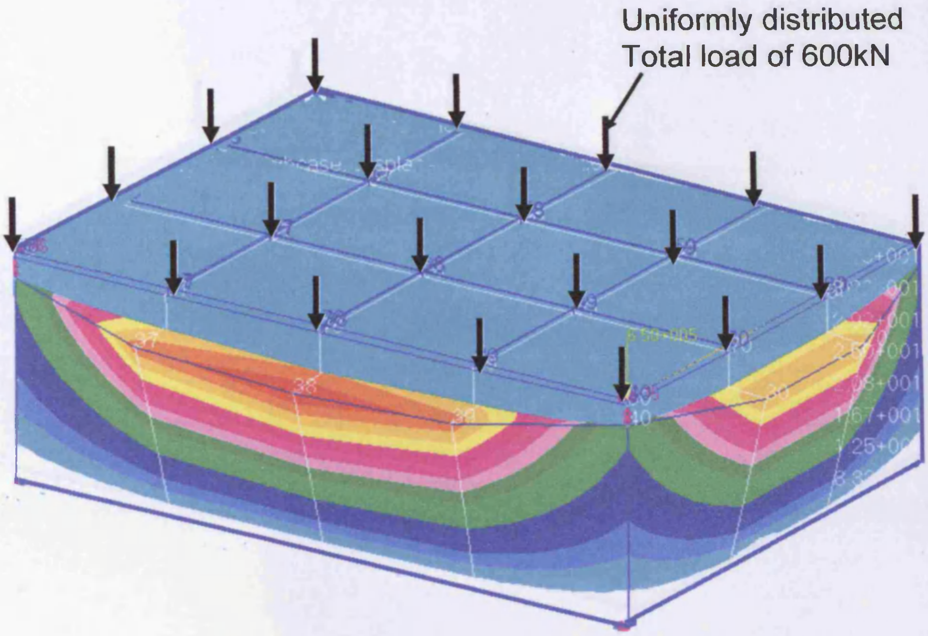


Figure 7.3h Fringe carton plot of deformation for $B_{1.5}$

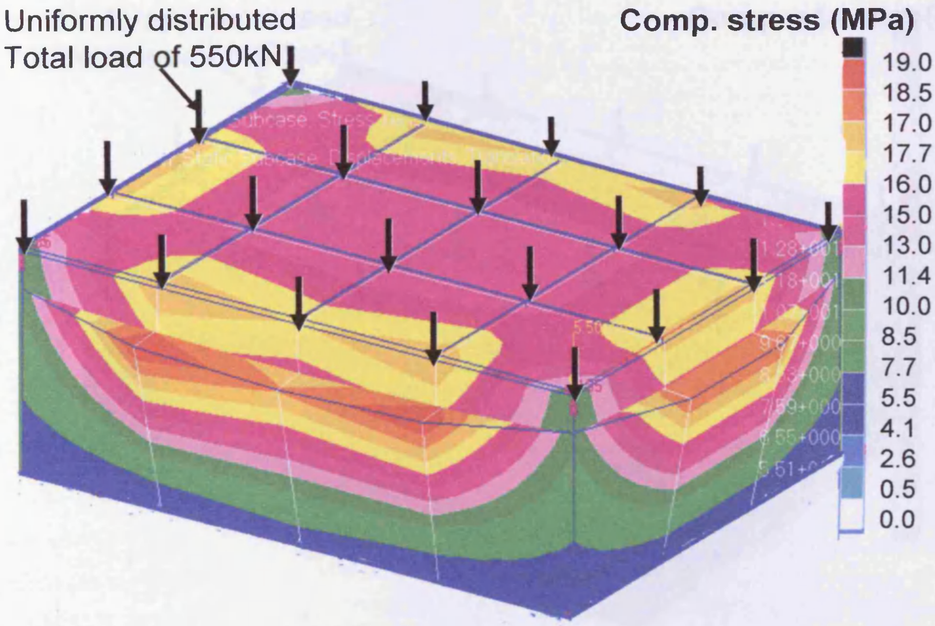


Figure 7.3i Fringe carton plot of stress for $C_{0.75}$

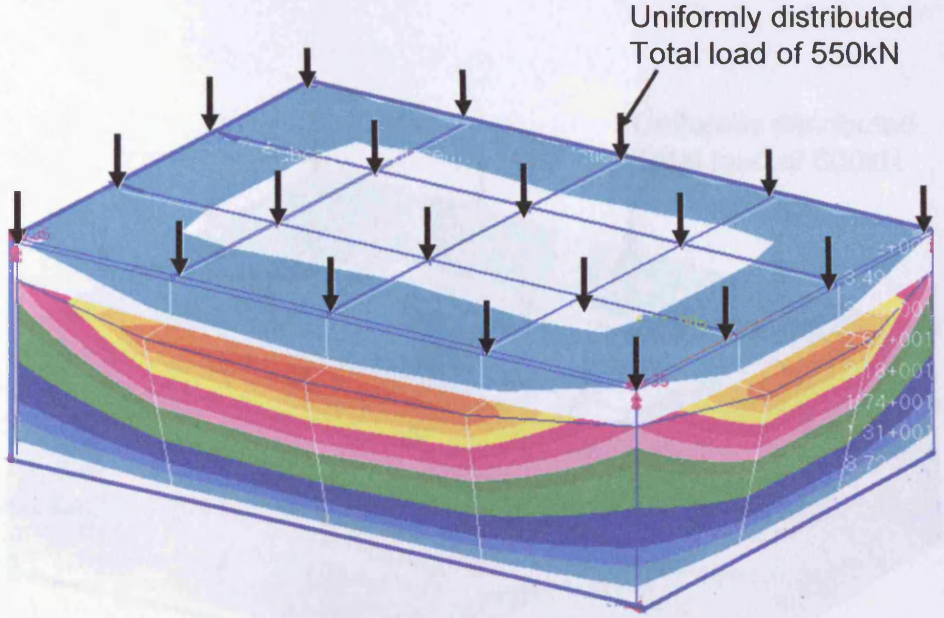


Figure 7.3j Fringe carton plot of deformation for $C_{0.75}$

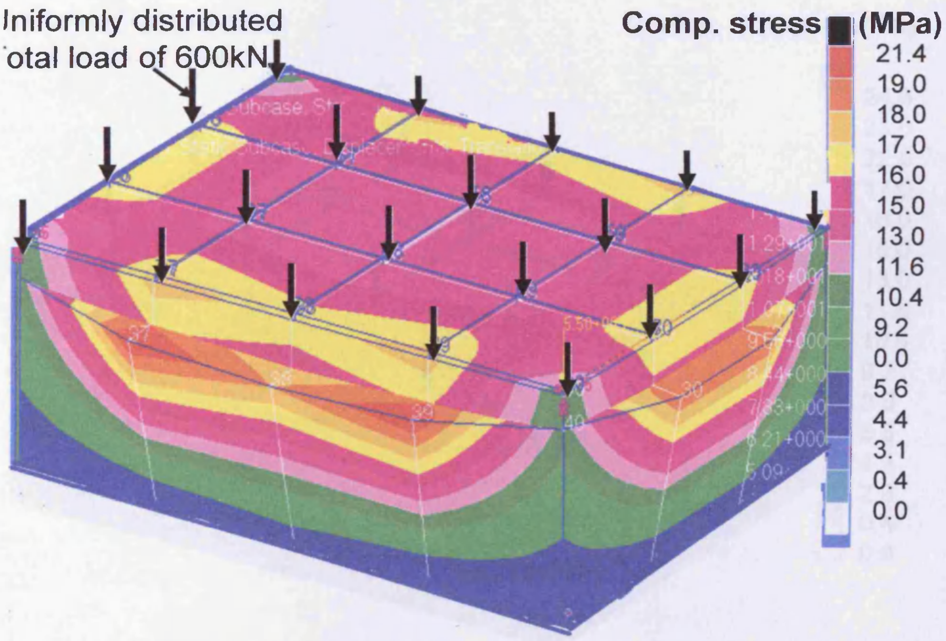


Figure 7.3k Fringe carton plot of stress for $C_{1.0}$

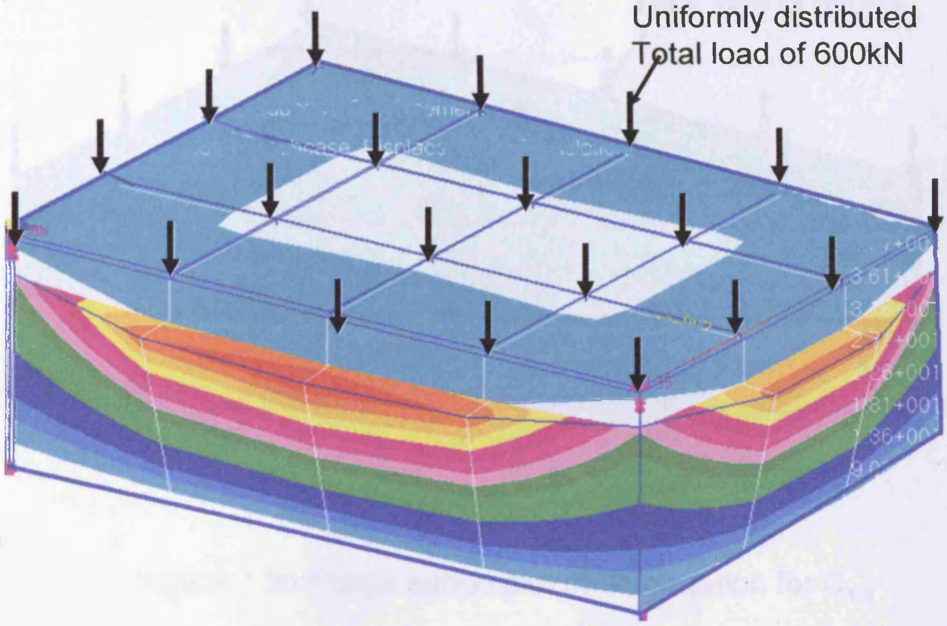


Figure 7.3l Fringe carton plot of deformation for $C_{1.0}$

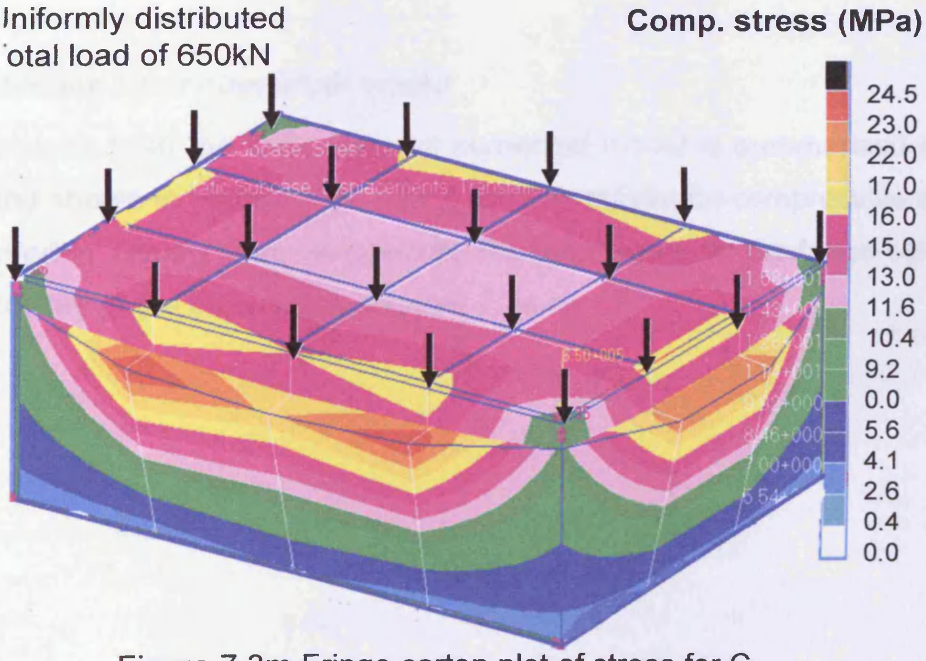


Figure 7.3m Fringe carton plot of stress for $C_{1.5}$

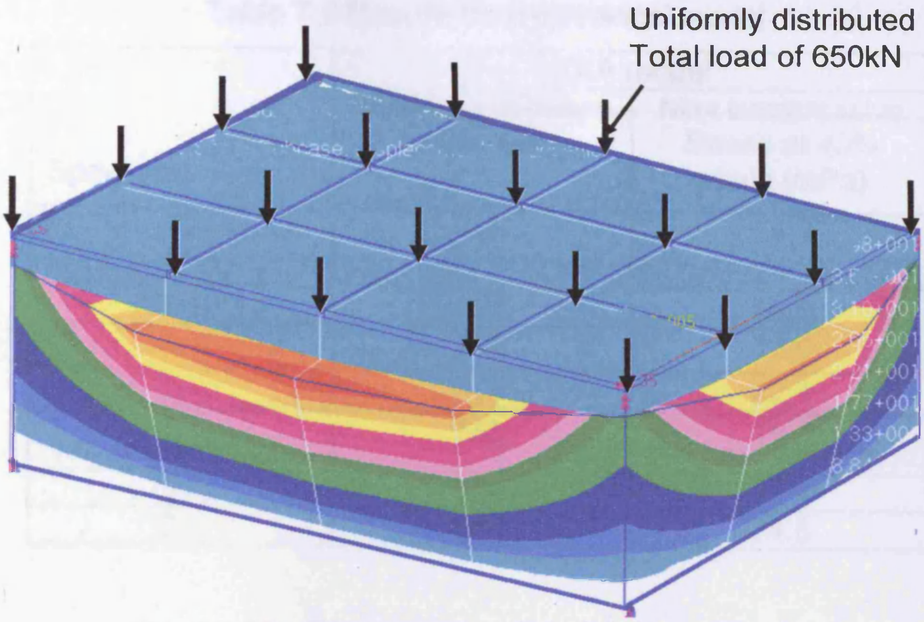


Figure 7.3n Fringe carton plot of deformation for $C_{1.5}$

7.2 Results from numerical model

The results from the finite element numerical model is summarised in Table 7.3 and shown in Figures 7.4a and 7.4b. The maximum compressive stresses recorded in Table 7.3 as indicated by the red colours on the fringe carton plot of stresses of the blocks (see Figures 7.3a-n)

Table 7.3 Results from numerical model

Specimen	FEA model	
	Max load at failure strain (kN)	Max compressive Stress at 40% strain (MPa)
A	400	14.3
$B_{0.75}$	550	19.0
$B_{1.0}$	550	19.6
$B_{1.5}$	600	22.4
$C_{0.75}$	550	19.0
$C_{1.0}$	600	21.4
$C_{1.5}$	650	24.5

Similar to the experimental studies, thermoplastic carton soil block model without addition of fibres as an enhancement has the lowest compressive strength of 14.3 MPa as compared to those with fibre addition. In the case of fibre enhanced soil blocks, the compressive strength increased with increase in weight fraction of fibre content, for both type of fibres as shown in Table 7.3 and Figures 7.4a and 7.4b.

The compressive strength was 24.7% higher for the lowest level of 0.75% fibre addition compared to the block without fibre. At 1.0% levels of fibre addition, the compressive strength increased by 27% and 33% respectively for oil palm fibre (B_x) and plastic fibre (C_x) enhanced thermoplastic carton soil block model. At 1.5% levels of fibre addition, the corresponding figures were increases of 36% and 46%.

For increase in fibres content from 0.75% to 1.5% (i.e. an increase of 100% of fibre content) the compressive strength increased only by about 15%

and 22% for B_x and C_x thermoplastic carton soil blocks The increase in fibre content has both qualitatively and quantitatively significant effect on the strength of the block in both cases of soil blocks enhanced with fibres. Although, the deformed shaped of the modelled blocks did not exactly resemble the deformed shape of the observed block from the experimental studies, there are similarities in terms of lateral expansion of the block along the length as shown in Figures 7.4. and 7.5. The next section describes the validation of the experimental studies described in Chapter six.

