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Utilization of coconut fibres and ropes as concrete reinforcement in mortar-free construction with novel interlocking blocks

Considering its application in earthquake resistant housing

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A thesis submitted in fulfilment of the requirements for
the Degree of Doctor of Philosophy

Supervised by Associate Professor Nawawi Chouw

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In the Name of ALLAH, Most Gracious and Most Merciful
Abstract

Safe housing in earthquake prone regions is a priority for all. No one wants to compromise on this, but at the same time, there arises a question of ability to finance. Many people, particularly those who live in rural areas of developing and underdeveloped countries, cannot afford expensive houses. The overall aim of the research is to develop natural fibre reinforced concrete structures, which can not only withstand earthquakes, but also are low cost and easy to construct. Natural fibres are cheap and locally available in many countries.

For this study, coconut fibres were selected because of their highest toughness amongst all natural fibres. Coconut fibre is extracted from the husk of coconut fruit. The material characterization of coconut fibre reinforced concrete (CFRC) was carried out because limited information was available about it. Fibre properties like fibre length and content were optimised for obtaining the best properties of CFRC. During the standard static testing, cracks were produced in CFRC cylinders (unlike the spalling/splitting of plain concrete) while performing compressive and splitting tensile strength tests. CFRC beamlets did not break into two pieces after a crack was produced, reflecting the advantage of coconut fibres. While performing dynamic testing, it was observed that the damping of CFRC increased with increasing fibre content, and this increment was considerable after cracking at different damage stages. These observations confirmed that CFRC can be utilized as a construction material. The investigations on bond strength between (i) fibres and concrete and (ii) rope and CFRC were also studied. The considered parameters include embedded length, diameter, pre-treatment and matrix strength. The results of both bond strength tests provided insightful information about the influential variables to be taken care of for increasing the structural stability at local and global levels.

Mortar-free construction can lead to energy dissipation during strong ground motions. Therefore, interlocking blocks were invented to facilitate the construction of mortar-less walls. The blocks were prepared with CFRC. The blocks can move relative to each other during an earthquake, and also have self-centring ability during and after the ground motion. The shear and compressive capacities of the interlocking blocks were experimentally determined. Mortar-free structures using these blocks and rope reinforcement were tested under harmonic and earthquake loadings. The dynamic
performance was analysed and it was found that the damping increased with increasing load amplitude because of block uplift. Finally, future recommendations are provided for achieving the overall goal of the research, i.e. affordable earthquake-resistant housing.
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<th>Description</th>
</tr>
</thead>
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<tr>
<td>CFRC</td>
<td>Coconut fibre reinforced concrete</td>
</tr>
<tr>
<td>$C_f$</td>
<td>Fibre content in % by mass of concrete materials</td>
</tr>
<tr>
<td>$D_f$</td>
<td>Fibre diameter in fibre pullout test</td>
</tr>
<tr>
<td>$D_r$</td>
<td>Rope diameter in rope pullout test</td>
</tr>
<tr>
<td>$E_f$</td>
<td>Energy required to pull out fibre from plain concrete</td>
</tr>
<tr>
<td>$E_{fibre}$</td>
<td>Elastic modulus of coconut fibre in GPa</td>
</tr>
<tr>
<td>$E_r$</td>
<td>Energy required to pull out rope from CFRC</td>
</tr>
<tr>
<td>$E_{static}$</td>
<td>Static modulus of elasticity in GPa</td>
</tr>
<tr>
<td>$E_{dynamic}$</td>
<td>Dynamic modulus of elasticity in GPa</td>
</tr>
<tr>
<td>$L_f$</td>
<td>Fibre embedment length in cm in fibre pullout test</td>
</tr>
<tr>
<td>$L_r$</td>
<td>Rope embedment length in cm in rope pullout test</td>
</tr>
<tr>
<td>MOR</td>
<td>Modulus of rupture in MPa</td>
</tr>
<tr>
<td>$M_f$</td>
<td>Mix design ratios (cement: sand: aggregates) considered for fibre pullout tests</td>
</tr>
<tr>
<td>$M_r$</td>
<td>Mix designs ratios (cement: sand: aggregates) considered for rope pullout tests</td>
</tr>
<tr>
<td>$N$</td>
<td>Numbers of knot in rope pullout tests</td>
</tr>
<tr>
<td>PC</td>
<td>Plain concrete</td>
</tr>
<tr>
<td>$P_{crack}$</td>
<td>Cracking load i.e. the load taken by fibres and part of concrete after the first visible crack is formed.</td>
</tr>
<tr>
<td>$P_f$</td>
<td>Pre-treatment of fibres in fibre pullout test</td>
</tr>
<tr>
<td>$P_r$</td>
<td>Pre-treatment of ropes in rope pullout test</td>
</tr>
<tr>
<td>$P_{impact}$</td>
<td>Impact load applied by hammer at mid-span</td>
</tr>
<tr>
<td>$P_{static}$</td>
<td>Static four-point loading for producing damage</td>
</tr>
<tr>
<td>Q</td>
<td>Base shear</td>
</tr>
<tr>
<td>STS</td>
<td>Splitting tensile strength in MPa</td>
</tr>
<tr>
<td>$T_c$</td>
<td>Compressive toughness in MPa</td>
</tr>
<tr>
<td>$T_f$</td>
<td>Flexural toughness</td>
</tr>
<tr>
<td>$T_{fibre}$</td>
<td>Total toughness of coconut fibre in MPa</td>
</tr>
</tbody>
</table>
TTI  Total toughness index
W/C  Water cement ratio
X  An exponential factor for estimating $\tau_r$
$X_d, Y_d, Z_d$  Constants corresponding to fibre length $L_f$ for estimating $E_{\text{dynamic}}$
$X_s, Y_s, Z_s$  Constants corresponding to fibre length $L_f$ for estimating $E_{\text{static}}$
Y  An exponential factor for estimating $E_r$

a, b, c  Constants for estimating $\tau_f$
$c_f$  Fibre content in % by mass of cement for estimating $E_{\text{dynamic}}$ or $E_{\text{static}}$
$f$  Fundamental frequency in Hz
$f_n$  Natural frequency in Hz
$f_{cs}$  Compressive strength of single interlocking block
$f_{cm}$  Compressive strength of multiple interlocking blocks
k  Stiffness
$l_f$  Fibre length in cm
t, v, w  Constants for estimating $\tau_r$
u  Top relative displacement
$u, u_g, u_s$  In-plane horizontal displacement at top of structure (column/wall), foundation and shake table, respectively
p, q, r, s  Constants for estimating $E_r$
x, y, z  Constants for estimating $E_r$

$\alpha$  Reinforcement factor for estimating $\tau_f$
$\beta$  Matrix factor for estimating $\tau_f$
$\alpha_r, \beta_r, \gamma_r, \lambda_r$  Factors for embedment length, pre-treatment, mix design and fibre content of matrix, respectively for estimating $\tau_r$
$\Delta$  Mid-span deflection (in mm) corresponding to MOR
$\Delta_{\text{max}}$  Maximum mid-span deflection (in mm)
$\varepsilon$  Strain corresponding to compressive strength
$\zeta$  Damping ratio
$\phi$  Fibre diameter dimensions
$\gamma$ Reinforcement factor for estimating $E_t$

$\lambda$ Matrix factor for estimating $E_t$

$\sigma$ Compressive strength in MPa

$\sigma_{\text{fibre}}$ Tensile strength of coconut fibre in MPa

$\sigma_{\text{rope}}$ Ultimate tensile strength of rope in MPa

$\tau_f$ Bond strength between single coconut fibre and PC

$\tau_r$ Bond strength between rope and CFRC

$\eta, \psi, \mu, \omega$ Factors for embedment length, pre-treatment, mix design and fibre content of matrix, respectively for estimating $E_t$
List of publications

1. Journal articles:

   Published:


2. Refereed conference papers:

   Published:


   Intended:

Chapter 1.

Introduction

1.1. Prologue

The importance of economical earthquake-resistant housing has been recognised in recent years due to frequent seismic activities causing loss of life and property. The available technologies utilize heavy steel reinforcement to achieve overall ductility of the structures. However, steel reinforcement is still unaffordable for many communities. Poor people do not have engineered houses, especially in rural areas. Mostly, such houses are built by masons or contractors without any engineering supervision, because to construct well supervised earthquake-resistant houses costs significantly more. An alternate approach for such communities is therefore required, especially in under-developed and developing countries. The new technology should be cheap as well as easy to construct with available local resources. Plant fibres (here after called natural fibres) are abundantly available, renewable and low-cost (Aziz et al., 1981; Asasutjarit et al., 2007).

Natural fibres as reinforcement in composites (such as cement paste, cement sand mortar and/or concrete) have been studied by many researchers, but only for non-structural members (Cook et al., 1978; Ramakrishna and Sundararajan, 2005a; Asasuțjarit et al., 2007). These composites have been tested for plastering and as roofing materials, corrugated slabs and boards in different parts of the world. Significant improvement in the properties of these composites has been obtained by the insertion of fibres. Their potential as an earthquake-resistant structural material needs to be considered.

In the latest edition of the book “Guidelines for earthquake-resistant non-engineered construction” (page 28), Arya et al. (2012) state:

“Desirable properties of earthquake-resistant design include ductility, deformability and robustness. Ductility and deformability are interrelated
1. Introduction

concepts signifying the ability of a structure to sustain large deformations without collapse. Robustness refers to the ability of a structure to undergo substantial damage, without partial or total collapse. This is desirable because it means that structures can absorb more damage, and because it permits the deformations to be observed for repairs or evacuation to proceed, prior to collapse. In this sense, a warning is received and lives are saved. Formally, ductility refers to the ratio of the displacement of a building just prior to ultimate displacement or collapse to the displacement at first damage or yield. Deformability is a less formal term referring to the ability of a structure to displace or deform substantial amounts without collapsing. Ductility is a term applied to both material and structures, while deformability is applicable only to structures. Robustness is also a desirable quality for construction, and refers to the ability of a structure to undergo substantial damage, without partial or total collapse.”

They also mentioned the following parameters as the most important from the point of view of structural seismic design:

a) Building material properties
   - Strength in compression, tension and shear, including dynamic effects
   - Unit weight (density)
   - Modulus of elasticity

b) Dynamic characteristics of the building system, including periods, modes of vibration and damping.

c) Load-deflection characteristics of building components.(Arya et al., 2012; Page 10).

Therefore, material properties of coconut fibre reinforced concrete (CFRC), the bond between fibre and concrete, the bond between rope and CFRC, and dynamic and load-deflection characteristics of the proposed system (i.e. mortar-free construction) are evaluated experimentally. The bond between reinforcing materials and matrix plays an important role in the overall strength of the composite. CFRC can be used in blocks and parking pavements e.g. for avoiding cracks due to shrinkage. It can also be used in normal reinforced concrete to improve its behaviour during an earthquake. But CFRC with steel
rebars needs to be properly investigated before implementation. It is important to mention that the research presented in this thesis is different to capacity and strength based design because energy absorption is achieved via material damage as well as through relative movement in the whole structure.

1.2. Research motivation

Earthquakes have caused mass destruction of buildings because of non-engineered archaic construction. This ultimately results in significant loss of life and finance. One example is the October 2005 Kashmir Earthquake which the author personally experienced and observed damage on a very large scale. This earthquake (magnitude 7.6 on the Richter scale) was spread over an area of approximately 28,000 km$^2$. It killed more than 73,000 people, injured 128,000, damaged or destroyed more than 600,000 houses, 6,298 education institutions and 782 health facilities. Ninety percent of the destroyed or damaged houses were in rural areas (Mumtaz et al., 2008). The observations emphasize the need for new techniques for economical and safe housing in earthquake prone rural areas. This motivated the author to explore new construction technologies. Natural fibre reinforced concrete structures can be one solution, but they require suitable quantities of fibres in the concrete. A certain quantity of fibres can be beneficial for enhancing the properties of plain concrete, but it may be noted that all properties may not be improved. The addition of fibres may improve certain properties while, at the same time, there may be some compromise on other properties. Therefore, fibres in an appropriate quantity should be selected. An effort to diversify and encourage the use of natural fibres for construction is made in this research. Natural fibres (e.g. coconut/coir, sisal, bamboo, flax and hemp) are cheaper than conventional steel fibres and are locally available in many countries. The use of natural fibres costs little, and the resulting product is also much cheaper and has equal or even better properties than the locally available commercial products (Cook et al., 1978). Therefore, their use is increasing day by day. Coconut fibre reinforced concrete (CFRC) and coconut fibre rope is investigated for their potential in low-cost housing in under-developed and developing countries.
1.3. Research objectives and methodology

The overall aim of the research is to develop a new construction technology for economical seismic-resistant houses. The specific objective of this doctoral research is to consider the suitability of coconut fibres and rope as concrete reinforcement for housing. The experimental research objectives are as follows:

- to optimize the length and content of coconut fibres in CFRC for obtaining its best mechanical and dynamic properties
- to determine the tensile strength and elastic modulus of coconut fibres and to study bond behaviour between fibre and concrete
- to find out the tensile strength and elongation of coconut-fibre ropes and to study bond behaviour between rope and CFRC
- to optimize the mix design of CFRC for block preparation and to determine compressive and shear strength of the novel interlocking blocks
- to understand the dynamic response of mortar-free interlocking structures.

Coconut fibres are selected on the basis of their highest toughness amongst all natural fibres as reported by Munawar et al. (2007) for fibre bundles and by Satyanarayana et al. (1990) for single fibres. This thesis presents results of an extensive experimental programme, and with the obtained knowledge from the laboratory testing, empirical equations are developed. CFRC is studied in detail. Ropes made of coconut fibres are used as vertical reinforcement along with CFRC for building structural components in this new construction technology. The dynamic properties of CFRC members are determined at different considered damage stages. The basic static properties of CFRC are determined using standard procedures that are usually used for plain concrete i.e. NZS 3112: Part2: 1986. ASTM standard D3822-07 and ASTM International (2009) were taken as the guide for the testing of coconut fibres and coconut-fibre ropes. The investigations on bond strength between (i) fibres and concrete and (ii) rope and CFRC are focussed on the influence parameters embedded length, content, pre-treatment and matrix strength. The pullout tests were performed to study the bond strength. Mortar-free construction can lead to a higher damping because of energy loss due to relative movements between adjacent interlocking blocks. An innovative interlocking block is invented for earthquake-resistant houses. The compressive and shear capacities of these blocks are determined experimentally. Mortar-free structures using these blocks and rope reinforcement are
tested using shake table under harmonic and earthquake loadings to investigate their seismic performance.

The scope of this research is limited to material characterization (i.e. properties of CFRC, fibre properties, the bond between coconut fibre and concrete, rope properties and the bond between rope and CFRC) and structural aspects (i.e. interlocking blocks and mortar-free construction). Following this research, the work can be extended to testing of full-scale wall with diaphragm or even 3D structure. The other materials like fly ash, rice husk ash and palm oil fuel ash, coconut shells, palm kernel, pumice and other natural fibres can also be studied depending upon their local availability. The details are provided at the end of the thesis.

1.4. Thesis outline

The thesis consists of eight chapters, of which Chapters 3 to 7 are aimed to write independent journal articles but here are accordingly slightly modified to maintain coherence in the thesis and to avoid repetition of the certain parts from the original paper format. The experimental research presented in this thesis can be classified into two main parts: (1) material characterization i.e. properties of CFRC, the bond between coconut fibre and concrete, and the bond between rope and CFRC; and (2) structural aspects, i.e. interlocking blocks and mortar-free construction. The thesis is organised into the following main chapters:

Chapter 2 presents a literature review about the usage of coconut fibres as a construction material in different composites for civil engineering applications. Coconut fibre is a natural fibre abundantly available in tropical regions, and is extracted from the husk of coconut fruit. Not only the physical, chemical and mechanical properties of coconut fibres are shown, the properties of composites (cement pastes, mortar and/or concrete etc), in which coconut fibres are used as reinforcement, are also discussed. The research conducted and the conclusions drawn by different researchers in the last few decades are also briefly presented. The relationships between different properties are shown. Coconut fibre reinforced composites have been used as cheap and durable non-structural elements. This review provides an overview of how it can be utilized for structural purposes.
Chapter 3 focuses on investigation of the mechanical properties of CFRC. In addition to static properties, damping ratios and fundamental frequencies of simply supported CFRC beams were determined experimentally. A comparison between the static and dynamic modulii was conducted. The influences of 1%, 2%, 3% and 5% fibre contents by mass of cement and fibre lengths of 2.5, 5 and 7.5 cm were investigated. To evaluate the effect of coconut fibres in improving the properties of concrete, the properties of plain concrete were used as a reference.

In Chapter 4, the effects of fibre embedment lengths, diameters, pre-treatment conditions and concrete mix design ratios on the bond strength between single coconut fibre and concrete are investigated. Fibres were prepared and categorized manually. Fibre diameters were measured by a stereomicroscope. Fibre and concrete properties were also determined experimentally. Single fibre pullout tests were conducted to determine load-slippage curves with the help of an Instron tensile machine. Bond strengths and energy required to pull out fibre are calculated from the experimental data. The single fibre-concrete interfacial bond strength is calculated by dividing the maximum pullout load by the surface area of the embedded portion of a fibre. Pullout energy is calculated as the area under the load-slippage curve. With the obtained knowledge, empirical equations are developed to determine the bond strength and energy required for pullout.

In Chapter 5, the effects of rope embedment lengths, rope diameters, pre-treatment conditions, concrete mix design ratios, fibre content and knots on the bond strength between rope and CFRC are investigated. The higher pullout load (i.e. rope tension) generated due to an earthquake loading may govern the embedment length of the ropes inside the foundation. Therefore, pullout load against different parameters (as stated earlier) is investigated. The rope tension during dynamic loading is considered in Chapter 7. The generated rope tension should be less than the rope tensile capacity and the pullout force for avoiding structure collapse. Thus, the tensile strength and elongation of coconut fibre rope were also determined considering parameters like rope diameter and pre-treatment. Rope pullout tests were performed to determine load-slippage curves. Bond strength and the energy required for rope pull out are calculated from the experimental data. The rope-CFRC interfacial bond strength is calculated by dividing the maximum pullout load by the surface area of the embedded portion of the rope. Pullout energy is
calculated as the area under the load-slippage curve. With the obtained knowledge, empirical equations are also developed to determine the bond strength between rope and CFRC, and the energy required for rope pullout.

Chapter 6 presents the concept of invented interlocking blocks and the proposed construction technique. The block is provided with special bottom and top interconnecting profiles so that it will interlock with upper, lower and adjacent blocks. Additional bottom- and top-smooth profiles, as well as half blocks are developed to facilitate the construction of a wall. The selection of mix design (cement: sand: aggregates) for preparing CFRC was made before block production because of invented block shape. The compressive capacity of single (standard, bottom, top and half blocks) and multiple blocks (standard as well as a combination of top, standard and bottom blocks) were determined experimentally. The relationship between compressive strength of individual and multiple standard blocks, and the in-plane and out-of-plane shear capacities of interlocking mechanism, are investigated.

In Chapter 7, the investigation on mortar-free structures, made of interlocking blocks with movability at the interface, is presented. Four structures were considered: two columns (without and with rope) and two walls (without and with rope). Mortar-free construction can facilitate more energy dissipation during a seismic event, because of relative movement. This chapter describes the in-plane behaviour of the mortar-free structures under different loadings (i.e. push over, snap back, impact, harmonic and earthquake loadings). The influences of rope reinforcement on fundamental frequency, damping ratio, induced accelerations, top relative displacement and block uplift are characterized.

The study is concluded with recommendations for the future in Chapter 8, followed by a list of references.
1. Introduction
Chapter 2.

Background: coconut fibres and its composites

Related papers:

2.1. Introduction

Researchers have used natural fibres as an alternative to steel or synthetic fibres in composites such as cement paste, mortar and concrete (Slate, 1976; Cook et al., 1978; Das Gupta et al., 1978 and 1979; Aziz et al., 1981 and 1984; Ramaswamy et al., 1983; Paramasivam et al., 1984; Mwamila, 1985; Aggarwal, 1992; Al-Oraimi and Seibi, 1995; Agopyan et al., 2005; John et al., 2005; Luisito et al., 2005; Mohammad, 2005; Ramakrishna and Sundararajan, 2005a and 2005b; Toldedo et al., 2005; Li et al., 2006; Asasutjarit et al., 2007; Baruah and Talukdar, 2007; Reis, 2007). These natural fibres include coconut, sisal, jute, hibiscus cannabinus, eucalyptus grandis pulp, malva, ramie bast, pineapple leaf, kenaf bast, sansevieria leaf, abaca leaf, vakka, date, bamboo, palm, banana, hemp, flax, cotton and sugarcane fibres. Natural fibres are cheaper than artificial fibres (Paramasivam et al. 1984) and are locally available in many countries. Their cost as a construction material for improving the properties of composites is only a fraction of the total cost of composites. Compared to steel fibres, they are also easier to handle because of their high flexibility, especially when a large volume of fibres is involved. However, in
such a case, a methodology for casting needs to be developed. Terms such as volume fraction and fibre content are often used for expressing the quantities of fibres (Slate, 1976; Das Gupta et al., 1978 and 1979; Cook et al., 1978; Aziz et al., 1981 and 1984; Asasutjarit et al., 2007; Baruah and Talukdar, 2007). Volume fraction of fibres is the percentage of fibres by volume of composite materials or any of its ingredients, e.g. 1% fibre per m$^3$ of concrete. Similarly, fibre content is the percentage of fibres by mass of composite materials or any of its ingredients, e.g. 1% fibres by mass of concrete materials or 5% fibres by mass of cement. Researchers often investigated the optimum quantity and length of fibres to achieve maximum strength of the composite, as any increase or decrease in volume fraction and/or fibre length over the optimum level may be detrimental.

Coconut fibre is extracted from the outer shell of a coconut. The common name, scientific name and plant family of coconut fibre are coir, cocos nucifera and arecaceae (Palm), respectively. There are two types of coconut fibres: brown fibres extracted from matured coconuts and white fibres extracted from immature coconuts. Brown fibres are thick, strong and have high abrasion resistance, while white fibres are smoother and finer, but also weaker. Coconut fibres are commercially available in three forms, namely bristle (long fibres), mattress (relatively short) and decorticated (mixed fibres). These different types of fibres have different uses depending upon the requirement. In engineering, brown fibres are mostly used (Gu, 2009). Of the 55 billion coconuts harvested every year in the world, only 15% of the coconut fibres are recovered for use (Wei and Gu, 2009). According to the official website of International Year for Natural Fibres 2009, approximately 500,000 tonnes of coconut fibres are produced annually worldwide, with an approximate value of $100 million. India and Sri Lanka are the main exporters, followed by Thailand, Vietnam, the Philippines and Indonesia. Around half of the coconut fibres produced is exported in the form of raw fibre. The general advantages of coconut fibres are that they are moth-proof, resistant to fungi and rot, provide excellent insulation against temperature and sound, flame-retardant, unaffected by moisture and dampness, tough and durable, resilient, and spring back to shape even after constant use (Hemsri et al. 2012).
2.2. Properties of coconut fibres

2.2.1. Physical and mechanical properties

The physical and mechanical properties of coconut fibres are presented in Table 2.1, and the conditions specifically mentioned by the researchers are given at the end of the table. Coconut fibres were investigated by many researchers for different purposes. There is a large variation in tensile strength of fibre for the same diameter; for example, compare tensile strength of coconut fibres mentioned by Ramakrishna and Sundararajan (2005a) and Toledo et al. (2005) in Table 2.1. Also, the range shown for a particular property is quite wide; Toledo et al. (2005) mentioned the density of coconut fibre as 0.67-10.0 g/cm³. These values seem to be unrealistic, and real values may be 0.67-1.00 g/cm³. There are significant variations in the properties of coconut fibres which make their adoption as a construction material challenging. The purpose of this compilation of data of the fibre properties was to establish a benchmark, but a large variation was noticed. Therefore, there is a need to develop standards for such variations, similar to standards for sand and aggregates.

Figure 2.1 shows the stress-strain relationship for coconut fibres as reported by some researchers (Paramasivam et al., 1984; Munawar et al., 2007; Satyanarayana et al., 1990). Coconut fibre is the toughest fibre (21.5 MPa) amongst all natural fibres (Munawar et al. 2007) when tested in bundles. Toughness of a fibre bundle is taken as the area under stress-strain curve. Coconut fibres are capable of taking strains 4-6 times more than those taken by other fibres, as shown in Figures 2.1(b) and 2.1(c). Fibre dimensions of the various individual cells are said to be dependent on the type of species, location and maturity of the plant. The flexibility and rupture strength of the fibre is affected by the length-to-diameter ratio (aspect ratio) of the fibre. The shape and size of central hollow cavity, lumen, depends on (i) the thickness of the cell wall and (ii) the source of the fibre. The hollow cavity serves as an acoustic and thermal insulator because its presence decreases the bulk density of the fibre (Flower et al., 2006 as cited by Abiola, 2008).
2. Background: Coconut fibres and its composites

### Table 2.1: Physical and mechanical properties of coconut fibres

<table>
<thead>
<tr>
<th>Diameter (mm)</th>
<th>Length (mm)</th>
<th>Tensile strength (MPa)</th>
<th>Specific tensile strength (MPa)</th>
<th>Average tensile modulus (GPa)</th>
<th>Specific Tensile modulus (GPa)</th>
<th>Tensile strain (%)</th>
<th>Elongation (%)</th>
<th>Elastic modulus (GPa)</th>
<th>Specific elastic modulus (GPa)</th>
<th>Toughness (MPa)</th>
<th>Permeable voids (%) **</th>
<th>Moisture content (%)</th>
<th>Water absorption saturation (%)</th>
<th>Density (Kg/m³)</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.40-0.10</td>
<td>60-250</td>
<td>15-327</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>75</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Ramakrishna and Sundararajan 2005a</td>
</tr>
<tr>
<td>0.21 a, b</td>
<td>-</td>
<td>107c</td>
<td>-</td>
<td>-</td>
<td>37.7d, e</td>
<td>2.8f</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>56.6-73.1</td>
<td>-</td>
<td>93.8-161</td>
<td>1104-1370</td>
<td>Agopyan et al. 2005 c</td>
</tr>
<tr>
<td>0.3</td>
<td>-</td>
<td>69.3f</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>2.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
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<td>-</td>
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</tr>
<tr>
<td>-</td>
<td>-</td>
<td>50.9g</td>
<td>-</td>
<td>-</td>
<td>17.6h</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
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<td>-</td>
<td>-</td>
<td>180h</td>
<td>-</td>
<td>Ramakrishna and Sundararajan 2005b</td>
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<td>0.27±0.073</td>
<td>50±10</td>
<td>142±36</td>
<td>-</td>
<td>-</td>
<td>24±10k</td>
<td>2.0±0.3</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>10m</td>
<td>24</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Li et al. 2007</td>
</tr>
<tr>
<td>0.11-0.53</td>
<td>-</td>
<td>108-252</td>
<td>-</td>
<td>-</td>
<td>13.7-41a</td>
<td>2.5-4.5</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>85.0-135</td>
<td>670-1000</td>
<td>-</td>
<td>-</td>
<td>Toledo et al. 2005</td>
</tr>
<tr>
<td>0.12±0.005</td>
<td>-</td>
<td>137±11</td>
<td>158</td>
<td>-</td>
<td>3.7±0.6</td>
<td>4.2</td>
<td>21.5±2.4</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>11.4p</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Munawar et al. 2007</td>
</tr>
<tr>
<td>-</td>
<td>-</td>
<td>50.00</td>
<td>0.43h</td>
<td>2.5</td>
<td>2.17i</td>
<td>0.00</td>
<td>30</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>11.4p</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Rao and Rao 2007</td>
</tr>
<tr>
<td>-</td>
<td>-</td>
<td>175</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>4-6</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Fernandez 2002</td>
</tr>
<tr>
<td>0.1-0.4</td>
<td>-</td>
<td>174</td>
<td>-</td>
<td>-</td>
<td>10-25</td>
<td>16-26</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Reis 2006</td>
</tr>
<tr>
<td>0.1-0.4</td>
<td>50-250</td>
<td>100-130</td>
<td>-</td>
<td>-</td>
<td>10-26</td>
<td>19-26</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>130-180</td>
<td>145-280</td>
<td>-</td>
<td>-</td>
<td>Aggarwal 1992</td>
</tr>
<tr>
<td>0.10-0.45</td>
<td>-</td>
<td>106-175</td>
<td>-</td>
<td>-</td>
<td>17-47</td>
<td>4-6</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Satyanarayana et al. 1990</td>
</tr>
</tbody>
</table>

*a Coefficients of variation frequently over 50%  
*b Determinations of thickness by scanning electron microscopy  
*c Brazilian Standard NBR-9778  
*d Elongation on rupture  
*e Authors took other researchers’ data  
*f Ultimate value  
*g (Unit: mm) Maximum Value and it does not agree with the generally accepted value which may be due to the test conditions adopted by Ramakrishna and Sundararajan (2005b)  
*h In 24hrs  
*i - used natural dry condition of fibres  
+j - width  
+k - At break  
+l Water absorption ratio (100 % humidity)  
+m At 20°C  
*n Strain at failure  
*o Data for mechanical properties are given as averages and 95% confidence interval  
*p Percentage moisture present on weight basis at normal atmospheric conditions  
*q MPa / (Kg/m³)  
**By Vol.  
*By mass
Figure 2.1: Stress-strain curves for natural fibres; (a) Paramasivan et al. (1984), (b) Munawar et al. (2007) * and (c) Satyanarayana et al. (1990).

(Graphs are reproduced here for better quality)

*Samples are fibre bundles
Abiola (2008) evaluated the mechanical properties (load-extension and stress–strain curves, elastic modulus, yield stress, stress and strain at break) of inner and outer coconut fibres experimentally, and the results were verified by a finite element method using a commercial software ABAQUS. The author found that the inner coconut fibre had a higher mechanical strength compared to that of outer fibre, but the outer coconut fibre had a higher elongation property which enabled it to absorb or withstand a higher stretching force.