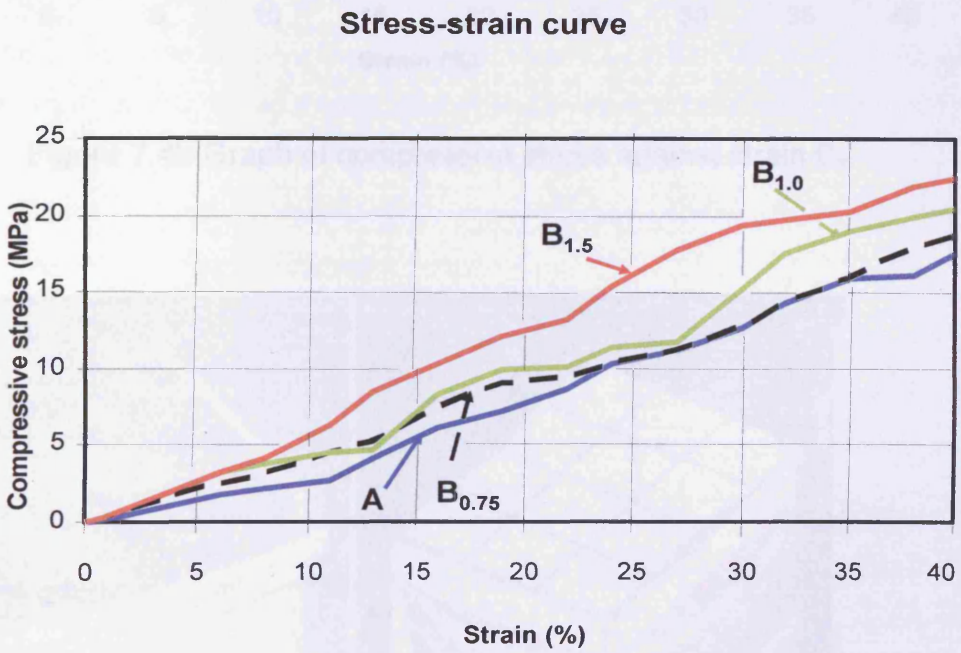


Figure 7.4a Graph of compressive stress against strain B_x

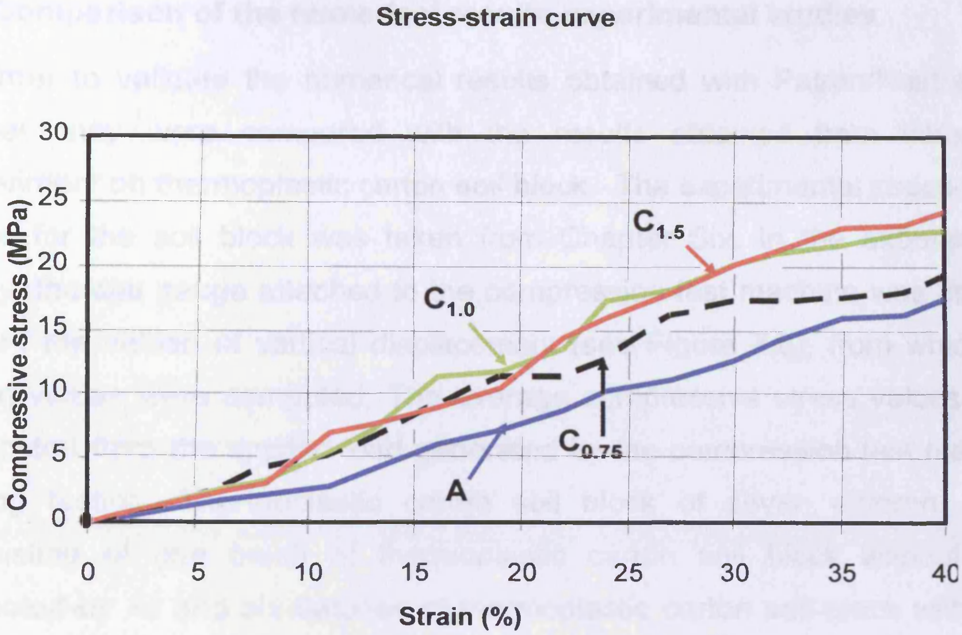


Figure 7.4b Graph of compressive stress against strain C_x

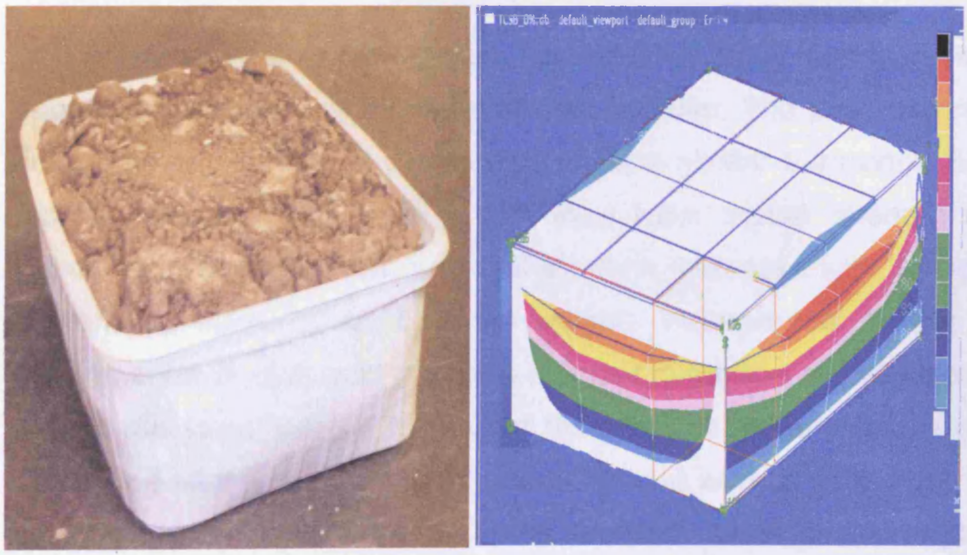


Fig. 7.5 The deformed shape of soil block
a) experimentally measured
b) FE numerical model

7.3 Comparison of the numerical results experimental studies

In order to validate the numerical results obtained with Patran/Nastran FE model, they were compared with the results obtained from laboratory experiment on thermoplastic carton soil block. The experimental stress-strain curve for the soil block was taken from Chapter Six. In the experimental study, the dial gauge attached to the compression test machine was used to record the values of vertical displacement (see Figure 7.6), from which the strain values were computed. The average compressive stress values were computed from the applied load generated by the compression test machine during testing. Thermoplastic carton soil block of seven different types consisting of one batch of thermoplastic carton soil block without fibre (denoted by A) and six batches of thermoplastic carton soil block with fibre (denoted by B and C) were made, tested and evaluated in an effort to validate the numerical approached with the required performance characteristic. Details of FEA model prediction and experimental values are in Tables 7.4 and graphically shown in Figures 7.7a-g.

The measured results from experimental study clearly demonstrate an important positive consequence of fibre addition. The predicted results from finite element numerical model also show a similar behaviour. Both sets of results indicated that TCSBs with fibre have higher strength than those without any fibre addition, and that strength increases with increase in fibre content (at least up to 1.5% by weight). Furthermore, results from both experimental studies and the finite element numerical model prediction show that at the same weight fraction of fibre, blocks enhanced with plastic fibre performed slightly better than those enhanced with oil palm fibre. This could be expected since the plastic fibres are both stiffer and stronger than the natural palm fibres.

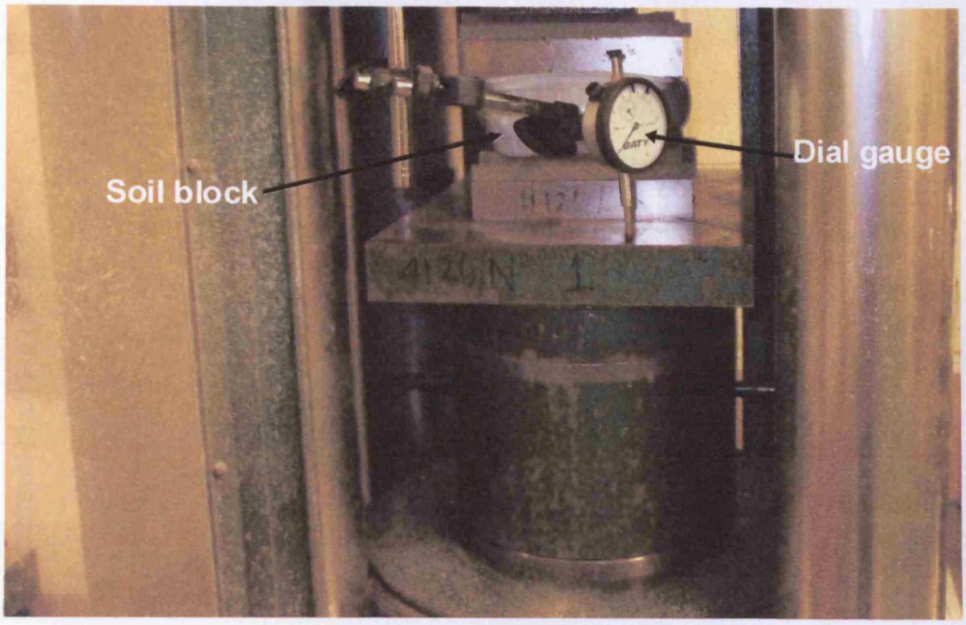


Figure 7.6 Compression testing machine with dial gauge

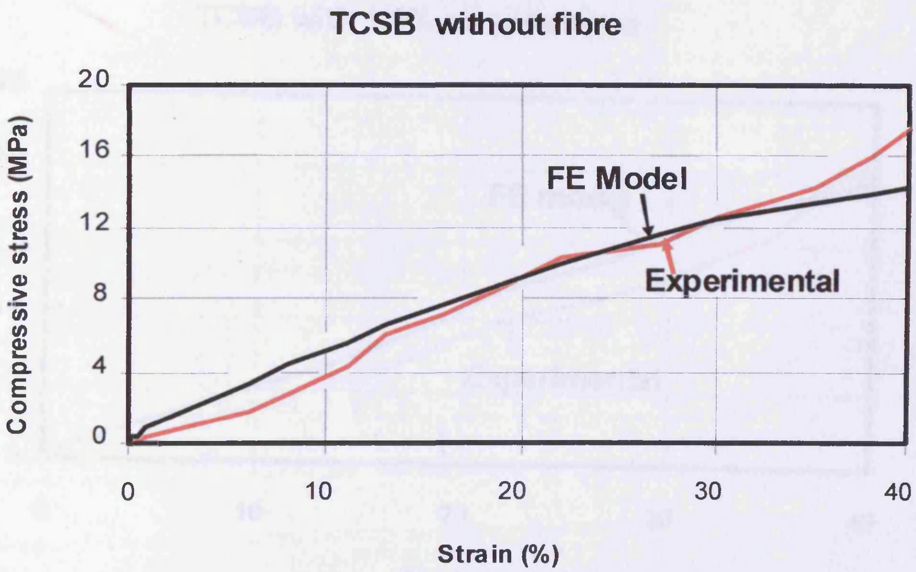


Figure 7.7a Comparison of FEA Model with test data (A)

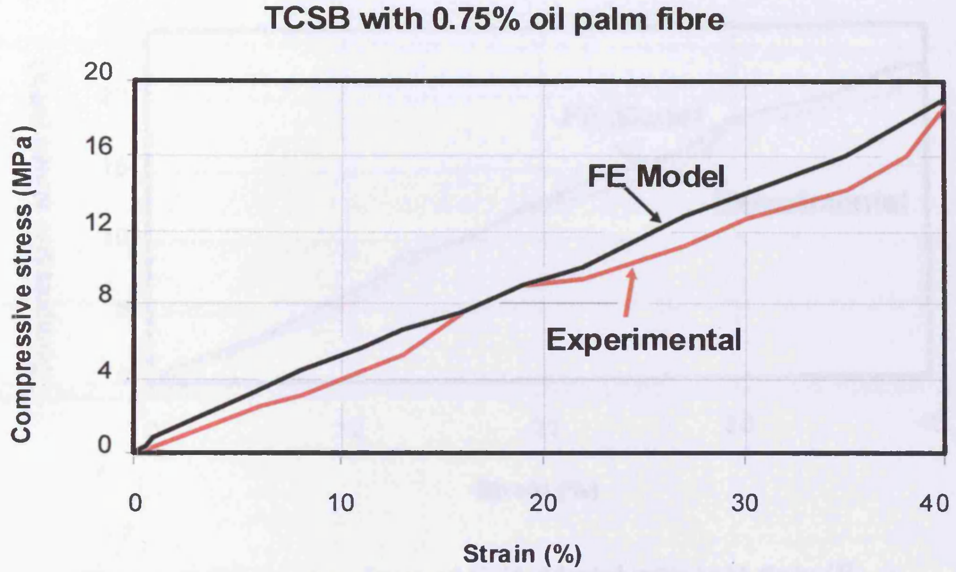


Figure7.7b Comparison of FEA Model with test data ($B_{0.75}$)

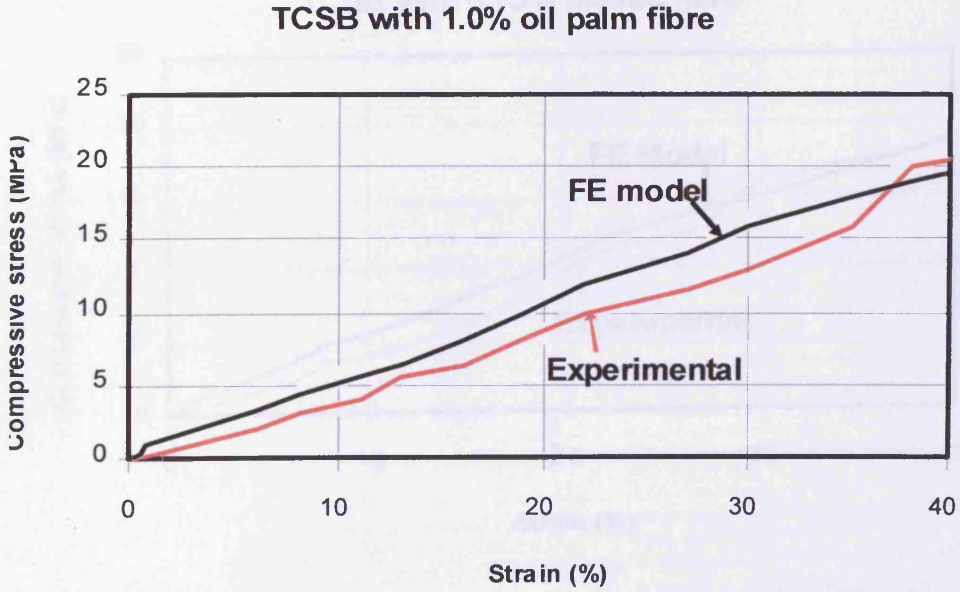


Figure 7.7c Comparison of FEA Model with test data ($B_{1.0}$)

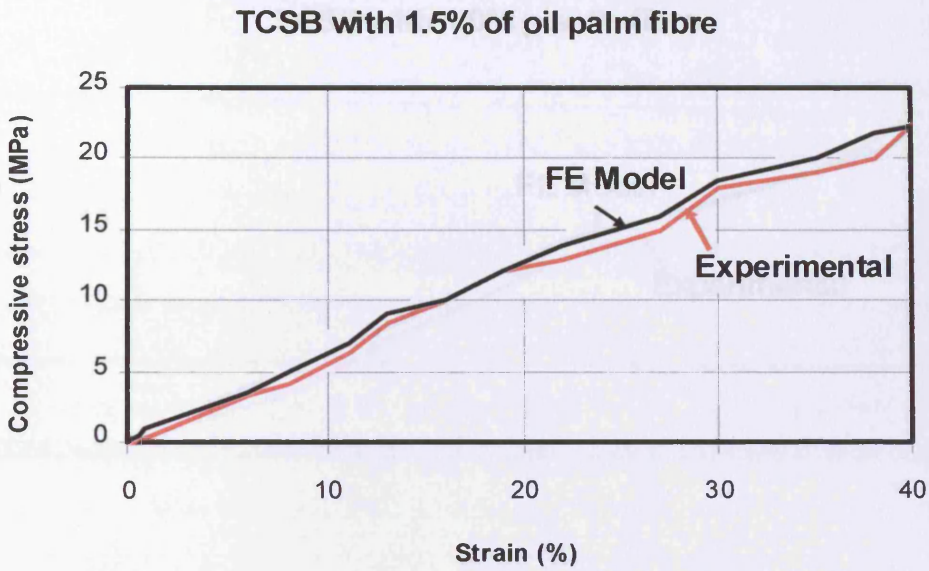


Figure 7.7d Comparison of FEA Model with test data ($B_{1.5}$)

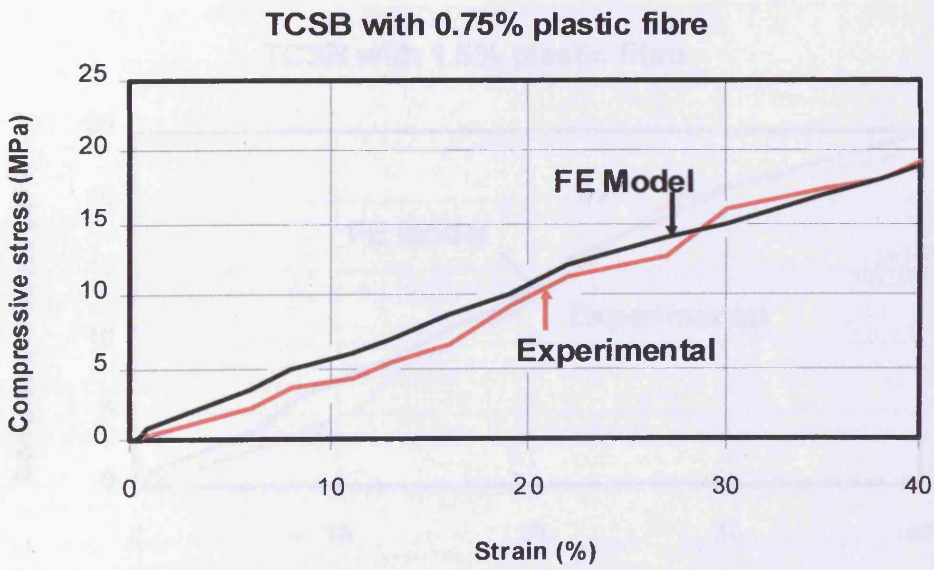


Figure 7.7e Comparison of FEA Model with test data ($C_{0.75}$)

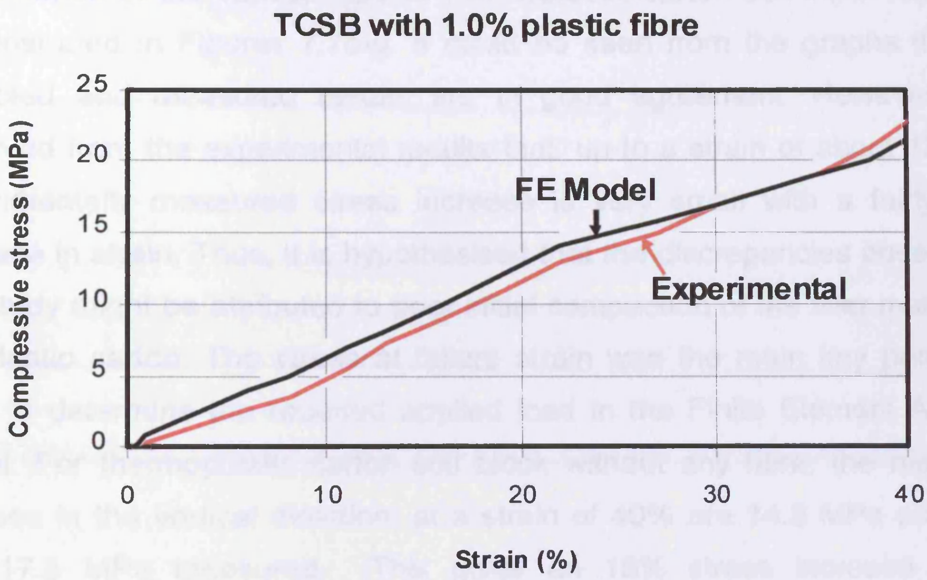


Figure 7.7f Comparison of FEA Model with test data ($C_{1.0}$)

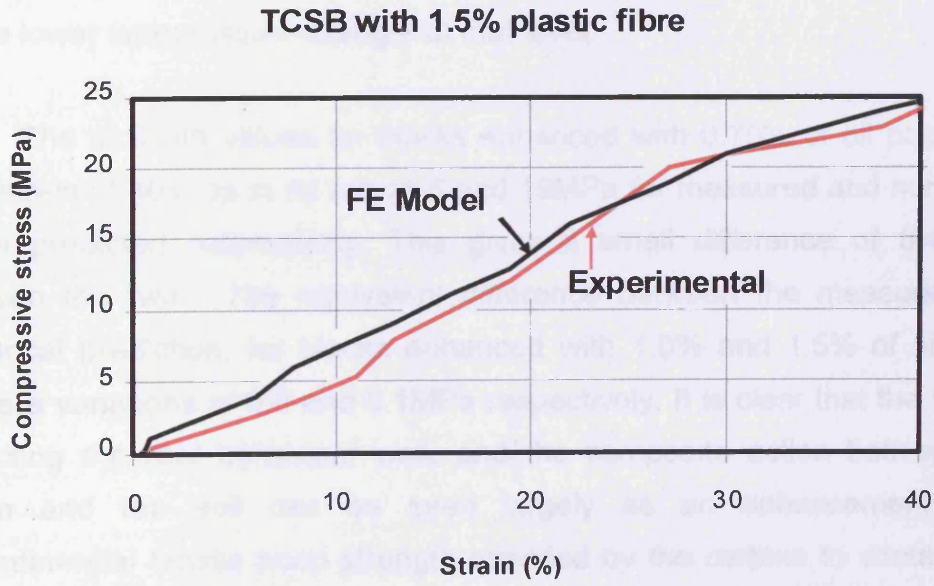


Figure 7.7g Comparison of FEA Model with test data ($C_{1.5}$)

Very good agreement between the numerical model and measured performance of the various type of thermoplastic carton soil block has been demonstrated in Figures 7.7a-g. It could be seen from the graphs that the predicted and measured results are in good agreement. However, it is observed from the experimental results that, up to a strain of about 13% the experimentally measured stress increase is very small with a fairly rapid increase in strain. Thus, it is hypothesised that the discrepancies observed in this study might be attributed to poor initial compaction of the filler material in the plastic carton. The stress at failure strain was the main key parameter used to determine the required applied load in the Finite Element Analysis model. For thermoplastic carton soil block without any fibre, the maximum stresses in the vertical direction, at a strain of 40% are 14.3 MPa predicted and 17.5 MPa measured. This gives an 18% stress increase of the experimental value over that of predicted by the numerical model (see Fig 7.7a) though for 75% of the loading range, the experimentally measured stress was lower. This might be attributed to the fact that up to about 75% loading range the soil in the thermoplastic carton was not fully compacted, hence lower compressive strength at that level.

The strength values for blocks enhanced with 0.75% of oil palm fibre (at a strain of 40% as in A) are 18.6 and 19MPa for measured and numerical model predicted respectively. This gives a small difference of 0.4 MPa between the two. The equivalent difference between the measured and numerical prediction, for blocks enhanced with 1.0% and 1.5% of oil palm fibre are variations of 0.9 and 0.1MPa respectively. It is clear that the FEA is predicting the real behaviour well, and the composite action between the carton and the soil can be seen largely as an enhancement of a circumferential tensile hoop strength provided by the cartons to contain and retain the soil (which is weak in lateral tension) in place to continue to carry the compressive force.

The same trend occurred in blocks enhanced with plastic fibre. The differences here are 0.3, 1.3 and 0.3MPa between experimental and numerical results for fibre contents of 0.75%, 1.0% and 1.5% respectively. The agreement between the predicted and experimental results is clearly good. However, the applied loads for the numerical results are generally between 15 and 36% higher than the experimentally applied load to obtain the same failure strain of 40 %. This again can be due to initial lack of good compaction in the experimental sample.

The blocks with the highest weight fraction of fibre were found to provide the best correlation between the FEA and the experimental observation in relation to both the loading path and maximum strength, see Figure 7.7d and 7.7g although, thermoplastic carton soil block enhanced with 0.75% plastic fibre by weight of soil showed excellent correlation in relation to the maximum compressive strength (see Figure 7.8).

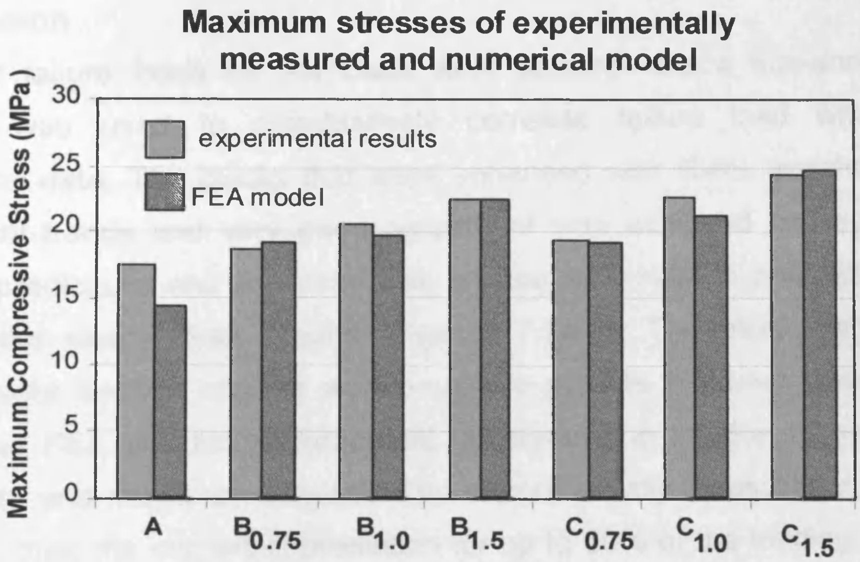


Figure 7.8 Comparison of predicted and measured maximum compressive strengths of soil block

The fibre additions to the soil did not only achieve higher strengths but also higher stresses at lower strains were achieved. The advantage is thus higher stiffness were generated by the fibre enhancement. In this respect, the use of plastic fibres as opposed to the palm fibres consistently produced the higher stiffness, as indicated in Figure 7.8. For the same fibre content of 0.75%, 1.0% and 1.5% by weight of soil, the strength at 40% strain of blocks C_x is averagely about 5.4%, 11% and 6% respectively higher than blocks B_x. This could be expected since the plastic fibres are both stiffer and stronger than the natural palm fibres.

7.4 Conclusion

A range of failure loads for the block were studied, and a trial-and-error procedure was used to quantitatively correlate failure load with the experimental data. The blocks that were enhanced with fibres matched the experimental trends and very good agreement was achieved between the numerical predictions and experimentally measured results in both size and shape of the stress-strain graphs (Figures 7.7a-g). The blocks with the highest weight fraction of fibre were found to provide the best correlation between the FEA and the experimental observation in relation to both the loading path and maximum strength. The experimentally measured stiffness were lower than the numerical prediction for up to 30% of the loading range, for all soil block samples. This might be attributed to the fact that in this loading range the soil in the thermoplastic carton was not fully compacted, and hence lower stiffness at the earlier loading range was observed.

It is also clear that the FEA is predicting the real behaviour well, and the composite action between the carton and the soil can be seen largely as an enhancement through a circumferential tensile hoop strength provided by the cartons to contain and retain the soil in place to continue to carry the compressive force.

The finite element analysis model has been shown to be a good method to rapidly investigate effects of changes to the soil carton block geometry, material and configuration. Work should continue with the validated numerical model tool to design a thermoplastic crate soil block with required performance characteristic for practical use in construction. However there is a limitation on the modelling of the interface between the plastic and the soil. This needs to be examined further before recommending the finite element model, as a reliable tool for validation of the thermoplastic crate soil block.

CHAPTER EIGHT

CONCLUSION AND RECOMMENDATIONS

8.0 Conclusions and recommendations

The research conducted during this research has increased our understanding in several areas related to the production of low-cost building materials using waste materials from agricultural and industrial waste. This chapter aims to draw together the different aspects of work done on concrete enhanced with natural fibre, cement stabilised soil block, performance of thermoplastic carton soil block and finite element model analysis to validate the thermoplastic carton soil blocks. This chapter is divided into three sections and each section makes recommendations for further work.

8.1 Natural fibre enhanced concrete

The finding of experimental investigations on the strength characteristics of concrete enhanced with rock wool and coconut fibres were reported in Chapter Four. The following conclusions can be summarised.

There was unexpected depreciation of toughness, and tensile strength for concrete matrix with addition of rock wool fibre as an enhancement. It was unexpected because earlier studies in understanding the mechanisms of fibre-reinforced cement-based material conducted by Romualdi et at. (1996) and Suredra et al. (2004), indicated that fibre addition improves the tensile strength and toughness of the concrete. Although they used steel fibres which are clearly stronger and stiffer than rock wool fibres, it was nonetheless thought that use of rock wool fibres would not actually lead to deteriorating of the properties.

The addition of coconut fibres to concrete significantly improved many of the engineering properties the concrete, notably torsion, toughness and tensile strength. The ability to resist cracking and spalling were also enhanced. However, the addition of fibres adversely affected the compressive strength, as expected, due to difficulties in compaction which consequently lead in increase of voids.

When coconut fibre was added to plain concrete, the torsional strength increased (by up to about 25%) as well as the energy-absorbing capacity, but there is an optimum weight fraction (0.5% by weight of cement) beyond which the torsional strength started to decrease again.

Similar results were also obtained for different fibre aspect ratios, where again results showed there was an optimum aspect ratio (125). An increase in fibre weight fraction provided a consistent increase in ductility up to the optimum content (0.5%) with corresponding fibre aspect ratio of 125.

Work still needs to be done to develop acceptance criteria. The major cause for concern in the use of natural fibres as enhancement to concrete or mortar is probably the durability of the material when embedded in concrete. The highly alkaline environment degrades natural fibres. Work has concentrated on developing alkali-resistant glass and on using carbon and aramid fibres, but little attention has been paid to the resin. Ways of assessing the durability of the materials are urgently needed.

From previous researchers like Gram (1983), Le Huu Do et al. (1995) Romildo et al.(2000) Savastano (2000), and Ramkrisha et al. (2004) on natural fibres in cement and concrete composite, the following disadvantages have been identified:

- high water absorption of natural fibre causes unstable volume and low cohesion between fibre between fibre and matrix and
- Natural fibre decomposes rapidly in the alkaline environment of cement and concrete.

Based on the above disadvantages the future experiments on coconut fibre-reinforced concrete and mortar should concentrate on the limitation of these disadvantages.

Given the variety of fibre materials, the number of mix constituent and method of production, it is evident that product development should be the prime future research objective. Economic methods of natural fibre production, handling, and economical and automated methods of dispensing fibres at a batching plant is needed if large quantities of fibres are going to be used in construction.

8.2 Performance of cement stabilised soil block

A local soil was chemically stabilised by cement. A better compressive strength at the dry state, and after two hours of immersion in water, was obtained with chemical stabilisation, and best values were obtained at cement content of 4%. Optimal water content was sought to get higher strength and higher durability. The highly decreased compressive strength after two hours of immersion in water, even with higher cement content, indicated that appropriate construction specification is necessary to prevent stabilised soil blocks from coming into any prolonged direct contact with rainwater.

Research in the production of building blocks over the years has revealed that, generally unrendered low-cement (<6%) and low-density (<1800kg/m^3) CSSB exhibit an unacceptably low tolerance to humid conditions and will deteriorate in less than 10 years. This deterioration is typically in the form of spalling of the exterior surface. By increasing the optimum cement content, from 4% to 8% the stability of the blocks might be greatly enhanced and becomes more acceptable for use in humid areas. Previous research into CSSB production indicated that for suitable soils doubling the cement content more than doubles the wet compressive strength (Montgomery, 2002).

The following is recommended:

- Higher cement content would certainly produce more expensive blocks for low cost housing in the deprived rural communities in Western Africa sub-region. The cement content could actually be reduced to say 6% without harming the performance of the blocks by developing a block making mould that could significantly increase in density of the blocks.
- To make CSSB a more environmentally and socially acceptable alternative building material, its production and use should be carefully controlled by increasing the density of the blocks which would lead to the use of modest amount of cement and hence low-energy requirement in its production and subsequent erection of building walls.
- It is also recommended that, taller, interlocking and hollow blocks should be explored for further reduction of cement requirement of the blocks and hence it erection.

8.3 Performance of thermoplastic carton soil block

The compressive strength obtained from the laboratory experiments on thermoplastic carton soil block was very promising. Such strength values of around 20MPa is about four times higher than chemically stabilised soil blocks stabilised without plastic cartons (from earlier work in Section 8.2). This is true even when the soil blocks had as high as 5% of cement content (by weight) which is approaching the economic limit. There is thus clearly a case for a larger scale study, and taking mitigating steps to ensure the durability of the TCSB as an alternative building material in construction of low cost housing, especially for disaster purposes and for low income earners in developing world.

It should also be noted that the proposed thermoplastic cartons and plastic fibres are actually environmentally friendly in that they are to be made from recycled plastic, which would otherwise be a waste material. Furthermore, there is no stringent specification for these cartons or fibres, since the current tests were conducted with disposable food cartons and recycled waste plastic fibres. The current newly proposed scheme of using plastic cartons with soil block thus achieves considerable improvements over the plain soil blocks (without plastic cartons) and at the same time provides a use for plastic waste which is abundant worldwide.

A range of failure loads for the block were studied, and a trial-and-error procedure was used to quantitatively correlate failure load with the experimental data. The blocks that were enhanced with fibres matched the experimental trends and very good agreement was achieved between the numerical predictions and experimentally measured results in both size and shape of the stress-strain graphs (Figures 7.7a-g). The blocks with the highest weight fraction of fibre were found to provide the best correlation between the FEA and the experimental observation in relation to both the loading path and maximum strength. At least up to 30% of the loading range, the experimentally measured loads were lower than the numerical prediction for all soil block samples. This might be attributed to the fact that in this loading range the soil in the thermoplastic carton was not fully compacted, and hence the observed lower compressive strength at the earlier loading range.

It is also clear that the FEA is predicting the real behaviour well, and the composite action between the carton and the soil can be seen largely as an enhancement of a circumferential tensile hoop strength provided by the cartons to contain and retain the soil in place to continue to carry the compressive force.

The finite element analysis model has shown to be a good method to rapidly investigate effects of changes to the soil carton block geometry, material and configuration. Work should continue with the validated numerical model tool to design a thermoplastic crate soil block with required performance characteristic for practical use in construction.

For the practical implementation of this research, ultraviolet stabilised carbon black systems are recommended for the manufacturing of the waste plastic containers which are intended to be used as the thermoplastic crate. Furthermore, it is clear that production of interlocking rectangular shape plastic crates from waste plastic containers, using injection moulding machine capable of moulding according to specifications would result in a better building block product where the individual blocks would have some additional mechanical connections between themselves. Such simple mechanical interlocking would also produce a wall more resistant to dynamic loading, as in the case of an earthquake. The interlocking shapes of these plastics soil block could also help to reduce the skill level needed for homeowners to build their own homes. In addition, several layers of blocks could be placed in the wall at a time, reducing construction time. This could be helpful in cases of shelter provision post a natural disaster.

Formulation of plastics with a minimum of two percent finely dispersed carbon black fibres, would greatly increase the weather resistance of the compound and give sufficient protection for continuous outdoor service.