### 2.2.2. Chemical properties

Coconut fibres contain cellulose, hemi-cellulose and lignin as major components. These compositions affect the different properties of coconut fibres. The pre-treatment of fibres changes the composition and ultimately alters not only its properties but also the properties of composites. Sometimes it improves the behaviour of fibres but sometimes its effect is not favorable. The chemical composition of coconut fibres is presented in Table 2.2.

<table>
<thead>
<tr>
<th>Fibre</th>
<th>Hemi-cellulose (%)</th>
<th>Cellulose (%)</th>
<th>Lignin (%)</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coir</td>
<td>31.1     (^a)</td>
<td>33.2     (^a)</td>
<td>20.5     (^a)</td>
<td>Ramakrishna and Sundararajan (2005a)</td>
</tr>
<tr>
<td></td>
<td>15 - 28(^b)</td>
<td>35 - 60(^b)</td>
<td>20 - 48(^b)</td>
<td>Agopyan et al. (2005)</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>43</td>
<td>45</td>
<td>Satyanarayana et al. (1990)</td>
</tr>
<tr>
<td></td>
<td>16.8</td>
<td>68.9</td>
<td>32.1</td>
<td>Asasutjarit et al. (2007)</td>
</tr>
<tr>
<td></td>
<td>0.15 – 0.25</td>
<td>36 - 43</td>
<td>41 – 45</td>
<td>Corradini et al. (2008)</td>
</tr>
</tbody>
</table>

\(^a\) The compositions are % by weight of dry and powdered fibre sample.

\(^b\) Chemical compositions are % by mass and the author took other researchers’ data.

### 2.3. Coconut fibre reinforced composites

#### 2.3.1. Paste

Aziz et al. (1981) cited the work of Das Gupta et al. (1978; 1979) who studied the mechanical properties of cement paste composites reinforced with different lengths and volume fractions of coconut fibres. Aziz et al. (1981) reported that the tensile strength and modulus of rupture of the cement paste increased when fibres up to 38 mm long fibre and
2. Background: Coconut fibres and its composites

4% volume fraction were used. A further increase in length or volume fraction reduced the strength of the composite. The tensile strengths of the cement paste composite were 1.9, 2.5, 2.8, 2.2 and 1.5 MPa when reinforced with 38 mm long coconut fibre at volume fractions of 2%, 3%, 4%, 5% and 6%, respectively. The corresponding modulus of rupture was 3.6, 4.9, 5.45, 5.4 and 4.6 MPa, respectively. With 4% volume fraction, they also studied the tensile strength of cement paste reinforced with different lengths of coconut fibres. With the fibre lengths of 25, 38 and 50 mm, the reported tensile strength was 2.3, 2.8 and 2.4 MPa, respectively.

2.3.2. Mortar

Slate (1976) investigated compressive and flexural strengths of coconut fibre reinforced mortar. Two cement-sand ratios by weight were considered: 1:2.75 with water cement ratio of 0.54, and 1:4 with water cement ratio of 0.82. Fibre content was 0.08%, 0.16% and 0.32% by total weight of cement, sand and water. The mortars for both design mixes without any fibres were also tested as a reference. Cylinders of 50 mm diameter and 100 mm height and beams of 50 mm width, 50 mm depth and 200 mm length were tested. The curing was done for 8 days only. It was found that, compared to that of plain mortar of both mix designs, all strengths were increased in the case of fibre reinforced mortar with all considered fibre contents. However, a decrease in the strength of mortar with an increase of fibre content was also observed.

2.3.3. Concrete

Reis (2006) performed third-point loading tests to investigate the flexural strength, fracture toughness and fracture energy of epoxy polymer concrete reinforced with coconut, sugarcane bagasse and banana fibres. The investigation revealed that fracture toughness and energy of the coconut fibre reinforced polymer concrete were the highest, and an increase of flexural strength up to 25% was observed with coconut fibres.

Baruah and Talukdar (2007) investigated the mechanical properties of plain concrete (PC) and fibre reinforced concrete (FRC) with different fibre volume fractions ranging from 0.5% to 2%. Steel, synthetic, jute and coconut fibres were used. Here, the discussion is limited to the coconut fibres reinforced concrete (CFRC) only. The
2. Background: Coconut fibres and its composites

cement:sand:aggregate ratio for plain concrete was 1:1.67:3.64, and the water cement ratio was 0.535. Coconut fibres having lengths of 4 cm and an average diameter of 0.4 mm with volume fractions of 0.5%, 1%, 1.5% and 2% were added to prepare CFRC. The sizes of specimens were (1) 150 mm diameter and 300 mm height for cylinders (2) 150 mm width, 150 mm depth and 700 mm length for beams, and (3) 150 mm cubes having a cut of 90 mm x 60 mm in cross-section and 150 mm high for L-shaped shear test specimens. All specimens were cured for 28 days. The compressive strength, splitting tensile strength, modulus of rupture using four point load test and shear strength are shown in Table 2.3 for PC and CFRC. It can be seen that CFRC with 2% fibres showed the best overall performance amongst all volume fractions. The compressive strength, splitting tensile strength, modulus of rupture and shear strength of CFRC with 2% fibres by volume fraction were increased by 13.7%, 22.9%, 28.0% and 32.7%, respectively, compared to those of PC. Their research indicated that all these properties were also improved for CFRC with 0.5%, 1% and 1.5% volume fraction in comparison to that of PC. Even for CFRC with a small fibre volume fraction of 0.5%, the corresponding properties were increased up to 1.3%, 4.9%, 4.0% and 4.7%, respectively.

<table>
<thead>
<tr>
<th>Fibre volume fraction (%)</th>
<th>Compressive Strength (MPa)</th>
<th>Split Tensile Strength (MPa)</th>
<th>Modulus of Rupture (MPa)</th>
<th>Shear Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>-</td>
<td>21.42</td>
<td>2.88</td>
<td>3.25</td>
<td>6.18</td>
</tr>
<tr>
<td>0.5</td>
<td>21.70</td>
<td>3.02</td>
<td>3.38</td>
<td>6.47</td>
</tr>
<tr>
<td>1.0</td>
<td>22.74</td>
<td>3.18</td>
<td>3.68</td>
<td>6.81</td>
</tr>
<tr>
<td>1.5</td>
<td>25.10</td>
<td>3.37</td>
<td>4.07</td>
<td>8.18</td>
</tr>
<tr>
<td>2.0</td>
<td>24.35</td>
<td>3.54</td>
<td>4.16</td>
<td>8.21</td>
</tr>
</tbody>
</table>

Islam et al. (2012) conducted an experimental investigation on the properties of coir and steel fibres reinforced normal-strength and high-strength concrete. Mix design was performed as per ACI method. Here, the discussion is limited to CFRC only. The workability test, compressive strength test, indirect tensile strength test, and flexural strength test were performed. The fibre volume fractions of 0%, 0.5%, and 1.0% were considered for a constant fibre length of 30 mm. In both normal strength concrete (NSC) and high strength concrete (HSC), the workability significantly reduced as the fibre dose increased. This research showed that 0.5% and 1.0% coir fibres improved flexural strength of NSC by 60%, and 0.5% coir fibre gives improved flexural and tensile...
2. Background: Coconut fibres and its composites

strengths of HSC by 6% only. Also, the addition of coir fibres in both types of concrete increased the ductility and toughness of concrete, whereas reduction of compressive strength was observed in the case of HSC.

Hasan et al. (2012) studied experimentally the physical and mechanical characteristics of CFRC. The mix design was 1:2:3 with a water-cement ratio of 0.4. Adhesive “seal frost” was also used for quick setting of the concrete, with a dosage of 70 grams per kg of cement. The volume percentages of fibres were 1, 3, 5 and 7 and the fibre length was 15-35 mm. The results showed that the compressive strength decreased as the fibres’ volume percentage increased. The density of CFRC decreased with an increase in fibre percentage in the concrete. It was also found that the coconut fibre concrete performed satisfactorily on the growth of cracks and crack widths compared with the conventional plain concrete. Finally, it was concluded that the use of coconut fibre has great potential in the production of structurally lightweight concrete, especially in the construction of low-cost concrete structures.

2.4. Application of coconut fibre reinforced composites in civil engineering

2.4.1. Roofing material

Cook et al. (1978) reported the use of coconut fibre reinforced cement composites as low cost roofing materials. The parameters studied were fibre lengths (2.5, 3.75 and 6.35 cm), fibre volumes (2.5%, 5%, 7.5%, 10% and 15%) and casting pressure (from 1 to 2 MPa with an increment of 0.33 MPa). They concluded that the optimum composite consisted of fibres with a length of 3.75 cm, a fibre volume fraction of 7.5% and was cast under a pressure of 1.67 MPa. A comparison revealed that this composite was much cheaper than locally available roofing materials.

Paramasivam et al. (1984) conducted a feasibility study on coconut fibre reinforced corrugated slabs measuring 915 mm long x 460 mm wide x 10 mm thick for low-cost housing. A cement–sand ratio of 1:0.5 and a water–cement ratio of 0.35 were used. Tests for flexural strength using third point loading were performed. A fibre length of 2.5 cm, a volume fraction of 3%, and a casting pressure of 0.15 MPa were recommended for
producing slabs with a nominal flexural strength of 22 MPa. The thermal conductivity and absorption coefficient for low frequency sound were comparable with those of asbestos boards.

Ramakrishna and Sundararajan (2005b) performed experiments on the impact resistance of slabs using a falling weight of 0.475 kg from a height of 200 mm. The slabs consisted of 1:3 cement–sand mortar with dimensions of 300 mm x 300 mm x 20 mm. They were reinforced with coconut, sisal, jute and hibiscus cannabinus fibres, having four different fibre contents of 0.5%, 1.0%, 1.5% and 2.5% by weight of cement and three fibre lengths of 20, 30 and 40 mm. A fibre content of 2% and a fibre length of 40 mm of coconut fibres showed the best performance by absorbing 253.5 J of impact energy. At ultimate failure, all except the coconut fibres exhibited fracture, while the coconut fibres showed fibre pullout. The failure was determined based on the number of blows required to open a crack, sufficient to propagate it through the entire depth of the specimen.

Li et al. (2007) studied fibre volume fractions and fibre surface treatment with a wetting agent for coir mesh reinforced mortar using nonwoven coir mesh matting. They performed a four-point bending test and concluded that cementitious composites, reinforced by three layers of coir mesh with a low fibre content of 1.8%, resulted in a 40% improvement in the maximum flexural strength. The composites were 25 times stronger in flexural toughness and about 20 times higher in flexural ductility. There was 20% improvement in flexural strength by fibre surface treatment.

Agopyan et al. (2005) studied coconut and sisal fibres as a substitute for asbestos in roofing tiles. The dimensions of the tiles were 487 mm x 263 mm x 6 mm. Three-point bend test specimens with 2% total fibre volume fraction, support span of 350 mm, and deflection rate of 5 mm/min were employed for the determination of the maximum load. After the ageing periods of 16 and 60 months, the corresponding maximum loads taken by coir tiles were 235 N and 248 N, respectively, while that by sisal tiles were 237 N and 159 N, respectively. The major benefit of reinforced tiles was their higher energy absorption (22%) than that of unreinforced tiles. This capability can help to avoid fragile rupture of tiles during transportation or installation.
2. Background: Coconut fibres and its composites

2.4.2. Wall panels / boards

Mohammad (2005) tested wall panels made of gypsum cement and coconut fibre. Bending and compressive strength, moisture content, density and water absorption were investigated. As expected, coconut fibres did not contribute to bending strength of the tested wall panels, but compressive strength increased with the addition of coconut fibres. There was no considerable change of moisture content with coconut fibres, although moisture content increased with time. Water absorption of panels was not significantly affected with an increase in fibre content.

Asasutjarit et al. (2007) determined the physical (density, moisture content, water absorption and thickness swelling), mechanical (modulus of elasticity, modulus of rupture and internal bond) and thermal properties of coir-based light weight cement board after 28 days of hydration. The physical and mechanical properties were measured by Japanese Industrial Standard JIS A 5908-1994, and the thermal properties were tested according to JIS R 2618. The parameters studied were fibre length, coir pre-treatment and mixture ratios. Boiled and washed 6 cm long fibres with the optimum cement:fibre:water weight ratio of 2:1:2 gave the highest modulus of rupture and internal bond amongst the tested specimens. The board also had a thermal conductivity lower than other commercial flake board composites.

Luisito et al. (2005) invented coconut fibre boards (CFBs) for applications such as tiles, bricks, plywood and hollow blocks. These boards were used for internal and exterior walls, partitions and ceilings (Figure 2.2). CFBs consisted of 70% cement and 30% fibre by weight. CFBs had a density of 550-650 kg/m³, bending strength (modulus of rupture) of 8.3 kg/cm² and water absorption of 32%.
Abdullah et al. (2011) investigated the physical and mechanical properties of cement panels reinforced with treated and untreated coconut fibres. The treatment of coconut fibres was done by immersing in water, washing repeatedly in running tap water and then drying in solar radiation. The cement:sand:fibre ratios were 1:1:0, 1:0.97:0.03, 1:0.94:0.06, and 1:0.91:0.09 with a water-cement ratio of 0.55. The samples were 160 mm long x 40 mm wide x 40 mm deep for the compression test, and 100 mm long x 100 mm wide x 40 mm deep for density, moisture content and water absorption tests. The samples were cured for 28 days. The compressive strength and moisture content of panels increased with increasing content of treated coconut fibre. The compressive strength of panels reduced and water absorption increased with increasing content of untreated coconut fibre. The density of panels decreased with the addition of untreated and treated coconut fibre. Abdullah et al. (2012) studied the modulus of rupture (MOR) of cement panels reinforced with untreated fibres having the same mix design ratios for curing periods of 7, 14 and 28 days. The MOR increased with increasing fibre content and with the duration of curing time.
2. Background: Coconut fibres and its composites

2.5. Durability of coconut fibres in cementitious composites

The durability of natural fibres remained a topic of interest for many researchers. A detailed literature research is presented here about the durability of coconut fibres and its composites. Ramakrishna and Sundararajan (2005) investigated the variations in chemical composition and tensile strength for four natural fibres (namely, coconut, sisal, jute and hibiscus cannabinus fibres) when subjected to alternate wetting and drying and continuous immersion for 60 days in water, saturated lime and sodium hydroxide. The chemical composition of all fibres changed because of immersion in the considered solutions. Continuous immersion was found to be critical due to the loss of their tensile strength. However, coconut fibres were reported best for retaining a good percentage of its original tensile strength in all tested conditions.

Toledo Filho (2000) studied the durability of coconut fibres as strength loss occurred over 420 days for three types of treatments: fibres stored in tap water of pH 8.3; fibres stored in a solution of calcium hydroxide of pH 12; and fibres stored in a solution of sodium hydroxide of pH 11. A significant reduction in strength was observed for calcium hydroxide solution. It has been found that coconut fibres retained 58.7% of their original strength after 210 days. Coconut fibres immersed in sodium hydroxide retained 60.9% of their initial strength after 420 days. The higher alkaline attack by calcium hydroxide is probably associated with crystallisation of lime in the pores of the fibres. The loss of strength over time was least for fibres immersed in tap water.

Toledo Filho et al. (2003) found the reduction in strength of natural fibre (sisal and coconut) reinforced composites when subjected to ageing conditions (stored in water with temperature of about 18°C, controlled cycles of wetting and drying and London open air weathering). Accordingly, Toledo Filho et al. (2003) studied several approaches to improve the durability of natural fibre reinforced composites. These included: carbonation of the matrix in a CO₂-rich environment; the immersion of fibres in slurried silica fume (SF) prior to incorporation in the Ordinary Portland Cement (OPC) matrix; and partial replacement of OPC by undensified SF or blast furnace slag. The behaviour was analysed in terms of effects of aging in water, exposure to cycles of wetting and drying and open air weathering on the flexural behaviour. The immersion of natural fibres in a SF slurry before adding them to the cement based composites was found to be an
2. Background: Coconut fibres and its composites

Effective means of reducing embrittlement of the composite in the considered conditions. The early cure composites in a CO$_2$-rich environment and the partial replacement of OPC by undensified silica fume were also efficient approaches in obtaining natural fibres with improved durability.

John et al. (2005) studied the coconut fibre reinforced blast-furnace slag cement mortar taken from the internal and external walls of a 12 year old house. The panel of the house was produced using 1:1.5:0.504 (binder: sand: water, by mass) mortar reinforced with 2% of coconut fibres by volume. The cement was fully carbonated after considered duration. Fibres removed from the old samples were reported to be undamaged. No significant difference was found in the lignin content of fibres removed from external and internal walls, confirming the durability of coconut fibres in cement composites.

Li et al. (2006) studied untreated and alkalized coconut fibres of length 40 mm as reinforcement in cement composites for normal curing and accelerated ageing. Mortar was mixed in a laboratory mixer at a constant speed of 30 rpm, with a cement:sand:water: super plasticizer ratio of 1:3:0.43:0.01 by weight, and fibres were slowly put into the running mixer. For accelerated ageing in the last two days of curing, the specimens were taken out of the water tank, air dried, and then frozen at −$10^{\circ}$C for 24 hours, followed by thawing the specimens at $24^{\circ}$C for 2 hours and baking them in a forced draft oven at $90^{\circ}$C for 22 hours. The resulting mortar with treated fibres had lesser flexural strength (0.8%) and ductility (4%) but greater toughness (19%) than mortar with untreated fibres for normal cured specimens. However, for accelerated ageing specimens, treated fibres reinforced mortar had a lesser flexural strength (38%) but greater toughness (44%) and ductility (73%) than that of untreated fibres reinforced mortar.

Sivaraja et al. (2010) determined the static strength properties of coconut fibre reinforced concrete (CFRC) at an interval of 3 months for a period of 2 years under alternate wetting and drying conditions. The specimens had initial curing of 28 days. The wetting was given by immersing the specimens for three days underwater and then allowing them to dry in the open air for four days. This was repeated until the testing time. The fibre dose was 1.5% volume fraction of concrete volume. It was found that the compressive strength increased from 27.8 to 30.3 MPa, the splitting tensile strength increased from 3.28 to 3.58 MPa, and the modulus of rupture increased from 4.79 to 4.85 MPa at the ages of 28 days.
to 2 years, respectively. This clearly shows that the alternate wetting and drying for a period up to 2 years has not affected the mechanical properties of CFRC, elaborating signs of fibre durability to some extent.

2.6. Summary

The versatility and applications of coconut fibres in cement composites was discussed in detail. Coconut fibres are reported as the toughest and most energy absorbent material of all fibres. Coconut fibres are cheaper than artificial fibres and are renewable. It is concluded that coconut fibres have the potential to be used in composites for different purposes. Various aspects of many coconut fibre reinforced composites have already been investigated: the resulting products are economical and better than the commercially available products as reported by many researchers. The studies of John et al. (2005) and Sivaraja et al. (2010) also showed the durability of coconut fibre reinforced composites with no encountered problems over a time of 12 and 2 years, respectively. However, the investigations of Li et al. (2006) and Toledo et al. (2000) showed concerns regarding the durability of coconut fibre reinforced composites when subjected to severe ageing conditions. This may have limitations for the use of coconut fibre reinforced composites in marine areas. Toledo et al. (2003) also recommended some solutions for increasing its durability. In civil engineering, coconut fibres have been used as reinforcement in cement composites, mainly for nonstructural components. There is a need for investigating the static and dynamic behaviours of coconut fibre reinforced concrete for its application as a structural material in beams, columns, shear walls and blocks.

Steel reinforced concrete is expensive for most people residing in the rural areas of developing countries. Thus, in those areas, the usage of steel reinforced masonry is not widely adopted for earthquake-resistant housing. A safe economical solution for housing in earthquake prone areas will surely help in avoiding structure collapse, ultimately resulting in the saving of many lives. The price of reinforcing materials will automatically be cut down if local resources are used. As stated earlier, there are a wide range of natural fibres, namely sisal, bamboo, coir (coconut fibre), jute, and many others. The researchers focused on finding the optimum fibre length and content for the composites. Studies showed that natural fibres are good alternatives because they are not only cost effective
but also promote sustainable buildings. The researchers investigated cement composites with natural fibres for non-structural members like boards for partition walls, corrugated and simple slab panels for roofing. To reduce construction time and cost of structures and to make construction easy, researchers have also suggested the use of interlocking blocks to replace the normal bricks and eliminate mortar from masonry construction for resisting static loading. To the best knowledge of author, the research of interlocking structures for resisting dynamic loading is very limited. Investigation on structures with CFRC interlocking blocks and rope reinforcement under dynamic loading is unknown. Therefore, to achieve the overall goal of developing cheap earthquake resistant housing, new interlocking blocks were invented and prepared with CFRC. The purpose of adding coconut fibres in concrete is to enhance its compressive and flexural toughness.
Chapter 3.

Properties of coconut fibre reinforced concrete

Related papers:


3.1. Introduction

It has been mentioned previously that coconut fibres have the highest toughness amongst natural fibres. Concrete is strong in compression, and its other properties like tensile strength, toughness, and damping can be improved significantly by incorporating coconut fibres. Since the properties of a building material is important from the point of view of seismic design, the mechanical and dynamic properties of coconut fibre reinforced concrete (CFRC) members need to be well understood. In this chapter, in addition to their mechanical properties, the damping ratios and fundamental frequencies of simply supported CFRC beams were determined experimentally. A comparison of the static and dynamic modulus was conducted. The influences of 1%, 2%, 3% and 5% fibre contents by mass of cement and fibre lengths of 2.5, 5 and 7.5 cm were investigated. Higher fibre content is not considered because of the expected balling effect which can reduce the compressive strength to a larger extent. To evaluate the effects of coconut fibres in
improving the properties of a fibre-concrete composite, the properties of plain concrete were used as a reference.

To the best knowledge of the author, no literature study has reported dynamic properties of CFRC. Dynamic tests were performed only for concrete reinforced by other fibres, such as polyolefin fibres (Yan et al., 2000) or rubber scrap (Zheng et al., 2008). To determine the effects of fibre parameters on CFRC properties, thorough investigations involving different fibre lengths and contents are required in order to have reliable insights.

3.2. Experimental work

3.2.1. Materials used

Ordinary Portland cement, sand, aggregates, tap water and imported brown coconut fibres were used for preparation of CFRC. The maximum size of the aggregates was 12 mm (passing through a 12 mm sieve and retained at a 10 mm sieve). The mean diameter of the coconut fibres was 0.25 mm.

3.2.2. Fibre preparation

The fibre preparation was developed by the author after different hit and trail methods keeping in mind the capability of non-skilled labour and available resources in developing countries. Since fibres were in a hydraulic compressed form, preparation of the fibres into the required length was time consuming and laborious. Different approaches were taken to get fibres into the required length quickly without much success. Finally, coconut fibres were loosened and soaked in tap water for 30 minutes to soften the fibres and to remove coir dust. Fibres were then washed and soaked again for 30 minutes. Washing and soaking were repeated three times. Fibres were then straightened manually and combed with a steel comb. To accelerate the drying process, wet long fibres were put in an oven at 30 °C for 10-12 hours where for the most part moisture was removed. The fibres were further dried in the open air to avoid burning of fibres in the oven, combed again and finally cut into the required length with a guillotine. It may be noted that the precut fibres are also available commercially at relatively high cost, as they are prepared for special
purposes such as brushes and mats. This cost can be reduced if fibres are mechanically prepared on a large scale.

3.2.3. Mix design

The immediate target of new technology, in terms of implementation, is in rural areas where the availability of skilled labour is also a problem. The selection of a mix design was made as simple as possible. Cement was the only binding agent which was responsible for the strength of the concrete. It was kept the same and other quantities were increased as multiples of the cement quantity. For plain concrete (PC), the mix design ratio for cement, sand and aggregates was 1:2:2, with a water-cement (W/C) ratio of 0.48. The mix design for CFRC was the same as for plain concrete, except that (1) more water was added (stepwise to avoid bleeding) to make CFRC workable because of the fibre addition, and (2) different lengths and contents of fibres were added and the same amount of aggregates was deducted from the total mass of aggregates. All materials were taken by mass of cement. It is well known that the W/C ratio has an influence on properties of concrete, but compaction is also an important factor. The increased W/C ratios for CFRC were to ensure its proper compaction with workable mix so that a good strength could be achieved. The obtained properties of CFRC with respective W/C ratio can be taken as the optimum one, because any further addition of water will cause bleeding, ultimately reducing CFRC strength in a hardened state. Reduced W/C ratio can lead to improper compaction, again resulting in lower strength.

3.2.4. Casting procedure

A pan type concrete mixer was used in preparing the plain concrete. All materials were put in the mixer pan along with the water, and the mixer was rotated for three minutes. The slump was 50 mm. For preparing CFRC, a layer of coconut fibres was spread in the pan, followed by spreading of aggregates, cement and sand. The first layer of fibres was hidden under the dry concrete materials with the help of a spade. Then, another layer of coconut fibres was spread, followed by layers of aggregates, cement and sand. This process was repeated until the remaining materials were put into the mixer pan. Approximately, three quarters of the water (according to a water cement ratio of 0.48, which was the same as that of plain concrete) was added, and the mixer was rotated for 2
3. Properties of coconut fibre reinforced concrete

minutes. The remaining water was then added and the mixer was again rotated for 2 minutes. All CFRCs were not workable at this stage, so more water was added in small increments to make the CFRC workable. The mixer was rotated for one minute after each increment of water. The water cement (W/C) ratio of CFRC varied from 0.49 to 0.62, which was to make sure that there should not be any bleeding. Finally, the mixer was rotated for three minutes to ensure the CFRC was well mixed.

The effects of the fibre content and length on the required W/C ratio of CFRC are shown in Figure 3.1. It can be observed that increasing fibre content or length resulted in an increased W/C ratio. The W/C ratio of all CFRCs was greater than that of PC.

A slump test for PC and CFRC was always performed before pouring into moulds. The slumps for CFRCs were 10–40 mm, but CFRC was workable despite this low slump. The reason for this low slump was the presence of fibres in the concrete. Further addition of water could lead to bleeding which can result in low compressive strength. The slump is usually decreased with an increase in fibre content. The slump of CFRC with 5 cm long fibres increased when compared to the 2.5 cm long fibres for all considered fibre contents. The slump decreased when the fibre length increased from 5 to 7.5 cm. For CFRC with 2.5 cm fibres, more fibres were available for holding surrounding fresh concrete, causing low slump. In Figure 3.2, the effects of fibre content and length on slump are shown. The slump of all CFRCs was less than that of PC.

![Figure 3.1: Effect of fibre (a) content and (b) length on water cement ratio](image)
CFRC was poured into the moulds in layers. Each layer was compacted first by tamping 25 times with a 16 mm diameter rod and then the moulds were lifted up to a height of approximately 200–300 mm and dropped to the floor for self compaction of the fibre concrete and to remove air voids from CFRC. All specimens were cured for 28 days before testing.

![Figure 3.2: Influence of fibre (a) content and (b) length on slump](image)

3.2.5. Specimens

Cylinders 100 mm in diameter and 200 mm in height and beamlets 100 mm wide, 100 mm deep and 500 mm long were prepared for PC and CFRC. Beams with the dimensions of 100 mm width, 100 mm depth and 915 mm length, were prepared only for CFRC, because the PC beams broke during demoulding. The cylinders and beamlets were used for their material properties while the beams were used to determine the fundamental frequency, damping ratio and dynamic modulus of elasticity. A set of three samples for each particular test was produced. A total of 6 cylinders, 3 beamlets and 3 beams were prepared for one combination of a particular length and fibre content. Table 3.1 shows the specimens labeling scheme. In total 72 cylinders, 36 beamlets and 11 beams were tested.

A, B and C along with labels depicted the mark of each specimen (for example, for A1 combination, the marks were A1A, A1B and A1C for considered specimens. A1D, A1E and A1F were the marks for cylinders in splitting tensile strength tests). All specimens were white-washed before testing to enable a clear identification of cracks.
3. Properties of coconut fibre reinforced concrete

### Table 3.1: Specimens labelling scheme

<table>
<thead>
<tr>
<th>Fibres content *</th>
<th>Fibre length</th>
<th>0 cm</th>
<th>2.5 cm</th>
<th>5 cm</th>
<th>7.5 cm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Symbol</td>
<td>O</td>
<td>1</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>0 %</td>
<td>O</td>
<td>O</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1 %</td>
<td>A</td>
<td>-</td>
<td>A1</td>
<td>A2</td>
<td>A3</td>
</tr>
<tr>
<td>2 %</td>
<td>B</td>
<td>-</td>
<td>B1</td>
<td>B2</td>
<td>B3</td>
</tr>
<tr>
<td>3 %</td>
<td>C</td>
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<td>C1</td>
<td>C2</td>
<td>C3</td>
</tr>
<tr>
<td>5 %</td>
<td>D</td>
<td>-</td>
<td>D1</td>
<td>D2</td>
<td>-</td>
</tr>
</tbody>
</table>

*By mass of cement

### 3.2.6. Testing procedure

#### 3.2.6.1. Cylinder and beamlet tests

The properties of the matrix (PC/CFRC) were determined using standard procedures (NZS 3112: Part2: 1986). All cylinders were tested in a compression testing machine to determine static modulus of elasticity $E_{\text{static}}$, compressive strength $\sigma$, corresponding strain $\varepsilon$, compressive toughness $T_c$ and splitting tensile strength STS. Each cylinder was capped with plaster of Paris for uniform distribution of the load before testing of the $E_{\text{static}}$, $\sigma$ and $T_c$.

All beamlets were tested in a universal testing machine with a capacity of 100 kN using a 4-point loading to obtain modulus of rupture MOR, corresponding deflection $\Delta$, flexural toughness $T_f$ and cracking load $P_{\text{crack}}$. Cracking load is the load taken by the fibres and part of the concrete after the first visible crack is formed.