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APPENDIX A

TABLES OF EXPERIMENTAL RESULTS FOR NATURAL FIBRE AS AN ENHANCEMENT OF CONCRETE

Table A1 load displacement values of rock wool fibre enhanced concrete

Applied torque (kN)					
Plain concrete	30% Rock wool content	25% rock wool content	20% rock wool content	10% rock wool content	Displacement (mm)
0.10	0.03	0.04	0.04	0.07	0.0000
0.11	0.05	0.08	0.06	0.08	0.0002
0.13	0.06	0.09	0.09	0.08	0.0005
0.13	0.08	0.11	0.08	0.08	0.0007
0.14	0.10	0.13	0.09	0.11	0.0009
0.15	0.12	0.15	0.09	0.12	0.0010
0.16	0.13	0.16	0.10	0.13	0.0012
0.17	0.15	0.19	0.10	0.13	0.0011
0.17	0.16	0.22	0.11	0.13	0.0012
0.17	0.17	0.23	0.11	0.13	0.0013
0.17	0.18	0.23	0.11	0.14	0.0015
0.19	0.20	0.26	0.13	0.15	0.0017
0.20	0.21	0.29	0.14	0.18	0.0020
0.20	0.24	0.31	0.14	0.18	0.0022
0.22	0.24	0.33	0.15	0.20	0.0024
0.23	0.27	0.34	0.15	0.23	0.0026

Continuation of Table A1

Applied torque (kN)					
Plain concrete	30% Rock wool content	25% rock wool content	20% rock wool content	10% rock wool content	Displacement (mm)
0.28	0.34	0.45	0.18	0.25	0.0034
0.28	0.35	0.47	0.19	0.26	0.0036
0.29	0.36	0.48	0.19	0.26	0.0038
0.29	0.38	0.53	0.20	0.27	0.0040
0.30	0.40	0.50	0.21	0.26	0.0042
0.32	0.42	0.54	0.21	0.26	0.0044
0.33	0.44	0.55	0.22	0.30	0.0046
0.36	0.47	0.59	0.25	0.30	0.0048
0.37	0.48	0.61	0.25	0.30	0.0050
0.37	0.48	0.62	0.25	0.34	0.0052
0.37	0.49	0.63	0.26	0.34	0.0054
0.38	0.51	0.66	0.27	0.35	0.0056
0.39	0.53	0.67	0.28	0.35	0.0058
0.40	0.53	0.69	0.29	0.38	0.0060
0.41	0.55	0.71	0.29	0.39	0.0062
0.42	0.55	0.72	0.30	0.40	0.0064
0.42	0.58	0.74	0.31	0.40	0.0066
0.43	0.59	0.77	0.30	0.41	0.0068
0.44	0.61	0.78	0.32	0.41	0.0070
0.45	0.64	0.79	0.33	0.43	0.0072
0.46	0.64	0.81	0.33	0.44	0.0074
0.47	0.65	0.83	0.33	0.44	0.0076
0.48	0.65	0.85	0.34	0.45	0.0078
0.49	0.66	0.87	0.35	0.46	0.0080
0.50	0.66	0.88	0.37	0.46	0.0082
0.52	0.67	0.91	0.37	0.47	0.0084
0.52	0.70	0.93	0.38	0.47	0.0086
0.53	0.71	0.94	0.39	0.48	0.0088
0.54	0.73	0.96	0.39	0.49	0.0090
0.55	0.74	0.97	0.41	0.49	0.0092
0.56	0.79	1.01	0.44	0.52	0.0098
0.57	0.79	1.03	0.45	0.53	0.0100
0.59	0.80	1.05	0.45	0.53	0.0101
0.59	0.82	1.07	0.46	0.54	0.0102
0.60	0.83	1.08	0.46	0.54	0.0104
0.61	0.84	1.10	0.48	0.54	0.0106
0.62	0.85	1.12	0.48	0.55	0.0108

Table A2. Torsion and twist values of rock wool fibre enhanced concrete.

Torsion (Nm)			Twist		
Plain concrete	30% rock wool content	25% rock wool content	20% rock wool content	10 % rock wool content	($x10^{-3}$ radian)
0.0	0.0	0.0	0.0	0.00	0.000
1.4	0.5	3.4	3.4	0.80	0.004
16.9	7.8	11.7	10.4	11.70	0.008
16.9	10.4	14.3	10.4	10.40	0.012
18.2	13.0	16.9	14.3	11.70	0.016
19.5	15.6	19.5	15.6	12.35	0.020
20.8	16.9	20.8	16.9	13.00	0.024
22.1	19.5	24.7	16.9	13.00	0.028
24.7	26.0	33.8	19.5	16.90	0.044
26.0	31.2	40.3	23.4	18.20	0.052
28.6	31.2	42.9	26.0	19.50	0.056
29.9	35.1	44.2	29.9	19.50	0.060
31.2	2.6	48.1	27.3	22.10	0.064
33.8	39	53.3	29.9	22.10	0.072
35.1	41.6	55.9	29.9	23.40	0.076
37.7	49.4	68.9	35.1	26.00	0.092
39.0	52.0	65.0	33.8	27.30	0.096
41.6	54.6	70.2	33.8	27.30	0.100
42.9	57.2	71.5	39.0	28.60	0.104
46.8	61.1	76.7	39.0	32.50	0.108
48.1	62.4	79.3	39.0	32.50	0.110
49.4	66.3	85.8	45.5	35.10	0.124
50.7	68.9	87.1	45.5	36.40	0.128
52.0	68.9	89.7	49.4	37.70	0.132
53.3	71.5	92.3	50.7	37.70	0.136
54.6	75.4	96.2	52.0	40.30	0.144
55.9	76.7	100.1	53.3	39.00	0.148
57.2	79.3	101.4	53.3	41.60	0.152

Continuation of Table A2

Torsion (Nm)					Twist ($x10^{-3}$ radian)
Plain concrete	30% rock wool content	25% rock wool content	20% rock wool content	10 % rock wool content	
63.7	85.8	113.1	59.8	45.5	0.172
65	85.8	114.4	59.8	48.1	0.176
72.8	102.7	131.3	67.6	57.2	0.208
74.1	102.7	133.9	68.9	58.5	0.212
76.7	106.6	139.1	70.2	59.8	0.220
78	107.9	140.4	70.2	59.8	0.224
79.3	109.2	143	70.2	62.4	0.228
80.6	110.5	145.6	71.5	62.4	0.232
84.5	114.4	152.1	75.4	65.0	0.244
94.9	131.3	178.1	85.8	74.1	0.288
96.2	133.9	180.7	88.4	76.7	0.296
97.5	135.2	182	89.7	78.0	0.300
104	143	195	96.2	80.6	0.324
105.3	144.3	197.6	94.9	79.3	0.329
107.9	148.2	204.1	100.1	85.8	0.340
110.5	148.2	205.4	100.1	87.1	0.460
111.8	149.5	205.4	101.4	87.1	0.348
113.1	150.8	208	102.7	92.3	0.352
113.1	152.1	211.9	104	93.6	0.360
115.7	154.7	215.8	104	96.2	0.368
124.8	166.4	234	114.4	104.0	0.404
131.3	175.5	247	117	110.5	0.432
131.3	175.5	248.3	117	110.5	0.436
132.6	175.5	250.9	124.8	111.8	0.440
133.9	178.1	249.6	124.8	111.8	0.444
133.9	178.1	252.2	127.4	114.4	0.448
135.2	179.4	256.1	127.4	114.4	0.452
136.5	180.7	256.1	127.4	114.4	0.456
137.8	182	257.4	127.4	117.0	0.460
145.6	187.2	265.2	133.9	123.5	0.484

Continuation of Table A2

Torsion (Nm)					Twist ($x10^{-3}$ radian)
Plain concrete	30% rock wool content	25% rock wool content	20% rock wool content	10 % rock wool content	
171.6	208.0	281.0	162.5	148.2	0.600
172.9	209.3	285.0	163.8	150.8	0.608
174.2	210.6	284.0	163.8	150.8	0.612
175.5	210.6	280.0	165.1	153.4	0.624
252.2	228.8	225.0	219.7	231.4	0.630
260.0	161.2	124.8	223.6	241.8	0.640
262.6	143.0	87.1	227.5	245.7	0.660
273.0	123.5	57.2	237.9	258.7	0.700
282.1	114.4	40.3	241.8	266.5	0.720
292.5	104.0	29.9	293.0	275.6	0.760
304.2	85.8	18.2	263.0	286.0	0.800
306.8	76.7	16.9	239.2	289.9	0.850
315.9	63.7	13.0	230.0	299.0	0.900
323.7	57.2	10.4	226.2	306.8	0.950
140.4			123.0	72.0	6.400
136.5			112.0	56.0	6.600
135.2			91.0	28.0	7.000
132.6			90.0	27.0	7.400
131.3			83.0	26.0	7.600
128.7			71.0	24.0	8.000

Table A3a Experimental data of coconut fibre enhanced concrete (WF 0.25%)

l/d =150		l/d =125		l/d=75		Plain Concrete	
Torque (kN)	Disp (mm).	Torque (kN)	Disp (mm).	Torque (kN)	Disp (mm).	Torque (kN)	Disp. (mm)
0.31	1.4516	0.16	0.0018	0.16	0.0018	0.53	0.0123
0.35	1.4508	0.23	0.0028	0.23	0.0028	0.56	0.0134
0.38	1.4500	0.28	0.0042	0.28	0.0042	0.60	0.0146
0.41	1.4492	0.33	0.0056	0.33	0.0056	0.63	0.0165
0.45	1.4485	0.40	0.0069	0.40	0.0069	0.66	0.017
0.48	1.4473	0.45	0.0079	0.45	0.0079	0.69	0.0179
0.54	1.4462	0.49	0.0089	0.49	0.0089	0.71	0.0182
0.58	1.4454	0.54	0.0099	0.54	0.0099	0.76	0.0188
0.61	1.4445	0.61	0.0112	0.61	0.0112	0.78	0.0204
0.64	1.4437	0.66	0.0123	0.66	0.0123	0.82	0.0207
0.68	1.4428	0.72	0.0134	0.72	0.0134	0.84	0.0217
0.71	1.4419	0.76	0.0142	0.76	0.0142	0.87	0.0238
0.74	1.4408	0.80	0.015	0.80	0.0150	0.91	0.0242
0.78	1.4399	0.84	0.0158	0.84	0.0158	0.94	0.0245
0.81	1.4390	0.88	0.0166	0.88	0.0166	0.98	0.0258
0.85	1.4378	0.91	0.0173	0.91	0.0173	1.02	0.0269
0.87	1.4363	0.95	0.019	0.95	0.0190	1.04	0.0277
0.91	1.4356	1.00	0.0202	1.00	0.0202	1.08	0.0294
0.95	1.4345	1.03	0.021	1.03	0.0210	1.11	0.0302
0.99	1.4336	1.07	0.022	1.07	0.0220	1.14	0.0306
1.02	1.4325	1.10	0.0234	1.10	0.0234	1.17	0.0315

Continuation of Table A3a

l/d =150		l/d =125		l/d=75		Plain Concrete	
Turque (kN)	Disp (mm).	Torque (kN)	Disp (mm).	Torque (kN)	Disp (mm).	Torque (kN)	Disp. (mm)
1.20	1.4269	1.26	0.0279	1.26	0.0279	1.30	0.0366
1.24	1.4250	1.30	0.0288	1.30	0.0288	1.32	0.0362
1.29	1.4241	1.33	0.0295	1.33	0.0295	1.35	0.0371
1.31	1.4232	1.35	0.0305	1.35	0.0305	1.37	0.0378
1.35	1.4223	1.38	0.0313	1.38	0.0313	1.39	0.0385
1.38	1.4210	1.42	0.0322	1.42	0.0322	1.41	0.0395
1.42	1.4198	1.42	0.0328	1.42	0.0328	1.43	0.0413
1.46	1.4184	1.46	0.0337	1.46	0.0337	1.46	0.0409
1.49	1.4174	1.49	0.0344	1.49	0.0344	1.47	0.0426
1.53	1.4162	1.52	0.0351	1.52	0.0351	1.49	0.0431
1.58	1.4149	1.54	0.0358	1.54	0.0358	1.54	0.0441
1.62	1.4128	1.58	0.0368	1.58	0.0368	1.58	0.0447
1.67	1.4114	1.62	0.0376	1.62	0.0376	1.61	0.0460
1.71	1.4103	1.66	0.0388	1.66	0.0388	1.65	0.0482
1.76	1.4088	1.70	0.0400	1.70	0.0400	1.69	0.0496
1.80	1.4071	1.74	0.0414	1.74	0.0414	1.71	0.0510
1.85	1.4055	1.79	0.0430	1.79	0.0430	1.74	0.0524
1.90	1.4039	1.83	0.0446	1.83	0.0446	1.78	0.0531
1.94	1.4021	1.87	0.0458	1.87	0.0458	1.81	0.0549
1.98	1.4007	1.93	0.0471	1.93	0.0471	1.85	0.0570
2.03	1.3992	1.98	0.0487	1.98	0.0487	1.89	0.0592
2.08	1.3972	2.04	0.0502	2.04	0.0502	1.93	0.0610
2.13	1.3951	2.10	0.0519	2.10	0.0519	1.97	0.0617
2.19	1.3921	2.15	0.0535	2.15	0.0535	2.01	0.0632
2.24	1.3905	2.20	0.0555	2.20	0.0555	2.04	0.0649
2.29	1.3884	2.26	0.0574	2.26	0.0574	2.08	0.0679
2.34	1.3847	2.31	0.0592	2.31	0.0592	2.11	0.0684
2.39	1.3813	2.36	0.0615	2.36	0.0615	2.15	0.0711
2.44	1.3779	2.42	0.0641	2.42	0.0641	2.19	0.0735
2.49	1.3738	2.47	0.0660	2.47	0.0660	2.22	0.0774
2.53	1.3676	2.52	0.0681	2.52	0.0681	2.26	0.0801
2.56	1.3157	2.72	0.0771	2.72	0.0771	0.78	0.2858
2.48	1.2819	2.77	0.0795	2.77	0.0795	0.59	0.3282
0.31	0.5266	2.80	0.0823	2.80	0.0823	0.54	0.3438
0.28	0.4871	2.83	0.0851	2.83	0.0851	0.51	0.3574
0.27	0.4613	2.86	0.0880	2.86	0.0880	0.47	0.3722

Continuation of Table A3a

l/d =150		l/d =125		l/d=75		Plain Concrete	
Turque (kN)	Disp (mm).	Torque (kN)	Disp (mm).	Torque (kN)	Disp (mm).	Torque (kN)	Disp. (mm)
0.17	0.2409	0.67	0.0619	0.67	0.0619	0.37	0.4545
0.16	0.2196	0.65	0.079	0.65	0.079	0.36	0.4646
0.15	0.1942	0.61	0.0969	0.61	0.0969	0.36	0.4733
0.14	0.1703	0.59	0.1128	0.59	0.1128	0.34	0.4825
0.13	0.1215	0.56	0.1407	0.56	0.1407	0.33	0.4993
0.12	0.1004	0.54	0.1536	0.54	0.1536	0.33	0.5066
0.11	0.0727	0.53	0.1645	0.53	0.1645	0.31	0.5137
0.11	0.0438	0.52	0.1733	0.52	0.1733	0.30	0.5196
0.10	0.0184	0.51	0.1821	0.51	0.1821	0.29	0.5258
0.06	0.1692	0.45	0.2403	0.45	0.2403	0.25	0.5797
0.05	0.1945	0.45	0.2482	0.45	0.2482	0.23	0.5876
0.05	0.2195	0.45	0.2551	0.45	0.2551	0.23	0.5939
0.04	0.2463	0.44	0.2643	0.44	0.2643	0.23	0.6007
0.04	0.2693	0.44	0.2716	0.44	0.2716	0.23	0.6073
0.04	0.2960	0.42	0.2806	0.42	0.2806	0.23	0.6146
0.03	0.3202	0.43	0.2877	0.43	0.2877	0.22	0.6199
0.03	0.3442	0.43	0.2947	0.43	0.2947	0.21	0.6280
0.04	0.3638	0.41	0.3022	0.41	0.3022	0.22	0.6360

Table A3b Experimental data of coconut fibre enhanced concrete (WF 0.5%)

l/d =150		l/d =125		l/d=75		Plain Concrete	
Torque (kN)	Disp (mm).	Torque (kN)	Disp (mm).	Torque (kN)	Disp (mm).	Torque (kN)	Disp. (mm)
0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000
0.20	0.0030	0.30	0.0050	0.40	0.0101	0.36	0.0087
0.22	0.0038	0.34	0.0056	0.45	0.0115	0.39	0.0094
0.26	0.0048	0.37	0.0060	0.49	0.0125	0.43	0.0099
0.29	0.0056	0.40	0.0065	0.53	0.0136	0.46	0.0114
0.32	0.0065	0.44	0.0072	0.58	0.0149	0.49	0.0111
0.35	0.0075	0.47	0.0079	0.62	0.0161	0.53	0.0123
0.38	0.0084	0.52	0.0086	0.68	0.0171	0.56	0.0134
0.42	0.0093	0.55	0.0092	0.71	0.0182	0.60	0.0146
0.43	0.0104	0.58	0.0097	0.75	0.0193	0.63	0.0165
0.47	0.0112	0.61	0.0105	0.80	0.0203	0.66	0.0170
0.50	0.0119	0.64	0.0126	0.84	0.0217	0.69	0.0179
0.53	0.0126	0.68	0.0141	0.88	0.0228	0.71	0.0182
0.55	0.0134	0.72	0.0145	0.93	0.0238	0.76	0.0188
0.58	0.0142	0.74	0.0152	0.98	0.0252	0.78	0.0204
0.61	0.0148	0.78	0.0161	1.00	0.0261	0.82	0.0207
0.64	0.0155	0.81	0.0170	1.06	0.0275	0.84	0.0217
0.67	0.0161	0.84	0.0179	1.11	0.0286	0.87	0.0238
0.69	0.0170	0.88	0.0191	1.15	0.0299	0.91	0.0242
0.73	0.0176	0.93	0.0199	1.19	0.0309	0.94	0.0245
0.75	0.0192	0.95	0.0209	1.23	0.0320	0.98	0.0258
0.79	0.0199	1.00	0.0222	1.28	0.0331	1.02	0.0269
0.82	0.0207	1.04	0.0238	1.31	0.0339	1.04	0.0277
0.86	0.0215	1.07	0.0248	1.34	0.0349	1.08	0.0294
0.89	0.0224	1.08	0.0250	1.36	0.0358	1.11	0.0302
0.93	0.0233	1.11	0.0252	1.41	0.0367	1.14	0.0306
0.97	0.0244	1.13	0.0260	1.44	0.0375	1.17	0.0315
1.00	0.0255	1.16	0.0268	1.46	0.0384	1.20	0.0327
1.04	0.0262	1.2.0	0.0275	1.49	0.0391	1.23	0.0330
1.07	0.0273	1.23	0.0281	1.53	0.0401	1.26	0.0355
1.10	0.0284	1.24	0.0290	1.56	0.0411	1.28	0.0361
1.14	0.0303	1.27	0.0299	1.61	0.0421	1.30	0.0366
1.17	0.0308	1.31	0.0309	1.66	0.0437	1.32	0.0362
1.21	0.0319	1.33	0.0323	1.70	0.0452	1.35	0.0371
1.24	0.0329	1.36	0.0331	1.74	0.0465	1.37	0.0378