#### 3.2.6.2. Beam tests

The experimental setup for testing beams is shown in Figure 3.3. A small impact load $P_{\text{impact}}$ was applied three times at the mid-span of the beam with the help of a calibrated hammer. The response was recorded by accelerometers, located near to mid-span (Figure 3.3a). Then the same beam was put under a static load $P_{\text{static}}$ of 1 kN in the universal testing machine. Deflection was noted using a linear variable differential transformer (LVDT), and Figure 3.4 shows the typical load–displacement curves. Again, a small
impact load was applied three times and the response was recorded. The static load was increased by 1 kN, followed by the impact load for recording response. This procedure was repeated until the first crack in the beam appeared. The static load before producing the first crack was taken as the reference for the just before crack stage. The application of the load before this stage is not shown in Figure 3.4(b) because the beam remained in the elastic range. Note that the impact load was applied three times to obtain the average value of a particular dynamic property. The magnitude of the impact load was kept small so that no additional damage was produced since the goal was to identify the fundamental frequency and damping at different damage stages. Four stages were considered: (i) Uncracked beam [S1], (ii) just before cracking [S2], (iii) cracked beam [S3] and (iv) after cracks occurred following 2–3 cycles of static load [S4a, S4b and S4c]. Crack development is shown in Figure 3.5. Each cycle of loading consists of applying a static load on the cracked beam up to a certain deflection and then releasing the load for measuring its dynamic properties. The results of the dynamic testing are presented in section 3.4.

Figure 3.3: Experimental setup for dynamic tests (a) Hitting with calibrated hammer at mid-span and (b) introducing damage by four-point static loading
3. Properties of coconut fibre reinforced concrete

![Image of load displacement curves at different damage stages of CFRC beams]

**Figure 3.4:** Typical load displacement curves at different damage stages of CFRC beams

![Images of crack development](a) At cracking [S3] (b) After crack - cycle 1 [S4a] (c) After crack - cycle 2 [S4b]

**Figure 3.5:** Crack development

3.3. Mechanical properties of CFRC

3.3.1. Static modulus of elastic $E_{static}$

$E_{static}$ is calculated as the ratio of stress change to strain change in the elastic range. Stress-strain curves of PC and CFRC with 5% and 5 cm long fibres are shown in Figure 3.6(a). The stress-strain relationship for each sample shows the average strain readings taken by two LVDTs attached to the specimens. Crushed PC and CFRC cylinder specimens with fibre content 5% and 5 cm long fibres for determining $E_{static}$ and $\sigma$ are shown in Figure 3.6(b). It can be noted that the spalling of concrete was observed in the case of PC cylinders, whereas only cracks were produced for CFRC cylinders.
3. Properties of coconut fibre reinforced concrete

Figure 3.6: Compressive test

Figure 3.7: Influence of fibre (a) content and (b) length on static modulus of elasticity $E_{\text{static}}$

Figure 3.7 shows the influence of fibre content and length on $E_{\text{static}}$. The solid straight line is $E_{\text{static}}$ of PC. $E_{\text{static}}$ of CFRCs decreased with increasing fibre content and length. However, $E_{\text{static}}$ of CFRC having 2.5 cm long fibres with increasing fibre contents showed a different trend: it first increased and then decreased (Figure 3.7(a)), and these values were higher than that of PC. This might be because of shorter length of fibres causing less strain in CFRC which ultimately resulted in higher elastic modulus. With lower fibre percentage, the $E_{\text{static}}$ of CFRC is greater, whereas the $E_{\text{static}}$ of CFRC with high fibre content is smaller than that of PC (Figure 3.7(b)). Compared to PC value, adding fibres caused about 15% increase or decrease of $E_{\text{static}}$ of CFRC.

The mathematical models have been proposed based on statistical tools for the predicting the mechanical properties of steel fibre reinforced concrete (Pawade et al. 2011; Song and...
3. Properties of coconut fibre reinforced concrete

Hwang, 2004; Xu and Shi, 2009). Taking these studies as a guide, the following simple equation is proposed for estimating static modulus of elasticity in GPa:

\[ E_{\text{static}} = X_s + Y_s c + Z_s c^2 \]  \hspace{1cm} (3.1)

where \( c \) is the fibre content parameter of the values of 0, 1, 2, 3 or 5 and \( X_s, Y_s \) and \( Z_s \) are constants corresponding to fibre length \( l_f \). The values can be taken from Table 3.2. The percentage error in \( E_{\text{static}} \) with Equation (3.1) is less than one percent (Table 3.4).

<table>
<thead>
<tr>
<th>Fibre length ((l_f))</th>
<th>( X_s )</th>
<th>( Y_s )</th>
<th>( Z_s )</th>
</tr>
</thead>
<tbody>
<tr>
<td>-</td>
<td>33</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2.5</td>
<td>32.6</td>
<td>-2.2</td>
<td>0.37</td>
</tr>
<tr>
<td>5</td>
<td>34</td>
<td>-1</td>
<td>0</td>
</tr>
<tr>
<td>7.5</td>
<td>33</td>
<td>3.6</td>
<td>-0.66</td>
</tr>
</tbody>
</table>

**Table 3.2:** Constants for static modulus of elasticity \( E_{\text{static}} \)

---

3.3.2. Compressive strength \( \sigma \)

The maximum stress value of the stress–strain curves is taken as compressive strength \( \sigma \). Figure 3.8 shows the influence of fibre content and length on \( \sigma \). The straight line is the compressive strength of PC. The compressive strength of CFRC decreased with higher fibre content, but it first increased and then decreased with increasing fibre length. Because of the fibres, compressive strength increased up to 24%.

**Figure 3.8:** Influence of fibre (a) content and (b) length on compressive strength \( \sigma \)
3. Properties of coconut fibre reinforced concrete

Compared to that of PC, in most cases, compressive strength increased. Only with 7.5 cm long fibres having 2% or 3% fibre content, $\sigma$ of CFRC was less than that of PC. This may be caused by creation of air voids because of long fibres with relatively high fibre content.

3.3.3. Compressive toughness $T_c$

$T_c$ is calculated as the total area under the stress–strain curve (Mindess and Zhang, 2009; Libre et al., 2011). Other researchers have taken it as the area after the maximum stress up to 1% strain (Chan and Bindigana vile, 2010). Figure 3.9 shows the effect of fibre content and length on $T_c$. The solid straight line indicates $T_c$ of PC. Compressive toughness increased with higher fibre content. CFRC with 5% fibre content and 5 cm long fibres has the highest toughness of 0.32 MPa.

![Figure 3.9: Influence of fibre (a) content and (b) length on compressive toughness $T_c$](image)

3.3.4. Splitting tensile strength STS

Load-time histories, obtained during STS tests of PC and CFRC with 5% and 5 cm long fibres, are shown in Figure 3.10(a). The maximum load from these curves is taken for the calculation of STS. PC cylinders broke into two halves at the maximum load, while CFRC cylinders did not break (the two halves held together) after cracking even when the test was continued up to more than 800 seconds in order to observe the post peak load behaviour. The two pieces were held together until the end of the test. One of the tested PC and CFRC cylinder specimens are shown in Figure 3.10(b).
Figure 3.11 shows the influence of fibre content and length on STS of CFRC. The solid straight line is STS of PC. STS decreases with higher fibre content. However it first increased and then slightly reduced with increasing fibre length. In the case of 1% fibre content, STS kept on increasing with the length of fibre (Figure 3.11(b)). Compared to the STS of PC, adding fibres can increase or decrease the splitting tensile strength up to 11%.

![Figure 3.10: Splitting tensile test](image1)

(a) Load-time relation from STS test  
(b) Cylinders after STS test

**Figure 3.10: Splitting tensile test**

![Figure 3.11: Influence of fibre (a) content and (b) length on splitting tensile strength STS](image2)

(a)  
(b)

**Figure 3.11: Influence of fibre (a) content and (b) length on splitting tensile strength STS**

### 3.3.5. Modulus of rupture MOR

Figure 3.12(a) displays the load–displacement curves of PC and CFRC with 5% fibre content and 5 cm long fibres. The maximum load from these curves is taken for the calculation of MOR. PC beams broke into two pieces at the maximum load (Figure 3.12(b)), but CFRC beams held together even after the maximum load (Figure 3.12(c)).
3. Properties of coconut fibre reinforced concrete

Figure 3.12(d) shows the cross-section of a CFRC beam. CFRC beams were intentionally broken into two halves to observe the fibre failure. Two types of fibre failure were observed: (i) fibre breaking and (ii) fibre pullout. The fibres were randomly distributed in the concrete to bridge the crack. Those fibres which had sufficient embedment across the crack on both sides broke, but those fibres which had relatively less embedment at one side of the crack pulled out. More pulled out fibres were observed with higher fibre content, whereas this pullout decreased as expected with increasing fibre length.

![Load-displacement curves for MOR](image1)

(a) Load-displacement curves for MOR  
(b) Tested PC beam

![Tested CFRC beam](image2)

(c) Tested CFRC beam  
(d) Cross-section of a tested CFRC beam

**Figure 3.12:** Beamlet test

![Influence of fibre content and length on MOR](image3)

(a) Influence of fibre content on modulus of rupture MOR

![Influence of fibre length on MOR](image4)

(b) Influence of fibre length on modulus of rupture MOR

**Figure 3.13:** Influence of fibre (a) content and (b) length on modulus of rupture MOR
3. Properties of coconut fibre reinforced concrete

Figure 3.13 displays the influence of the fibre content and length on the \textit{MOR}. The solid line is the \textit{MOR} of PC. With a higher fibre content and longer fibres, the \textit{MOR} increases. However, \textit{MOR} of PC beams is mostly higher than that of CFRC beams. Compared to PC value \textit{MOR} of CFRC with 5\% fibre content and 5 cm long fibres increased slightly up to 4\%.

3.3.6. Flexural toughness

Flexural toughness is measured as the total toughness index (\textit{TTI}). It is the ratio of the area under the load–displacement curve up to the maximum deflection to the area under the curve up to the load corresponding to the first crack. In Figure 3.12(a), it is the ratio of the area under the curve up to 18 mm deflection to the area under the curve up to 1.5 mm deflection. Usually, the toughness index is taken as the area under the curve up to 3, 5.5 or 10.5 times the first-crack deflection to the area under the curve at the first-crack deflection, and these toughness indices are notated as \textit{I}_5, \textit{I}_{10} and \textit{I}_{15}, respectively (Richardson et al., 2010).

Figure 3.14 shows the effects of fibre content and length on \textit{TTI}. The solid straight line is \textit{TTI} of PC. The \textit{TTI} increases with increasing fibre content (Figure 3.14(a)). As far as fibre length is concerned, for fibre contents of 1\%, 2\% and 3\%, the \textit{TTI} increases a little when fibre length changes from 2.5 to 5 cm and then it decreases slightly when fibre length increases to 7.5 cm (Figure 3.14(b)). The possible reason can be explained as follows: (i) when fibre length was 2.5 cm, more fibres were available for bridging the crack, however a shorter fibre embedment resulted in pullout of fibres; (ii) when fibre length was 5 cm, relatively fewer fibres were available but sufficient embedment length was there to hold the cracks. This results in a higher \textit{TTI} compared to that with 2.5 cm long fibres; (iii) when the fibre length is 7.5 cm, the number of fibres is further reduced, resulting in a lower \textit{TTI} compared to that with 5 cm long fibres. The CFRC with 5\% fibre content and 5 cm long fibres has the highest toughness index of 10.1.
3. Properties of coconut fibre reinforced concrete

3.3.7. Density

Figure 3.15 shows the effects of fibre content and length on the density of CFRC. The solid straight line indicates the PC density. As expected, the density of CFRC decreased with the higher fibre content and increased with the shorter fibre length. In general, the density of CFRC decreases up to 4% when compared to PC. A smaller density is significant because less inertia forces will be activated in earthquakes and, thus, smaller structural dimensions are required to withstand the reduced earthquake impact.

Figure 3.14: Influence of fibre (a) content and (b) length on total toughness index TTI

Figure 3.15: Influence of fibre (a) content and (b) length on density
3. Properties of coconut fibre reinforced concrete

3.4. Dynamic properties of CFRC simply supported beams

3.4.1. Damping ratio $\xi$ and fundamental frequency $f$

Four damage stages were considered: (i) uncracked beam [S1], (ii) just before cracking [S2], (iii) cracked beam [S3] and (iv) after cracks occurred following some cycles of static loading [S4a, S4b and S4c] (see Figure 3.5 and 3.6). Typically recorded acceleration time-histories at stages S1, S3 and S4b are shown in Figure 3.16.

![Figure 3.16: Typical recorded acceleration time histories of CFRC beam](image)

Figure 3.16: Typical recorded acceleration time histories of CFRC beam

![Figure 3.17: Effect of fibre content on dynamic properties of CFRC beams with 7.5 cm long fibres at different damage stages](image)

Figure 3.17: Effect of fibre content on dynamic properties of CFRC beams with 7.5 cm long fibres at different damage stages
3. Properties of coconut fibre reinforced concrete

![Graphs showing damping ratio and fundamental frequency](image)

**Figure 3.18:** Influence of fibre (a) content and (b) length on dynamic properties of CFRC beams
A logarithmic decrement is used for calculating the damping ratio of simply supported CFRC beams. \( f \) is calculated from the period of the recorded acceleration time histories. \( \xi \) of CFRC beams increased and \( f \) decreased with the formation of cracks. The effects of fibre content on \( \xi \) and \( f \) are displayed in Figure 3.17 for CFRC beams having a fibre length of 7.5 cm at different damage stages. CFRC beam with 5% fibre content has the highest damping and the lowest fundamental frequency in an uncracked and cracked stage when compared to that of CFRC beams with fibre contents of 1% and 2%.

The effects of fibre content and length on the dynamic properties of CFRC beams before and after cracks are shown in Figure 3.18. As expected, compared to the properties at a damaged stage, the fibre effect on the dynamic properties before damage is not so pronounced. However, as anticipated, after cracking \( \xi \) increases and \( f \) decreases considerably.

### 3.4.2. Dynamic modulus of elasticity \( E_{\text{dynamic}} \)

Fundamental frequency is used to define the actual beam dynamic elastic modulus \( E_{\text{dynamic}} \) (Zheng et al., 2008). Figure 3.19 shows a comparison of \( E_{\text{dynamic}} \) of CFRC for considered parameters. It decreases with increasing fibre content and length.

![Figure 3.19: Influence of fibre content and length on dynamic modulus of elasticity \( E_{\text{dynamic}} \)](image)
Similar to equation 3.1, the following empirical equation is proposed to predict dynamic modulus of elasticity in GPa:

\[ E_{\text{dynamic}} = X_d + Y_d \cdot c + Z_d \cdot c^2 \]  

(3.2)

where \( c \) is fibre content (0, 1, 2, 3 or 5) and \( X_d, Y_d \) and \( Z_d \) are constants corresponding to fibre length \( l_f \). The values are given in Table 3.3. The percentage error in \( E_{\text{dynamic}} \) with Equation (3.2) is less than 2% (Table 3.4).

### Table 3.3: Constants for dynamic modulus of elasticity \( E_{\text{dynamic}} \)

<table>
<thead>
<tr>
<th>Fibre length</th>
<th>Constants</th>
</tr>
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<td>( l_f )</td>
<td>( X_d )</td>
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<td>2.5</td>
<td>39.2</td>
</tr>
<tr>
<td>5</td>
<td>37</td>
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<tr>
<td>7.5</td>
<td>29.7</td>
</tr>
</tbody>
</table>

### Table 3.4: Comparison of modulus of elasticity

<table>
<thead>
<tr>
<th>Concrete</th>
<th>Fibre content (%)</th>
<th>Modulus of elasticity (GPa)</th>
<th>% Difference (A and D)</th>
<th>% Difference (A and B)</th>
<th>% Difference (D and E)</th>
<th>% Difference (A and D)</th>
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</thead>
<tbody>
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<td>PC</td>
<td>-</td>
<td>A: 33.10</td>
<td>B: 33.00</td>
<td>C: -0.30</td>
<td>D: 33.10</td>
<td>E: 33.00</td>
</tr>
<tr>
<td>CFRC (2.5 cm)</td>
<td>1</td>
<td>38.56</td>
<td>38.92</td>
<td>-0.94</td>
<td>35.85</td>
<td>35.94</td>
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<tr>
<td></td>
<td>2</td>
<td>38.80</td>
<td>38.08</td>
<td>1.86</td>
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</tr>
<tr>
<td>CFRC (5 cm)</td>
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<td>34.46</td>
<td>34.54</td>
<td>0.24</td>
<td>32.94</td>
<td>33.00</td>
</tr>
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<td>0.05</td>
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<td>31.00</td>
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<td>0.47</td>
<td>29.28</td>
<td>29.00</td>
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<td>CFRC (7.5 cm)</td>
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</tr>
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<td></td>
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<td>29.16</td>
<td>0.26</td>
<td>29.31</td>
<td>29.33</td>
</tr>
</tbody>
</table>
3. Properties of coconut fibre reinforced concrete

### Table 3.5: Consequence of CFRC for mechanical properties

<table>
<thead>
<tr>
<th>Concrete type</th>
<th>(E_{\text{static}}) (GPa)</th>
<th>(\sigma) (MPa)</th>
<th>STS (MPa)</th>
<th>MOR (MPa)</th>
<th>Density (kg/m(^3))</th>
<th>(T_c) (MPa)</th>
<th>TTI ((^{-}))</th>
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</thead>
<tbody>
<tr>
<td>PC</td>
<td>33.1</td>
<td>34.7</td>
<td>3.82</td>
<td>4.34</td>
<td>2338</td>
<td>0.265</td>
<td>1</td>
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<tr>
<td>CFRCs with minimum values</td>
<td>29.38</td>
<td>31.3</td>
<td>3.42</td>
<td>3.41</td>
<td>2242</td>
<td>0.23</td>
<td>3.21</td>
</tr>
<tr>
<td>CFRCs with maximum values</td>
<td>37.8</td>
<td>43.2</td>
<td>4.27</td>
<td>4.51</td>
<td>2298</td>
<td>0.32</td>
<td>10.1</td>
</tr>
<tr>
<td>Recommended CFRC</td>
<td>29.38</td>
<td>36.1</td>
<td>3.74</td>
<td>4.43</td>
<td>2242</td>
<td>0.32</td>
<td>10.1</td>
</tr>
</tbody>
</table>

(5 % fibre content, 5 cm long fibres)

### Table 3.6: Consequence of CFRC for dynamic properties

<table>
<thead>
<tr>
<th>Concrete type</th>
<th>Uncracked CFRC beams</th>
<th>Cracked CFRC beams</th>
<th>(E_{\text{dynamic}}) (GPa)</th>
<th>(E_{\text{static}}) (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(\xi) (%)</td>
<td>(f) (Hz)</td>
<td>(\xi) (%)</td>
<td>(f) (Hz)</td>
</tr>
<tr>
<td>PC</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>CFRCs with minimum values</td>
<td>2.5</td>
<td>109.3</td>
<td>5.8</td>
<td>84.3</td>
</tr>
<tr>
<td>CFRCs with maximum values</td>
<td>6.9</td>
<td>145.2</td>
<td>14.1</td>
<td>120.4</td>
</tr>
<tr>
<td>Recommended CFRC</td>
<td>6.9</td>
<td>109.3</td>
<td>14.1</td>
<td>84</td>
</tr>
</tbody>
</table>

(5 % fibre content and 5 cm long fibres)
3.5. CFRC with best overall properties

All static and dynamic properties of CFRC are summarized in Tables 3.5 and 3.6, respectively. The maximum values of different properties are generated with different combinations of fibre length and contents, as can be observed from the tables. Since the CFRC is prepared with a particular fibre length and content, that combination for CFRC should be selected, for which most properties are better than the plain concrete and the CFRC with other fibre lengths and contents. From the obtained results, CFRC with a fibre length of 5 cm and a fibre content of 5% has the best overall properties. By using fibres in concrete, the compressive strength, compressive toughness, modulus of rupture and total toughness index can be increased from 34.7 MPa, 0.265 MPa, 4.34 MPa and 1 to 36.1 MPa, 0.32 MPa, 4.43 MPa and 10.1, respectively. It also has the highest damping. The damping of damaged beams is approximately more than twice compared to that of uncracked one.

3.6. Summary

Experiments were performed to investigate the static and dynamic properties of CFRC. The mechanical properties investigated were the static modulus of elasticity $E_{static}$, compressive strength $\sigma$, compressive toughness $T_c$, splitting tensile strength $STS$, modulus of rupture $MOR$, total toughness index $TTI$ and density. These properties were also compared with those of plain concrete. The dynamic properties investigated were damping ratio $\xi$, fundamental frequency $f$, and dynamic modulus of elasticity $E_{dynamic}$ of CFRC beams. The considered fibre lengths were 2.5, 5 and 7.5 cm and the fibre contents were 1%, 2% and 3% for all fibre lengths, and 5% for 2.5 and 5 cm long fibres. Three specimens of CFRC were tested for each combination of fibres to obtain reliable average results.

The static investigation revealed:

- The properties can improve or worsen depending on fibre length and content, and CFRC strengths can be greater or smaller than that of plain concrete.
3. Properties of coconut fibre reinforced concrete

- The testing confirmed that the inclusion of coconut fibres in concrete can improve the flexural toughness of concrete considerably for all considered cases.
- The CFRC with 5 cm long fibres and 5% fibre content had an increased $\sigma$, $T_e$, MOR and TTI up to 4%, 21%, 2% and 910%, respectively, and decreased $E_{\text{static}}$, STS and density up to 6%, 2% and 3%, respectively, when compared to that of plain concrete.

The dynamic tests revealed:
- Damping of CFRC beams had, as expected, a growing trend and the fundamental frequency had a declining trend with the increasing damage.
- Increasing fibre content resulted in a higher damping ratio and a lower fundamental frequency. The change was more pronounced after cracking.
- CFRC beams with 5 cm long fibres had higher damping when compared to those with other fibre lengths.
- The static and dynamic modulii of elasticity generally decreased with an increase in fibre content or fibre length.
- Of the considered cases, CFRC with 5 cm long fibres and 5% fibre content had the best overall static and dynamic properties.
- Only 7% difference was observed in static and dynamic modulus of elasticity. This small difference indicated that non-destructive modal testing may be used to determine the modulus of elasticity of built members.

It is important to note that the free vibration testing of the uncracked CFRC beams gives an indication of changes in material damping due to the inclusion of fibres of different lengths and contents. However, these measurements are for low amplitude excitations, and behaviours would likely be different during a seismic event. Therefore, further research on configurations and behaviours of CFRC structures under earthquake loading is necessary.
Chapter 4.

Bond strength between coconut fibre and plain concrete

Related paper:

4.1. Introduction

Coconut fibre reinforced concrete was investigated in detail for its possible use in civil engineering applications. The work presented in this chapter investigates the bond strength between coconut fibre and concrete, primarily because the bond between the reinforcing material and matrix contributes to the better behaviour of the ensuing composite. The outcome of this work can help in selecting the optimum fibre properties and pre-treatment for increasing the bond with concrete, ultimately resulting in increased strength of the composite. Therefore, the effects of fibre embedment lengths, diameters, pre-treatment conditions and concrete mix design ratios on the bond strength between a single coconut fibre and concrete is considered. In this experiment, fibres were prepared and categorized manually and fibre diameters were measured by a stereomicroscope. Fibre and concrete properties were also determined experimentally. Single fibre pullout tests were performed to determine load-slippage curves with the help of an Instron tensile machine having load cell. Bond strengths and the energy for fibre pullout were calculated from the experimental data. The single fibre-concrete interfacial bond strength was calculated by dividing the maximum pullout load by the surface area of the embedded portion of a fibre. Pullout energy was calculated as the area under the load-slippage curve.
4. Bond strength between coconut fibre and plain concrete

With the obtained knowledge, empirical equations were also developed to determine the bond strength and energy required for pullout.

4.2. Background

Plain concrete is a brittle material with low tensile strength. There has been a steady increase in the use of short and randomly distributed natural fibres to reinforce the matrix (paste, mortar and concrete). Fibres alter the behaviour of concrete when a crack occurs by bridging the cracks (Figure 4.1), and thus can provide some post-cracking toughness. Fibres crossing the crack guarantee a certain level of stress transfer between both faces of crack, providing a residual strength to the composite. The magnitude of residual strength depends on the fibre, matrix and fibre-matrix interface (Vitor et al. 2010). Therefore, the strength properties of the fibre, matrix and fibre-matrix interface should be well understood in order to predict the behaviour of the resulting composite, particularly after cracking.

Figure 4.1: Coconut fibres bridging a crack

As stated earlier, the bond strength plays an important role in the overall strength of the composites. Its experimental investigations are very diverse. Single fibre pullout tests, which measure the load required to pull out a fibre embedded in a matrix under uni-axial tension, were commonly used to investigate the fibre-matrix bond behaviour. These tests were usually performed at room temperature with the help of an Instron tensile machine having a load cell. The rate of pullout was controlled to be constant. The pullout rate was chosen in such a way that fibres would not break during the test. Many researchers used typical pullout rates between 1-5mm/min to pull out synthetic and natural fibres (e.g.
4. Bond strength between coconut fibre and plain concrete

cocoanut, sisal, and banana fibres) from the matrix (cement paste, polyethylene or polyester) (Mani and Satyanarayana, 1990; Aggarwal, 1992; Jianxin, 1994; Yan et al., 2005; Brahmakumar et al., 2005; Asasutjarit et al., 2007, Nirmal et al., 2011). The higher the pullout rate, the higher the probability that a fibre may break during pullout. It was found that for a short fibre embedded length, the bond strength was less than the fibre tensile strength and fibres could be pulled out from specimens; for a long fibre embedded length, as expected, the bond strength could exceed the fibre tensile strength and fibres were broken during pullout. A brief study on the bond strength between coconut fibres and the cement paste showed a critical fibre embedded length of about 30 mm (Aggarwal, 1992). The diameter of a fibre was usually measured by a microscope. Each fibre was measured in two perpendicular directions to improve the accuracy of the measurements. A minimum of eight measurements were taken along the length of a fibre, and results were averaged (Ramaswamy et al., 1983; Yuan et al., 2002).

In most practical situations, raw natural fibres are not able to provide adequate interfacial bond strength. A considerable amount of work is required to improve fibre surfaces for increasing bond strength with the surrounding matrix and for increasing the tensile strength of fibres. These pre-treatments can be achieved by physical and chemical modifications of fibre surfaces. An easy-to-use and cheap chemical adopted by some researchers was sodium hydroxide. Gu (2009) treated brown coconut fibres by NaOH solution with concentrations from 2% to 10% and found that fibre tensile strength decreased with treatment of increasing NaOH concentration. Coconut fibres were soaked in sodium hydroxide solution (pH=11) for 28 days and then dried. It was found that the tensile strength was reduced by 5% (Ramaswamy et al., 1983). When coconut fibres were pre-treated with 5% NaOH (Mani and Satyanarayana, 1990), there was a 10% decrease in the ultimate tensile strength of fibres. In another study (Ramakrishna and Sundararajan, 2005), fibres were exposed to alternate wetting in NaOH solution for 24 hours and drying for 24 hours. This was repeated over a total period of 60 days. Coconut fibres retained 40% - 60% of their tensile strength. Mani and Satyanarayana (1990) found that the combined use of sodium alginate and calcium chloride effectively improved the ultimate tensile strength of fibres by 18%. Compared to the sodium hydroxide application, the main advantage is that fibres do not lose their tensile strength. In addition, sodium alginate is a weak acidic food gel and calcium chloride is almost neutral, so both of them are safe to use and are environmentally friendly. The main constituents of the surface of a
coconut fibre are lignin and cellulose. Boiled and washed fibres are stiffer and tougher compared to raw fibres because they have high lignin and cellulose contents due to washout of extractives (Asasutjarit et al., 2007). The chemical process could also remove a part of the extraneous fibre surface components which may resist the formation of a bond between the fibres and cement paste. Thus, washed and boiled fibres could effectively increase the fibre-matrix bond strength and tensile strength. Internal bond tests were performed by applying a tension load on coconut fibre reinforced cement board. The load at fracture was divided by thickness and width of the board to obtain the internal bond. It was shown that, compared to fibres without pre-treatment, the internal bond between washed and boiled fibres and the cement paste can be doubled. The cement content of the matrix influences the fibre-matrix interfacial bond strength significantly. Asasutjarit et al. (2007) showed that the internal bond of boiled and washed coconut fibre reinforced cement boards was directly proportional to the cement content. This is because, as the cement content increases, more cement crystals will form to interlock fibres with the surrounding matrix.

Investigations by other researchers so far focused on either the effect of pre-treatment on the internal bond of coconut fibre reinforced boards consisting of cement paste and a number of fibres (Asasutjarit et al., 2007) or only the influence of embedment length on the bond between a single coconut fibre and polyester or cement paste (Mani and Satyanarayana, 1990; Aggarwal, 1992). To the best of the author knowledge, research on pullout of coconut fibres from concrete has not been conducted. It is also important to understand how interfacial bonding behaves under different parameters pertaining to fibre and matrix characteristics. This information can help in the selection of optimized fibre and matrix properties for the resulting composite. Therefore, the pullout behaviour of the coconut fibre from concrete is investigated in this chapter.

4.3. Experimental procedures

The objective is to determine the influence of the most significant factors on the bond strength ($\tau_f$) between coconut fibres and concrete and the energy ($E_f$) required to pull out fibres from the concrete. The variables are:

- the mix design ratio of concrete ($M_f$),
4. Bond strength between coconut fibre and plain concrete

- embedment length of fibre in concrete \( (L_f) \),
- fibre diameter \( (D_f) \) and
- pre-treatment of the fibre \( (P_f) \).

Table 4.1 summarizes the scope of this work. The bond strength of the fibre with the concrete was determined by fibre pullout tests. Much care had to be taken for this testing because some fibres could break during demoulding. Sixteen fibres for a particular parameter were cast in a specimen (see details in section 4.3.2). The average of successfully pulled out fibres was taken to represent the bond strength between the coconut fibre and concrete, and the energy required to pull out the fibre from the concrete.

<table>
<thead>
<tr>
<th>Considered factors</th>
<th>Cases</th>
<th>Numbers of specimens**</th>
<th>Reference specimen</th>
</tr>
</thead>
</table>
| Mix design* ‘\( M_f \)’ | i) 1:2:2  
 ii) 1:3:3  
 iii) 1:4:4 | 2 | |
| Embedment length ‘\( L_f \)’ | i) 10 mm  
 ii) 20 mm  
 iii) 30 mm  
 iv) 40 mm | 2 | \( M_f = 1:3:3 \)  
 \( L_f = 20 \text{ mm} \)  
 \( D_f = \text{medium} \)  
 \( P_f = \text{Soaked} \) |
| Fibre diameter ‘\( D_f \)’ | i) Thin, \( \phi 0.15 \sim 0.20 \text{ mm} \)  
 ii) Medium, \( \phi 0.20 \sim 0.30 \text{ mm} \)  
 iii) Thick, \( \phi 0.30 \sim 0.35 \text{ mm} \) | 2 | |
| Fibre pre-treatment ‘\( P_f \)’ | i) Soaked fibres  
 ii) CaAl fibres  
 iii) Boiled fibres | 2 | |

Note: (i) Highlighted one is the reference specimen. For other variable magnitude, only that value will be altered and other parameters remain the same.
* Cement : sand: aggregates with water cement ratio of 0.48.
** See Figures 4.4 and 4.5.

4.3.1. Fibre preparation

The fibres were prepared according to the procedure described in section 3.2.2. It is important to mention that relatively longer fibres were required for the fibre pullout tests.
4. Bond strength between coconut fibre and plain concrete

It was quite time consuming to get long fibres from hydraulically compressed fibres. These prepared fibres are, hereinafter, named as soaked fibres.

4.3.1.1. Fibre sampling

Because of the restriction of the fibre pullout test apparatus, lengths of selected fibres needed to be at least 250 mm. Initial selections of fibres were based on the naked eye; special care was taken to make sure that only those fibres which have minimum variation along the length were selected. The following observations are important:

- There were few fibres which were either very thin (diameter less than 0.15 mm) or very thick (diameter more than 0.35 mm). These extremely thin and thick fibres could not be used to represent the population of fibre diameters, thus they were not selected.
- A single fibre had a variable diameter along its length (Figure 4.2(a)). The fibre was generally thicker near one end and thinner near its other end. Care was taken to ensure that the diameter of the part of the fibre which would be embedded into the concrete was approximately the same, and generally the diameter variance along the length of the fibre was minimal.
- About 30% of the fibres split into several thinner fibres near their tips (Figure 4.2(b)). These tips were cut.

![Figure 4.2](image_url)

**Figure 4.2:** Long coconut fibres; (a) Fibre with variable diameter along its length and (b) split of fibre into smaller fibres at one end.

Each coconut fibre had a different property because of a variation in diameter among the fibres and also along the fibre length. The selected sampling represents all coconut fibres...
to be used. Three samples of different diameters of fibres were selected, namely thin, medium and thick (Table 4.1). Diameters of selected fibres were measured with the help of an electronic microscope (Figure 4.3). A minimum of eight measurements were taken along the length of a fibre, and results were averaged. As expected, fibre diameter varied along the length. However, the selected fibre should not vary much from its averaged value. The averaged properties of a single fibre were assumed constant along its length.

![Fibre diameter measurement](image)

**Figure 4.3:** Fibre diameter measurement; (a) The microscope, (b) a fibre under measurement, and (c) fibre under microscope.

4.3.1.2. Fibre pre-treatment

Only soaked and medium fibres were chosen for further pre-treatments, keeping in mind Asasutjarit et al. (2007) and Mani and Satyanarayana (1990) studies as a guide, as follows:

**i. Treatment with boiling water and washing:** The soaked fibres were put in boiling water for 2 hours. They were then washed with tap water until the color of the water was clear. The fibres were then dried in the same manner as the soaked fibres. These treated fibres are called boiled fibres.
4. Bond strength between coconut fibre and plain concrete

**ii. Treatment with chemicals:** The soaked fibres were dipped for 30 minutes into 0.25% Sodium Alginate (NaC₆H₇O₆) solution prepared using distilled water. Fibres were removed from the solution and then soaked in 1% Calcium Chloride (CaCl₂) solution for 90 minutes. Fibres were finally dried at 70°C for 60 minutes. These chemically treated fibres are called CaAl fibres.

### 4.3.2. Specimens

Ordinary Portland cement, fine aggregates (sand), coarse aggregates and water were put into a mixer pan, and it was rotated for 5 minutes. The maximum size of aggregates was 12 mm. The water-cement ratio was taken as 0.48. Specimens with 100 mm width, 100 mm depth and 500 mm length with two layers of fibres were cast (Figure 4.4). Thin plastic sheets were wrapped on the sides of the moulds in order to avoid the flow of concrete towards the outside. First, concrete was filled up to 30 mm depth and fibres were inserted on both sides of the mould for a predefined embedment length. Another layer of concrete was filled up to 70 mm depth and fibres were inserted in a similar manner. The advantage of this casting procedure was that fibres would remain levelled during the concrete pouring stages. The levelling of fibres was necessary to ensure that the pullout load would be parallel to the fibre embedded length during tests. Specimens were demoulded after 48 hours. Figure 4.5 shows a demoulded specimen.

![Figure 4.5](image-url)  
**Figure 4.4:** Single fibre casting for pullout test (cross-section of Figure 4.5)
4. Bond strength between coconut fibre and plain concrete

For each concrete batch, six cylinders (100 mm diameter and 200 mm height) and three beamlets (100 mm x 100 mm x 500 mm) were also cast in order to determine the concrete properties. All specimens were cured for 28 days.

4.3.3. Testing

All cylinders and beamlets were tested in the same manner as described in section 3.2.5.1 for determining the material properties of plain concrete. ASTM standard D3822-07 was taken as the guide for the testing of coconut fibres. Fibres were tested to determine their tensile strength, strain, elastic modulus and toughness. Since no standard method is available for a single fibre pullout test, previous studies as described in section 4.2 are taken as a guide. Single fibre pullout tests were performed in a computer-controlled Instron machine having a load cell with a pulley diameter of 50 mm and a crosshead speed of 2.5 mm/min. The test setup is shown in Figure 4.6.

Figure 4.5: Specimen with fibres

Figure 4.6: Single fibre pullout test setup
4. Bond strength between coconut fibre and plain concrete

4.4. Results and analysis

4.4.1. Properties of plain concrete with considered mix designs

For interpretation of the results obtained from pullout tests, concrete properties were determined using standard procedures as described in section 3.3. The average of six readings was taken for plain concrete with 1:3:3 because this mix was used in many specimens. The average of three readings was taken for concrete with other mix designs.

The effect of mix design ratio on concrete properties is presented in Table 4.2. As expected, all properties have a growing trend with an increase in cement content. The compressive strength, toughness, splitting tensile strength and modulus of rupture increased by 218, 85, 153, and 146%, respectively, when the mix design ratio was changed from 1:4:4 to 1:2:2. It is well known that the greater the compressive strength, the higher the bond strength will be, but how much is the increase? This is what being also investigated in this chapter.

**Table 4.2: Plain concrete properties**

<table>
<thead>
<tr>
<th>Matrix Mix design</th>
<th>Cylinder testing</th>
<th>Beam-let testing</th>
<th>Density (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>σ (MPa)</td>
<td>ε (%)</td>
<td>E_{static} (GPa)</td>
</tr>
<tr>
<td>1:2:2</td>
<td>34.71</td>
<td>0.188</td>
<td>33.08</td>
</tr>
<tr>
<td>1:3:3</td>
<td>18.8</td>
<td>0.181</td>
<td>18.23</td>
</tr>
<tr>
<td>1:4:4</td>
<td>10.9</td>
<td>0.169</td>
<td>10.14</td>
</tr>
</tbody>
</table>

Note: The average of six readings is taken for plain concrete with 1:3:3 because it is used in many specimens. The average of three readings is taken for other matrices.

4.4.2. Properties of coconut fibres

Table 4.3 shows the properties of tested coconut fibres; only fibre diameter and pre-treatment were taken into account. The average of 10 readings is taken for medium and soaked fibres because it is used in many specimens. The average of five readings is taken for other fibres. The elastic modulus of fibres is calculated as the ratio of stress change to strain change in the elastic range of the fibres. The stress at break (tensile strength) and its corresponding strain were also noted. The total area under the stress-strain curve is taken as the fibre toughness. It is important to mention that the mean peak tensile load for
4. Bond strength between coconut fibre and plain concrete

Pre-defined thin (ϕ 0.15 ~ 0.20 mm), medium (ϕ 0.20 ~ 0.30 mm) and thick fibres (ϕ 0.30 ~ 0.35 mm) are 1.14, 3.25 and 6.59 N, respectively, and their standard deviations are 0.2, 0.9 and 1.2 N, respectively. The averaged maximum tensile loads of soaked, boiled and CaAl fibres are 3.25, 4.01, and 2.72 N, respectively, and their standard deviations are 0.9, 1.1, and 0.8 N, respectively. The large magnitude of standard deviation shows that there is a huge variation in the properties of fibres within a particular category. The reason for such discrepancy could be the variation of cross-section dimension along the fibre length and also among the fibres. Similar divergent results were also observed by Abiola (2008). The researcher investigated the mechanical properties of inner and outer fibres taken from a coconut shell. It was found that the inner coconut fibre had a 44% higher stress at break compared to that of the outer fibre, but the outer coconut fibre had a 107% higher elongation capability which enabled it to absorb or withstand higher stretching energy. In fact, there was also a huge variation in the properties of inner and outer fibres. For example, the averaged stress at failure for inner fibre was 67 MPa with a standard deviation of 34 MPa, and that for outer fibre was 47 MPa with a standard deviation of 18 MPa.

Table 4.3: Properties of coconut fibres

<table>
<thead>
<tr>
<th>Diameter</th>
<th>Pre-treatment</th>
<th>Peak load (N)</th>
<th>Stress at break (MPa)</th>
<th>Strain at break (%)</th>
<th>Modulus of elasticity (GPa)</th>
<th>Total toughness (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thin</td>
<td>Soaked</td>
<td>1.14±0.2</td>
<td>47.3±11.5</td>
<td>19.5±3.1</td>
<td>0.58±0.12</td>
<td>6.2±1.6</td>
</tr>
<tr>
<td>Medium</td>
<td>Soaked</td>
<td>3.25±0.9</td>
<td>66.3±14.5</td>
<td>27.8±3.6</td>
<td>0.74±0.17</td>
<td>12.8±2.4</td>
</tr>
<tr>
<td>Thick</td>
<td>Soaked</td>
<td>6.59±1.2</td>
<td>81.9±17.1</td>
<td>34.8±4.5</td>
<td>1.04±0.20</td>
<td>23.7±2.9</td>
</tr>
<tr>
<td>Medium</td>
<td>Boiled</td>
<td>4.01±1.1</td>
<td>88.6±17.6</td>
<td>31.1±3.7</td>
<td>0.99±0.18</td>
<td>19.8±2.5</td>
</tr>
<tr>
<td>Medium</td>
<td>CaAl</td>
<td>2.72±0.8</td>
<td>51.2±12.2</td>
<td>21.8±3.3</td>
<td>0.65±0.16</td>
<td>8.0±1.7</td>
</tr>
</tbody>
</table>

Note: The average of 10 readings is taken for medium and soaked fibres because it is used in many specimens & their properties are shown in italic. The average of five readings is taken for other fibres.

It can be observed from the Table 4.3 that the tensile strength of fibres increases from 47 to 82 MPa with an increase in diameter. It may be noted that the tensile load of thick fibre is 478% higher than that of thin fibres, but the stress of the former is only 73% more compared to that of the latter. The reason is that the maximum applicable tensile load is not proportional to the fibre diameter. The strain at break and the total toughness increase with larger fibre diameters. There is a considerable decrease of about 23% in tensile
4. Bond strength between coconut fibre and plain concrete

strength, 22% in strain and 38% in toughness due to chemical treatment when compared to the soaked process. However, Mani and Satyanarayana (1990) found a different result with the same chemical treatment i.e. an increase in tensile strength was observed. This might be because the fibres were of different origins. However, in general, the tensile force, strength, strain and toughness of boiled and washed fibres were greater (23, 34, 12, and 57%, respectively) compared to those of soaked fibres.

![Graphs showing comparison of experimental and empirical results](image)

**Figure 4.7:** Comparison of experimental and empirical results; (a) fibre tensile stress, (b) and (c) fibre elastic modulus, and (d) fibre toughness.

Using Kulkarni et al. (1981) and Xu and Shi (2009) studies as a guide for simple statistical approaches, the following equations have been developed using the experimental results for the estimation of tensile strength ($\sigma_{\text{fibre}}$) in MPa, elastic modulus ($E_{\text{fibre}}$) in GPa and total toughness ($T_{\text{fibre}}$) in MPa of the soaked fibres:

\[
\sigma_{\text{fibre}} = 225 \left( D_f \right)^{0.9} \\
E_{\text{fibre}} = 2.82 \left( D_f \right)^{0.93} \\
E_{\text{fibre}} = 0.011 \left( \sigma_{\text{fibre}} \right)^{1.02}
\]  

(4.1)  
(4.2)  
(4.3)
4. Bond strength between coconut fibre and plain concrete

\[ T_{\text{fibre}} = 263 \left(D_f\right)^{2.16} \]  \hspace{1cm} (4.4)

where \( D_f \) = fibre diameter in mm. These equations are valid for soaked fibres only. This is because pre-treatment of fibres has modified properties differently: the boiling and washing treatment have increased the tensile strength but decreased the elastic modulus. The comparison of experimental and empirical results is presented in Figure 4.7. The empirical results are in good agreement with the experimental results.

4.4.3. Fibre pullout behaviour

4.4.3.1. Typical load-slippage curves

Figure 4.8 shows the typical load-slippage curves of some selected fibres. In the displayed case, two fibres were broken during pullout (which shows that the pullout load was more than the tensile load) and two are successfully pulled out (which shows that the tensile load was more than the pullout load). The maximum load required to pull out fibre from the specimen is called the pullout load. The maximum axial load required to break fibre in tension is called the tensile load. The reason for fibre breaking during the pullout test might be the variation in the cross-section of the fibre. The single fibre-concrete interfacial bond strength is calculated by dividing the maximum pullout load by the surface area of the embedded portion of a fibre. Pullout energy is calculated as the area under the load-slippage curve. An average is taken for successfully pulled out fibres.

![Figure 4.8: Typical load-slippage curves for selected fibre](image-url)
4. Bond strength between coconut fibre and plain concrete

4.4.3.2. Effects of considered factors on bond strength

The effects of considered parameters (i.e. fibre embedment, diameter, pre-treatment and concrete mix design) on bond strength between fibre and concrete are presented in Figure 4.9. It may be noted that the scattered data is obtained for a particular permutation, and the number of successful pulled out fibres is different for different combinations. For example, the number of successfully pulled out fibres decreases with an increase in embedment length \( L_f \). The reason is that the pullout load becomes higher than the tensile load for most cases as the \( L_f \) increases, and also because the miniature variations in the cross-section of fibres along the fibre length means more fibres are broken for higher \( L_f \) and fewer fibres are broken for smaller \( L_f \). Also, the data of bond strength is strewn for a particular embedment length. The reasons are that: (1) the average cross-section of fibres varies slightly from fibre to fibre within a predefined category, e.g. medium fibres have diameters of 0.2 – 0.3 mm; and (2) the slight variations in cross-section along the considered fibre length. The pullout load is not linearly proportional to the fibre diameter, resulting in speckled data of bond strength.

Considering the effect of embedded length on bond strength, the bond strength becomes higher as the \( L_f \) increases from 10 to 30 mm, and then there is a 10% drop as the length increases from 30 to 40 mm (upper left graph in Figure 4.9). This indicates that the critical fibre embedded length is 30 mm and the concrete mix design ratio is 1:3:3. The work of Aggarwal (1992) also ended with a critical fibre embedded length of 30 mm in cement paste as the matrix. The current result reveals the same critical length for fibre in concrete.

On the other hand, as the fibre diameter increases from the medium to thick category, the bond strength is more than double (upper right graph in Figure 4.9). This trend is expected because an increase of the interface between the diameter fibre and surrounding concrete provides more bonding, thus a larger debonding force is needed to overcome the bond strength. All thin fibres are broken because of their low tensile load and strain, although the pullout rate is reduced piece-wise from 2.5 to 0.5 mm/min.

The pre-treatment of fibres has also altered the bond strength considerably. Boiled fibres have enhanced the bond strength by 184%, and CaAl fibres have decreased the bond
4. Bond strength between coconut fibre and plain concrete

strength by 25% compared to soaked fibres, as can be seen in the bottom left graph in Figure 4.9. Asasutjarit et al. (2007) also found that the boiled and washed fibres increased the fibre-matrix bond strength because of their increased lignin and cellulose content due to the boiling pre-treatment. Their investigation was done on coconut fibre reinforced boards with cement paste as the matrix. However, the present study is based on concrete as the matrix. Both studies have the same outcome, as cement is used as a binding agent.

**Figure 4.9:** Influence of considered parameters on the bond strength between fibre and concrete.
The bond strength of fibre with concrete is proportional to the mix design ratio i.e. the content of cement (bottom right graph in Figure 4.9). It is shown in Table 4.2 that the strength of concrete is proportional to the mix design ratio. This indicates that, as the concrete strength increases, the bond strength also increases. The study of Asasutjarit et al. (2007) also concluded that the internal bond increased with an increase in cement content. It is important to mention that the primary purpose of cement, in their work, is to hold fibres only and to give strength to the resulting composite (coconut fibre boards (CFB)). However, in the present study, the function of cement in the fibre-concrete is to hold fibres, sand and aggregates and to give strength to the resulting composite (coconut fibre reinforced concrete (CFRC)). Therefore, a proportionate increase of bond in CFB and CFRC would be different. The fibres embedded in the concrete with high cement content (1:2:2) are all broken because the bond strength is too high, compared to the fibre tensile strength. This means that the selected concrete strength should not be too high in the design of CFRC if concrete members with higher toughness are intended. The addition of fibres to concrete can increase toughness drastically, which can prevent brittle failures of protective concrete structures (e.g. in blast loading).

**4.4.3.3. Empirical equation for estimating bond strength**

Taking Yan and Li (2013) study as a guide and using statistical tools, the following simplified equation has been developed, using experimental results, for determining the bond strength “τf” between the coconut fibre and concrete in MPa:

\[
\tau_f = \alpha \beta \left[ a L_f^3 + b L_f^2 + c (L_f - 1) \right]
\]  

(4.5)

where \(L_f\) is the embedment length (in cm) of coconut fibre in concrete; \(a, b\) and \(c\) are its constants having values of -0.0166, 0.112 and -0.188, respectively; \(\alpha\) and \(\beta\) are reinforcement and matrix factors, respectively. The reinforcement factor \(\alpha\) is 1 for ‘medium and soaked fibres’, 2.85 for ‘thick and soaked fibres’ or ‘medium and boiled fibres’, or 7 for ‘thick and boiled fibres’. The matrix factor \(\beta\) is 0.43 for 1:4:4 and 1 for 1:3:3 mix design ratios. The comparison of empirical and experimental results of bond strength for different fibre embedded lengths is shown in Figure 4.10. The numerical values of bond strength between a single coconut fibre and the concrete are in good
agreement with that of experimental values (Table 4.4). The error is less than 3%.

![Comparison of empirical and experimental results of bond strength.](image)

**Figure 4.10**: Comparison of empirical and experimental results of bond strength.

<table>
<thead>
<tr>
<th>Matrix and reinforcement parameters</th>
<th>Bond strength</th>
<th></th>
<th></th>
<th>Difference (A and B)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fibre embedment length “mm”</td>
<td>Experimental</td>
<td>Equation (4.5)</td>
<td>“MPa” “MPa” “%”</td>
</tr>
<tr>
<td></td>
<td></td>
<td>A</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td>Medium and soaked fibres in mix design 1:3:3</td>
<td>10</td>
<td>0.096</td>
<td>0.0954</td>
<td>0.63</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>0.128</td>
<td>0.1272</td>
<td>0.62</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>0.184</td>
<td>0.1838</td>
<td>0.11</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>0.166</td>
<td>0.1656</td>
<td>0.24</td>
</tr>
<tr>
<td>Medium and soaked fibres in mix design 1:4:4</td>
<td>20</td>
<td>0.055</td>
<td>0.0547</td>
<td>0.55</td>
</tr>
<tr>
<td>Thick and soaked fibres in mix design 1:3:3</td>
<td>20</td>
<td>0.372</td>
<td>0.3625</td>
<td>2.55</td>
</tr>
<tr>
<td>Medium and boiled fibres in mix design 1:3:3</td>
<td>20</td>
<td>0.363</td>
<td>0.3625</td>
<td>0.13</td>
</tr>
</tbody>
</table>
4.4.3.4. Effects of considered parameters on energy required for fibre pullout

The effects of fibre embedment, diameter, pre-treatment and concrete mix design on the energy required to pull out fibres from concrete are presented in Figure 4.11. Again, the scattered data is observed as in the case of the bond strength, and the same reasons apply here. Based on average values, it is concluded that as the fibre embedded length increases from 10 to 40 mm, the pullout energy increases from 8.7 to 120.1 N.mm. The pullout energy is increased by 171% when the fibre diameter increases from the medium to thick category. Compared to the reference case of soaked fibres, boiled fibres have enhanced the pullout energy by 165% and CaAl fibres have decreased it by 34%.

Figure 4.11: Effect of considered factors on energy E for fibre pullout.
4.4.3.5. **Empirical equation for estimating energy required for fibre pullout**

Similar to equation 4.5, the following simplified equation has been developed, using experimental results, to numerically determine the energy “$E_f$” in N.mm required to pull out fibres from concrete:

\[
E_f = \gamma \lambda \left[ p L_f^3 + q L_f^2 + r L_f + s \right]
\] (4.6)

where $L_f$ is the embedment length (in cm) of coconut fibre in concrete; $p$, $q$, $r$ and $s$ are its constants having values of -10.4, 81, -150 and 88, respectively; $\gamma$ and $\lambda$ are reinforcement and matrix factors, respectively. The reinforcement factor $\gamma$ is 1 for ‘medium and soaked fibres’, 2.65 for ‘thick and soaked fibres’ or ‘medium and boiled fibres’, or 7 for ‘thick and boiled fibres’. The matrix factor $\lambda$ is 0.35 for 1:4:4 and 1 for 1:3:3 mix design ratios.

The comparison of empirical and experimental results of energy required to pull out fibre from concrete for different fibre embedded lengths is shown in Figure 4.12. The numerical values of energy required for fibre pullout are in good agreement with that of the experimental values (Table 4.5). The error is less than 3%.

![Figure 4.12: Comparison of empirical and experimental results of energy required for fibre pullout.](image)
4. Bond strength between coconut fibre and plain concrete

Table 4.5: Experimentally and numerically obtained energy for fibre pullout

<table>
<thead>
<tr>
<th>Matrix and reinforcement parameters</th>
<th>Fibre embedment length</th>
<th>Energy</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>“mm”</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Experimental</td>
<td>Equation (4.6)</td>
</tr>
<tr>
<td></td>
<td>A “N.mm”</td>
<td>B “N.mm”</td>
</tr>
<tr>
<td>Medium and soaked fibres in mix design 1:3:3</td>
<td>10</td>
<td>8.7</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>29.1</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>87.3</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>120.1</td>
</tr>
<tr>
<td>Medium and soaked fibres in mix design 1:4:4</td>
<td>20</td>
<td>10.1</td>
</tr>
<tr>
<td>Thick and soaked fibres in mix design 1:3:3</td>
<td>20</td>
<td>78.9</td>
</tr>
<tr>
<td>Medium and boiled fibres in mix design 1:3:3</td>
<td>20</td>
<td>77.2</td>
</tr>
</tbody>
</table>

4.5. Summary

The effects of fibre embedment length, diameter, pre-treatment condition and mix design ratio on bond strength between coconut fibres and concrete were investigated experimentally. The energy required for fibre pullout was also taken into account. Based on the conducted investigation, empirical equations are developed to determine the bond strength between coconut fibre and concrete and the energy required for fibre pullout. The properties of concrete (compressive strength, modulus of elasticity, compressive toughness, splitting tensile strength and modulus of rupture) and fibres (tensile load, stress and strain at break, modulus of elasticity and toughness) were determined using standard procedures. The simplified equations are developed for estimating fibre tensile strength, elastic modulus and toughness. All concrete properties improved as expected by increasing the cement content. Single fibre pullout tests were performed to determine the
bond strength and the energy required for pullout. The experiment on fibre tensile and fibre-concrete bond strength revealed that:

- thick (0.30 ~ 0.35 mm diameter) and boiled fibres had higher tensile strengths compared to thin (0.15 ~ 0.20 mm diameter), medium (0.20 ~ 0.30 mm diameter), soaked and chemically treated fibres.
- the bond strength increased with embedment length and had the highest value with a 30 mm embedment.
- the pullout energy increased with an increase in embedment length.
- all thin fibres in concrete with a 1:3:3 mix design ratio and medium fibres in concrete with a 1:2:2 mix design ratio were broken because the pullout load was higher than the fibre tensile load.
- the thick fibres had bond strength and tensile strength of 0.37 and 82 MPa, respectively.
- the fibre tensile strength, fibre toughness and fibre-concrete bond strength increased by 34%, 55% and 184%, respectively, when fibres were boiled and washed.
- chemical pre-treatment caused a decrease of bond strength and tensile strength by 25% and 23%, respectively.
4. Bond strength between coconut fibre and plain concrete
Chapter 5.

Bond strength between rope and CFRC

Related paper:

5.1. Introduction

The aim of this study is to develop a new construction technology using coconut fibre reinforced concrete (CFRC) and rope reinforcement. Ropes, made of coconut fibres, are intended to be used as the ungrouted vertical reinforcement for the mortar-free walls (see proposed sketch in Figure 6.1). The embedment of ropes into the foundation and the top tie-beams should be selected such that there is no pullout. The pullout force (i.e. rope tension) generated due to an earthquake loading may govern the embedment length of the ropes in the foundation. The rope tension during dynamic loading is considered in Chapter 7. The generated rope tension should be less than the pullout force and the rope tensile capacity for avoiding the structure collapse. Therefore, the effects of rope embedment length, rope diameter, pre-treatment condition and concrete mix design ratio with different fibre contents on the bond strength between rope and CFRC are investigated in this chapter. The influence of the knot inside CFRC on bond strength was also studied. Even though the effect of rope knot in Chapter 7 is not considered the outcome might be useful for the future development of this construction technique. Rope pullout tests were carried out to determine load-slippage curves. Bond strength and the energy required to pull out the rope were calculated from the experimental data. The rope-CFRC interfacial bond strength was calculated by dividing the maximum pullout
load by the surface area of the embedded portion of the rope. Pullout energy was calculated as the area under the load-slippage curve. The tensile strength and elongation of coconut fibre rope were determined considering parameters like rope diameter and pre-treatment. Stress-strain relationship of ropes was also investigated experimentally. With the obtained knowledge, empirical equations are also developed to determine the bond strength between rope and CFRC, the energy required for rope pullout and the rope tensile strength.

5.2. Background

Other alternatives to existing seismic-resistant housing are under investigation, because of the increased demand due to earthquake damage. The urgent need for finding other solutions is the high cost of current existing earthquake-resistant structures, because many people living in rural seismic-prone regions cannot afford high-cost housing. Apart from this fact, new housing colonies at low price are also in high demand because of urbanization and increased population (Bardhan, 2011). This need is a huge issue for many developing countries. Therefore, keeping all these requirements in mind, new methods principally have to focus on using local materials. Natural fibres are good alternatives because of their lower cost and at the same time they promote sustainable development (Ramakrishna and Sandararajan, 2005). Development of composites using natural fibres also helps in protecting the environment (Asasujarit et al., 2007). For these reasons, coconut fibre reinforced concrete (CFRC) and coconut-fibre rope reinforcement are selected. It is well known that the bond strength between reinforcing material and concrete plays an important role in the overall behaviour of the composite (Yang et al., 2011). Investigation of the rope/matrix interface (rope pullout test) is significant because the mechanical properties of the composites depend strongly not only on the properties of the rope and the matrix but also on the rope/matrix interface inside the foundation. The role of rope, as longitudinal reinforcement, will be more significant in mortar-free construction as it will increase the structural stability.

Since pullout tests on rope cast in CFRC have never been carried out before, standard bond strength test methods used for rebars made of steel or fibre reinforced polymer cast in plain concrete were taken as a guide to investigate bond strength between rope and
5. Bond strength between rope and CFRC

CFRC. ASTM International (2009) is a standard test method to determine the bond strength of steel reinforcing bars within plain concrete. Uni-directional load is used to measure the load-slip curve. The loading device must be able to measure forces that are within 2% of the applied load. The slippage at the loaded and free ends of rebar must be measured relative to the loaded and free ends of the concrete specimen using linear variable differential transducers (LVDTs). The loading rate should be such that the specimen does not fail before 3 minutes after the specimen has been loaded, and at least ten load and displacement readings should be taken before failure of the specimen. The width, height and length of matrix specimens should also be determined according to ASTM International (2009). The width of each specimen should be greater than the sum of the bar diameter and two times the required concrete cover. The minimum length of each specimen should be five times this value. The height of each specimen should be equal to the embedment length of the rope. Each specimen should be cured in a curing compound or by using wetting cloth and covering with plastic sheets to avoid rapid evaporation of water. A minimum of two cylinders is required from each sample of concrete. The test cylinders should be cured in the same manner as the specimens have been cured. Guadagnini et al. (2004) determined the bond strength between fibre reinforced polymer (FRP) bars and concrete, using two testing methods: a pullout test and a splitting test (eccentric pull out). The pullout test was used to determine the bond strength of bars in confined conditions, whereas the splitting test was used to find the bond strength of FRP bars near the surface of the concrete. Their research also showed that pullout tests generally produced far more consistent results than splitting tests, as the natural inconsistencies in the concrete had a significant effect on splitting behaviour. Therefore, pullout tests using uni-directional loading are relatively easy, quick and more reliable for having the first hand information about the bond strength between the rope and the concrete.

5.3. Experimental procedures

The objective of the experiment is to determine how different parameters affect the bond strength ($\tau_r$) between rope and coconut fibre reinforced concrete (CFRC), and the energy ($E_r$) required to pull out the rope from CFRC. The considered variables are:

- embedment length of rope,
5. Bond strength between rope and CFRC

- rope diameter,
- pre-treatment condition,
- the mix design ratio,
- fibre content, and
- number of knots.