Continuation of Table A3b

l/d =150		l/d =125		l/d=75		Plain Concrete	
Torque (kN)	Disp (mm).	Torque (kN)	Disp (mm).	Torque (kN)	Disp (mm).	Torque (kN)	Disp. (mm)
1.33	0.0359	1.44	0.0348	1.90	0.0509	1.43	0.0413
1.37	0.0373	1.47	0.0357	1.95	0.0520	1.46	0.0409
1.41	0.0385	1.51	0.0365	2.00	0.0541	1.47	0.0426
1.44	0.0393	1.54	0.0371	2.04	0.0561	1.49	0.0431
1.49	0.0410	1.57	0.0382	2.11	0.0574	1.54	0.0441
1.52	0.0422	1.60	0.0395	2.15	0.0594	1.58	0.0447
1.57	0.0441	1.65	0.0407	2.21	0.0612	1.61	0.0460
1.62	0.0452	1.67	0.0418	2.25	0.0625	1.65	0.0482
1.68	0.0470	1.72	0.0437	2.31	0.0649	1.69	0.0496
1.73	0.0489	1.76	0.0444	2.36	0.0670	1.71	0.0510
1.78	0.0515	1.81	0.0459	2.40	0.0686	1.74	0.0524
1.82	0.0531	1.84	0.0469	2.44	0.0709	1.78	0.0531
1.89	0.0547	1.86	0.0478	2.47	0.0729	1.81	0.0549
1.93	0.0562	1.89	0.0489	2.50	0.0747	1.85	0.0570
1.98	0.0583	1.93	0.0504	2.52	0.0764	1.89	0.0592
2.04	0.0604	1.98	0.0519	2.56	0.0785	1.93	0.0610
2.09	0.0628	2.01	0.0534	2.59	0.0803	1.97	0.0617
2.15	0.0672	2.05	0.0549	2.60	0.0826	2.01	0.0632
2.20	0.0705	2.10	0.0580	2.54	0.0852	2.04	0.0649
2.24	0.0739	2.13	0.0588	2.39	0.0862	2.08	0.0679
2.28	0.0765	2.16	0.0596	2.21	0.0802	2.11	0.0684
2.31	0.0788	2.19	0.0609	2.04	0.0670	2.15	0.0711
2.35	0.0812	2.23	0.0625	1.40	0.0136	2.19	0.0735
2.39	0.0838	2.28	0.0642	1.28	0.0006	2.22	0.0774
2.43	0.0868	2.30	0.0656	1.21	0.0101	2.26	0.0801
2.46	0.0910	2.34	0.0672	1.14	0.0187	2.28	0.0861
2.49	0.0964	2.37	0.0688	1.08	0.0272	2.28	0.0941
2.39	0.1250	2.42	0.0710	1.01	0.0377	2.26	0.1049
0.85	0.6118	2.46	0.0735	0.98	0.0437	0.78	0.2858
0.71	0.7391	2.50	0.0766	0.96	0.0481	0.59	0.3282
0.64	0.8146	2.54	0.0788	0.93	0.0531	0.54	0.3438
0.61	0.8553	2.58	0.0819	0.90	0.0574	0.51	0.3574
0.60	0.8889	2.60	0.0851	0.87	0.0619	0.47	0.3722
0.58	0.9261	2.66	0.0893	0.84	0.0664	0.46	0.3816

Continuation of Table A3b

l/d =150		l/d =125		l/d=75		Plain Concrete	
Torque (kN)	Disp (mm).	Torque (kN)	Disp (mm).	Torque (kN)	Disp (mm).	Torque (kN)	Disp. (mm)
0.48	1.6055	0.49	0.8499	0.42	0.2236	0.23	0.5939
0.47	1.6286	0.48	0.8633	0.42	0.2292	0.23	0.6007
0.48	1.6542	0.47	0.8744	0.41	0.2341	0.23	0.6073
0.47	1.6763	0.45	0.8842	0.41	0.2398	0.23	0.6146
0.47	1.7010	0.44	0.8929	0.40	0.2455	0.22	0.6199
0.43	2.0223	0.35	0.9948	0.36	0.3140	0.18	0.7082
0.43	2.0440	0.34	1.0025	0.35	0.3186	0.17	0.7141
0.43	2.0658	0.34	1.0099	0.35	0.3234	0.18	0.7205
0.43	2.0926	0.34	1.0171	0.35	0.3279	0.17	0.7278
0.42	2.1153	0.33	1.0249	0.34	0.3322	0.17	0.7338
0.42	2.1354	0.33	1.0346	0.34	0.3362	0.16	0.7412
0.42	2.1556	0.33	1.0439	0.34	0.3401	0.16	0.7477
0.42	2.1787	0.32	1.0504	0.33	0.3447	0.16	0.7542
0.42	2.2007	0.31	1.0482	0.33	0.3494	0.16	0.7610
0.42	2.2205	0.31	1.0546	0.34	0.3539	0.15	0.7675
0.42	2.2426	0.30	1.0617	0.33	0.3578	0.14	0.7733
0.42	2.2647	0.29	1.0703	0.33	0.3616	0.14	0.7796
0.41	2.2867	0.30	1.0766	0.33	0.3654	0.14	0.7860
0.42	2.3059	0.28	1.0843	0.33	0.3695	0.14	0.7935
0.40	2.5162	0.24	1.1525	0.33	0.4097	0.13	0.8553
0.41	2.5329	0.23	1.1596	0.32	0.4137	0.12	0.8612
0.41	2.6747	0.21	1.2079	0.32	0.4400	0.10	0.9063
0.39	2.6902	0.21	1.2154	0.32	0.4435	0.09	0.9123
0.39	2.7128	0.21	1.2209	0.32	0.4463	0.10	0.9189
0.40	2.7352	0.21	1.2275	0.32	0.4500	0.10	0.9246
0.32	3.6202	0.14	1.4784	0.36	0.5422	0.06	1.0951
0.31	3.6413	0.14	1.4842	0.36	0.5444	0.07	1.1013
0.30	3.6620	0.15	1.4893	0.35	0.5471	0.06	1.1088
0.27	3.7162	0.13	1.5350	0.39	0.5638	0.05	1.1703
0.26	3.7168	0.11	1.5493	0.40	0.5712	0.05	1.1786
0.26	3.7169	0.11	1.5486	0.40	0.5714	0.05	1.1787

Table A3c Experimental data of coconut fibre enhanced concrete (WF 0.5%)

l/d =150		l/d =125		l/d=75		Plain Concrete	
Torque (kN)	Disp (mm).	Torque (kN)	Disp (mm).	Torque (kN)	Disp (mm).	Torque (kN)	Disp. (mm)
0.00	0.0000	0.00	0.000	0.00	0.0000	0.00	0.0000
0.01	0.0003	0.03	0.0001	0.04	0.0001	0.03	0.0029
0.01	0.0003	0.04	0.0001	0.04	0.0001	0.03	0.0041
0.01	0.0003	0.04	0.0001	0.05	0.0001	0.03	0.0051
0.01	0.0003	0.04	0.0001	0.04	0.0001	0.04	0.0059
0.03	0.0001	0.03	0.0001	0.04	0.0001	0.03	0.0067
0.05	0.0007	0.04	0.0001	0.04	0.0001	0.03	0.0065
0.07	0.0013	0.02	0.0001	0.05	0.0001	0.03	0.0087
0.10	0.0019	0.03	0.0001	0.05	0.0004	0.03	0.0094
0.14	0.0029	0.03	0.0001	0.05	0.0004	0.03	0.0099
0.18	0.0038	0.03	0.0001	0.05	0.0004	0.03	0.0114
0.21	0.0046	0.03	0.0001	0.10	0.0020	0.03	0.0111
0.24	0.0057	0.03	0.0001	0.15	0.0026	0.02	0.0123
0.27	0.0064	0.05	0.0001	0.21	0.0042	0.03	0.0134
0.30	0.0069	0.10	0.0013	0.26	0.0049	0.02	0.0146
0.32	0.0077	0.15	0.0024	0.31	0.0069	0.04	0.0165
0.36	0.0084	0.20	0.0034	0.34	0.0078	0.05	0.0170
0.39	0.0090	0.24	0.0047	0.38	0.0080	0.09	0.0179
0.41	0.0098	0.29	0.0060	0.42	0.0098	0.12	0.0182
0.45	0.0108	0.33	0.0071	0.48	0.0114	0.17	0.0188
0.48	0.0114	0.37	0.0081	0.53	0.0125	0.20	0.0204
0.5	0.0124	0.43	0.0099	0.56	0.0134	0.24	0.0207
0.54	0.0133	0.46	0.0112	0.60	0.0142	0.27	0.0217
0.56	0.0147	0.51	0.0122	0.62	0.0145	0.30	0.0238
0.60	0.0152	0.55	0.0134	0.66	0.0159	0.33	0.0242
0.62	0.0158	0.60	0.0145	0.69	0.0167	0.36	0.0245
0.65	0.0167	0.63	0.0158	0.72	0.0166	0.39	0.0258
0.68	0.0176	0.67	0.0169	0.76	0.0176	0.43	0.0269
0.72	0.0185	0.71	0.0180	0.79	0.0185	0.46	0.0277
0.75	0.0194	0.75	0.0192	0.82	0.0194	0.49	0.0294
0.78	0.0202	0.79	0.0203	0.85	0.0204	0.53	0.0302
0.81	0.0213	0.83	0.0216	0.88	0.0220	0.56	0.0306
0.84	0.0223	0.87	0.0231	0.92	0.0225	0.60	0.0315
0.88	0.0232	0.91	0.0243	0.95	0.0232	0.63	0.0327

Continuation of Table A3c

l/d =150		l/d =125		l/d=75		Plain Concrete	
Torque (kN)	Disp (mm).	Torque (kN)	Disp (mm).	Torque (kN)	Disp (mm).	Torque (kN)	Disp. (mm)
1.24	0.0358	1.40	0.0382	1.44	0.0363	1.08	0.0441
1.26	0.0365	1.43	0.0391	1.46	0.0371	1.11	0.0447
1.29	0.0372	1.46	0.0401	1.49	0.038	1.14	0.0460
1.31	0.0381	1.48	0.0408	1.53	0.0391	1.17	0.0482
1.34	0.0389	1.51	0.0412	1.55	0.0399	1.20	0.0496
1.36	0.0398	1.52	0.0417	1.60	0.0422	1.23	0.0510
1.38	0.0405	1.55	0.0424	1.64	0.0422	1.26	0.0524
1.41	0.0413	1.59	0.0435	1.69	0.0441	1.28	0.0531
1.41	0.0419	1.63	0.0452	1.75	0.0454	1.30	0.0549
1.44	0.0434	1.67	0.0461	1.81	0.0474	1.32	0.0570
1.46	0.0437	1.71	0.0472	1.86	0.0489	1.35	0.0592
1.49	0.0445	1.76	0.0484	1.90	0.0501	1.37	0.0610
1.52	0.0450	1.79	0.0493	1.93	0.0517	1.39	0.0617
1.55	0.0458	1.83	0.0507	2.01	0.0537	1.41	0.0632
1.57	0.0467	1.87	0.0521	2.05	0.0557	1.43	0.0649
1.60	0.0480	1.93	0.0541	2.10	0.0572	1.46	0.0679
1.63	0.0490	1.97	0.0552	2.14	0.0589	1.47	0.0684
1.67	0.0502	2.01	0.0568	2.19	0.0599	1.49	0.0711
1.70	0.0519	2.05	0.0584	2.24	0.0615	1.54	0.0735
1.75	0.0530	2.10	0.0609	2.29	0.0632	1.58	0.0774
1.78	0.0558	2.13	0.0622	2.34	0.0651	1.61	0.0801
1.81	0.0568	2.17	0.0641	2.39	0.0667	1.65	0.0861
1.85	0.0578	2.20	0.0658	2.44	0.0685	1.69	0.0941
1.88	0.0597	2.23	0.068	2.49	0.07	1.71	0.1049
1.90	0.0615	2.28	0.0698	2.53	0.0717	1.74	0.2858
1.93	0.0635	2.31	0.0725	2.57	0.0738	1.78	0.3282
1.97	0.0655	2.33	0.0755	2.60	0.0763	1.81	0.3438
2.01	0.0673	2.37	0.0776	2.62	0.0798	1.85	0.3574
2.04	0.0699	2.40	0.0804	2.63	0.0842	1.89	0.3722
2.07	0.0727	2.42	0.0839	2.02	0.1422	1.93	0.3816
2.10	0.0774	2.44	0.0885	1.00	0.2269	1.97	0.3933
2.13	0.0792	2.46	0.0925	0.80	0.2849	2.01	0.4022
2.16	0.0828	2.47	0.0981	0.69	0.3345	2.04	0.4113

Continuation of Table A3c

l/d =150		l/d =125		l/d=75		Plain Concrete	
Torque (kN)	Disp (mm).	Torque (kN)	Disp (mm).	Torque (kN)	Disp (mm).	Torque (kN)	Disp. (mm)
0.42	0.6517	0.57	0.6684	0.51	0.5016	0.78	0.4993
0.41	0.6652	0.55	0.6881	0.50	0.5182	0.59	0.5066
0.40	0.6768	0.54	0.7055	0.48	0.5342	0.54	0.5137
0.39	0.6867	0.53	0.7233	0.46	0.5498	0.51	0.5196
0.38	0.6993	0.51	0.7422	0.45	0.5665	0.47	0.5258
0.37	0.7125	0.51	0.7604	0.45	0.5834	0.46	0.5332
0.36	0.7263	0.50	0.7775	0.43	0.6013	0.44	0.5403
0.37	0.7412	0.49	0.7927	0.42	0.6185	0.44	0.5475
0.36	0.7552	0.48	0.8098	0.40	0.6349	0.42	0.5534
0.36	0.7703	0.47	0.8283	0.39	0.6482	0.41	0.5599
0.34	0.7849	0.46	0.8441	0.37	0.6597	0.40	0.5665
0.34	0.8005	0.45	0.8642	0.36	0.6743	0.39	0.5738
0.34	0.8144	0.43	0.8833	0.35	0.689	0.38	0.5797
0.34	0.8280	0.43	0.9030	0.35	0.7054	0.37	0.5876
0.34	0.8393	0.42	0.9254	0.34	0.7204	0.36	0.5939
0.33	0.8541	0.41	0.9450	0.33	0.7347	0.36	0.6007
0.33	0.8688	0.40	0.9644	0.32	0.7504	0.34	0.6073
0.32	0.8825	0.40	0.9780	0.31	0.7662	0.33	0.6146
0.32	0.8976	0.40	0.9946	0.30	0.783	0.33	0.6199
0.33	0.9121	0.39	1.0114	0.30	0.7968	0.33	0.6280
0.31	0.9280	0.39	1.0271	0.29	0.8082	0.31	0.6360
0.30	0.9425	0.38	1.0417	0.29	0.8203	0.30	0.6411
0.30	0.9574	0.37	1.0573	0.29	0.8333	0.29	0.6475
0.31	0.9727	0.36	1.0719	0.28	0.8459	0.29	0.6556
0.30	0.9889	0.36	1.0858	0.28	0.8598	0.28	0.6627
0.30	1.0049	0.36	1.0981	0.26	0.8717	0.27	0.6696
0.29	1.0216	0.35	1.1101	0.27	0.885	0.26	0.6772
0.28	1.0353	0.36	1.1249	0.26	0.898	0.26	0.6838
0.29	1.0489	0.35	1.1366	0.25	0.9087	0.25	0.6897
0.28	1.0614	0.35	1.1496	0.25	0.9221	0.25	0.6956
0.28	1.0749	0.34	1.1650	0.25	0.9354	0.25	0.702
0.28	1.0874	0.34	1.1828	0.24	0.949	0.23	0.7082
0.29	1.1022	0.33	1.2018	0.24	0.963	0.23	0.7141
0.28	1.1142	0.33	1.2184	0.23	0.9754	0.23	0.7205
0.27	1.3439	0.26	1.3732	0.17	1.2119	0.18	0.840
0.26	1.3571	0.26	1.3947	0.17	1.2272	0.17	0.8461

Table A4a Computational results of torsion and twist for coconut fibre enhanced concrete (WF=0.25%)