Table 5.1 summarizes the scope of the work. The bond strength of rope with CFRC is determined by rope pullout tests. The average of three readings is taken for a particular parameter.

<table>
<thead>
<tr>
<th>Variables</th>
<th>Cases</th>
<th>Number of specimens</th>
<th>Reference specimen</th>
</tr>
</thead>
<tbody>
<tr>
<td>Embedment length ($L_r$)</td>
<td>i) 100 mm</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>ii) 150 mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>iii) 200 mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rope diameter ($D_r$)</td>
<td>i) ~ 18 mm</td>
<td>2</td>
<td>$L_r = 100$ mm</td>
</tr>
<tr>
<td></td>
<td>ii) ~ 27 mm</td>
<td></td>
<td>$D_r = ~ 36$ mm</td>
</tr>
<tr>
<td></td>
<td>iii) ~ 36 mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pre-treatment ($P_r$)</td>
<td>i) Soaked</td>
<td>2</td>
<td>$P_r = $ Soaked</td>
</tr>
<tr>
<td></td>
<td>ii) Boiled</td>
<td></td>
<td>$M_r = 1:3:3$</td>
</tr>
<tr>
<td></td>
<td>iii) CaAl</td>
<td></td>
<td>$C_f = 0.5%$</td>
</tr>
<tr>
<td>Mix design ratio* ($M_r$)</td>
<td>i) 1:2:2</td>
<td>2</td>
<td>$N = 0$</td>
</tr>
<tr>
<td></td>
<td>ii) 1:3:3</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>iii) 1:4:4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fibre content ($C_f$)</td>
<td>i) Plain concrete (0%)</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>ii) 0.5%</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>iii) 1.0%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Number of knot ($N$)</td>
<td>i) 0</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>ii) 1</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: (i) Highlighted is the reference specimen. For other variables, only that value is altered and other parameters remain the same. (ii) Fibre content is % by mass of concrete materials.
* Cement: fine aggregates (sand) : coarse aggregates. The water-cement ratio for concrete with 0, 0.5 and 1% fibre content is 0.48, 0.54 and 0.64, respectively.
5. Bond strength between rope and CFRC

5.3.1. Rope pre-treatment

Coconut fibre ropes were soaked in tap water for 4 hours to remove coir dust and then ropes were dried in the open air. These prepared ropes are, hereinafter, called soaked ropes. These ropes were chosen for further pre-treatment as follows:

a. Treatment with boiling water and washing: The soaked ropes were put in boiling water for 2 hours. They were then washed with tap water until the color of water became clear. The ropes were then dried in the same manner as soaked ropes. These treated ropes are called boiled ropes.

b. Treatment with chemicals: The soaked ropes were dipped in 0.25% Sodium Alginate (NaC₆H₇O₆) solution prepared by distilled water for 30 minutes. Ropes were removed from the solution and then soaked in 1% Calcium Chloride (CaCl₂) solution for 90 minutes. The ropes were finally dried. These chemically treated ropes are called CaAl ropes.

5.3.2. Production of fibres, CFRC and specimens

The fibres and CFRC / plain concrete were prepared as per procedures described in sections 3.2.1 and 3.2.3, respectively. For preparing CFRC, the fibre content is 0.5% by mass of concrete material. Specimens with 150 mm width, 600 mm length and height equivalent to rope embedment length were cast. The centre to centre distance of rope was 150 mm. Before pouring the CFRC into moulds, the diameter of rope was measured at three locations: (i) near the loaded end, (ii) the middle of the embedded length, and (iii) near the unloaded end. At each location, four diameter readings were taken at an angle of 45°. Thus, the average diameter of the rope in the embedded part was actually the mean of 12 readings. Three ropes were cast in each specimen. For each CFRC or plain concrete batch, six cylinders (100 mm diameter and 200 mm height) and three beamlets (100 mm x 100 mm x 500 mm) were also cast in order to determine the matrix properties. All specimens were cured for 28 days.
5. Bond strength between rope and CFRC

5.3.3. Testing

All cylinders and beamlets were tested in a similar manner as described in section 3.2.6.1 for determining the material properties of CFRC and plain concrete (PC). Ropes were tested in an Avery machine to determine their tensile strength and elongation. Rope pullout tests were performed in the same machine. The test setup is shown in Figure 5.1. The load was applied in the upward direction. Slippage was measured with the help of LVDTs at the loaded and unloaded ends of the specimen. One LVDT was attached with the jaw of Avery to measure the rope elongation. Elongation of the rope was taken as the difference of readings of LVDTs at the jaw and loaded end of the specimen. It may be noted that the slippage at the loaded end is the summation of slippage and rope elongation in the embedded part. It is impossible to separate these two parameters, and the slippage is more pronounced than the elongation. Therefore, hereinafter, it is taken as slippage at the loaded end.

Figure 5.1: Rope pullout test setup
5. Bond strength between rope and CFRC

5.4. Results and analysis

5.4.1. Matrix properties

For interpretation of the results obtained from the pullout tests, matrix properties were determined using standard procedures (also explained in section 3.3). Material properties of the considered matrices are shown in Table 5.2. An average of nine and three readings is taken for CFRC (0.5% fibre content) matrix and all other matrices, respectively. CFRC (0.5% fibre content) matrix is given more weight for the average values because, being a reference matrix, it was cast in many batches for the rope-pullout specimens.

As expected, all properties have a growing trend with an increase in cement content. The $\sigma$, $E$, $T_c$, $STS$, $MOR$, and $TTI$ are considerably increased by 241%, 262%, 87%, 136%, 123%, and 92%, respectively, when the mix design ratio is changed from 1:4:4 to 1:2:2 in fibre-concrete with only 0.5% fibre content.

On the other hand, the $\sigma$, $E$, $T_c$, $STS$, $MOR$, and $TTI$ of CFRC (with mix design of 1:3:3 and 0.5% fibre content) are increased by 4%, 5%, 11%, 15%, 12%, and 294%, respectively, compared to that of plain concrete (1:3:3). However, with same mix design ratio, CFRC (1% fibre content) has generated different results with comparison to CFRC (0.5% fibre content). The $\sigma$, $E$, and $STS$ of CFRC (1%) are decreased by 2%, 3%, and 7%, respectively, whereas $T_c$, $MOR$, and $TTI$ are increased by 9%, 8%, and 10%, respectively, compared to that of CFRC (0.5%). However, all strength properties of CFRC (1%) are more than that of plain concrete.
5. Bond strength between rope and CFRC

**Table 5.2:** Properties of plain concrete and coconut fibre reinforced concrete with considered mix designs

<table>
<thead>
<tr>
<th>Matrix</th>
<th>Cylinder testing</th>
<th>Beam-let testing</th>
<th>Density (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>σ (MPa)</td>
<td>ε (%)</td>
<td>E (GPa)</td>
</tr>
<tr>
<td>Plain concrete (1:3:3, 0%)*</td>
<td>18.8</td>
<td>0.18</td>
<td>18.2</td>
</tr>
<tr>
<td>CFRC (1:2:2, 0.5%)*</td>
<td>39.6</td>
<td>0.23</td>
<td>37.6</td>
</tr>
<tr>
<td>CFRC (1:3:3, 0.5%)*</td>
<td>19.5</td>
<td>0.19</td>
<td>19.2</td>
</tr>
<tr>
<td>CFRC (1:4:4, 0.5%)*</td>
<td>11.6</td>
<td>0.17</td>
<td>10.4</td>
</tr>
<tr>
<td>CFRC (1:3:3, 1%)*</td>
<td>19.1</td>
<td>0.22</td>
<td>18.6</td>
</tr>
</tbody>
</table>

* Mix design ratio and fibre content (% by mass of concrete material) are shown in brackets. The water-cement ratio for concrete with 0, 0.5 and 1% fibre content is 0.48, 0.54 and 0.64, respectively.

Note: The average of 6 readings is taken for CFRC (1:3:3, 0.5%) because it is used in many specimens. The average of three readings is taken for other matrices.
5. Bond strength between rope and CFRC

5.4.2. Rope properties

Figure 5.2 shows the typical stress-strain relation for coconut-fibre rope. It was a 36 mm diameter soaked rope, made of small diameter curled ropes. The maximum load is taken for the calculation of the ultimate strength of the rope and its corresponding strain is noted from the curve. The elongation up to the peak load and the total elongation were also measured. It can be observed from the figure that the stress-strain curve has three slopes (OA, AB, BC) before the maximum load is achieved. It means that the stress-strain relation is non-linear before the peak stress, but the curve OC can easily be divided into three slopes. The transition between these slopes is not a definite point but, in fact, an arc. Since rope of a particular diameter was made of small diameter curled-ropes, it can be observed in the stress-strain curve. When the rope was put under tensile load, the stresses in the curled-ropes were not the same. Initially, one curled rope with maximum stress was broken at point C and stress was suddenly dropped to point D. The remaining curled-ropes took the tension load and the stress started increasing from point D to E and then from E to F, showing two slopes DE and EF having a smooth arc between them. At point F, one of the curled ropes was broken and the stress again dropped suddenly to point G. The rope tensile test continued in similar manner, breaking the remaining curled ropes and having two slopes GH and HI for the third and one slope JK for the last curled-rope. The stress-strain relationship after the peak load depends on the number of curled ropes present in a particular diameter rope.

Figure 5.2: Typical stress-strain relationship for coconut-fibre rope
Table 5.3 shows the results from the rope tensile strength tests. An average of ten and five readings is taken for 36 mm diameter soaked ropes and all other ropes, respectively. The 36 mm diameter soaked ropes are given more weight for the average values because, being a reference rope, it was used in many specimens.

<table>
<thead>
<tr>
<th>Diameter (mm)</th>
<th>Pre-treatment</th>
<th>Ultimate strength (MPa)</th>
<th>Corresponding strain (-)</th>
<th>Elongation at peak load (%)</th>
<th>Total elongation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>≈ 18</td>
<td>Soaked</td>
<td>23.5</td>
<td>0.42</td>
<td>142</td>
<td>167</td>
</tr>
<tr>
<td>≈ 27</td>
<td>Soaked</td>
<td>18.1</td>
<td>0.49</td>
<td>149</td>
<td>174</td>
</tr>
<tr>
<td>≈ 36</td>
<td>Soaked</td>
<td>15.3</td>
<td>0.63</td>
<td>163</td>
<td>190</td>
</tr>
<tr>
<td>≈ 36</td>
<td>Boiled</td>
<td>17.6</td>
<td>0.79</td>
<td>179</td>
<td>206</td>
</tr>
<tr>
<td>≈ 36</td>
<td>CaAl</td>
<td>12.2</td>
<td>0.51</td>
<td>151</td>
<td>179</td>
</tr>
</tbody>
</table>

*Note: The average of 10 readings is taken for ~36 mm Ø soaked ropes because it is used in many specimens and their properties are shown in italic. The average of five readings is taken for other ropes.*

It can be seen from the table that the ultimate tensile strength of rope decreased and the corresponding strain and the elongation increased with an increase in rope diameter. The tensile strength was reduced by 35% and the corresponding strain was increased by 50% when the rope diameter was increased from ~18 mm to ~36 mm. It may be noted that the higher tensile load was required for the thicker rope compared to the thinner rope. The tensile force was not proportionally increased compared to the rope diameter, causing a reduction in tensile strength.

It was also found that the tensile strength and the corresponding strain of chemically treated ropes decreased by 20% and 19%, respectively, and that of boiled and washed ropes increased by 15% and 25%, respectively. It was observed by hand that the chemical and boiled treatment had made the rope surface somewhat brittle/rough and smooth, respectively. This might have caused the decrease and increase in tensile strength for CaAl and boiled ropes, respectively, when compared to that of soaked ropes.
5. Bond strength between rope and CFRC

Similar to equation 4.1, the following simple equation has been developed using the experimental results for the rope tensile strength ($\sigma_{\text{rope}}$) in MPa:

$$\sigma_{\text{rope}} = 130 (D_r)^{0.6}$$ (5.1)

where $D_r =$ rope diameter in mm. This equation is valid for soaked ropes only. The comparison of experimental and empirical results is presented in Figure 5.3. The empirical results are in good agreement with the experimental results. The error is less than 2.5%.

![Figure 5.3: Comparison of experimental and empirical results of the rope tensile strength](image)

**Figure 5.3:** Comparison of experimental and empirical results of the rope tensile strength

5.4.3. Rope pullout behaviour

5.4.3.1. Different stages during rope pullout

Different stages of rope pullout, marked as ‘a’ to ‘e’, are elaborated in Figure 5.4. Since the rope was made of coconut fibres, which have the ability of taking high strains, the upward load of 0.5 kN was applied in order to tighten the rope between the loaded end and jaw of the Avery machine (stage a). When the load was applied to pull out the rope from the specimen, the elongation was initially observed (stage b). Then the slippage of the rope started at the loaded end (stage c). When slippage at the loaded end was about 15-20% of the embedded length $L$ and the load was near to its peak value, the slippage at the unloaded end started (stage d). At peak load, the slippage at the loaded end was approximately 20-30% of $L$. 

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5. Bond strength between rope and CFRC

When the slippage at the loaded end was equal to L, approximately 30-40% of L remained in the specimen showing that elongation in the embedded part had occurred (stage e). It is important to mention that this embedded part was taking only a negligible pullout load. Therefore only L is taken in calculating the embedded surface area of the rope, and slippage at the loaded end is taken as the reference for determining the pullout energy. At the end of the test, no cracks in CFRC around the rope periphery were observed. The matrix specimen was intentionally broken into two halves through the rope embedded part. It was observed with the naked eye that the ribs of the ropes were marked in the matrix, some fibres from the rope were also stuck within the concrete at rib marks and some minute fragments of concrete were stuck to the rope.

5.4.3.2. Typical load-slippage curves

Figure 5.5 shows the typical load-slippage curves for a particular combination. The full and dotted lines show load-slippage at the loaded and unloaded ends, respectively. It can be observed that the slippage at the unloaded end was delayed with respect to that at the loaded end because of the elongation in the embedded part. Since coconut fibres are
5. Bond strength between rope and CFRC

capable of taking high strains, this is why the elongation of the coconut fibre rope was also observed in the embedded part. The debonding between rope and CFRC was gradual from the loaded to unloaded end, because when the load was started, it did not reach at the unloaded end due to elongation in the embedded part. The slippage at the unloaded end is approximately zero even the applied load was near its peak value. This shows that the stress distribution along the embedment length is nonlinear. The rope pullout phenomenon has been explained in detail in section 5.4.3.1. For calculation of the bond strength, it is assumed that there is an equivalent linear distribution. The rope-CFRC interfacial bond strength is calculated by dividing the maximum pullout load by the surface area of the embedded portion of a rope. Pullout energy is calculated as the area under the load – slippage at loaded end curve. An average of three readings is taken for a particular combination.

![Load-slipage curves](image)

**Figure 5.5:** Typical load-slipage curves for a particular combination.

5.4.3.3. Effects of considered parameters on bond strength

The effects of the rope embedment length, rope diameter, pre-treatment condition and concrete mix design ratio with different fibre contents on the bond strength between rope
and CFRC are presented in Figure 5.6. The bond strength reduces from 0.144 to 0.128 MPa as the rope embedment length increases from 100 to 200 mm, (upper left graph of Figure 5.6). It is important to note that the pullout load increases from 1.58 to 2.84 kN as the rope embedment length increases. However, due to a larger embedment, the interfacial surface area increases, ultimately resulting in low bond strength. This means that the pullout force is not proportionally increased compared to the embedment length. Okelo et al. (2005) investigated the bond strength of rebars (aramid fibre reinforced polymer rebars, carbon fibre reinforced polymer rebars, glass fibre reinforced polymer rebars and steel rebars) in normal strength concrete and found that the pullout load increased as the embedment length increased. It was also observed that the average bond strength decreased as the rebar’s embedment length increased. Achillides et al. (2004) studied the bond behaviour of fibre reinforced polymer rebars with the same conclusion that as the embedment length of rebar increased the maximum average bond stress of rebar with concrete decreased. The observations of Okelo et al. (2005) and Achillides et al. (2004) match with the current finding regarding different embedment lengths of ropes in CFRC for their bond strength.

As the rope diameter decreases from ~36 to ~18 mm, the bond strength increases from 0.144 to 0.196 MPa (upper right graph Figure 5.6). It is worth mentioning that a higher pullout load (1.58 kN) is required for a larger rope diameter (~36 mm). Only 1.14 kN pullout load is required to pull out ~18 mm diameter rope. However, due to the increased diameter, the interfacial surface area increases, causing reduced bond strength. Again, the pullout force is not proportionally increased with the rope diameter. Previously, Ichinose et al. (2004) considered the effects of deformed bar diameter on the bond strength and found that an increase in bar diameter resulted in a decrease in bond strength. Guadagnini et al. (2004) also concluded that fibre reinforced polymer rebars with smaller diameters had higher bond strength. The studies of Ichinose et al. (2004) and Guadagnini et al. (2004) verify the present trends of bond strength between rope of different diameters and CFRC. The bond strength between reinforcing rebars and concrete can be increased due to the presence of ribs (Hao et al., 2009), it can be assumed that the ribs of ropes have contributed towards an increase in bond strength.

Boiled treatment of ropes enhanced the bond strength from 0.144 to 0.165 MPa and CaAl ropes reduced bond strength from 0.144 to 0.109 MPa compared to soaked ropes
5. Bond strength between rope and CFRC

(refERENCE CASE), as can be seen in the middle left graph of Figure 5.6. It is also noted in previous studies that boiling and washing coconut fibres was able to increase the tensile strength and the bond strength between coconut fibres and the cement (Filho, 2003; Asasujarit et al., 2007; Hirunlabh et al., 2007). This is also observed in the case of coconut fibre rope and CFRC. Mani and Satyanarayana (1990) found that the combined use of sodium alginate and calcium chloride effectively improved the ultimate tensile strength of fibres by 39.6%. However, a decrease in tensile strength and the bond strength between rope and CFRC is observed in the present case.

The bond strength between rope and concrete is proportional to the mix design ratio (middle right graph in Figure 5.6). Table 5.2 shows that the strength of concrete is proportional to the mix design ratio. This indicates that as the concrete strength increases from 11.6 to 32.4 MPa, the bond strength also increases from 0.105 to 0.199 MPa. Lee et al. (2008) also showed that as the compressive strength of concrete increased from 25.6 MPa to 92.4 MPa, the bond strength between glass fibre reinforced polymer (GFRP) bars and concrete increased. Dancygier et al. (2009) found similar results that the bond between steel reinforcement and high-strength concrete specimens was higher than for normal strength concrete specimens. These trends, observed in Lee et al. (2008) and Dancygier et al. (2009), match with the present study. The addition of fibres in the concrete can also increase its bond with ropes from 0.128 to 0.180 MPa (bottom left graph in Figure 5.6). Campione et al. (2005) and Won et al. (2008) indicated that when the percentage of steel fibres was increased in concrete, the bond strength also increased. It is in accordance with the current trend.

When a knot was provided in CFRC, the ropes were broken during rope pullout. This means that the anchorage/bond is greater than the tensile strength of the rope. Some slippage occurred at the loaded end, but it can be taken as the elongation in the embedded part between the top of the knot and the loaded end.
5. Bond strength between rope and CFRC

**Figure 5.6:** Dependency of bond strength between rope and CFRC
5. Bond strength between rope and CFRC

5.4.3.4. Empirical equation for estimating bond strength

Similar to equations developed in previous chapter, the following empirical equation has been developed using experimental results to determine the bond strength \( \tau_r \) (kPa) between coconut-fibre rope and CFRC:

\[
\tau_r = (\alpha_r \beta_r \gamma_r \lambda_r)^X [t(D_r)^2 + vD_r + w]
\]

where \( D_r \) is the rope diameter in mm; \( t, v \) and \( w \) are constants having values of 0.15, -11 and 344, respectively; \( \alpha_r, \beta_r, \gamma_r \) and \( \lambda_r \) are factors for embedment length, pretreatment, mix design and fibre content of matrix, respectively. \( \alpha_r \) is 1, 0.91 and 0.89 for embedment length of 100, 150 and 200 mm, respectively. \( \beta_r \) is 1, 1.15 and 0.76 for soaked, boiled and CaAl ropes, respectively. \( \gamma_r \) is 0.73, 1 and 1.39 for mix design of 1:4:4, 1:3:3 and 1:2:2, respectively. \( \lambda_r \) is 0.89, 1 and 1.25 for matrix with fibre content of 0, 0.5 and 1%, respectively. \( X \) is 1 if all factors \( \alpha_r, \beta_r, \gamma_r \) and \( \lambda_r \) are equal to 1, or only one factor is other than 1, otherwise it is \( 7/8 \).

The numerical values of bond strength between rope and the considered matrices are in good agreement with that of the experimental values (Table 5.4). The error is less than 1%.
5. Bond strength between rope and CFRC

<table>
<thead>
<tr>
<th>Constants</th>
<th>Variable</th>
<th>Name</th>
<th>Value</th>
<th>Experimental A “kPa”</th>
<th>Equation 5.2 B “kPa”</th>
<th>Difference (A and B) C “%”</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 mm embedded length ($\alpha_r = 1$) Soaked ropes ($\beta_r = 1$) 1:3:3 mix design ($\gamma_i = 1$) 0.5 % fibre content ($\lambda_i = 1$)</td>
<td>Rope diameter</td>
<td>~18 mm $\phi (D_r = 18)$</td>
<td>196.4</td>
<td>194.6</td>
<td>0.92</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>~27 mm $\phi (D_r = 27)$</td>
<td>157.3</td>
<td>156.4</td>
<td>0.86</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>~36 mm $\phi (D_r = 36)$</td>
<td>143.8</td>
<td>142.4</td>
<td>0.97</td>
<td></td>
</tr>
<tr>
<td>~ 36 mm $\phi$ rope ($D_r = 36$) Soaked ropes ($\beta_r = 1$) 1:3:3 mix design ($\gamma_i = 1$) 0.5 % fibre content ($\lambda_i = 1$)</td>
<td>Embedded length</td>
<td>150 mm ($\alpha_i = 0.91$)</td>
<td>130.5</td>
<td>129.6</td>
<td>0.70</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>200 mm ($\alpha_i = 0.89$)</td>
<td>128.0</td>
<td>126.7</td>
<td>0.99</td>
<td></td>
</tr>
<tr>
<td>~ 36 mm $\phi$ rope ($D_r = 36$) 100 mm embedded length ($\alpha_r = 1$) 1:3:3 mix design ($\gamma_i = 1$) 0.5 % fibre content ($\lambda_i = 1$)</td>
<td>Pre-treatment</td>
<td>Boiled ($\beta_i = 1.15$)</td>
<td>165.1</td>
<td>163.8</td>
<td>0.81</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>CaAl ($\beta_i = 0.76$)</td>
<td>109.3</td>
<td>108.2</td>
<td>0.98</td>
<td></td>
</tr>
<tr>
<td>~ 36 mm $\phi$ rope ($D_r = 36$) 100 mm embedded length ($\alpha_r = 1$) Soaked ropes ($\beta_r = 1$) 0.5 % fibre content ($\lambda_i = 1$)</td>
<td>Mix design</td>
<td>1:4:4 ($\gamma_i = 0.73$)</td>
<td>104.9</td>
<td>103.9</td>
<td>0.90</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>1:2:2 ($\gamma_i = 1.39$)</td>
<td>199.4</td>
<td>197.9</td>
<td>0.73</td>
<td></td>
</tr>
<tr>
<td>~ 36 mm $\phi$ rope ($D_r = 36$) 100 mm embedded length ($\alpha_r = 1$) Soaked ropes ($\beta_r = 1$) 1:3:3 mix design ($\gamma_i = 1$)</td>
<td>Fibre content</td>
<td>0 % ($\lambda_i = 0.89$)</td>
<td>127.9</td>
<td>126.7</td>
<td>0.91</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>1 % ($\lambda_i = 1.25$)</td>
<td>179.7</td>
<td>178.0</td>
<td>0.95</td>
<td></td>
</tr>
</tbody>
</table>

The bold data is for the reference case.
5. Bond strength between rope and CFRC

5.4.3.5. Effects of the considered parameters on energy required to pull out the rope

The effects of the considered factors (i.e. rope embedment lengths, rope diameters, pre-treatment conditions and concrete mix design ratios with different fibre contents) on the energy required to pull out ropes are presented in Figure 5.7. It can be easily observed from the figure that the energy required to pull out the rope from the matrix (PC/CFRC) increased from 63.8 to 114.5, 46 to 63.8, 45.7 to 96.5, 56.9 to 88.4 Nm with an increase in embedment length from 100 to 200 mm, rope diameter from ~18 to ~36 mm, cement from 1:4:4 to 1:2:2 and fibre content in matrix from 0 to 1%, respectively. Compared to the reference case of soaked ropes, boiled ropes have increased the pullout energy from 63.8 to 72.3 Nm, and it is reduced from 63.8 to 51.3 Nm with CaAl ropes.

5.4.3.6. Empirical equation for estimating energy required to pull out the rope

Similar to equation 5.2, the following simplified equation has been developed using experimental results to determine the energy “\(E_r\)” (in N.mm) required to pull out the ropes from the concrete:

\[
E_r = (\eta \psi \mu \omega)^Y [x D_r^2 + y D_r + z]
\]

(5.3)

where \(D_r\) is the rope diameter in mm; \(x, y\) and \(z\) are constants having values of -0.003, 1.15 and 26.2, respectively; \(\eta, \psi, \mu, \) and \(\omega\) are factors for embedment length, pre-treatment, mix design and fibre content of matrix, respectively. \(\eta\) is 1, 1.4 and 1.8 for embedment length of 100, 150 and 200 mm, respectively. \(\psi\) is 1, 1.1 and 0.8 for soaked, boiled and CaAl ropes, respectively. \(\mu\) is 0.7, 1 and 1.5 for mix design of 1:4:4, 1:3:3 and 1:2:2, respectively. \(\omega\) is 0.9, 1 and 1.4 for matrix with fibre content of 0, 0.5 and 1 %, respectively. \(Y\) is 1 if all factors \(\alpha, \beta, \gamma\) and \(\lambda\) are equal to 1, or only one factor is other than 1, otherwise it is 7/8.

The numerical values of the energy required to pull out the rope from the considered matrices are in good agreement with that of the experimental values (Table 5.5). The error is less than 1\%.

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5. Bond strength between rope and CFRC

**Figure 5.7:** Effect of considered factors on energy E required for rope pullout
Table 5.5: Percentage difference between experimental and numerical values of energy required to pull out the rope from matrix

<table>
<thead>
<tr>
<th>Constants</th>
<th>Variable</th>
<th>Name</th>
<th>Value</th>
<th></th>
<th></th>
<th></th>
<th>Experimental</th>
<th>Equation 5.3</th>
<th>Difference (A and B)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 mm embedded length ($\eta = 1$)</td>
<td></td>
<td>~18 mm $\phi$ ($D_r = 18$)</td>
<td></td>
<td>Rope diameter</td>
<td></td>
<td>46.1</td>
<td>45.9</td>
<td>0.37</td>
<td></td>
</tr>
<tr>
<td>Soaked ropes ($\psi = 1$)</td>
<td></td>
<td>~27 mm $\phi$ ($D_r = 27$)</td>
<td></td>
<td>55.2</td>
<td>55.1</td>
<td>0.25</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1:3:3 mix design ($\mu = 1$)</td>
<td></td>
<td>~36 mm $\phi$ ($D_r = 36$)</td>
<td></td>
<td>63.8</td>
<td>63.7</td>
<td>0.14</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.5 % fibre content ($\omega = 1$)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>~36 mm $\phi$ rope ($D_r = 36$)</td>
<td></td>
<td>150 mm ($\eta = 0.91$)</td>
<td></td>
<td>Embedded length</td>
<td></td>
<td>89.8</td>
<td>89.2</td>
<td>0.67</td>
<td></td>
</tr>
<tr>
<td>Soaked ropes ($\psi = 1$)</td>
<td></td>
<td>200 mm ($\eta = 0.89$)</td>
<td></td>
<td>115.2</td>
<td>114.7</td>
<td>0.45</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1:3:3 mix design ($\mu = 1$)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.5 % fibre content ($\omega = 1$)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>~36 mm $\phi$ rope ($D_r = 36$)</td>
<td>Pre-treatment</td>
<td>Boiled ($\psi = 1.15$)</td>
<td></td>
<td>70.7</td>
<td>70.1</td>
<td>0.87</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>100 mm embedded length ($\eta = 1$)</td>
<td></td>
<td>CaAl ($\psi = 0.76$)</td>
<td></td>
<td>51.3</td>
<td>51.0</td>
<td>0.64</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1:3:3 mix design ($\mu = 1$)</td>
<td>Mix design</td>
<td>1:4:4 ($\mu = 0.73$)</td>
<td></td>
<td>44.9</td>
<td>44.6</td>
<td>0.67</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.5 % fibre content ($\omega = 1$)</td>
<td></td>
<td>1:2:2 ($\mu = 1.39$)</td>
<td></td>
<td>96.1</td>
<td>95.6</td>
<td>0.55</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>~36 mm $\phi$ rope ($D_r = 36$)</td>
<td></td>
<td>0 % ($\omega = 0.89$)</td>
<td></td>
<td>57.8</td>
<td>57.3</td>
<td>0.79</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>100 mm embedded length ($\eta = 1$)</td>
<td>Fibre content</td>
<td>1 % ($\omega = 1.25$)</td>
<td></td>
<td>89.5</td>
<td>89.2</td>
<td>0.34</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: The bold data is for the reference case.
5. Bond strength between rope and CFRC

5.5. Summary

The effects of rope embedment length, diameter, pre-treatment condition, concrete mix design ratio, fibre content and knot on the bond strength between coconut-fibre rope and coir fibre reinforced concrete (CFRC) were experimentally investigated. The energy required to pull out rope from CFRC was also taken into account. Rope pullout tests were performed. Based on the conducted study, empirical equations were developed to determine the bond strength between rope and CFRC and the energy required for rope pullout. The experimental and empirical results are in good agreement. The higher pullout load (i.e. rope tension) generated due to earthquake loading may govern the foundation depth. The rope tension during dynamic loading is investigated further in Chapter 7. The rope tension should be less than the rope tensile capacity and the pullout load for avoiding structure collapse. Thus, the tensile strength and elongation of coconut fibre rope were also determined considering parameters like rope diameter and pre-treatment in addition. The considered properties of concrete/CFRC, i.e. compressive strength, modulus of elasticity, splitting tensile strength, modulus of rupture, compressive and flexural toughness, were determined by physical tests using standard procedures. All concrete properties improved, as expected, by increasing the cement content.