Twist ($x 10^{-3}$ rad)	l/d =150 Torsion (Nm)	l/d =125 Torsion (Nm)	l/d=75 Torsion (Nm)	Plain concrete Torsion (Nm)
0.0000	0.0	0.0	0.0	0.0
0.0752	50.7	20.8	10.4	10.4
0.0792	55.9	24.7	10.4	10.4
0.0912	59.8	31.2	13.0	13.0
0.0984	68.9	40.3	20.8	20.8
0.1072	72.8	45.5	29.9	29.9
0.1168	78.0	49.4	36.4	36.4
0.1320	81.9	53.3	42.9	42.9
0.1360	85.8	58.5	52.0	52.0
0.1432	89.7	62.4	58.5	58.5
0.1456	92.3	70.2	63.7	63.7
0.1504	98.8	75.4	70.2	70.2
0.1632	101.4	79.3	79.3	79.3
0.1656	106.6	83.2	85.8	85.8
0.1736	109.2	88.4	93.6	93.6
0.1904	113.1	92.3	98.8	98.8
0.1936	118.3	96.2	104.0	104.0
0.1960	122.2	101.4	109.2	109.2
0.2064	127.4	105.3	114.4	114.4
0.2152	132.6	110.5	118.3	118.3
0.2216	135.2	113.1	123.5	123.5
0.2352	140.4	118.3	130.0	130.0
0.2416	144.3	123.5	133.9	133.9
0.2448	148.2	128.7	139.1	139.1
0.2520	152.1	132.6	143.0	143.0
0.2616	156.0	137.8	145.6	145.6
0.2640	159.9	143.0	150.8	150.8
0.2840	163.8	146.9	156.0	156.0
0.2888	166.4	152.1	159.9	159.9
0.2928	169.0	156.0	163.8	163.8
0.2896	171.6	161.2	169.0	169.0
0.2968	175.5	167.7	172.9	172.9
0.3024	178.1	170.3	175.5	175.5

Continuation of Table A4a

Twist (x 10^{-3} rad)	l/d =150 Torsion (Nm)	l/d =125 Torsion (Nm)	l/d=75 Torsion (Nm)	Plain concrete Torsion (Nm)
0.3408	191.1	193.7	193.7	193.7
0.3448	193.7	198.9	197.6	197.6
0.3528	200.2	205.4	200.2	200.2
0.3576	205.4	210.6	205.4	205.4
0.3680	209.3	217.1	210.6	210.6
0.3856	214.5	222.3	215.8	215.8
0.3968	219.7	228.8	221.0	221.0
0.4080	222.3	234.0	226.2	226.2
0.4192	226.2	240.5	232.7	232.7
0.4248	231.4	247.0	237.9	237.9
0.4392	235.3	252.2	243.1	243.1
0.4560	240.5	257.4	250.9	250.9
0.4736	245.7	263.9	257.4	257.4
0.4880	250.9	270.4	265.2	265.2
0.4936	256.1	276.9	273.0	273.0
0.5056	261.3	284.7	279.5	279.5
0.5192	265.2	291.2	286.0	286.0
0.5432	270.4	297.7	293.8	293.8
0.5472	274.3	304.2	300.3	300.3
0.5688	279.5	310.7	306.8	306.8
0.5880	284.7	317.2	314.6	314.6
0.6192	288.6	323.7	321.1	321.1
0.6408	293.8	328.9	327.6	327.6
0.6888	296.4	334.1	334.1	334.1
0.7528	296.4	334.1	340.6	340.6
0.8392	293.8	335.4	348.4	348.4
2.2864	101.4	332.8	353.6	353.6
2.6256	76.7	322.4	360.1	360.1
2.7504	70.2	40.3	364.0	364.0
2.8592	66.3	36.4	367.9	367.9
2.9776	61.1	35.1	371.8	371.8
3.0528	59.8	33.8	373.1	373.1
3.1464	57.2	32.5	377.0	377.0
3.2176	57.2	31.2	375.7	375.7

Continuation of Table A4a

Twist (x 10^{-3} rad)	l/d =150 Torsion (Nm)	l/d =125 Torsion (Nm)	l/d=75 Torsion (Nm)	Plain concrete Torsion (Nm)
3.7864	46.8	19.5	79.3	79.3
3.8600	44.2	18.2	76.7	76.7
3.9328	42.9	15.6	74.1	74.1
3.9944	42.9	16.9	72.8	72.8
4.0528	42.9	15.6	70.2	70.2
4.1096	40.3	14.3	68.9	68.9
4.1568	39.0	14.3	67.6	67.6
4.2064	37.7	13.0	66.3	66.3
4.2656	37.7	10.4	65.0	65.0
4.3224	36.4	11.7	63.7	63.7
4.3800	35.1	10.4	63.7	63.7
4.4272	33.8	9.1	62.4	62.4
4.4792	33.8	9.1	62.4	62.4
4.5320	32.5	7.8	61.1	61.1
4.5904	32.5	7.8	61.1	61.1
4.6376	32.5	7.8	58.5	58.5
4.7008	29.9	6.5	58.5	58.5
4.7512	29.9	6.5	58.5	58.5
4.8056	29.9	5.2	57.2	57.2
4.8584	29.9	5.2	57.2	57.2
4.9168	29.9	5.2	54.6	54.6
4.9592	28.6	3.9	55.9	55.9
5.0240	27.3	3.9	55.9	55.9
5.0880	28.6	5.2	53.3	53.3
5.1288	27.3	3.9	53.3	53.3
5.1800	26.0	2.6	52.0	52.0
5.2448	24.7	2.6	50.7	50.7
5.3016	24.7	2.6	50.7	50.7
5.3568	26.0	2.6	50.7	50.7
6.1400	19.5		44.2	44.2
7.1040	10.4		41.6	41.6
8.9736	6.5		40.3	41.6
9.0280	5.2		39.0	40.3

Table A4b Computational results of torsion and twist for coconut fibre enhanced concrete (WF=0.5%)

Twist	l/d =150	l/d =125	l/d=75	Plain concrete
(x 10^{-3} rad)	Torsion (Nm)	Torsion (Nm)	Torsion (Nm)	Torsion (Nm)
0.0000	0.0	0.00	0.0	0.0
0.0536	16.9	29.9	41.6	39.0
0.0520	20.8	32.5	46.8	42.9
0.0696	26.0	39.0	52.0	46.8
0.0752	28.6	44.2	58.5	50.7
0.0792	33.8	48.1	63.7	55.9
0.0912	37.7	52.0	68.9	59.8
0.0888	41.6	57.2	75.4	63.7
0.0984	45.5	61.1	80.6	68.9
0.1072	49.4	67.6	88.4	72.8
0.1656	79.3	101.4	130.0	106.6
0.1736	83.2	105.3	137.8	109.2
0.1904	87.1	109.2	144.3	113.1
0.1936	89.7	114.4	149.5	118.3
0.1960	94.9	120.9	154.7	122.2
0.2064	97.5	123.5	159.9	127.4
0.2152	102.7	130.0	166.4	132.6
0.2216	106.6	135.2	170.3	135.2
0.2352	111.8	139.1	174.2	140.4
0.2416	115.7	140.4	176.8	144.3
0.2448	120.9	144.3	183.3	148.2
0.2520	126.1	146.9	187.2	152.1
0.2616	130.0	150.8	189.8	156.0
0.2640	135.2	156.0	193.7	159.9
0.2840	139.1	159.9	198.9	163.8
0.2888	143.0	161.2	202.8	166.4
0.2928	148.2	165.1	209.3	169.0
0.2896	152.1	170.3	215.8	171.6
0.2968	157.3	172.9	221.0	175.5
0.3024	161.2	176.8	226.2	178.1
0.3080	163.8	180.7	232.7	180.7

Continuation of Table A4b

Twist ($x 10^{-3}$ rad)	l/d =150 Torsion (Nm)	l/d =125 Torsion (Nm)	l/d=75 Torsion (Nm)	Plain concrete Torsion (Nm)
0.3304	172.9	187.2	247.0	185.9
0.3272	178.1	191.1	253.5	189.8
0.3408	183.3	196.3	260.0	191.1
0.3448	187.2	200.2	265.2	193.7
0.3528	193.7	204.1	274.3	200.2
0.3576	197.6	208.0	279.5	205.4
0.3680	204.1	214.5	287.3	209.3
0.3856	210.6	217.1	292.5	214.5
0.3968	218.4	223.6	300.3	219.7
0.4080	224.9	228.8	306.8	222.3
0.4192	231.4	235.3	312.0	226.2
0.4248	236.6	239.2	317.2	231.4
0.4392	245.7	241.8	321.1	235.3
0.4560	250.9	245.7	325.0	240.5
0.4736	257.4	250.9	327.6	245.7
0.4880	265.2	257.4	332.8	250.9
0.4936	271.7	261.3	336.7	256.1
0.5056	279.5	266.5	338.0	261.3
0.5192	286.0	273.0	330.2	265.2
0.5432	291.2	276.9	310.7	270.4
0.5472	296.4	280.8	287.3	274.3
0.5688	300.3	284.7	265.2	279.5
0.5880	305.5	289.9	182.0	284.7
0.6192	310.7	296.4	166.4	288.6
0.6408	315.9	299.0	157.3	293.8
0.6888	319.8	304.2	148.2	296.4
0.7528	323.7	308.1	140.4	296.4
0.8392	310.7	314.6	131.3	293.8
2.2864	110.5	319.8	127.4	101.4
2.6256	92.3	325.0	124.8	76.7
2.7504	83.2	330.2	120.9	70.2
2.8592	79.3	335.4	117.0	66.3
2.9776	78.0	338.0	113.1	61.1
3.0528	75.4	345.8	109.2	59.8
3.1464	72.8	348.4	102.7	57.2

Continuation of Table A4b

Twist ($x\ 10^{-3}$ rad)	l/d =150 Torsion (Nm)	l/d =125 Torsion (Nm)	l/d=75 Torsion (Nm)	Plain concrete Torsion (Nm)
3.2176	70.2	351.0	97.5	57.2
3.2904	70.2	349.7	92.3	54.6
3.3624	70.2	323.7	88.4	53.3
3.4312	70.2	141.7	84.5	52.0
3.4952	68.9	123.5	83.2	50.7
3.5616	68.9	114.4	80.6	49.4
3.8600	67.6	97.5	74.1	44.2
4.0528	66.3	89.7	70.2	42.9
4.1096	66.3	87.1	68.9	40.3
4.1568	66.3	85.8	66.3	39.0
4.2064	65.0	83.2	65.0	37.7
4.2656	66.3	80.6	63.7	37.7
4.3224	65.0	79.3	63.7	36.4
5.1288	59.8	53.3	50.7	27.3
5.1800	59.8	52.0	50.7	26.0
5.2448	59.8	50.7	49.4	24.7
5.3016	58.5	50.7	49.4	24.7
5.3568	58.5	49.4	49.4	26.0
5.4176	58.5	48.1	48.1	24.7
5.4704	55.9	46.8	49.4	23.4
5.9816	54.6	42.9	44.2	20.8
6.0336	54.6	41.6	42.9	20.8
6.0880	54.6	40.3	42.9	20.8
6.1400	54.6	40.3	44.2	19.5
6.1864	54.6	39.0	42.9	18.2
6.7688	53.3	32.5	42.9	15.6
6.7992	53.3	31.2	41.6	14.3

Table A4c Computational results of torsion and twist for coconut fibre enhanced concrete (WF=0.75%)

Twist (x 10^{-3} rad)	l/d =150 Torsion (Nm)	l/d =125 Torsion (Nm)	l/d=75 Torsion (Nm)	Plain concrete Torsion (Nm)
0.0000	0.0	0.0	0.0	0.00
0.0520	6.5	5.2	5.2	3.75
0.0696	9.1	2.6	6.5	3.75
0.0752	13.0	3.9	6.5	3.75
0.0792	18.2	3.9	6.5	3.75
0.0912	23.4	3.9	6.5	3.75
0.0888	27.3	3.9	13.0	3.75
0.0984	31.2	3.9	19.5	2.50
0.1072	35.1	6.5	27.3	3.75
0.1168	39.0	13.0	33.8	2.50
0.1320	41.6	19.5	40.3	5.00
0.1360	46.8	26.0	44.2	6.25
0.1432	50.7	31.2	49.4	11.25
0.1456	53.3	37.7	54.6	15.00
0.1504	58.5	42.9	62.4	21.25
0.1632	62.4	48.1	68.9	25.00
0.1656	65.0	55.9	72.8	30.00
0.1736	70.2	59.8	78.0	33.75
0.1904	72.8	66.3	80.6	37.50
0.1936	78.0	71.5	85.8	41.25
0.1960	80.6	78.0	89.7	45.00
0.2064	84.5	81.9	93.6	48.75
0.2152	88.4	87.1	98.8	53.75
0.2448	105.3	107.9	114.4	70.00
0.2520	109.2	113.1	119.6	75.00
0.2640	117.0	123.5	130.0	82.50

Continuation of Table A4c

Twist (x 10^{-3} rad)	l/d =150 Torsion (Nm)	l/d =125 Torsion (Nm)	l/d=75 Torsion (Nm)	Plain concrete Torsion (Nm)
0.2928	127.4	137.8	144.3	95.00
0.2896	132.6	143.0	149.5	97.50
0.2968	135.2	148.2	152.1	102.50
0.3024	139.1	154.7	157.3	105.00
0.3080	141.7	157.3	161.2	108.75
0.3160	145.6	162.5	166.4	113.75
0.3304	148.2	166.4	170.3	117.50
0.3272	153.4	170.3	174.2	122.50
0.3408	156.0	172.9	179.4	127.50
0.3448	158.6	178.1	182.0	130.00
0.3528	161.2	182.0	187.2	135.00
0.3576	163.8	185.9	189.8	138.75
0.3680	167.7	189.8	193.7	142.5
0.3856	170.3	192.4	198.9	146.25
0.3968	174.2	196.3	201.5	150.00
0.4080	176.8	197.6	208.0	153.75
0.4192	179.4	201.5	213.2	157.50
0.4248	183.3	206.7	219.7	160.00
0.4392	183.3	211.9	227.5	162.5
0.4560	187.2	217.1	235.3	165.00
0.4736	189.8	222.3	241.8	168.75
0.4880	193.7	228.8	247.0	171.25
0.4936	197.6	232.7	250.9	173.75
0.5056	201.5	237.9	261.3	176.25
0.5192	204.1	243.1	266.5	178.75
0.5432	208.0	250.9	273.0	182.50
0.5472	211.9	256.1	278.2	183.75
0.5688	217.1	261.3	284.7	186.25
0.5880	221.0	266.5	291.2	192.50
0.6192	227.5	273.0	297.7	197.50
0.6408	231.4	276.9	304.2	201.25
0.6888	235.3	282.1	310.7	206.25

Continuation of Table A4c

Twist ($\times 10^{-3}$ rad)	l/d =150 Torsion (Nm)	l/d =125 Torsion (Nm)	l/d=75 Torsion (Nm)	Plain concrete Torsion (Nm)
3.2176	276.9	319.8	104.0	251.25
3.2904	280.8	321.1	89.7	255.00
3.3624	282.1	321.1	81.9	260.00
3.4312	284.7	118.3	79.3	263.75
3.4952	270.4	100.1	76.7	268.75
3.5616	202.8	92.3	75.4	273.75
3.6360	63.7	88.4	74.1	277.50
3.7168	61.1	84.5	70.2	282.50
3.7864	57.2	79.3	70.2	285.00
3.8600	57.2	78.0	68.9	285.00
3.9328	55.9	75.4	67.6	282.50
3.9944	54.6	74.1	66.3	97.50
4.0528	53.3	71.5	65.0	73.75
4.1096	52.0	70.2	62.4	67.50
4.1568	50.7	68.9	59.8	63.75
4.2064	49.4	66.3	58.5	58.75
4.2656	48.1	66.3	58.5	57.50
4.3224	46.8	65.0	55.9	55.00
4.3800	48.1	63.7	54.6	55.00
4.4272	46.8	62.4	52.0	52.5.0
4.4792	46.8	61.1	50.7	51.25
4.5320	44.2	59.8	48.1	50.00
4.5904	44.2	58.5	46.8	48.75
4.6376	44.2	55.9	45.5	47.50
4.7008	44.2	55.9	45.5	46.25
4.7512	44.2	54.6	44.2	45.00
4.8056	42.9	53.3	42.9	45.00
4.8584	42.9	52.0	41.6	42.50
4.9168	41.6	52.0	40.3	41.25
4.9592	41.6	52.0	39.0	41.25

Continuation of Table A4c

Twist	l/d =150	l/d =125	l/d=75	Plain concrete
(x 10^{-3} rad)	Torsion (Nm)	Torsion (Nm)	Torsion (Nm)	Torsion (Nm)
5.1288	39	49.4	37.7	37.50
5.1800	39	48.1	37.7	36.25
5.2448	40.3	46.8	36.4	36.25
5.3016	39.0	46.8	36.4	35.00
5.3568	39.0	46.8	33.8	33.75
5.4176	37.7	45.5	35.1	32.5
5.4704	36.4	46.8	33.8	32.5
5.5176	37.7	45.5	32.5	31.25
5.5648	36.4	45.5	32.5	31.25
5.6160	36.4	44.2	32.5	31.25
5.6656	36.4	44.2	31.2	28.75
5.7128	37.7	42.9	31.2	28.75
5.7640	36.4	42.9	29.9	28.75
6.5776	33.8	35.1	22.1	23.75
6.6288	33.8	33.8	23.4	22.5
6.6744	33.8	33.8	23.4	21.25
6.7200	35.1	33.8	22.1	22.5
6.7688	33.8	33.8	22.1	21.25
6.7992	35.1	33.8	22.1	21.25
6.8424	35.1	33.8	20.8	20.00
6.8896	33.8	32.5	20.8	20.00
6.9464	35.1	33.8	22.1	20.00
6.9984	33.8	32.5	20.8	20.00
7.0456	35.1	32.5	20.8	18.75
7.1040	35.1	32.5	20.8	17.50
7.1512	35.1	32.5	20.8	17.50
7.2096	33.8	32.5	19.5	17.50
7.2504	33.8	32.5	19.5	17.50
7.2984	35.1	31.2	19.5	16.25
7.3512	35.1	32.5	19.5	16.25
7.3968	33.8	31.2	19.5	16.25
7.4408	33.8	31.2	18.2	16.25
7.4952	33.8	29.9	18.2	16.25
8.0160	33.8	26.0	15.6	12.50