The experiments on rope tensile and rope-CFRC bond strength revealed that:

- thin and boiled ropes had higher tensile strength and elongation than that obtained with thick, soaked and chemically treated ropes. However, higher tensile load was required for thick ropes compared to thin ropes.
- the bond strength decreased by 11% and 27%, and pullout energy increased by 44% and 28% with an increase in embedment length from 100 to 200 mm and rope diameter from ~18 to ~ 36 mm, respectively.
- the bond strength increased by 41% and 90% and pullout energy enhanced by 35% and 52% with an increase in fibre from 0% to 1% and cement content in the matrix from 1:4:4 to 1:2:2, respectively.
- the bond strength and pullout energy reduced by 24% and 25%, respectively, with chemical treatment and increased by 15% and 13%, respectively, with boiled ropes compared to soaked ropes.
- the ropes broke during rope pullout when a knot was provided in the matrix.
The idea of using CFRC and rope reinforcement in the construction of earthquake-resistant houses is unique. This current research provides the fundamental information about the bond strength between the two materials along with the properties of concrete and main reinforcement. Keeping in mind the above presented results, achieving the ultimate goal (i.e. the development of a structure) will depend on the method of transfer of the load generated due to a ground excitation.
5. Bond strength between rope and CFRC
Chapter 6.

Capacity of interlocking blocks under monotonic loading

Related paper:

6.1. Introduction

As a contribution to low-cost earthquake-resistant buildings, an innovative interlocking block was invented. The block was fabricated with special bottom and top interconnecting profiles so that it will interlock with upper, lower and adjacent blocks. Additional bottom- and top-smooth profiles, as well as half blocks were also developed to facilitate the construction of a wall. The total construction cost can considerably be reduced when locally available materials are used. Therefore, coconut fibre reinforced concrete (CFRC) was utilized as a construction material. The selection of mix design (cement: sand: aggregates) for preparing CFRC was also made before block production because of the invented block shape. The compressive capacities of single (standard, bottom, top and half blocks) and multiple blocks (standard as well as combination of top, standard and bottom blocks) were determined experimentally. The relationship between compressive strength of individual and multiple standard blocks is developed. The in-plane and out-of-plane shear capacities of interlocking mechanism are also investigated.

6.2. Background

Many technologies are available for constructing earthquake-resistant houses. However, most of them are too expensive for the majority of people, especially those in developing countries. The available construction procedures are also too complex. People in rural areas do not hire skilled labor. They even build non-engineered structures by themselves,
just to reduce construction cost. Often they simply adopt old traditional construction technologies, which are not earthquake resistant resulting in collapses and ultimately causing fatalities and financial loss (Munshi, 2009). A number of major earthquakes in the past (e.g. Nepal earthquake in 2011, Haiti earthquake in 2010, Pakistan earthquake in 2005 and Sumatra earthquake in 2004) clearly indicate the need for developing a new construction technology to be adopted in earthquake prone regions. It should be affordable to normal people so that they are able to construct their own houses with the available local resources and little guidance.

The price of reinforcing materials will automatically be cut if the local resources are used. Studies, as described in Chapter 2, have shown that natural fibres are good alternatives because they are not only cost-effective but also promote sustainable buildings as they are recyclable materials. Natural fibres are generally abundantly available in most developing countries where the need for proper and cheap housing construction is in high demand (Gunasekaran et al., 2011).

To reduce construction time and the cost of structures, researchers have also suggested the use of interlocking blocks to replace the normal bricks, thus eliminating mortar from masonry construction (Anand and Ramamurthy, 1999). Ramamurthy and Nambiar (2004) compiled the history of development of interlocking blocks till 2004 and explained their shape geometry, purposes and methods of construction. Some other interlocking blocks were also developed from 2004 to date. Most of these blocks were hollow (Anand and Ramamurthy, 2000 and 2003; Jaafar, 2006), some were solid (Nazar and Sinha, 2007) and curved (Dedek et al., 2012), and in a few cases, with provisions of holes for reinforcement. These blocks can be prepared mechanically or manually, but in some cases these required very complicated moulds and manual casting. Usually, the material was concrete (Anand and Ramamurthy, 2000 and 2003; Jaafar 2006) but there was also stabilized soil (Smith, 2010; Dedek et al., 2012) and fly ash (Nazar and Sinha, 2007; Uygunoglu et al., 2012). The thickness of these blocks also varied, making them suitable either for load bearing, partition or cladding walls. Interlocking mechanisms were provided either by horizontal, vertical or both interconnecting keys for in-plane and out-of-plane directions. The main purpose of these blocks was to make precise alignment and quick construction. It may be noted that the interlocking keys of hollow blocks alone were normally not sufficient to resist the stresses of design load for an assembled wall in...
a structure due to the elimination of mortar layers (Thanoon et al., 2004). This might be because of limited key projection. To overcome this problem, normal reinforced concrete was used at regular intervals in the holes provided in the hollow blocks. This made the structure a little bit expensive. In some cases, relatively less mortar (compared to that required in normal brick masonry) was used with the interlocking blocks (Smith, 2010). Their main objective considered so far was an easier, fast and cost-effective construction mainly for resisting static loading.

To the best of the author’s knowledge, there is only one study, conducted by Elvin and Uzoegbo (2011), on the seismic response of a dry-stack masonry structure made with hydra-form interlocking bricks and minimum steel reinforcement. The details of testing are discussed in Chapter 7. However, the following points are carefully noted from their study: 1. use of minimum steel reinforcement in the structure; 2. blocks made of earth soil, which resulted in splitting and crushing of bricks during dynamic loading; 3. insufficient longitudinal interlocking key height and lack of transverse keys; and 4. earthquakes with vertical components may have adverse effect on energy dissipation. These points may be the shortcomings for efficient performance of the proposed mortar-free structures. By comparison, for the author’s invention: 1. there is no use of steel reinforcement, only ropes are used; 2. blocks are made of CFRC, which avoids splitting and crushing of materials and also gives material damping; 3. sufficient interlocking key heights in both directions; and 4. earthquakes with vertical components are considered in the design of interlocking blocks as energy is dissipated through uplifts (i.e. the relative movement at the interface).

In the following sections the compressive and shear resistance of the CFRC interlocking block is discussed. These strengths of block represent the capacity of a structure in bearing a vertical static load and forces at the interface of the interlocking blocks induced by a horizontal earthquake loading.

6.3. Invented CFRC interlocking blocks

Mortar-free construction, capable of reducing earthquake impact during a seismic event, is considered for seismic-resistant housing. As found earlier in Chapter 3, coconut fibre
reinforced concrete (CFRC) members with cracks have more damping than those without cracks. To enhance the damping capability of the structure, CFRC interlocking blocks are used in the mortar-free construction. Figure 6.1 shows the proposed wall under gravity, and the in-plane and out-of-plane earthquake loadings. Ropes, made of coconut fibres, are utilized as the ungrouted vertical reinforcement of the wall to avoid its ultimate collapse by limiting the block movement up to the key height. The ropes are post-tensioned by anchoring in foundation and top tie-beams. The pullout behaviour of ropes from CFRC and the rope tensile strength are investigated in Chapter 5. The gravity loading is to be taken by the compressive strength of the blocks. Because of the movability of all blocks relative to each other, the earthquake forces induced into the structure will be reduced. The activated lateral forces are then resisted by the interlocking keys of the blocks in both in-plane and out-of-plane directions. This resistance will depend on the shear strength of the interlocking keys. Thus, the invented CFRC interlocking block will lead to the overall objective: easy-to-build, economical and earthquake-resistant housing.

Figure 6.1: Sketch of a mortar-free wall assembly with interlocking blocks under gravity and earthquake loadings. (a) In-plane and (b) out-of-plane directions.
6. Capacity of interlocking blocks under monotonic loading

The shapes of the developed CFRC interlocking blocks are shown in Figure 6.2. These blocks, named as standard, bottom, top and half blocks, can be utilized in constructing a wall (Figure 6.1). The bottom and top layers of the wall should be made with bottom and top blocks (Figures 6.2(b) and (c)), respectively. The block shown in Figure 6.2(a) has overall dimensions of 400 mm long x 200 mm wide x 195 mm high with in-plane and out-of-plane interlocking keys of 45 mm height. Since the blocks are to be used in load bearing and earthquake-resistant houses, the width is taken as a reference (equal to wall width), the length is twice the width and the height is a little less than the width. This size is also selected because of (i) the provision of holes for rope reinforcement, (ii) relatively large interlocking keys, and (iii) the use of fibre reinforced concrete (FRC) as smaller block size may create problems in compaction of FRC. The weights of the standard, bottom, top and half blocks are 24 kg, 21 kg, 20 kg and 12 kg, respectively. The characteristic of the developed blocks is their ability to move relatively to neighbouring blocks during an earthquake because of the mortar-free construction. This relative movement then leads to energy dissipation. Because of the inclined key, the blocks will come back to their original position. The maximum uplift should be restricted to the key height, which is ensured by the presence of post-tension ropes that vertically run through the two holes provided in the block.

As part of the development process, the present study, explained in this chapter, was conducted to determine the compressive and shear capacities of the CFRC interlocking blocks. Type (standard, bottom, top and half blocks) and number of blocks (single and three blocks) were considered in the compressive tests. In-plane and out-of-plane shear tests were also performed. The mix design ratio of cement, sand, aggregates and fibres was optimized for preparing CFRC. This current work was one step towards earthquake-resistant (EQR) buildings using interlocking blocks and rope reinforcement. The testing of mortar-free columns and walls is explained in the next chapter in order to understand their behaviour under seismic loading. This will help in improving the technology for the EQR houses.
Figure 6.2: Innovative CFRC interlocking blocks; (a) standard, (b) bottom, (c) top and (d) half block.
6.3.1. Optimization of CFRC mix design ratio for blocks

A total of six mix design ratios were taken into account. First, the mix design ratio (cement:sand:aggregates) was kept constant (i.e. 1:3:3) while the fibre content was taken as 0, 0.5 and 1% by mass of concrete materials. The water-cement ratio for CFRC with 0.5% and 1% fibre content was 0.54 and 0.64, respectively. The length of medium soaked coconut fibres was 5 cm in all combinations, as this length was optimum for CFRC (from Chapter 3). The fibre content was then kept constant (i.e. 0.5%) while the mix design ratio was taken as 1:2:2, 1:3:3 and 1:4:4. In the next constellation, the sand content was increased compared to the amount of aggregates (i.e. the mix design was changed from 1:3:3 to 1:4:2 having 1% fibre content) so that more mortar was available for grabbing the fibres. Better compaction could be done with the mix ratio of 1:4:2 (i.e. reducing air voids in fresh state and having the smooth finished surface in hardened state) particularly for the interlocking keys of the blocks. Plain concrete (PC) properties were taken as a reference for a comparison with CFRC. For PC, the mix design ratio for cement, sand and aggregates was 1:3:3 with a water-cement (W/C) ratio of 0.48.

The materials, CFRC preparation and procedure for filling cylinders and beamlet moulds have already been explained in Chapter 3. However, a special wooden mould was prepared in such a way that the interlocking block was cast inverted. The mould was filled in four layers: approximately up to (i) 75 mm, (ii) 150 mm, (iii) 175 mm and (iv) 195 mm. The same compaction procedure as was done for CFRC cylinders and beamlets was adopted for each layer. The blocks were also cured for 28 days, then dried for saturated surface dry conditions and finally white-washed before testing to enable a clear identification of cracks.

6.3.1.1. Properties of matrices with considered mix design ratios

The testing and analysis procedures for cylinders and beamlets are explained in detail in Chapter 3. The properties of PC and CFRC with considered mix design ratios, except for 1:4:2, are presented in Table 5.2. For the cases of constant mix design ratio 1:3:3 having fibre contents of 0, 0.5 and 1%, compressive and splitting tensile strength decreased while modulus of rupture, compressive and flexural toughness increased with an increase in fibre content. For the cases with a constant fibre content of 0.5% in mix design ratios
(cement:sand:aggregates) of 1:2:2, 1:3:3 and 1:4:4, as expected, all strength properties increased with higher cement content. The compressive strength, $E$, $T_c$, STS, MOR and TTI of CFRC were reduced by 71, 72, 46, 58, 55 and 48 % when the mix design changed from 1:2:2 to 1:4:4 with the same fibre content of 0.5%.

One of the main purposes of the new construction technology is to make it simple and easy, so that unskilled labour or even lay people should be able to construct their earthquake-resistant houses by themselves. Therefore, at this stage, single storey houses are the core target, for which relatively high strength concrete (around 40 MPa with 1:2:2) is not considered and the strength of around 12 MPa with 1:4:4 may be a little low for constructing houses. So emphasis is on concrete strength of approximately 20 MPa, which can be achieved with the mix design of 1:3:3. The properties of CFRC can be improved by adding more fibres (i.e. 1% fibres) or even by slight change of mix design, say from 1:3:3 to 1:4:2. The addition of fibres in 1:3:3 may cause some voids and thus reduce the compressive strength, which can be avoided in the case of 1:4:2. The compressive strength, $E$, $T_c$, STS, MOR and TTI were increased by 8, 4, 9, 10, 3 and 4% when the mix design was changed from 1:3:3 to 1:4:2 with the same fibre content (1% by mass of concrete materials). This shows that the properties of CFRC are improved when the quantities of sand and aggregates are optimized from 1:3:3 to 1:4:2 for the same fibre content. The improvement is because of avoiding honey combing due to better compaction and allowing more mortar for grabbing the fibres. The properties of CFRC (with 1:4:2 mix design and 1% fibre content) are presented in Table 6.1. An average of three readings was taken.

| Table 6.1: Properties of CFRC (with 1:4:2 mix design and 1% fibre content by mass of concrete material) for preparing interlocking blocks |
|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|
| $\sigma$ (MPa) | $\varepsilon$ (%) | $E$ (GPa) | $T_c$ (MPa) | STS (MPa) | $\Delta$ (mm) | $\Delta_{\text{max}}$ (mm) | $P_{\text{crack}}$ kN | TTI (-) |
| 20.7 | 0.24 | 19.3 | 0.25 | 2.75 | 3.6 | 1.21 | 3.3 | 11.8 | 4.51 | 2128 |

Density (kg/m$^3$)
6. Capacity of interlocking blocks under monotonic loading

6.4. Block capacities

For investigating the compressive and shear capacity of interlocking blocks, a mix design of 1:4:2 with a fibre content of 1 % by mass of concrete materials and length of 5 cm with water-cement ratio of 0.64 was selected.

6.4.1. Compressive capacity

CFRC interlocking blocks were tested in a 2000 kN compression testing machine for their compressive strength, elastic modulus, total compressive toughness and Poisson’s ratio. The experimental setup is shown in Figure 6.3. The block was placed between two steel plates; one with a spherical projection so that the applied stress was uniform during the test. Four portal strain gauges were attached at the corners to measure the vertical deformation. An average of these four readings was taken for vertical strain. Portal gauges were also attached at the front and rear of the block to measure the lateral strain. The same setup was used for testing the bottom, top, half and multiple blocks. In the case of multiple blocks, the vertical deformation was measured with the help of linear variable differential transducers (LVDTs) instead of portal gauges. The measurements taken by the portal gauges, LVDT and applied load were then recorded through a computer connected to the test setup throughout the process. Multiple blocks consisted of three blocks. This was selected to check the compressive load carrying capability of the invented interlocking blocks stacked together.

Figure 6.3: Test setup for compressive capacity
6. Capacity of interlocking blocks under monotonic loading

Load-deformation curves were recorded during the tests, which were then converted to average stress-strain curves for determining block properties. For a single standard block test, the maximum load was 370 kN and the corresponding deformation was 2.28 mm. It was observed that cracks were initiated at the bottom interlocking keys of the block, with the first crack appearing in one of the corner interlocking keys on the bottom side of the standard block before the ultimate load was reached, at approximately 90% of its peak load, propagating upwards. The reason could be a smaller bearing area of the bottom interlocking keys in comparison to the upper keys, especially at the corners. The appearance of the first crack was always in one of the corner interlocking key at the bottom of the block, but not in the respective key of the other samples/specimens (as three specimens were tested). The reason could be the uneven compaction in the interlocking keys as the blocks were prepared manually. More cracks appeared in the other interlocking keys on the bottom side as the load approached its maximum value. When the load decreased after reaching its peak value, the cracks continued to propagate upwards and widen. The cracks were bridged by the fibres and parts of the block were in contact with each other at the end of the test.

For bottom, top and half blocks, the maximum loads were 751, 269 and 115 kN, respectively, and the corresponding displacements were 5.22, 7.61 and 4.15 mm, respectively. During these tests, as expected, the cracks were initiated in the bottom interlocking keys of top and half blocks propagating upwards, and in the top interlocking keys of bottom blocks propagating downwards.

For a test of multiple standard blocks, the maximum load was 344 kN and the corresponding deformation was 5.46 mm. Cracks initiated at the bottom interlocking keys of the standard block at the bottom, and propagated upward. The first crack appeared after the ultimate load had been reached. As the load continued to decrease, the length and width of the crack increased and more cracks developed propagating upwards. Only tiny cracks were observed on the top and middle standard blocks, although the bottom interlocking keys of the bottom standard block had been squashed quite severely. The cracks in the top standard block were more than those in the middle standard block. The reason for this is that, after the bottom keys of the lowest standard block were completely crushed, the top interlocking keys of the upper most standard block became the smallest contact area with the load. This induced the highest stress on the top standard block,
causing more cracks compared to the standard block in the middle. At the end of the test, all standard blocks had their integrity because of the presence of fibres, with the exception of the bottom keys of the lowest standard blocks. This indicates that when walls are built, the bottom and top layers of the wall should be made with bottom and top blocks, as shown in Figures 6.2(b) and 6.2(c), respectively. It was the reason that the tests on multiple blocks with top, standard and bottom blocks were also performed. In this test, bottom and top blocks had the same contact area with the applied load, whereas the standard block in the middle had relatively larger contact interface with upper and lower blocks. The maximum load was 710 kN and the corresponding vertical displacement was 10.13 mm. Cracks initiated in the bottom interlocking keys of the top block propagating upwards. After that, cracks appeared in the bottom interlocking keys of the middle standard block, followed by the cracks in the top interlocking keys of the bottom block. At the end of the tests, it was observed that relatively fewer cracks were present compared to that in the three standard blocks testing.

An average stress-strain curve of an interlocking block was analysed in a similar manner to the compressive properties of CFRC and PC cylinders. It is important to note that the block has different top and bottom contact areas, resulting in different top and bottom stresses. There is no guideline available for which stress is to be taken to represent the compressive strength of the interlocking block and to calculate other engineering properties (modulus of elasticity and compressive toughness). The bottom and top contact areas of the tested specimens are shown in Figure 6.4. These areas are used to calculate the bottom and top stresses and then their average is taken to represent the compressive strength of the block. The vertical strains at the four corners are slightly different. An average of the top and bottom stresses, and four vertical (or axial) strains are taken as the average-stress and average-strain, respectively. From each standard single and multiple blocks test, the average stress-strain curves are shown in Figure 6.5. The peak stress of a single block is larger than that of multiple blocks, but the corresponding vertical strain of the multiple blocks is higher than that of a single block. The reason for the larger strain could be the small gap between the two blocks in case of multiple blocks. The reason for the lower stress of multiple blocks is their slenderness (i.e. ratio of height to least dimension; in this case it is 2.9). The compressive toughness and modulus of elasticity are calculated using these curves. For single blocks, Poisson’s ratio (i.e. the ratio of lateral to
6. Capacity of interlocking blocks under monotonic loading

vertical strains) is calculated. The lateral (horizontal or transverse) strain is the average of strain measured at the front and rear sides of the blocks.

![Specimens for compressive tests](image1)

**Figure 6.4:** Definition of contact areas for calculating compressive stresses.

![Average stress-strain curves](image2)

**Figure 6.5:** Average stress-strain curves from single and multiple blocks under compressive loading.
The properties of single and multiple specimens are presented in Tables 6.2 and 6.3, respectively. An average of three readings was taken. The compressive strength of a single standard block was 16.48 MPa with the corresponding axial and transverse strain of 1.3 and 0.02%, respectively. Its modulus of elasticity, compressive toughness and Poisson’s ratio were 2.34 GPa, 0.56 MPa and 0.015, respectively. The compressive strength of bottom block (17.02 MPa) was more than that of other tested blocks. It may be noted that the strength of the top block (7.73 MPa) was less compared to that of the half block (8.66 MPa), but the average maximum load of the former (244 kN) was more than that of the latter (104.05 kN). Also, the contact areas of the top block are much larger than those of the half block (Figure 6.4). This illustrates that the peak load is not proportional to the contact area, resulting in less strength for the top block and more strength for the half block. The total compressive toughness of the bottom block is also more than that of other blocks (standard, top and half blocks). The reason is that the stress drop after the peak stress is more for other blocks compared to that of the bottom block. It is because the bottom interlocking keys of the standard, top and half blocks were relatively weak compared to the top ones, which initiated cracks in the bottom keys and ultimately provided less resistance to damage after peak stress in comparison to the bottom blocks with top interlocking keys only. On the other hand, the compressive strength, modulus of elasticity and total compressive toughness of multiple standard blocks are 15.8 MPa, 1.44 GPa and 0.56 MPa, respectively. 0.7 MPa compressive strength was reduced for multiple blocks when compared with that of a single block. It was observed that $f_{cm} = 0.96 f_{cs}$, where $f_{cm}$ and $f_{cs}$ stand for the compressive strengths of the multiple and single blocks, respectively. Jaafar et al. (2006) also observed for their developed interlocking hollow concrete blocks that $f_{cw} < f_{cp} < f_{cb}$, where $f_{cw}$, $f_{cp}$, $f_{cb}$ stands for compressive strength of a wall panel, prism with three blocks and unit block, respectively. Thus, the compressive strength of a wall panel made of CFRC interlocking blocks will also be lower because of its higher slenderness ratio, which is defined as the ratio of height to the least horizontal dimension. In the standard multiple blocks compressive test, the smallest stress was experienced by the middle block because it had the largest contact area with the lower and upper blocks. Therefore, the crack appeared in the top block after the cracks of the bottom block. This was also observed in lateral strains of the blocks i.e. top and bottom blocks had lesser and larger lateral strain, respectively. The average lateral strains of the top, middle and bottom blocks are 0.015, 0.012 and 0.019, respectively. The compressive toughnesses of the single and multiple standard blocks are the same (i.e. 0.56 MPa).
6. Capacity of interlocking blocks under monotonic loading

Table 6.2: Average compressive capacity of single blocks

<table>
<thead>
<tr>
<th>Block shape</th>
<th>Standard block</th>
<th>Bottom block</th>
<th>Top block</th>
<th>Half block</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum load (kN)</td>
<td>396.03</td>
<td>730.98</td>
<td>244</td>
<td>104.05</td>
</tr>
<tr>
<td>Compressive strength (MPa)</td>
<td>16.48</td>
<td>17.02</td>
<td>7.73</td>
<td>8.66</td>
</tr>
<tr>
<td>Corresponding vertical strain (%)</td>
<td>1.3</td>
<td>3.3</td>
<td>4.2</td>
<td>1.8</td>
</tr>
<tr>
<td>Corresponding horizontal strain (%)</td>
<td>0.02</td>
<td>0.016</td>
<td>0.029</td>
<td>0.023</td>
</tr>
<tr>
<td>Modulus of elasticity (GPa)</td>
<td>2.34</td>
<td>1.05</td>
<td>0.21</td>
<td>0.53</td>
</tr>
<tr>
<td>Total compressive toughness (MPa)</td>
<td>0.56</td>
<td>1.08</td>
<td>0.32</td>
<td>0.37</td>
</tr>
<tr>
<td>Poisson’s ratio (-)</td>
<td>0.015</td>
<td>0.005</td>
<td>0.007</td>
<td>0.013</td>
</tr>
</tbody>
</table>

Table 6.3: Average compressive capacity of multiple blocks

<table>
<thead>
<tr>
<th>Stacked blocks</th>
<th>Standard blocks</th>
<th>Top, standard and bottom blocks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum load (kN)</td>
<td>379.13</td>
<td>696.79</td>
</tr>
<tr>
<td>Compressive strength (MPa)</td>
<td>15.78</td>
<td>9.28</td>
</tr>
<tr>
<td>Corresponding vertical strain (%)</td>
<td>1.50</td>
<td>2.5</td>
</tr>
<tr>
<td>Corresponding horizontal strains (%)</td>
<td>0.015, 0.012, 0.019</td>
<td>0.024, 0.014, 0.018</td>
</tr>
<tr>
<td>Modulus of elasticity (GPa)</td>
<td>1.44</td>
<td>0.39</td>
</tr>
<tr>
<td>Total compressive toughness (MPa)</td>
<td>0.56</td>
<td>0.48</td>
</tr>
</tbody>
</table>

* Note: Strains of the top, middle and bottom blocks

Now, in comparison to multiple blocks with top, standard and bottom blocks, the compressive strength and total toughness index of multiple standard blocks are higher. The reason is that the former specimen has a larger contact area compared to the latter
6. Capacity of interlocking blocks under monotonic loading

(Figure 6.4). The average peak load for multiple blocks with top, standard and bottom blocks (696.79 kN) is also much larger than for multiple standard blocks (379.13 kN). Again, the peak load is not proportionate to their contact areas, resulting in more strength for multiple standard blocks and less strength for multiple blocks with top, standard and bottom blocks. It is also important to mention that the single and multiple blocks have smaller compressive strengths than those obtained from CFRC cylinder tests. This might be because of the blocks’ unique shape.

6.4.2. Shear capacity

The test setups for determining the in-plane and out-of-plane shear capacities of the interlocking mechanism are shown in Figure 6.6. Three blocks were stacked horizontally between two steel plates. These plates were connected using four steel rods and tightened using nuts. Four load cells were attached to the rods, one for each rod, to measure lateral load. For the in-plane test, the setup was put on a 2000 kN compressive testing machine so that the direction of the load was parallel to the longitudinal bed joints (Figure 6.6(a)). For the out-of-plane test, the setup was put on a 500 kN compressive testing machine (because of space limitation in the 2000 kN machine) so that the direction of the load was parallel to the transverse bed joints (Figure 6.6(b)). The load was applied to the middle block while steel blocks were placed under the left and right blocks. The displacement of the middle block relative to left and right blocks is measured by means of a LVDT.

The typical load-displacement curves recorded from in-plane and out-of-plane shear tests are shown in Figure 6.7. The maximum load and the corresponding deflection of in-plane are 165.2 kN and 12.9 mm, respectively, and that of out-of-plane shear are 145 kN and 7.8 mm, respectively.
Figure 6.6: Test setups for shear capacity; (a) in-plane and (b) out-of-plane loading.
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The shear-off areas of the in-plane and out-of-plane block testing are shown in Figure 6.8(a) and 6.8(b), respectively. The interlocking keys are numbered from 1 to 26, in order to explain the sequence of sheared-off keys. It may be noted that the sheared-off keys are not in a symmetrical pattern for the in-plane testing (shown in dotted rectangles in Figure 6.8(a)). This might be because of the lack of confined compaction in some sheared keys.

During the in-plane testing (refer to Figure 6.8(a)), the cracks first appeared in the interlocking keys 14 and 15 on the bottom side of the middle block because of keys 21 and 22 on the top side of the right block at a load of 58 kN, only 37% of the ultimate load. After reaching the maximum load, another crack formed in the interlocking key 9 on the top side of the middle block due to key 3 on the bottom side of the bottom block. As the load continued to decrease, cracks also appeared in the other blocks. Rather than spreading all over the block, the cracks in each block were confined to its interlocking keys. The interlocking keys 5 and 6 at the bottom of the left block are sheared-off by keys 11 and 12, respectively, at the top of the middle block. Similarly, the keys 14 and 15 at the bottom of the middle block were sheared-off by keys 21 and 22, respectively, at the top of the right block. On the other hand, some keys at the top of the middle and right blocks were also sheared-off by the keys at the bottom of the left and middle blocks, respectively. Because of friction between the two surfaces, some parts of keys 3, 16 and 17 at the bottom of the left and middle blocks were torn off, but the keys were not sheared. This might be due to a lack of proper compaction during manual casting in the

Figure 6.7: Load-displacement curves from selected shear tests.
bottom interlocking keys of the block. However, the key 4 at the bottom of the left block was sheared-off by key 8 at the top of the middle block. This was not observed for the interface between the bottom side of the middle block and the top side of the right block. Compaction during casting could be the reason for this non-uniform behaviour. This is claimed because, among many factors, compaction is the one which also affects the strength of the concrete. Since other factors were the same for the considered case and compaction was done manually, there might be variations for different blocks, even though the same method of compaction was used for all blocks. This may be taken as human carelessness. Also, compaction of the fibre reinforced concrete demands much more care compared to that of plain concrete, so it is very likely that this would have been the reason for the non-uniform behaviour. The key 7 on the top side of the middle block was sheared-off by the combined effect of friction with the central bottom sides of the left block and the sheared key 9 at the top of the middle block. Similarly, the key 26 at the top of the right block was sheared-off by the combined effect of friction with the central bottom sides of the middle block and the sheared keys 24 and 25 at the top of the right block. It can be said, in general, that the corner keys 1, 2, 5 and 6 (or 14, 15, 18 and 19) at the bottom were weak in comparison to keys 8, 9, 11 and 12 (or 21, 22, 24 and 25) at the top. Whereas, if proper compaction was done, middle keys 3 and 4 (or 16 and 17) at the bottom would be strong in comparison to keys 8, 9, 11 and 12 (or 21, 22, 24 and 25) at the top side for the in-plane case.

In the out-of-plane tests (refer to Figure 6.8(b)), alike patterns were observed for the interface between the bottom side of one block and the top side of the other block. Cracks were confined to the interlocking keys. The edged keys 2, 4 and 6 at the bottom of the left block were sheared-off by the out-of-plane keys 7, 10 and 13, respectively, at the top of the middle block. Similarly, the edged keys 14, 16 and 18 on the bottom side of the middle block were sheared-off by the out-of-plane keys 20, 23 and 26, respectively, on the top side of the right block.
6. Capacity of interlocking blocks under monotonic loading

**Figure 6.8:** Sheared interlocking keys highlighted in shaded black from selected tests due to (a) in-plane and (b) out-of-plane loading.
6. Capacity of interlocking blocks under monotonic loading

The maximum load from the load-displacement curves is divided by the total sheared-off area to get the shear capacity. The area under the load-displacement curve is taken as the energy required to cause shear failure of the interlocking keys. The author has defined a parameter shear index describing the shear toughness as the ratio of the total area under the load-displacement curve to the area under the load-displacement curve up to the maximum load. It is a measure of how much energy is required to cause further displacement after its strength decreases when subjected to shear loading. The shear capacity, total energy required to shear, and shear toughness of in-plane and out-of-plane testing are presented in Table 6.4. An average of three readings is taken. It can be observed that the out-of-plane shear strength (3.3 MPa) is more than the in-plane shear strength (2.65 MPa), but slightly more energy (3.61 kNm) is required to cause in-plane shear than out-of-plane shear (3.53 kNm). Actually, the sheared-off area of in-plane direction is much larger than that of out-of-plane (Figure 6.8), but the load required to cause in-plane shear (165.2 kN) is slightly more than that of out-of-plane (148.5 kN). This means that the load does not proportionally increase the sheared-off area (indicated in black in Figure 6.8) for in-plane testing when compared to out-of-plane testing. Therefore, the slight increase in the numerator and the considerable increase in denominator results in a lesser ratio, representing relatively low shear strength (i.e. 2.65 MPa).

![Table 6.4: Average shear capacity of interlocking keys](image)

<table>
<thead>
<tr>
<th>Shear</th>
<th>In-plane</th>
<th>Out-of-plane</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum load (kN)</td>
<td>165.2</td>
<td>148.5</td>
</tr>
<tr>
<td>Shear strength (MPa)</td>
<td>2.65</td>
<td>3.30</td>
</tr>
<tr>
<td>Total energy required to cause shear (kN.m)</td>
<td>3.61</td>
<td>3.53</td>
</tr>
<tr>
<td>Total shear toughness (-)</td>
<td>7.59</td>
<td>7.11</td>
</tr>
</tbody>
</table>

As the shear load (i.e. applied vertical load in Figure 6.6) increased during the tests, the variation of lateral reaction forces is shown in Figure 6.9. Parts of the curves from the in-plane shear tests are projected because the maximum displacement of the middle block cannot be measured due to space limitations in the 2000 kN compression machine.
It can be observed that the lateral reaction force at the lower part (bottom-left and right) is larger than that of the upper part (top-left and right) in both in-plane and out-of-plane cases. When considering either the lower or upper part, it can also be seen that there is a little difference between the left and right load cells readings. This might be caused by an initial minute gap between the block and steel plate and/or the development of very small gaps between sheared-off keys and the central block at a few locations within the
interface. Since the blocks were cast inverted, it was difficult to have a very smooth bottom surface of the block with fibre reinforced concrete. When the block was placed between the steel plates, very small (minute) gaps at some locations between the block bottom surface and the steel plate could be seen with the naked eye. These gaps are named as “initial minute gaps”. The blocks between the steel plates were tight enough to avoid movement/slippage of the blocks without any load. At this stage, the load measured was around 5kN. An effort was made to have minimum gaps. These gaps were not measured, but were kept in mind if the output detected these in terms of minor differences in readings of the left and right load cells. At these sheared portions, these small gaps were created for a very short duration due to falling particles of cement and sand, causing the slightly different readings in the left and right load cells.

For one of the in-plane tests, the corresponding lateral reaction force at the maximum vertical load for top-left, top-right, bottom-left and bottom-right locations was 3.18, 3.41, 5.76 and 6.0 kN, respectively. In one of the out-of-plane cases, the maximum lateral force at top-left, top-right, bottom-left and bottom-right locations was 4.25, 4.39, 5.30 and 5.34 kN, respectively. The lateral force at the maximum vertical load at the corresponding load cells was 3.48, 3.42, 5.05, and 4.61 kN, respectively. It may also be noted that the corresponding lateral force at the maximum vertical load for the upper portion (top-left and right) was less for the in-plane tests compared to the out-of-plane tests. The same for lower portion (bottom-left and right) was greater for the in-plane tests than the out-of-plane tests.

6.5. Summary

A novel interlocking block was presented. Different block shapes were developed in order to facilitate the wall construction. These blocks are named as standard, bottom, top and half blocks. Coconut fibre reinforced concrete (CFRC) was used in the manufacture of these blocks, which is a novel technique. The properties of CFRC were determined for different mix design ratios (i.e. cement:sand:aggregates) and fibre contents. The first case was 1:2:2, 1:3:3, and 1:4:4 with a constant fibre content of 0.5% by mass of concrete materials, the second case was 1:3:3 with fibre contents of 0 and 1%, and the third case was 1:4:2 with a fibre content of 1%. The 20 MPa strength of coconut fibre reinforced
concrete (CFRC) may be sufficient for single-storey earthquake-resistant houses. Higher strength (by using higher cement content) will increase the construction cost.

The compressive capacities of single (standard, bottom, top and half blocks) and three blocks (standard as well as combination of top, standard and bottom blocks) were determined experimentally. The in-plane and out-of-plane shear capacities of the blocks were investigated. An average of three readings was taken to represent a particular property. The shear toughness of the blocks is measured by the shear index, defined as the ratio of the total area under the load-displacement curve to the area under the curve up to maximum load. The following conclusions are drawn:

- CFRC with a mix design of 1:4:2 (cement:sand:aggregates), a W/C ratio of 0.64 and a fibre content of 1% by mass of concrete materials is recommended for manufacturing blocks, because of its better properties compared to those of CFRC with a mix design of 1:3:3 with 0, 0.5 or 1% fibre content.
- The compressive strength and total compressive toughness of the bottom block were higher than that of other blocks (standard, top and half blocks).
- The compressive strength of multiple standard blocks was only slightly less than that of the single standard block because of the slenderness effect, i.e. for standard blocks, \( f_{cm} = 0.96 f_{cs} \), where \( f_{cm} \) and \( f_{cs} \) stand for compressive strength of multiple and single blocks, respectively.
- Compressive toughness was the same for single and multiple standard blocks.
- The average of compressive peak load for multiple blocks with top, standard and bottom blocks was 84% larger than that for multiple standard blocks, but the compressive strength of the latter was 70% higher.
- The out-of-plane shear strength was 25% larger than the in-plane strength.
- More energy was required to generate an in-plane shear failure than the out-of-plane shear failure.
- The average in-plane shear index was 7% higher than the out-of-plane shear index.
- During manual casting, the compaction of CFRC in interlocking keys of the developed block should be done with great care to ensure uniform compaction.

This pilot study is the first step towards exploring the behaviour of the invented block for the deemed technology. Mechanical casting can help in proper compaction, and
6. Capacity of interlocking blocks under monotonic loading

production in large numbers will reduce the relative manufacturing costs. The blocks under cyclic loading should also be investigated. The interlocking blocks may provide a practicable solution for low-cost seismic-resistant housing.
Chapter 7.

Dynamic response of mortar-free structures

Related paper:

7.1. Introduction

Economical earthquake-resistant housing is desirable in the rural areas of underdeveloped and developing countries. These regions often suffer a significant loss of life because of lack of seismic-resistant housing. To enable an efficient and cost-effective solution, a new concept of construction (i.e. structures consisting of (i) interlocking blocks with movability at the interface and (ii) rope reinforcement) was investigated. Mortar-free construction can facilitate more energy dissipation during a seismic event, because of the relative movement at the interface. Four structures were considered: two columns (with and without ropes) and two walls (with and without ropes). This chapter describes the in-plane behaviour of the mortar-free structures under different loadings (i.e. push over, snap back, impact, harmonic and earthquake loadings). The influence of rope reinforcement on natural frequency, damping ratio, induced accelerations, top relative displacement, base shear, bending moment and block uplift is investigated.

7.2. Background

Earthquakes are one of the most hazardous threats that have caused collapse of houses and have resulted in fatalities. The reason is that the unreinforced masonry buildings are not good in bearing lateral load resulting from the strong ground motions (Turek et al.
7. Dynamic response of mortar-free structures

2007). One remedial measure is to reinforce the brick construction using stiffeners composed of steel reinforced concrete grout or fibre reinforced polymer sheets. This can considerably improve the strength of the construction against lateral earthquake loading (Thanoon et al. 2007; Munshi 2009; Turek et al. 2007). However, the problem lies on the economical aspect. The price of steel reinforced concrete is still quite expensive for most people, particularly those residing in rural areas of developing and underdeveloped countries. Thus, the usage of steel reinforced masonry is not adopted widely. An economical solution for housing in earthquake prone areas can surely help in avoiding structure collapse, ultimately resulting in the saving of many lives.

It is anticipated that the mortar-free construction can increase energy dissipation during an earthquake. Elvin and Uzoegbo (2011) studied the seismic response of a full-scale dry-stack masonry structure (3.9m x 3.9m x 2.76m high) made of hydraform interlocking bricks and minimum steel reinforcement with a 2560 kg top mass to simulate roof loading. Four earthquakes were applied in the following sequence: Northridge, Llolleo, El Centro and Llolleo for the second time. Applying one earthquake after the other to the test structure resulted in a small, but cumulative, damage. Two additional tests were performed to induce damage: i) harmonic base excitation that lasted for 6 s, had an amplitude of 9 mm and a frequency of 5 Hz, with a peak base acceleration of 0.9 g; and, ii) harmonic base excitation that lasted for 15 s, had an amplitude of 22 mm and a frequency of 3 Hz, with a peak acceleration of 0.8 g. As a result of these loadings, bricks were shifted and damaged in the plane of the wall to such an extent that brick-sized gaps opened up. Many bricks were split, cracked, crushed or dislocated. The cracking of plaster and spalling also occurred. A typical “X” damage pattern appeared in the two walls collinear with the direction of the base motion. Despite this severe damage, the test structure remained standing, carrying the roof load. The energies of the earthquake and harmonic loadings were dissipated through inter-brick friction and in some cases by brick cracking and crushing. The bricks were dry-stacked, which allowed them to move and, hence, dissipate energy. The ability of the test structure to withstand the applied earthquakes relied heavily on energy dissipation through friction. It should be noted that only horizontal ground motion was considered. If the vertical component of the ground accelerations was considered, the interaction between the bricks might be reduced.
To the best of the author’s knowledge, mortar-free construction without steel reinforcement has not been investigated so far for resisting earthquake loading. This study was conducted to understand the dynamic characteristics of mortar-free structures with the newly developed interlocking CFRC blocks described in Chapter 6.

7.3. Experimental procedures

7.3.1. Preparation of specimens

The materials, CFRC preparation and procedure for filling cylinders and beamlets have been explained in Chapter 3. The preparation of interlocking blocks is mentioned in Chapter 6. Four interlocking blocks, six cylinders and three beamlets were prepared from one batch of CFRC. A footing, 1500 mm long, 600 mm wide and 200 mm deep, was also prepared with CFRC (having a mix design ratio of 1:4:2 and 5 cm long fibres) but with a low fibre content (0.25%) because of the presence of steel reinforcement ($\phi 10\text{mm@100mmc/c}$ in both directions at bottom layer and $\phi 5\text{mm@100mmc/c}$ in both directions at top layer). The reason for using CFRC in the foundation was to observe damage (if any) because of friction between the blocks and foundation. Steel reinforcement was used so that the foundation could easily be shifted on and off the shake table; otherwise, steel reinforcement was not required. The holes were provided in the foundation for the insertion of the rope reinforcement so that the same foundation could be used for all structures to be tested; otherwise, the holes in the foundation were not required. There was no damage in the foundation after all structures had been tested. The depth of foundation would be governed by the rope tension (i.e. pullout load) generated during the dynamic loading. A groove (1200 mm long, 200 mm wide and 75 mm deep) was also provided in the foundation for holding the interlocking blocks. All specimens were cured for 28 days, then dried for 48 hours and finally whitewashed before testing to enable clear identification of cracks. Coconut fibre ropes of ~36 mm diameter were used as vertical reinforcement of the structures.

7.3.2. CFRC properties

The testing and analysis procedures for cylinders and beamlets are explained in detail in Chapter 3. The properties of CFRC used for preparing the blocks are presented in Table
7. Dynamic response of mortar-free structures

6.1, and those for preparing the foundation are shown in Table 7.1. An average of three readings is taken. The compressive capacities of single and multiple blocks, and their shear capacities, are presented in Tables 6.2, 6.3 and 6.4, respectively. The typical stress-strain curves and properties of coconut-fibre ropes are shown in Figure 5.2 and Table 5.3, respectively.

| Table 7.1: Properties of CFRC (with 1:4:2 mix design and 0.25% fibre content by mass of concrete material) for preparing foundation |
|-------------|-------------|-------------|-------------|-------------|-------------|-------------|--------------|
| Cylinder testing | Beam-let testing | Density (kg/m³) |
| σ (MPa) | ε (%) | E (GPa) | Tc (MPa) | STS (MPa) | MOR (MPa) | Δ (mm) | P_crack (kN) | Δ_max (mm) | TTI (-) |
| 20.9 | 0.21 | 19.6 | 0.20 | 2.85 | 3.2 | 1.07 | 2.4 | 8.2 | 4.12 | 2203 |

7.4. Testing and analysis of mortar-free interlocking columns

Two mortar-free columns, one without rope reinforcement and one with rope reinforcement, were considered for the determination of their dynamic characteristics. The columns consisted of 13 blocks with an overall height of 1.92 m (Figure 7.1). A mass of 150 kg was put at the top of the column to simulate it as a single degree of freedom system. The ropes were post-tensioned with a magnitude of 200 N. Different tests (i.e. push over, snap back, impact and shake table tests) were performed.

Figure 7.1: Mortar-free structures; (a) column without ropes, and (b) column with ropes.
7. Dynamic response of mortar-free structures

7.4.1. Push over tests

This test was performed to determine the approximate initial stiffness of the considered columns so that their natural frequencies could be estimated. With the help of this information (i.e. the approximate natural frequency), the structures can be excited with small amplitudes for many frequencies (including the natural frequency $f_n$) to obtain an approximate damping ratio $\zeta$ and relatively more accurate $f_n$ of the mortar-free structures.

A horizontal in-plane force was applied at the top of the column from one side (i.e. from the left side of the column in Figure 7.1(a)) and the lateral displacement was measured from the other side. The horizontal force and the lateral displacement were measured with the help of a 10 kN load cell and a wire displacement transducer, respectively. The force-displacement relationships for both columns are shown in Figure 7.2. The initial slope is shown by a straight dashed line. This is used in calculation of the initial stiffness $k$. For estimating natural frequencies, the mass is taken as the summation of top mass and the mass of the upper half column. The stiffnesses are 60561 N/m and 71229 N/m for the columns without and with ropes, respectively. Using these values, their natural frequencies were determined as 2.28 Hz and 2.48 Hz, respectively. The tests were repeated three times, and the average values are given in Table 7.2. As expected, the column with ropes is stiffer, and thus had a higher natural frequency than the column without ropes.

![Figure 7.2: Force – displacement curves for the (a) column without ropes and (b) column with ropes.](image-url)
7. Dynamic response of mortar-free structures

7.4.2. Snap back tests

These tests were performed to get an approximation of the damping ratios $\xi$ and natural frequencies of mortar-free column structures. An accelerometer was attached at the top of the column (i.e. block 13). The column was pulled at the top by 50 mm and suddenly released to allow it to vibrate freely. The recorded acceleration-time histories are shown in Figure 7.3. The logarithmic decrement method was used to calculate the damping ratios. These were 4.6% and 3.6% for the columns without and with ropes, respectively. Natural frequencies were calculated from the period of the acceleration-time histories. These came out to be 2.06 Hz and 2.30 Hz for the columns without and with ropes, respectively. The tests were repeated three times and the average values are shown in Table 7.2.

![Acceleration-time histories](image)

**Figure 7.3:** Acceleration-time histories for the (a) column without ropes and (b) column with ropes.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Push over</th>
<th>Snap back</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Column without ropes</td>
<td>Column with ropes</td>
</tr>
<tr>
<td>Stiffness $k$ (N/m)</td>
<td>59430</td>
<td>72753</td>
</tr>
<tr>
<td>Natural frequency $f_n$ (Hz)</td>
<td>2.26</td>
<td>2.50</td>
</tr>
<tr>
<td>Damping ratio $\xi$ (%)</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

**Table 7.2:** Average values of $k$, $f_n$, and $\xi$ from push over and snap back tests for the considered columns.
7. Dynamic response of mortar-free structures

7.4.3. Impact tests

These tests were performed to correlate the induced accelerations with the impact force, which may approximately be taken as the base shear. The experimental setup for the column without ropes is shown in Figure 7.4, and the same setup was used for the column with ropes. Four accelerometers were attached along the height of column (i.e. at blocks 2, 5, 9 and 13). The accelerations recorded by these accelerometers are named as \( \ddot{u}_2, \ddot{u}_5, \ddot{u}_9, \) and \( \ddot{u}_t, \) respectively. An impact force \( (P_2) \) was applied with the help of a calibrated hammer at the bottom block (i.e. block 2). The impact force and the induced accelerations were recorded. Three magnitudes of impact force, named as low, medium and high, were considered. Each magnitude was applied five times to obtain a reliable relationship between the force and activated acceleration. Since the impact force was applied manually, the magnitude of each level was not a constant value but a small range; low impact was between 0.4 and 1 kN, medium impact was between 1 and 2 kN, and high impact was between 2 and 3 kN. The accelerations induced by the low, medium and high impacts at block 2 are shown in Figure 7.5 for the column without ropes. These accelerations are normalised by the impulse (area under the impact force – time curve) to enable the comparison of the results due to different magnitudes of the impact force. These normalised accelerations are shown in Figure 7.6 for the column without ropes. Similarly, tests were repeated by applying the impact force \( (P_t) \) at the top block (i.e. block 13). The accelerations induced by the low, medium and high impacts at block 13 are shown in Figure 7.7 for the column without ropes, and their normalised accelerations are shown in Figure 7.8. It is observed from Figures 7.5 and 7.7 that the induced accelerations \( \ddot{u}_2 \) and \( \ddot{u}_t \) increased with an increase in the impact force when applied at blocks 2 and 13, respectively. The induced accelerations decreased along the height of the column, away from the location of the impact. When the impact was applied at the block 2, the accelerations recorded at block 13 were on average 86.4\% less compared to the accelerations at the impact location. On the other hand, when the impact was applied at the block 13, the accelerations recorded at block 2 were on average 91.1\% less compared to the accelerations at the impact location. It is also observed from Figures 7.7 and 7.8 that the peak normalised accelerations, at the locations where impact force was applied, are approximately the same, which implies that the accelerations can be correlated with the impact force. A correlation between induced accelerations and the impact forces at the bottom block (or the top block) is shown in Figure 7.9 for both structures. Impact force
was also applied at the mid height (i.e. at block 7) to evaluate the influence of the impact location on the induced accelerations. The accelerations were normalised in a similar manner. It is observed that the induced accelerations reduced along the column height either going upward or downward (Figure 7.8). When the impact was applied at the block 7 (Figure 7.10), the accelerations recorded at blocks 2 and 13 were on average 51.2% and 64.3%, respectively, smaller than the accelerations at the impact location.

**Figure 7.4:** Experimental setup – impact test for the column without ropes; (a) impact force at the bottom block, and (b) impact force at the top block.
Figure 7.5: Accelerations induced along the column height by the impact force at the bottom of the column without ropes.
7. Dynamic response of mortar-free structures

Figure 7.6: Accelerations along the column height normalised by the corresponding impulse due to an impact applied at the bottom of the column without ropes.
7. Dynamic response of mortar-free structures

![Diagram](image)

**Figure 7.7**: Accelerations induced along column height by the impact force at the top of the column without ropes.
7. Dynamic response of mortar-free structures

**Figure 7.8**: Accelerations along the column height normalised by the corresponding impulse due to an impact applied at the top of the column without ropes.
Figure 7.9: Correlation between induced accelerations and impact force for the considered column structures.
7. Dynamic response of mortar-free structures

![Graphs of induced and normalised accelerations along the column height when the impact force is applied at the mid-height of the column without ropes.](image)

**Figure 7.10:** Induced and normalised accelerations along the column height when the impact force is applied at the mid-height of the column without ropes.
7. Dynamic response of mortar-free structures

7.4.4. Shake table tests

The experimental setup for the mortar-free column with ropes mounted on the shake table along with the details of the instrumentation for measuring its response is shown in Figure 7.11. A mass of 150 kg was put at the top of the column. The column was instrumented with the following:

(i) six accelerometers along the column height for determining induced accelerations
   (at shake table, at foundation, one each at block 2, 5, 9 and 13 of the column)
(ii) ten portal strain gauges for quantifying the block relative movement, i.e. uplift
   (five on left and five on right sides)
(iii) three wire displacement transducers (one at the column top, one on the foundation
     and one on the shake table)
(iv) two load cells at column top for measuring the rope tension.

The same setup was used for the column without the rope reinforcement, with the exception that load cells mentioned in (iv) were not used. Loose ropes were used to ensure safety around the shake table during testing.

Figure 7.11: Mortar-free column with the rope reinforcement; schematic diagram showing instrumentation (left), and the actual setup (right).

The loadings employed with the shake table on the considered structures were in the following sequence:

a. Harmonic loading with a constant small amplitude and varying frequencies passing through the natural frequency. Three levels of amplitudes (i.e. 5, 10 and 15 mm) were applied. Each of these amplitudes was applied with exciting frequencies
ranged from 1 Hz to 2 Hz with an increment of 0.25 Hz. The corresponding peak table accelerations ranged from 0.02 g (for 5 mm and 1 Hz) to 0.24 g (for 15 mm and 2 Hz). Even though natural frequencies obtained from push over and snap back tests were higher, the structures had relatively lower natural frequencies in harmonic tests.
b. Earthquake loadings (Tabas 1978, Llolleo 1985, El Centro 1940, and Kobe 1995). These were applied from a PGA of 0.05 to 0.20 g. with an increment of 0.05 g
c. Harmonic loading with constant frequency of 1.5 Hz and varying amplitudes of 20, 30, 40 mm and so on until column failure. The selection of 1.5 Hz was based on the experience of test (a) (i.e. harmonic loading with constant amplitude and varying frequencies) as the response of structures at 1.5 Hz was believed to be near resonant frequency.

The shake table motion is taken as the ground motion in this thesis. It is important to note that the structures were tested under the repeated loading, any damage occurred at any stage was cumulative. The readings of wire displacement transducers at foundation and shake table confirmed that the shake table was capable of simulating the load accurately.

7.4.4.1. Column response under harmonic loading with constant amplitude and varying frequencies

a. Natural frequency and damping ratio

The half bandwidth method is used to determine the damping ratios and natural frequencies in spite of the fact that the considered column structures behave nonlinearly. The applied ground motion $u_g$ of 10 mm and the corresponding top displacement $u_t$ time histories of the column without ropes are shown in Figure 7.12 for different exciting frequencies. The column response is divided into three phases: A. when the column started its vibration until it attained the steady state, B. steady state response of the column, and C. free vibrations of the column. The duration of phase A varied considerably for different amplitudes and frequencies. The steady state response is used for the calculation of transmissibility using displacement and acceleration readings. The transmissibility using displacement readings vs exciting frequency ($TR_d$ vs $f$) curve is also shown in Figure 7.12. The damping ratio and natural frequency come out to be 17.9% and 1.48 Hz, respectively.
7. Dynamic response of mortar-free structures

Figure 7.12: Ground and top displacement – time history under 10 mm harmonic loading with varying frequencies and TR_d vs f curve for the column without ropes.

The ground acceleration $u_g$ and the top acceleration $u_t$ time histories of the column without ropes are shown in Figure 7.13 for different exciting frequencies. The calculated transmissibility using acceleration readings vs exciting frequency ($TR_d$ vs $f$)
curve is also shown. The damping ratio and natural frequency come out to be 18.3% and 1.49 Hz, respectively.

\[ f_n = 1.49 \text{ Hz} \]
\[ \xi = 18.3\% \]

**Figure 7.13:** Ground and top acceleration – time history under 10 mm harmonic loading with varying frequencies and TR\textsubscript{a} vs f curve for the column without ropes.
The column with ropes has, as expected, a higher average natural frequency (1.57 Hz) compared to the column without ropes (1.47 Hz). The comparison of \( TR_d \) vs \( f/f_n \) and \( TR_a \) vs \( f/f_n \) curves for all amplitudes of harmonic loading is shown in Figure 7.14 for both structures. It is noted that the curves become relatively flatter for high amplitude loading showing more damping for the structures. The damping ratios calculated using displacement and acceleration readings are close to each other (Figure 7.15). As expected, the column without ropes has more damping compared to the column with ropes (Figure 7.16).

A comparison of natural frequencies determined from different tests is shown in Figure 7.17. There is a considerable difference in the natural frequencies of the same structure. The higher frequencies were observed in push over and snap back tests compared to the harmonic tests. The reason could be the activation of the whole column mass in the harmonic tests. Natural frequencies obtained from the harmonic tests are more realistic.

**Figure 7.14**: Comparison of \( TR_d \) vs \( f/f_n \) and \( TR_a \) vs \( f/f_n \) curves for the considered column structures.
7. Dynamic response of mortar-free structures

**Figure 7.15:** Comparison of the damping ratio calculated using TR$_d$ and TR$_a$ for the (a) column without ropes and (b) column with ropes.

**Figure 7.16:** Comparison of the damping ratio for the considered column structures using (a) TR$_d$ and (b) TR$_a$.

**Figure 7.17:** Comparison of natural frequencies determined from different tests.
b. Base shear – displacement curves, energy dissipation and bending moment

Base shear ‘Q’ is calculated as $\Sigma (m_i \ddot{u}_i)$, where $\ddot{u}_i$ are the accelerations at the considered locations and $m_i$ are the contributed masses corresponding to $\ddot{u}_i$. The column mass is divided into the contributed masses in such a way that, along the column height, the centre of each mass is close to $\ddot{u}_i$. The purpose of this approximate calculation of base shear is to obtain the base shear - displacement relationship. Displacement ‘u’ is top relative displacement which is taken as the difference between the top displacement and the ground displacement. The base shear – displacement (Q-u) curves obtained from different frequencies under 10 mm harmonic loading are shown in Figure 7.18. The magnitudes of the base shear and the displacement are larger for the column without ropes compared to the column with ropes. These curves are further divided for column responses in the periods A, B and C as shown in Figure 7.19 to evaluate the behaviour during steady-state response. The area within one loop of a Q-u curve of the steady state response is taken as the energy dissipated. Energy dissipation increased up to a frequency near the resonant frequency and then it decreased (Figure 7.20). It also increased with a larger input amplitude. The column without ropes had more energy dissipation compared to the column with ropes.

![Figure 7.18: Q-u curves obtained from different exciting frequencies of 10 mm harmonic loading for the (a) column without ropes and (b) column with ropes.](image)

Base shear estimated from Figure 7.9 is 50% larger than that determined from the above procedure for the column with ropes. It is 65% larger in the case of the column without ropes.
ropes. The developed correlation is conservative and should be used only for rough estimation.

Figure 7.19: Q-u curves for responses A, B and C under 10 mm harmonic loading for the column without ropes.
The maximum bending moment of the column is calculated by multiplying the maximum base shear with the height from the top of the foundation to the centre of the total mass taken in calculation of the base shear. The comparison of bending moment is shown in Figure 7.21. The maximum bending moment usually first increased up to a frequency near to the resonant frequency and then decreased with an increase in exciting frequencies. It also increased with larger input amplitude. The increment is more non-linear at strong excitations compared to the low excitations. The column without ropes had a higher bending moment compared to the column with ropes because of larger top relative displacement in the former structure.
c. Block uplift and rope tension

The mortar-free column allows the vertical relative movement at the block interface during the applied harmonic loadings. The block uplifts along the column height during one cycle is displayed in Figure 7.22. When the column deflected towards one side (say left side, as shown in the figure), uplifts are observed at the right side and the top relative-displacement is on the left side. Regarding the rope stresses, considerably more tension is observed in the right rope compared to the left rope. It may be noted that for a higher load, there will be some tension in the left ropes because of significant uplift at the right side. All these observations are reversed when the column is deflected on the other side (i.e. the right side). The magnitude of the uplift along the column height varies (i.e. decreases), with the maximum uplift at the column base.

![Diagram of block uplift and rope tension](image)

**Figure 7.22:** Observations during deflections under harmonic and earthquake loadings.
(Note: Foundation and top-mass are not shown for clarity)

The uplifts between blocks 1 and 2 due to 1.5 Hz and 10 mm harmonic loading are shown in Figure 7.23 for both columns. The left and right uplifts occurred alternately. It is noted for the column without ropes that the left side has shorter uplift period compared to the right one. This could be because the interlocking keys are not perfectly identical, causing unsymmetrical uplift periods. As expected, the column without ropes had more uplift compared to the column with ropes. It is observed that the number of uplifts increased
with higher frequency. The left and right uplifts between blocks 1 and 2 during steady state at different frequencies under harmonic loading of 10 mm amplitude are shown in Figure 7.24 for the considered column structures. The magnitude of uplifts increased up to a frequency near to the resonant frequency and then it decreased.