APPENDIX B

DATA ON SOIL USED AND SOIL BLOCK

Table B1 Particle size distribution

Sieve opening (mm)	Retained Soil Wt (g)	Retained Soil (%)	Percentage Passing (%)	Soil classification	Soil fraction (%)
20.00	0	0.0	100	Gravel >2 mm	43
10.00	274	9.0	91		
5.00	604	20.0	71		
4.00	108	5.0	66		
2.36	303	9.0	57		
1.18	386	12.0	45	Sand silt 2-0.1 mm	46.5
0.60	367	11.5	33.5		
0.30	348	11.0	22.5		
0.15	174	6.0	16.5		
0.06	510	16.4	0.1	Fine	10.5

Table B2a. Dry Density/ moisture content relationships
Soil sample without stabiliser (data from compaction test)

Water content by weight of soil	6%	8%	10%	11%	12%	13%	14%	15%
Mass of mould + base + compacted specimen (M_2) g	6517	6576	6679	6700	6726	6738	6725	6720
Mass of mould + base(M_1) g	4648	4648	4648	4648	4648	4648	4648	4648
Mass of specimen (M_2-M_1) g	1869	1928	2031	2052	2078	2090	2077	2072
$\rho = \frac{(M_2 - M_1)x10^{-3}}{10^{-3}} k$	1869	1928	2031	2052	2078	2090	2077	2072
$\rho_d = \frac{100\rho}{100+w}$ Mg/m^3	1690	1740	1778	1797	1813	1786	1766	1750
Moisture content determination								
Container No.	Z18	Z19	E4	Z	5	12	L	Z1
Mass of container + wet specimen (M_3)	46.6	46.9	49.1	47.0	46.5	47.6	47.3	47.4
Mass of container + dry specimen Mass of container (M_4) g	43.8	43.9	45.9	43.5	42.8	43.2	42.8	42.7
Mass of container (M_5) g	17.4	17.2	21.7	17.1	17.6	17.4	17.3	17.4
Mass of moisture (M_3 -M_4) g	2.8	3.0	3.2	3.5	3.7	4.4	4.5	4.7
Mass of dry soil (M_4 –M_5) g	26.3	26.7	24.2	26.4	25.2	25.8	25.5	25.3
$w = \frac{M_3 - M_4}{M_4 - M_5} x100$ %	10.6	11.2	13.0	13.2	14.6	17	17.6	18.5

Table B2b Dry Density/ moisture content relationships
(Soil sample with 5% of cement as stabiliser)

Water content(by weight of soil)	6%	8%	10%	11%	12%	13%	14%	15%
Mass of mould + base + compacted specimen (M_2) g	6509	6569	6685	6719	6748	6738	6733	6725
Mass of mould + base (M_1) g	4648	4648	4648	4648	4648	4648	4648	4648
Mass of specimen g	1861	1921	2037	2071	2100	2090	2085	2077
$\rho = \frac{(M_2 - M_1)x10^{-3}}{10^{-3}}$ (kg/m^3)	1861	1921	2037	2071	2100	2090	2085	2077
$\rho_d = \frac{100\rho}{100+w}$ kg/m^3	1634	1684	1784	1813	1797	1781	1774	1719
Moisture content determination								
Container No.	17				Z5		Z2	E
Mass of container + wet specimen (M_3)	39.4	46.4	48.6	37.7	45.2	46.0	47.9	47.8
Mass of container + dry specimen Mass of container (M_4) (g)	36.2	42.9	44.9	34.2	41.2	41.8	43.4	42.5
Continuation of Table B2b								
Mass of container (M_5) g	16.2	18.1	18.9	17.2	17.4	17.4	17.8	17.1
Mass of moisture ($M_3 - M_4$)	3.2	3.5	3.7	3.5	4.01	4.21	4.5	5.3
Mass of dry soil ($M_4 - M_5$) g	23.0	24.8	26.0	23.6	23.8	24.4	25.6	25.4
$w = \frac{M_3 - M_4}{M_4 - M_5} x100$	13.9	14.1	14.2	14.8	16.8	17.3	17.5	20.8

Table B2c Dry Density/ moisture content relationships
(Soil sample with 4% of cement as stabiliser)

Test No.	6%	8%	10%	11%	12%	13%
Mass of mould + base + compacted specimen (M_2) g	6493	6595	6614	6668	6680	6672
Mass of mould + base (M_1) g	4648	4648	4648	4648	4648	4648
Mass of specimen g	1845	1947	1966	2020	2032	2024
$\rho = \frac{(M_2 - M_1)x10^{-3}}{10^{-3}} kg/m^3$	1845	1947	1966	2020	2032	2024
$\rho_d = \frac{100\rho}{100+w}$ kg/m^3	1724	1760	1730	1767	1766	1758
Moisture content determination						
Container No.	K	F	J	D	L	E
Mass of container + wet specimen (M_3)	44.3	45.5	45.2	45.7	45.7	46.3
Mass of container + dry specimen Mass of container (M_4) (g)	42.5	42.8	42.0	42.0	42.0	43.3
Mass of container (M_5) (g)	17.3	17.3	17.1	17.3	17.5	17.4
Mass of moisture (M_3 –M_4) g	1.78	2.7	3.4	3.6	3.7	3.9
Mass of dry soil (M_4 –M_5)	25.2	25.5	24.5	24.8	24.6	25.9
$w = \frac{M_3 - M_4}{M_4 - M_5} x100$	7.0	10.6	13.6	14.3	15.03	12.1

Table B2d Dry Density/ moisture content relationships
(Soil sample with 3% of cement as stabiliser)

Test No.	6%	8%	10%	11%	12%	13%
Mass of mould + base + compacted specimen(M_2 g	6509	6569	6621	6666	6659	6605
Mass of mould + base (M_1) g	4648	4648	4648	4648	4648	4648
Mass of specimen g	1861	1921	1973	2018	2011	1957
$\rho = \frac{(M_2 - M_1)x10^{-3}}{10^{-3}} kg/m^3$	1861	1921	1973	2018	2011	1957
$\rho_d = \frac{100\rho}{100 + w}$ kg/m^3	1634	1684	1743	1791	1755	1699
Moisture content determination						
Container No.	Z2	Z5	5	8	12	17
Mass of container + wet specimen (M_3)	39.4	46.4	45.2	45.1	45.6	45.2
Mass of container + dry specimen Mass of container (M_4) g	36.2	42.9	42.0	42.0	42.0	41.5
Mass of container (M_5) g	16.2	17.6	17.3	17.4	17.4	17.4
Mass of moisture (M_3 –M_4 g	3.2	3.5	3.22	3.14	3.6	4.21
Mass of dry soil (M_4 –M_5)	23.0	24.8	24.5	24.7	24.6	24.4
$w = \frac{M_3 - M_4}{M_4 - M_5} x100$	13.9	14.1	13.2	12.7	14.6	15.2

Table B2e Dry Density/ moisture content relationships
(Soil sample with 2% of cement as stabiliser)

Test No.	6%	8%	10%	11%	12%	13%
Mass of mould + base + compacted specimen (M_2) g	6509	6569	6653	6686	6674	6660
Mass of mould + base (M_1) g	4648	4648	4648	4648	4648	4648
Mass of specimen g	1861	1921	2005	2031	2026	2012
$\rho = \frac{(M_2 - M_1)x10^{-3}}{10^{-3}}$ kg/m^3	1861	1921	2005	2031	2026	2012
$\rho_d = \frac{100\rho}{100+w}$ kg/m^3	1634	1684	1699	1797	1647	1635
Moisture content determination						
Container No.	Q	P	E	F	H	I
Mass of container + wet specimen (M_3)	39.4	46.4	45.6	44.2	45.6	45.3
Mass of container + dry specimen Mass of container (M_4) g	36.2	42.9	42	41	42	40.1
Mass of container (M_5) g	16.2	18.1	17.2	16.9	17.1	17.4
Mass of moisture ($M_3 - M_4$) g	3.2	3.5	4.4	3.3	3.6	5.3
Mass of dry soil ($M_4 - M_5$)	23.0	24.8	23.9	24.1	25.0	22.7
$w = \frac{M_3 - M_4}{M_4 - M_5} x 100$	13.9	14.1	18.0	13.0	24.0	23.0

Table B3 Dry Density and Moisture content Relationship
of soil block (preliminary test)

Water content (%)	10	11	12
Average Mass of block (M_1, kg)	1.1	1.4	1.42
Bulk Density, ρ (kg/m^3) $\rho = \frac{M_1}{V}$	2417.6	3076.2	3120.8
Dry Density $\rho_d = \frac{100\rho}{100+w}$ (kg/m^3)	1981.6	2522.1	2457.4
Mass of dry block (M_2)	0.9	1.2	1.1
Mass of moisture (M_3)	0.2	0.3	0.3
Moisture content $w = \frac{M_3}{M_2} x 100$	22%	22%	27%

Table B4 Results of plastic limit determination

Plastic limit	1	2	Moisture content
Container no.	8	7	23.75%
Mass of wet soil + container (g)	29.28	22.82	
Mass of dry soil + container (g)	27.97	21.43	
Mass of container (g)	22.91	15.67	
Mass of moisture (g)	1.19	1.39	
Mass of dry soil (g)	5.06	5.76	
Moisture content (%)	23.50	24.00	

Table B5 Result of density against time of soil at natural state (preliminary test)

Specimen	Time (hrs) / Volume (m^3)	Density (kg/m^3) with time in hours								
		0hrs	2	4	6	12	18	24	30	36
A_{10}	0.0036	1844	1841	1833	1825	1811	1794	1788	1780	1780
	0.0035	1806	1803	1797	1789	1780	1774	1771	1769	1769
A_{11}	0.0031	1774	1771	1764	1761	1754	1748	1745	1745	1741
	0.0031	1777	1774	1767	1764	1754	1748	1748	1744	1744
A_{12}	0.0015	1760	1753	1747	1733	1727	1727	1727	1727	1727
	0.0015	1780	1773	1767	1760	1753	1753	1733	1733	1733

Table B6 Mechanical characteristics of stabilised soil block

Specimen	Dry Mass, (kg)	Volume mm^3	Dry density (kg/M^3)		Compressive strength (MPa)		
			Value	Average	Compressive Force (kN)		compressive strength (MPa)
					value	Ave	
A_{10}	2.73 2.73 2.79	140x140x80 140x140x79 140x140x80	1706 1706 1831	1748	77.9 79.8 85.3	81.00	4.13
A_{11}	2.64 2.72 2.64	138x140x76 138x140x76 140x140x76	1650 1700 1867	1739	68.9 68.7 68.2	68.60	3.61
A_{12}	2.86 2.86 2.67	142x140x76 138x140x76 137x140x76	1786 1626 1669	1713	86.2 92.1 93.2	90.50	4.62
B_{10}	3.04 3.00 2.85	140x140x87 140x140x82 140x140x80	1694 1725 1881	1813	94.6 103.0 90.9	96.17	4.91
B_{11}	2.71 2.76 3.01	140x140x80 140x140x86 140x140x80	1757 1694 1843	1767	56.5 59.1 57.51	56.93	2.90
B_{12}	2.85 2.71 2.90	140x140x80 140x140x82 140x140x80	1700 1906 1775	1765	88.7 74.5 57.8	63.67	3.20

Continuation Table B6

Specimen	Dry Mass (kg)	Volume m^3	Dry density (kg/m^3) Value	Ave.	Compressive strength (MPa) Compressive Force (kN) value	Ave,	Comp. strength (MPa)
C_{10}	2.72 3.05 2.84	140x140x80 140x140x80 140x140x80	1700 1906 1775	1793	79.6 84.9 83.1	82.5	4.2
C_{11}	2.99 2.90 2.93	140x140x80 140x140x85 140x140x80	1868 1759 1831	1819	100.6 95.7 98.2	98.2	5.01
C_{12}	2.88 2.91 3.31	140x140x85 140x140x80 140x140x80	1694 1818 1897	1802	66.9 61.9 65.4	63.4	3.24
D_{10}	2.96 3.0 3.01	140x140x80 140x140x79 140x140x80	1778 1681 1875	1778	77.7 71.2 68.2	72.4	3.69
D_{11}	3.02 2.69 3.00	140x140x83 140x140x80 140x140x85	1819 1875 1806	1855	100.2 100.6 101.5	100.7	5.14
D_{12}	2.60 2.73 2.87	140x140x86 140x140x86 140x140x86	1625 1706 1793	1707	65.5 68.7 64.4	67.2	3.40
E_{10}	2.93 2.93 2.80	140x140x80 140x140x80 140x140x80	1851 1831 1750	1811	87.8 84.7 93.3	93.0	4.75
E_{11}	3.04 3.07 3.06	140x140x80 140x140x85 140x140x80	1900 1918 1913	1910	106.6 107.7 108.8	107.0	5.46
E_{12}	2.91 2.92 2.99	140x140x80 140x140x85 140x140x80	1818 1825 1868	1837	77.3 82.2 80.5	80.0	4.80

Table B7 Mechanical characteristics of stabilised soil block

Specimen	Cement content	Moist. Content	Volume mm^3	Compressive Force (kN) value	Compressive Force (kN) ave	Wet compressive strength (MPa)
A_{10}	0	10	140x140x80 140x140x79 140x140x80	-	-	-
A_{11}	0	11	138x140x76 138x140x76 140x140x76	-	-	-
A_{12}	0	12	142x140x76 138x140x76 137x140x76	19.8 16.3 16.8	17.6	-
B_{10}	2	10	140x140x87 140x140x82 140x140x80	17.0 16.2 16.7	16.8	0.9
B_{11}	2	11	140x140x80 140x140x86 140x140x80	17.0 16.3 16.4	16.6	0.86
B_{12}	2	12	140x140x80 140x140x82 140x140x80	37.0 39.5 37.6	38.0	0.85
C_{10}	3	10	140x140x80 140x140x79 140x140x80	38.7 37.9 38.6	38	1.9
C_{11}	3	11	138x140x76 138x140x76 140x140x76	33.6 30.7 29.9	31	1.94
C_{12}	3	12	142x140x76 138x140x76 137x140x76	23.6 20.7 19.9	21	1.9

Continuation Table B7

Specimen	Cement content	Moist. Content	Volume mm^3	Compressive Force (kN)		Wet comp. strength (MPa)
				value	Ave.	
D_{10}	4	10	140x140x87 140x140x82 140x140x80	40.6 43.0 42.1	42.1	2.14
D_{11}	4	11	140x140x80 140x140x86 140x140x80	40.4 48.8 44.6	44.6	2.25
D_{12}	4	12	140x140x80 140x140x82 140x140x80	34.1 35.4 33.0	34.1	1.74
E_{10}	5	10	140x140x80 140x140x79 140x140x80	45.3 46.7 41.4	44.5	2.27
E_{11}	5	11	138x140x76 138x140x76 140x140x76	53.8 54.9 54.1	54.1	2.76
E_{12}	5	12	142x140x76 138x140x76 137x140x76	40.1 42.9 39.0	41.0	2.10

Table B8 Experimental result and determination of water absorption Capillary absorption determination

Specimen	Surface area (m^2)		Mass (M)				Mass of water absorbed(M_w - M_d	Water absorption coefficient (C_b)
			Dry (M_d)		Wet (M_w)			
	value	average	values	average	value	average		
A_{10}	0.026 0.025 0.025	0.026	6.39 6.20 6.30	6.30	6.48 6.38 6.47	6.44	0.14	16.8
A_{11}	0.023 0.022 0.022	0.022	5.41 5.33 5.36	5.37	5.57 5.50 5.52	5.52	0.15	21.3
A_{12}	0.011 0.011 0.011	0.011	2.59 2.62 2.59	2.60	2.61 2.66 2.65	2.64	0.06	17.1
B_{10}	0.011 0.011 0.011	0.011	2.89 2.82 2.87	2.86	2.94 2.87 2.92	2.91	0.05	14.2
B_{11}	0.012 0.012 0.012	0.012	2.89 2.74 2.82	2.82	2.96 2.81 2.88	2.89	0.07	19.9
B_{12}	0.011 0.011 0.011	0.011	2.53 2.91 2.71	2.72	2.59 2.97 2.77	2.78	0.06	15.6
C_{10}	0.012 0.012 0.012	0.012	3.12 3.05 3.08	3.09	3.17 3.09 3.13	0.13	0.04	11.3
C_{11}	0.012 0.012 0.012	0.012	3. 01 3.01 3.05	3.02	3.04 3.07 3.08	3.06	0.04	10.4
$C_{3/12}$	0.011 0.011 0.011	0.011	2.84 2.61 2.71	2.72	2.89 2.65 2.76	2.77	0.05	14.2

Continuation of Table B8

Specimen	Surface area (m^2)		Mass (M)				Mass of water absorbed(M_w - M_d	Water absorption coefficient (c_b)
			Dry (M_d)		Wet (M_w)			
	value	average	values	average	value	average		
D_{10}	0.011 0.011 0.011	0.011	2.85 2.61 2.70	2.72	2.89 2.65	2.77	0.05	14.2
D_{11}	0.013 0.013 0.013	0.013	3.10 2.85 2.95	2.97	3.13 2.88 2.98	3.00	0.03	7.2
D_{12}	0.013 1.012 0.012	0.012	2.79 2.91	2.85	2.84 2.97	2.91	0.06	15.6
E_{10}	0.011 0.011 0.013	0.012	2.72 3.00	2.86	2.76 3.04	2.9	0.04	10.4
E_{11}	0.013 0.013 0.013	0.013	2.64 2.91 2.76	2.78	2.67 2.94 2.79	2.81	0.03	7.2
E_{12}	0.012 0.011 0.011	0.011	2.84 2.71	2.78	2.88 2.75	2.82	0.04	11.36

Table B9 Experimental result and determination of Abrasive coefficient

Specimen	Mass of block before brushing (g) M_1		Mass of block after brushing (g) M_2		Mass of detached matter (g) (M_1-M_2)	Brushed surface area cm^2	Abrasive coeff., C_a (cm^2/g)
	values	Average	Values	Ave.			
A_{10}	6100 6400 6250	62500	6000 6300 6200	6200	50	2.5x29	1.45
A_{11}	5401 5210 5310	5310	5250 5100 5175	5175	139	2.5x29	0.52
A_{12}	2700 2400 2550	2550	2600 2300 2450	2450	100	2.5x13.5	0.34
B_{10}	2900 2950 2925	2925	2880 2900 2980	2980	25	2.5x14.5	1.47
B_{11}	2649 2650 2652	2650	2555 2522 2480	2520	55	2.5x12.5	0.57
B_{12}	2700 2800 2600	2700	2486 2752 2619	2619	81	2.5x12.5	0.41
C_{10}	3043 3106 2953	3106	3043 2925 2989	2989	35	2.5x14	0.96
C_{11}	2787 2531 2649	2649	2764 2503 2634	2634	15	2.5x13.5	2.25
C_{12}	3043 2753 2898	2898	2993 2687 2840	2840	58	2.5x14.5	0.63

Continuation of Table B9

Specimen	Mass of block before brushing (g) M1 values	Average	Mass of block after brushing (g) M2 Values	Ave.	Mass of detached matter (g) (M1-M2	Brushed surface area cm²	Abrasive coeff. C_a (cm^2 /g)
D_{10}	2899 2897 2898	2898	2877 2788 2968	2877	21.7	2.5x14	1.60
D_{11}	2864 2874 2855	2854	2854 2863 2845	2845	10.3	2.5x14	3.40
D_{12}	2866 2857 2844	2856	2843 2834 2823	2833	23.0	2.5x14	1.54
E_{10}	2845 2955 2655	2855	2821 2931 2631	2831	24.0	2.5x14	1.47
E_{11}	2875 2864 2886	2875	2865 2854 2876	2865	10.0	2.5x14	3.40
E_{12}	2855 2786 2924	2811	2833 2764 2902	2833	22.0	2.5x14	1.60

Table B10 Liquid limit (cone penetrometer) determination

Liquid limit	1		2		3		4		5	
Dial gauge reading	12	12.4	16.6	17	19.6	19.8	22.1	22.5	25.6	25.8
Average dial gauge reading	12.2		16.8		19.7		22.3		25.7	
Container no.	E4		Z15		R8		B3		R1	
Mass of wet soil + container g	10.59		44.93		44.82		39.93		40.53	
Mass of dry soil + container g	35		39		39		34		34	
Mass of container g	18.28		21.75		22.62		17.95		18.41	
Mass of moisture g	5.99		5.93		5.82		5.93		6.53	
Mass of dry soil g	16.72		17.25		16.38		16.05		15.57	
Moisture content	33		34		35		37		42	

Table B11 Linear shrinkage determination

Time (days)	Container no.	Original length (mm)	Oven-dried length (mm)	Percentage shrinkage	Ave. % shrinkage
	S1	130	128.77	0.946	
	S2	129	127.05	1.511	
	S3	130	128.46	1.184	
Day 1	S4	130	128.29	1.315	1.24
	S1	130	127.74	1.738	
	S2	129	126.23	2.147	
	S3	130	127.82	1.676	
Day2	S4	130	128.06	1.492	1.76
	S1	130	127.04	2.276	
	S2	129	125.74	2.525	
	S3	130	127.00	2.307	
Day3	S4	130	127.86	1.646	2.18
	S1	130	127.04	2.276	
	S2	129	125.74	2.525	
	S3	130	127.00	2.307	
Day4	S4	130	127.86	1.646	2.18
	S1	130	127.04	2.276	
	S2	129	125.74	2.525	
	S3	130	127.00	2.307	
Day5	S4	130	127.86	1.646	2.18

Table B 12 Triaxial saturation

Soil description medium clayey soil from Cardiff								
Test method clause 5.3/5.4 of BS 1377: Part 8: 1990								
Cell pressure and back pressure increments =50 kPa								
Cell pressure (kPa)		Back press. (kPa)	Pore pressure (kPa)		Coeff. of Saturation B	Volume change indicator		
σ_3	$\Delta\sigma_3$		U	Δu	$\Delta u/\Delta\sigma_3$	V_1	V_2	Δv
0		0	6.1			0	0	0
50	50	-	15.4	9.2	0.18	44.6	44.1	0.5
50		40	30.9					
100	50	-	35.8	20.3	0.20	44.5	44.3	0.2
100		90	80.1					
200	100	-	85.3	49	0.49	44.3	44.1	0.2
200			178.8					
300	100	290	184.1	98.9	0.99	44.0	43.8	0.2
300			284.9					
Saturation completed							Total	1.1
400			288.02		**For consolidation**			

Table B13 Triaxial consolidation (specimen 1)

Location :CARDIFF UNIVERSITY			
Soil description medium clayey soil from Cardiff			
Test method clause 6 of BS1377: Part 8: 1990			
Consolidation-undrained compression triaxial test			
With/without side drains	Date started 14/11/07	Date completed 14/11/07	
Required effective stress, σ'_3 400-287.88 =112 ≈110 kPa			
Cell pressure, σ_3 400 kPa		Initial diameter Do (mm)	
back pressure u_b 290 kPa		Initial length Lo(mm)	
Pore pressure after build-up, u_i(kPa)		Initial area Ao (mm^2)	
Excess pore pressure, ($u_i - u_b$) (kPa)		Initial volume Vo(cm^3)	

Consolidation date

Date	Time	Elapse Time(min)	√t	Volume change indicator		Pore pressure		
				v (mL)	Δv (mL)	u (kPa)	Δu (kPa)	Dissipation $U=\frac{u_i - u}{u - u_b}$
14/11/2007	12:25	¼	0.5	44.50	0.00	277.0	0.0	0.0%
		½	0.7	43.85	0.60	277.0	0.0	0.0%
		1	1.0	43.74	0.76	277.0	0.0	0.0%
		2 ¼	1.5	43.67	0.83	276.0	1.0	7.6%
		4	2.0	43.58	0.92	270.0	7.0	53.0%
		9	3.0	43.57	0.93	268.0	9.0	69.0%
	12:41	16	4.0	43.57	0.93	267.8	9.2	70.7%
	12:50	25	5.0	43.57	0.93	267.7	9.3	71.1%
	13:01	36	6.0	43.57	0.93	267.3	9.6	73.8%
	13:26	64	8.0	43.53	0.97	267.3	9.7	74.0%
	14:23	121	11.0	43.35	1.15	266.8	10.2	78.6%
	15:11	169	13.0	42.58	1.92	265.0	12	92.3%
	15:38	196	14.0	42.00	2.5o	264.2	12.8	98.0%

Tale B14 Consolidation undrained triaxial test with measurement of pore pressure (specimen 1)														
Start of compression Lc =75.65 mm Ac =1123.8 mm^2					Test method Clause 7 of BS 1377 : PART 8: 1990						Effective stress 110 kPa Cell pressure 400 kPa			
Axial strain				Area	Axial force (kN)				Pore Pressure		Deviator stress	Principal stresses (kPa)		
Time	L	ΔL	E	A_s	R	ΔR	C_r	P	U	ΔU	Б$_1$ –Б$_3$	Б$_1$	Б$'_1$	Б$'_3$
10:50	18.1	0	0	1124	47.6	4.4	3.613	0	250	0	0	400	150	150
	18.3	0.2	0.003	1127	52	5.4		15	252	2	3.3	403	151	148
	18.7	0.6	0.008	1132	53	7.9		19	253	3	6.8	407	153	147
	18.8	0.8	0.011	1134	55.5	9.9		28	258	8	14.7	417	156	142
11:30	19.8	1.74	0.023	1147	58	9.9		35	263	20.2	421	158	133	1.18
12:10	20.2	2.16	0.029	1157	60	12.4		44	267	30	430	163	133	1.23

Continuation of Table B14

Axial strain				Area	Axial force (kN)				Pore pressure		Deviator stress	Principal stresses (kPa)		
Time	L	ΔL	ε	A_s	R	ΔR	C_r	P	U	ΔU	$Б_1 - Б_3$	$Б_1$	$Б'_1$	$Б'_3$
12:35	20.4	2.3	0.03	1158	61	13.4	3.613	48	268	18	35	435	167	132
13:21	21.14	2.9	0.038	1167	75	27.4		989	270	20	745	475	205	130
13:40	21.19	3.1	0.041	1171	90	42.4		153	285	35	121	521	236	115
13:55	21.3	3.2	0.042	1172	100	52.4		189	292	42	152	551	256	105
14:14	23.6	5.5	0.072	1210	110	62.4		224	308	58	175	574.	267	92
14:30	23.9	5.8	0.072	1218	120	72.4		261	312	62	205	605	292	88

Appendix C

Tables of data for Chapter six–thermoplastic carton soil block

Table C1 Table of experimental results on thermoplastic carton soil block

Disp. (mm)	Force (kN)						
	TCSB	0.75% OPF	1.0% OPF	1.5% Opf	0.75% PF	1.0% PF	1.5% PF
0	0	0	0	0	0	0	0
0.5	2	7	7	6.9	5.4	7	9
1	5	10	10	8.6	6.8	9	12
2	9.6	12	12	10.8	9.1	11	16
3	17	16	20	12.7	9.8	13	20
4	19	20	23	14.6	12.1	15	24
5	23	22	26	15.3	15	19	27
6	27	27	29	18.2	17	27	34
7	30	30	36	32	24	37	38
8	34	35	52	36	31	45	44
9	38	45	56	49	43	79	50
10	40	48	65	54	61	95	65
11	44	54	68	80	84	126	77
12	68	79	97	114	104	136	147
13	73	87	102	124	124	143	152
14	81	99	107	134	152	170	186
15	90	102	115	143	172	190	215
16	102	108	130	152	180	210	254
17	114	124	150	186	190	240	279
18	120	168	215	215	191	260	297
19	134	197.6	249	259	210	280	300
20	158	224	310	290	260	294	322
21	187	261	406	360	310	300	361
22	264	292		446	356	340	392
23	347	317				370	432
24		372				450	479

Table C2 Table of values of stress and strain calculated from experimental results

Strain (%)	Compressive stress [MPa]						
	TCSB	0.75% OPF	1.0% OPF	1.5% OPF	0.75% PF	1.0% PF	1.5% PF
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.83	0.10	0.35	0.35	0.34	0.27	0.35	0.45
1.66	0.25	0.50	0.54	0.43	0.34	0.45	0.60
3.33	0.48	0.60	0.64	0.54	0.45	0.55	0.80
5.00	0.85	0.80	1.01	0.64	0.49	0.65	1.01
6.66	0.95	1.01	1.16	0.73	0.61	0.75	1.21
8.33	1.16	1.11	1.31	0.77	0.75	0.95	1.36
10.00	1.36	1.36	1.46	0.91	0.85	1.36	1.71
11.66	1.51	1.51	1.81	1.61	1.21	1.86	1.91
13.33	1.71	1.76	2.62	1.81	1.56	2.27	2.22
15.00	1.91	2.27	2.82	2.47	2.17	3.98	2.52
16.66	2.02	2.42	3.28	2.72	3.08	4.79	3.28
18.33	2.22	2.72	3.43	4.04	4.24	6.36	3.88
20.00	3.43	3.98	4.89	5.75	5.25	6.86	7.42
21.66	3.68	4.39	5.15	6.26	6.26	7.22	7.67
23.33	4.09	5.00	5.40	6.76	7.67	8.58	9.39
25.00	4.54	5.15	5.80	7.22	8.68	9.59	10.80
26.66	5.15	5.45	6.56	7.67	9.09	10.60	12.88
28.33	5.75	6.26	7.57	9.39	9.59	12.12	14.0
30.00	6.06	8.48	10.88	10.85	9.64	13.13	15.00
31.66	6.76	9.97	12.57	13.08	10.60	14.14	15.15
33.33	7.97	11.31	15.65	14.64	13.13	14.84	16.26
35.00	9.44	13.18	20.46	18.18	15.65	15.15	18.23
36.66	13.33	14.74	20.46	22.47	17.97	17.17	19.79
38.33	17.50	18.70	20.46	22.47	17.97	18.68	21.81
40.00	17.50	18.70	20.46	22.47	17.97	22.72	24.19

Table C3 Comparison of FEA model results with experimental results for TCSB with Oil Palm Fibre

Strain (%)	Experimental results Max Stress (MPa)				FEA model results Stress (MPa)			
	A	$B_{0.75}$	$B_{1.0}$	$B_{1.5}$	A	$B_{0.75}$	$B_{1.0}$	$B_{1.5}$
0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	0.3	0.4	0.4	0.4	4.7	4.4	2.8	2.0
6	1.8	2.5	3.2	3.2	5.3	5.5	4.0	2.5
8	2.2	3.1	3.8	4.2	6.0	6.5	5.6	5.0
11	2.8	4.5	4.5	6.4	7.7	7.6	6.2	6.3
13	4.2	5.3	4.8	8.5	8.5	8.6	7.3	7.7
16	6.1	7.5	8.3	10.3	9.2	9.7	8.4	9.2
19	7.2	9.0	10.0	12.2	9.8	10.7	9.6	12.0
22	8.7	9.4	10.2	13.3	10.4	11.3	10.7	13.0
24	10.3	10.3	11.5	15.4	11	12.8	12.0	15.0
27	11.2	11.2	11.8	17.7	11.7	13.8	13.0	16.0
30	12.6	12.6	15.3	19.4	12.4	15.0	14.0	17.7
32	14.3	14.3	17.6	19.7	13	15.5	15.0	19.1
35	16	16.0	20.4	20.2	13.6	17.0	16.3	20.5
38	16.2	18.4	20.5	22.4	14.0	18.0	18.5	22
40	17.5	18.7	20.5	22.5	14.3	19.0	19.6	22.4

Table C4 Comparison of FEA model results with experimental results for TCSB with Plastic Fibre

Strain (%)	Experimental results Max Stress (MPa)				From FEA model Stress (MPa)			
	A	$C_{0.75}$	$C_{1.0}$	$C_{1.5}$	A	$C_{0.75}$	$C_{1.0}$	$C_{1.5}$
0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	0.3	0.3	0.4	0.5	4.7	4.8	3.1	2.6
6	1.8	4.7	2.7	2.3	5.3	5.8	4.4	4.1
8	2.2	4.4	3.3	3.0	6.0	6.6	5.6	5.5
11	2.8	5.3	5.4	7.0	7.7	7.8	8.0	7.0
13	4.2	6.6	7.3	7.6	8.5	8.6	9.2	8.5
16	6.1	9.2	11.4	8.9	9.2	10.0	10.4	10.0
19	7.2	11.3	11.9	10.5	9.8	11.0	11.6	11.4
22	8.7	11.3	13.6	14.4	10.4	12.0	13.0	15.0
24	10.3	12.7	117.2	16.0	11.0	13.0	14.0	16.0
27	11.2	16.1	18.2	18.0	11.7	14.0	15.0	17.0
30	12.6	17.2	20.2	20.2	12.4	15.0	17.0	20.0
32	14.3	17.4	21.1	21.3	13	15.5	18.0	22.0
35	16.0	17.6	21.7	23.0	13.6	17.0	19.0	23.0
38	16.2	17.9	22.5	23.3	14.3	18.0	20.0	24.0
40	17.5	19.3	22.7	24.2	14.3	19.0	21.4	24.5

2.0 Modelling Procedure

1.0 File /New

New Database name → Plastic_crate

OK

Tolerance → Default

Analysis code → MSC/NASTRAN

Analysis type → Structural

OK

2. 0 Geometry

◊ ***Geometry***

Action : → create

Object: → solid

Method: → XYZ

Vector coordinate list → < 165 60 120>

Origin coordinate list → <0, 0, 0>

Apply

3.0 Load/BC

Action: → Create

Object: → Total load

Type: → Uniform-load

Target type → 3D

Input Data....

<f1 f2 f3>: → <0 -450 0>

OK

Select application Region

Geometry filter → Geometry

Select surface edges → Solid 1.4

Add

OK

Apply

◊ ***Load/BC***

Action : → Create

Object: → Displacement

Type: → Nodal

New set name → Fixed

Input data

<T1 T2 T3>: → <0,0,0>

<R1 R2 R3> → <0,0,0>

OK

Select application Region

Geometry filter → Geometry

Select surface edges → Solid 1.3

Add

OK

Apply

4.0 Material

Action: → Creat

Object: → 3D

Method: → Solid

Material Name: → Plastic

Input properties

Constitutive model: → Linear elastic

Elastic modulus → 170

Poisson Ratio → 0.3

Apply

◊ ***Material***

Action: → Create

Object: → 3D

Method: → Solid

Material Name: → Soil

Input properties

Constitutive model: → Linear elastic
Elastic modulus → 10.6
Poisson Ratio → 0.33

Apply

5.0 Properties

Action : → Create
Dimension: → 3D
Method: → Solid
Property set Name: → Prop_1

Input

Material name: → M: plastic

OK

Select member → Solid 1

Add
Apply

◊ ***Properties***

Action: → Create
Dimension: → 3D
Method: → Solid
Property set Name: → Prop 2

Input properties

Material name: → M: Soil

OK

Select member: → Solid 2

Add

Apply

6.0 Analysis

Action: → Analyze

Object: → Entire Model

Method: → Full Run

Apply

Attach the result file

◊ ***Analysis***

Action: → Assess Result

Object: → Attached XDB

Method: → Result Entities

Apply