**Figure 7.23**: Uplift-time history due to 1.5 Hz and 10 mm harmonic loading for the (a) column without ropes and (b) column with ropes.
As expected, the time history of the rope tension corresponds to the development of uplifts. The rope tension time history and the relationship between the rope tension and relative displacement are presented in Figure 7.25. The tension in the left and right ropes occurred alternately. The maximum tension of left and right ropes during steady-state response to the harmonic loading is shown in Figure 7.26. The magnitude of rope tension increased up to a frequency near to the resonant frequency and then it decreased. The maximum value of rope tension was far less than the pull out load determined in Chapter 5. A force of 1625 N is required to pull out ropes from a 100 mm deep foundation. This indicates that the ropes would not be pulled out during the considered harmonic loadings if the ropes were embedded in the foundation. The rope of 36 mm diameter is capable of taking a tensile load of 14 kN. The tension developed in the rope was also less than the rope tensile load, ensuring no possibility of the rope breaking.
7. Dynamic response of mortar-free structures

Figure 7.25: Rope tension due to 1.5 Hz and 10 mm harmonic loading; (a) rope tension – time history, and (b) rope tension – relative displacement relationship.

Figure 7.26: Maximum rope tension during steady-state response to the harmonic loadings for the (a) left and (b) right rope.
Figure 7.27: Considered earthquake loadings with PGA = 0.2 g and their corresponding response spectrum.
7. Dynamic response of mortar-free structures

7.4.4.2. Column response under earthquake loading

The considered earthquake loadings with a scaled PGA of 0.2 g and the corresponding response spectrum (with $\xi = 5\%$) are shown in Figure 7.27. The natural frequencies of both columns are marked in the enlarged view of the response spectrum to predict the behaviour of the considered structures. It is noted that the response spectrum is for the linear structures. For a linear structure with a natural frequency of 1.47 Hz, the response accelerations due to Tabas and Llolleo earthquake loadings would be close (i.e. 0.27 g and 0.28 g, respectively), and the response acceleration (0.51 g) due to Kobe earthquake loading would be larger than that (0.35 g) due to El Centro earthquake loading. Whereas, for a structure with a natural frequency of 1.57 Hz, the response acceleration due to El Centro and Kobe earthquake loadings would be close (i.e. 0.40 g and 0.41 g, respectively), and the response acceleration (0.36 g) due to Llolleo earthquake loading would be larger than that (0.24 g) due to Tabas earthquake loading. But the response of the considered mortar-free columns was expected to be nonlinear. It could be predicted that the Kobe earthquake loading would be critical, followed by the EL Centro, Llolleo and Tabas earthquake loadings for the considered structures and that the increment in the considered response parameters (i.e. induced acceleration, base shear, bending moment, block uplift and rope tension) would also be nonlinear. This is being observed in the considered parameters explained in the following sub-sections:

a. Induced accelerations

Induced vibrations were recorded using accelerometers attached along the height of the columns. The accelerations recorded at the blocks 2, 5, 9 and the column top (notated as $\ddot{u}_2$, $\ddot{u}_5$, $\ddot{u}_9$ and $\ddot{u}_t$, respectively) under 0.2 g El Centro 1940 loading are shown in Figure 7.28. It can be observed that the induced top acceleration was higher compared to the peak ground acceleration. The increase in peak response accelerations at any considered location, due to the applied incremental peak ground accelerations, was nonlinear for both columns with and without ropes. The reason for this could be the nonlinear activation of the inertial force within the columns due to the applied ground motions. The response spectrums for $\ddot{u}_2$, $\ddot{u}_5$, $\ddot{u}_9$ and $\ddot{u}_t$ are shown in Figure 7.28 for both columns. It can be observed that the response accelerations for the considered locations increased along the column height for frequencies up to 1 Hz and the behaviour is different for higher frequencies.
7. Dynamic response of mortar-free structures

(a) Column without ropes

(b) Column with ropes

Acceleration (g)

Time (sec)

\( \ddot{u}_t \)

\( \ddot{u}_9 \)

\( \ddot{u}_5 \)

\( \ddot{u}_2 \)
7. Dynamic response of mortar-free structures

**Figure 7.28:** Induced accelerations due to 0.2 g El Centro 1940 earthquake loadings for the considered columns; (a) & (b) time histories, and (c) & (d) their corresponding response spectrum.

**b. Base shear – displacement curves and maximum bending moment**

Base shear - displacement curves and the maximum bending moments are obtained in the same manner as for the harmonic loading. The base shear – displacement curves are shown in Figure 7.29. The column without ropes has relatively wider hysteretic loops compared to the column with ropes. This shows more energy dissipation for the former structure. The column without ropes experienced larger lateral displacement in comparison to the column with ropes. As stated earlier, the base shear estimated from Figure 7.9 is 50% more than that determined from the above procedure for the column with ropes. It is 65% more in the case of the column without ropes. The developed correlation is conservative and should be used only for rough approximation.

The maximum bending moment for both structures due to the considered earthquake loading is shown in Figure 7.30. The maximum bending moment is not proportional to any applied incremental earthquake loading. Kobe earthquake loading generated the highest bending moment followed by El Centro, Llolleo and Tabas earthquake loadings.
7. Dynamic response of mortar-free structures

<table>
<thead>
<tr>
<th>Earthquake Loading</th>
<th>Column without ropes</th>
<th>Column with ropes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tabas</td>
<td><a href="image">Graph</a></td>
<td><a href="image">Graph</a></td>
</tr>
<tr>
<td>Llolleo</td>
<td><a href="image">Graph</a></td>
<td><a href="image">Graph</a></td>
</tr>
<tr>
<td>El Centro</td>
<td><a href="image">Graph</a></td>
<td><a href="image">Graph</a></td>
</tr>
<tr>
<td>Kobe</td>
<td><a href="image">Graph</a></td>
<td><a href="image">Graph</a></td>
</tr>
</tbody>
</table>

**Figure 7.29:** Base shear - displacement curves obtained from the earthquake loadings for the considered column structures.
c. Block uplift and rope tension

The maximum block uplift between blocks 1 and 2 due to 0.2 g El Centro 1940 earthquake loading is shown in Figure 7.31. It can be seen that the left and right uplifts occurred alternately. As expected, more uplifts were observed in the column without ropes than in the column with ropes. The maximum block uplift for both structures is shown in Figure 7.32. The maximum block uplift is not proportional to any applied incremental earthquake loading. The reason for this could be the non-linear activation of inertial force within the column. Kobe earthquake loading generated the largest uplift, followed by El Centro, Llolleo and Tabas earthquake loadings.

The rope tension due to 0.2 g El Centro 1940 earthquake loading is shown in Figure 7.33 for the column with ropes. As expected, the left and right ropes were activated alternately. It can be observed that there was some magnitude of tension in one rope when the other was in full tension. This observation is already explained in Figure 7.22 (see stages b and d). The maximum rope tension due to the considered earthquake loadings is shown in Figure 7.34 for the column with ropes. The maximum rope tension is also not proportional to any applied incremental earthquake loading. Kobe earthquake loading generated most tension, followed by El Centro, Llolleo and Tabas earthquake loadings.
7. Dynamic response of mortar-free structures

**Figure 7.31**: Uplift between blocks 1 and 2 due to 0.2 g El Centro 1940 earthquake loading for the (a) column without ropes and (b) column with ropes.

**Figure 7.32**: Maximum uplifts between blocks 1 and 2 due to considered earthquake loadings for the (a) column without ropes and (b) column with ropes.

**Figure 7.33**: Rope tension due to 0.2 g El Centro 1940 earthquake loading for the column with ropes.
7. Dynamic response of mortar-free structures

7.4.4.3. Column response under harmonic loading with constant frequency and varying amplitudes

As mentioned earlier, the structures were tested under repeated loading, and any damage occurring at any stage was cumulative. No damage was apparently observed until all earthquake loadings had been applied. Finally, harmonic loadings with a constant frequency of 1.5 Hz and varying amplitudes (20, 30, 40, 50 and 60 mm) were employed to introduce damage in the structure. As expected, the block uplifts and top relative displacements increased with increased input load. The columns without and with ropes failed during harmonic loading of 50 mm and 60 mm, respectively. These ground displacements at 1.5 Hz caused peak table accelerations of 0.45 g and 0.54 g, respectively. Both columns failed in bending in the out-of-plane direction because one of the bottom interlocking keys cracked, causing instability in the columns. The response recorded at the column top in terms of displacement–time history and acceleration–time history, during which the column failed, are shown in Figure 7.35 for the column without ropes. The column failure and damaged interlocking key of the block are shown in Figure 7.36. From the discussion in the last chapter, it was predicted that the main cause of column failure would be damage in one of the bottom interlocking keys of the block. It is already recommended that careful compaction in the bottom interlocking keys can result

Figure 7.34: Maximum rope tension due to considered earthquake loadings for the column with ropes.
in higher capacity/strength of the block. The mechanically prepared blocks may also avoid such damage.

![Displacement and Acceleration Time History](image)

**Figure 7.35:** Column response at failure; (a) top displacement - time history, and (b) top acceleration - time history.

![Column Failure Images](image)

**Figure 7.36:** Typical column failure; (a) isometric view, (b) side view, and (c) damage of the bottom interlocking key.
7. Dynamic response of mortar-free structures

7.5. Testing and analysis of mortar-free interlocking walls

Two mortar-free walls (i.e. one without ropes and one with ropes) were considered. The walls consisted of bottom, standard and half interlocking blocks with an overall height of 2.07 m (Figure 7.37). A post-tension force of 200 N was applied in the ropes for the wall with vertical rope reinforcement. No mass was put at the top of the wall because of the limitation of the shake table payload. With the top mass, the results would likely be different. The testing of walls with a top mass or even with a diaphragm is highly recommended before the implementation of this new construction technology. The current investigation was a pilot study to explore the proposed technology. Different tests (i.e. push over, snap back, impact and shake table tests) were performed.

Figure 7.37: Mortar-free wall structures; (a) wall without ropes, and (b) wall with ropes.

7.5.1. Push over tests

This test was performed in the same way as for the columns to estimate the initial stiffness of the considered walls so that their natural frequencies could be determined. A horizontal in-plane force was applied at the top of the wall from one side (i.e. left side of the wall in Figure 7.37a) and the lateral displacement was measured from the other side. The force-displacement relationships for both walls are shown in Figure 7.38. It can be seen that the force-displacement relationship is not a smooth curve because of the mortar-
7. Dynamic response of mortar-free structures

Free construction. A polynomial curve (shown as dotted line in Figure 7.37) is first fitted and then the initial slope (shown as dashed line) is taken as the initial stiffness. The stiffness values are 169390 N/m and 311230 N/m for the walls without and with ropes, respectively. Using these stiffness values and the mass of top eight block layers, their natural frequencies come out to be 3.73 Hz and 5.06 Hz, respectively. The tests were repeated three times, and the average values are shown in Table 7.3. As expected, the wall with ropes had a higher stiffness and natural frequency than the wall without ropes.

![Force-displacement curves](image)

**Figure 7.38:** Force–displacement curves for the (a) wall without ropes and (b) wall with ropes.

### 7.5.2. Snap back tests

These tests were performed in the same manner as for the columns to get an approximation of the damping ratios and the natural frequencies of the mortar-free wall structures. An accelerometer was attached at the top of the wall. The wall top was pulled by 50 mm and suddenly released to allow free vibrations. The recorded acceleration-time histories for the considered walls are shown in Figure 7.39. The logarithmic decrement method is used to calculate the damping ratios. These are 9.6% and 5.5% for walls without and with ropes, respectively. Natural frequencies are calculated from the period of the acceleration-time histories, which come out to be 3.89 Hz and 4.62 Hz, respectively. The tests were repeated three times and the average values are shown in Table 7.3. As expected, the wall with ropes has a higher frequency than the wall without ropes.
7. Dynamic response of mortar-free structures

ropes. The damping of the latter structure is 70% higher to the former structure. The reason could be larger relative movement between the blocks in the wall without ropes.

![Figure 7.39: Acceleration-time histories for the (a) wall without ropes and (b) wall with ropes.](image)

Table 7.3: Average values of $k$, $f_n$ and $\xi$ from push over and snap back tests for the considered walls

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Push over</th>
<th>Snap back</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Wall without ropes</td>
<td>Wall with ropes</td>
</tr>
<tr>
<td>Stiffness $k$ (N/m)</td>
<td>198550</td>
<td>310977</td>
</tr>
<tr>
<td>Natural frequency $f_n$ (Hz)</td>
<td>4.03</td>
<td>5.05</td>
</tr>
<tr>
<td>Damping ratio $\xi$ (%)</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

7.5.3. Impact tests

These tests were performed to correlate the induced accelerations with the impact force, which may approximately be taken as the base shear. The experimental setup for the wall with ropes is shown in Figure 7.40, and the same setup was used for the wall without ropes. Four accelerometers were attached along the height of the wall. The accelerations recorded at the second, fifth, ninth and top layers of the wall are notated as $\ddot{u}_2$, $\ddot{u}_5$, $\ddot{u}_9$ and $\ddot{u}_t$, respectively. An impact force ($P_2$) was applied with the help of a calibrated impact hammer at the bottom of the wall, as shown in Figure 7.40a. The impact force and the induced accelerations were recorded. Three magnitudes of the impact force, named as
low, medium and high, were considered. Each magnitude was applied seven times to get a robust idea of the induced accelerations. Since the impact force was applied manually, the magnitude of each level was not a constant value but a small range; low impact was ranged between 0.3 and 0.7 kN, medium impact was ranged between 1 and 1.7 kN, and high impact was ranged between 2 and 3 kN. The accelerations induced by low, medium and high impact at the bottom are shown in Figure 7.41 for the wall with ropes. These accelerations are normalised by the impulse (area under the impact force – time curve) to enable a comparison of the results. The normalised accelerations are shown in Figure 7.42 for the wall with ropes. Similarly, tests were repeated by applying the impact force ($P_t$) at the top of the wall (Figure 7.40(b)). The accelerations induced by the low, medium and high impacts at the wall top are shown in Figure 7.43 for the wall with ropes, and their normalised accelerations are shown in Figure 7.44. As expected and as shown in Figures 7.41 and 7.43, the induced accelerations $\ddot{u}_2$ and $\ddot{u}_t$ increased with the magnitude of the impact force when applied at the bottom and top of the walls, respectively. The induced accelerations decreased along the height of the wall, away from the location of the impact. When the impact was applied at the block layer 2, the accelerations recorded at block layer 14 were on average 81.2% less compared to the accelerations at the impact location. On the other hand, when the impact was applied at the block layer 14, the accelerations recorded at block layer 2 were on average 82.3% less compared to the accelerations at the impact location. It is also observed from Figures 7.42 and 7.44 that the peak normalized accelerations are approximately the same, which implies that the accelerations can be correlated with the impact force. A correlation between induced accelerations and the impact forces at the wall bottom and the wall top is shown in Figure 7.45 for both walls. It is found that the impact force induced high accelerations at the wall bottom compared to those at the wall top. The induced accelerations were higher in the wall without ropes compared to the wall with ropes.
**Figure 7.40:** Experimental setup of the impact test for the wall with ropes; (a) impact force at the wall bottom, and (b) impact force at the wall top.
Figure 7.41: Accelerations induced along the wall height by the impact force at the bottom of the wall with ropes.
7. Dynamic response of mortar-free structures

**Figure 7.42:** Accelerations along the wall height normalised by the impulse resulting from an impact applied at the bottom of the wall with ropes.
Figure 7.43: Accelerations induced along the wall height by the impact force at the top of the wall with ropes.
Figure 7.44: Accelerations along the wall height normalised by the impulse due to an impact applied at the top of the wall with ropes.
Impact force was also applied at the mid height to see its influence on the induced accelerations. The accelerations were normalized in a similar manner to the bottom and top hits. It was observed that the induced accelerations reduced along the wall height either going upward or downward (Figure 7.46). When the impact was applied at the block layer 7, the accelerations recorded at the block layers 2 and 14 were on average 30.9% and 25.1%, respectively, smaller than the accelerations at the block layers 5 and 9.

**Figure 7.45**: Correlation between induced accelerations and the impact force for the considered walls.
7. Dynamic response of mortar-free structures

Impact force at mid height

Figure 7.46: Induced and normalised accelerations along the wall height when the impact force is applied at the mid-height of the wall with ropes.
7. Dynamic response of mortar-free structures

7.5.4. Shake table tests

The experimental setup for the mortar-free wall with ropes mounted on the shake table, along with the details of the attached equipment for measuring its response, is shown in Figure 7.47. As already mentioned, no mass was put at the top of the wall because of the limitation of the shake table. The tests were conducted to study the behaviour of the mortar-free walls under the dynamic loading. The wall was instrumented with the following:

(i) six accelerometers for determining induced accelerations at the shake table, at the foundation, one each at layers 2, 5, 9 and the top of the wall. The recorded accelerations are notated as \( \ddot{u}_s \), \( \ddot{u}_g \), \( \ddot{u}_2 \), \( \ddot{u}_5 \), \( \ddot{u}_9 \) and \( \ddot{u}_t \), respectively.

(ii) ten portal strain gauges for quantifying block relative movement (five on left and five on right sides).

(iii) three wire displacement transducers (one each at the wall top, on the foundation and the shake table).

(iv) two load cells at the wall top with the exterior ropes for measuring the tension.

The same setup was used for the wall without ropes with the exception that the load cells mentioned at (iv) were not used. Loose ropes were used to ensure safety around the shake table during the testing.

Figure 7.47: Mortar-free wall with the rope reinforcement; schematic diagram showing instrumentation (left), and the setup (right).
The loadings employed with the shake table on the considered walls were:

a. Harmonic loading with constant small amplitude and varying frequencies passing through the fundamental frequency. From the experience of column testing, it was anticipated that the fundamental frequencies of the wall structures would be less compared to that determined from the push over and snap back tests. Three levels of constant small amplitudes (5, 10 and 15 mm) were applied. Each of these amplitudes was applied with exciting frequencies from 1 Hz to 3 Hz with an increment of 0.5 Hz. These harmonic loadings had peak ground accelerations ranging from 0.02g to 0.54g.

b. Earthquake loadings (1978 Tabas earthquake, 1985 Llolleo earthquake, 1940 El Centro earthquake and 1995 Kobe earthquake). These were applied from a PGA of 0.05 to 0.30g with an increment of 0.05g.

c. Harmonic loading with constant frequency of 2 Hz and varying amplitudes of 20, 30, 40 mm and so on until wall failure. The selection of 2 Hz was based on the experience of test (a) (i.e. harmonic loading with constant amplitude and varying frequencies) as the response of the structures at 2 Hz was near resonant frequency.

It is important to note that the structures were tested under the repeated loading, and any damage occurred was cumulative.

7.5.4.1. Wall response under harmonic loading with constant amplitudes and varying frequencies

a. Natural frequencies and damping ratios

It has been mentioned that the half bandwidth method is used to determine the damping ratios and natural frequencies in spite of the fact that the considered walls behave nonlinearly. The applied amplitude of ground displacement $u_g$ (i.e. 10 mm) and the corresponding top displacement $u_t$ of the wall with ropes are shown in Figure 7.48 for different exciting frequencies. Similar to the column response, the wall response is divided into three phases: A. when the wall experienced its transient response until it attained steady-state response; B. steady-state response of the wall; and C. free vibrations of the wall. The duration of phase A varied considerably for different amplitudes and frequencies. The steady-state response of walls is used for the calculation of transmissibility using displacement and acceleration readings. The
7. Dynamic response of mortar-free structures

transmissibility using displacement readings vs exciting frequency (TR_d vs f) curve is also shown in Figure 7.48. The damping ratio and natural frequency of the wall with ropes comes out to be 18.9% and 2.02 Hz, respectively.

Figure 7.48: Ground and top displacement – time history under 10 mm harmonic loading with varying frequencies and TR_d vs f curve for the wall with ropes.
The ground acceleration $\ddot{u}_g$ and the top acceleration $\ddot{u}_t$ of the wall with rope reinforcement are shown in Figure 7.49 for different exciting frequencies. The calculated transmissibility using acceleration readings vs exciting frequency (TR$_a$ vs f)
curve is also shown in the figure. The damping ratio and natural frequency comes out to be 19.3% and 2.07 Hz, respectively.

As expected, the wall without ropes has a lower average natural frequency (1.93 Hz) compared to the wall without ropes (2.05 Hz). The comparison of \( TR_d \) vs \( f/f_n \) and \( TR_a \) vs \( f/f_n \) curves for all amplitudes of harmonic loading is presented in Figure 7.50. The curves become smooth/flatter for high amplitude loading, indicating higher damping of the walls. The damping ratios calculated using \( TR_d \) and \( TR_a \) are close to each other, as shown in Figure 7.51. As expected, the wall without ropes has higher damping ratios compared to the wall with ropes (Figure 7.52).

A comparison of natural frequencies determined from different tests is shown in Figure 7.53. There is a considerable difference in the natural frequencies of the same wall structure. The higher frequencies were observed in push over and snap back tests compared to the harmonic tests. The reason could be the activation of the whole wall mass in the harmonic tests. Natural frequencies obtained from the harmonic tests are more realistic for the wall structures as well.

**Figure 7.50:** Comparison of \( TR_d \) vs \( f/f_n \) and \( TR_a \) vs \( f/f_n \) curves for considered walls.
7. Dynamic response of mortar-free structures

Figure 7.51: Comparison of damping ratios calculated using $R_d$ and TR for the a. wall without ropes and b. wall with ropes.

Figure 7.52: Comparison of damping ratios for the considered structures using (a) $TR_d$ and (b) $TR_a$.

Figure 7.53: Comparison of natural frequencies obtained from different results.
b. Base shear – displacement curves, energy dissipation and bending moment

Base shear ‘Q’ is calculated as $\Sigma (m_i \ddot{u}_i)$, where $\ddot{u}_i$ are the accelerations at the considered locations and $m_i$ are the contributed masses corresponding to $\ddot{u}_i$. The wall mass is divided into the contributed masses in such a way that, along the wall height, the centre of each mass is close to $\ddot{u}_i$. The purpose of this approximate calculation of base shear is to obtain the base shear - displacement relationship. Displacement ‘$u$’ is top relative displacement which is taken as the difference between the top displacement of wall and the ground displacement. The base shear – displacement (Q-$u$) curves obtained from different frequencies under 10 mm harmonic loading are shown in Figure 7.54 for the considered walls. It can be observed that the magnitudes of the base shear and the displacement are more for the wall without ropes compared to the wall with ropes. These curves are further divided for wall responses A, B and C as shown in Figure 7.55 to evaluate the wall behaviour during the steady-state response. The area within one loop of Q-$u$ curve of the steady-state response reflects the energy dissipated. Energy dissipation increased up to a frequency near to the resonant frequency and then it decreased (Figure 7.56). It also increased with an increase in the input amplitude. The wall without ropes had higher energy dissipation compared to the wall with ropes.

Figure 7.54: Q-$u$ curves obtained from different exciting frequencies of 10 mm harmonic loading for the (a) wall without ropes and (b) wall with ropes.
Figure 7.55: Q-u curves for responses A, B and C under 10 mm harmonic loading for the wall with ropes.
Figure 7.56: Energy dissipation during harmonic loadings of the considered walls.

The maximum bending moment of the wall is calculated by multiplying the maximum base shear with the height from the foundation top to the centre of the total mass taken in the calculation of base shear. The comparison of bending moment for the considered structures is shown in Figure 7.57. The maximum bending moment increased up to a frequency near to the resonant frequency and then it decreased. It also increased with a larger input amplitude. The wall without ropes had a higher bending moment compared to the wall with ropes.

Figure 7.57: Maximum bending moment during harmonic loadings for the considered walls.
c. Block uplift and rope tension

The mortar-free wall allows the vertical relative movement at the block interface (i.e. uplift) during the applied harmonic loadings. The uplifts between block layers 1 and 2 at different frequencies of the harmonic loading with an amplitude of 10 mm are shown in Figure 7.58 for the wall with ropes.

Figure 7.58: Uplift between block layers 1 and 2 at different frequencies under 10 mm harmonic loading for the wall with ropes.
It can be observed that the left and right uplifts occurred alternately. The magnitude of uplifts increased with increasing frequency up to the resonant frequency and then it decreased. The number of uplifts also increased with higher frequency. The left and right uplifts between block layers 1 and 2 during steady-state response at different frequencies of 10 mm harmonic loadings are shown in Figure 7.59. As expected, the wall without ropes had more uplift compared to the wall with ropes and both structures had maximum values at a frequency near to the resonant frequency.

![Figure 7.59: Maximum uplifts between block layers 1 and 2 recorded during steady-state response of different exciting frequencies of 10 mm harmonic loading for the (a) left and (b) right side.](image)

The rope tension recorded at different exciting frequencies under harmonic loading of 10 mm has somewhat similar patterns as shown for uplifts in Figure 7.58. It is noted that the left and right rope tension occurred alternately. The number of times the ropes underwent tension increased with increasing frequency. The rope tension vs displacement history recorded during harmonic loadings of 10 mm amplitude is shown in Figure 7.60. The magnitude of rope tension first increased with increasing frequency up to the resonant frequency and then decreased. The maximum tension of the left and right ropes during steady-state response due to the harmonic loadings is shown in Figure 7.61. The structure had maximum rope tension at a frequency near to the resonant frequency. The maximum value of 875 N is far less than the tensile load of the ropes and the pull out load determined in Chapter 5. As mentioned earlier, a force of 1625 N is required to pull out a rope from a 100 mm deep foundation. The rope of ~36 mm diameter is capable of taking a tensile load of 14 kN. This indicates that the ropes will neither break nor pull out during the considered harmonic loadings.
7. Dynamic response of mortar-free structures

**Figure 7.60**: Rope tension vs displacement history recorded during harmonic loading of 10 mm amplitude.

**Figure 7.61**: Maximum rope tension during steady-state response of the harmonic loadings; (a) left, and (b) right rope.

### 7.5.4.2. Wall response under earthquake loading

The locations of the natural frequencies of both walls are indicated by the vertical lines in the enlarged view of the response spectrum of the considered earthquake loadings in Figure 7.62. It may be noted that the response spectrum is for the linear structures. For an assumed linear structure with a natural frequency of 1.93 Hz, the response spectral values of 0.47 g and 0.48 g due to El Centro and Kobe earthquake loadings are almost the same,
indicating that both earthquakes have almost the same impact on the structure. The response acceleration (0.43 g) due to Llolleo earthquake loading would be larger than that (0.35 g) due to Tabas earthquake loading. Whereas, for an assumed linear structure with a natural frequency of 2.05 Hz, the response accelerations due to Llolleo and El Centro earthquake loadings would be close (i.e. 0.44 g and 0.45 g). The response accelerations due to Tabas and Kobe earthquake loadings would be 0.36 g and 0.55 g, respectively. However, the response of the considered mortar-free walls was expected to be nonlinear. It could be predicted that the Kobe earthquake loading would be critical, followed by the EL Centro, Llolleo and Tabas earthquake loadings for the considered structures and consequently with a stronger impact a nonlinear structural behaviour can be expected. This is being observed in the considered parameters (i.e. induced acceleration, base shear, bending moment, block uplift and rope tension) explained in the following sub-sections:

**Figure 7.62:** Prediction of response accelerations due to considered earthquake loadings (with PGA = 0.2 g) for the wall structures.

### a. Induced accelerations

Induced vibrations were recorded using accelerometers attached along the height of the walls. The accelerations recorded at the block layers 2, 5, 9 and the wall top (notated as $\ddot{u}_2$, $\ddot{u}_5$, $\ddot{u}_9$ and $\ddot{u}_t$, respectively) under 0.2 g El Centro 1940 loading are shown in Figure 7.63. It is observed that the induced top acceleration was higher compared to the peak ground acceleration. It is also noted that the peak response accelerations at any considered location due to the applied incremental peak ground motions were nonlinear for both
walls. The reason for this could be the non-linear activation of inertial force within the wall due to the applied ground motions. The induced accelerations in the wall without ropes were more than those in the wall with ropes. The response spectrum for $\ddot{u}_2$, $\ddot{u}_5$, $\ddot{u}_9$ and $\ddot{u}_t$ is also shown in Figure 7.63 for the considered walls. It can be observed that the response accelerations for $\ddot{u}_t$ are higher compared to $\ddot{u}_2$, $\ddot{u}_5$ and $\ddot{u}_9$, and for frequencies higher than 2 Hz, the difference is more.
7. Dynamic response of mortar-free structures

Figure 7.63: Accelerometer records under 0.2 g El Centro 1940 earthquake loading for the considered walls; (a) & (b) induced accelerations, and (c) & (d) their corresponding response spectrum.

b. Base shear – displacement curves and maximum bending moment

The cumulative block uplifts along the wall height caused the relative lateral displacement. Base shear $Q$ and displacement $u$ are calculated in a similar manner as that for harmonic loading. The base shear – displacement curves are shown in Figure 7.64 for the considered wall structures. It may be noted that, usually, the wall without ropes has relatively wider hysteretic loops than the wall with ropes. This shows more energy dissipation for the former structure. The wall without ropes had more lateral displacement and base shear in comparison to the wall with ropes. The reason for this could be the free cantilever wall with no mass at its top. The maximum bending moment is also calculated in a similar manner to that for harmonic loading. The maximum bending moment for both structures during considered earthquake loadings is shown in Figure 7.65. As expected, the maximum bending moment is not proportional to any applied incremental earthquake loading because the structures behaved nonlinearly. The Kobe earthquake loading generated the higher bending moment, followed by El Centro, Llolleo and Tabas loadings.
7. Dynamic response of mortar-free structures

![Graphs showing base shear - displacement curves for Tiams, Llolleo, El Centro, and Kobe walls with and without ropes.](image)

Figure 7.64: Base shear - displacement curves obtained from earthquake loadings of the walls.
7. Dynamic response of mortar-free structures

![Graph showing maximum bending moment vs. PGA for Kobe, El Centro, Llolleo, and Tabas earthquake loadings.](image)

**Figure 7.65:** Maximum bending moment due to the considered earthquake loadings for the (a) wall without ropes and (b) wall with ropes.

c. **Block uplift and rope tension**

The maximum uplift between block layers 1 and 2 due to 0.2 g El Centro 1940 earthquake loading is shown in Figure 7.66 for the considered walls. It can be seen that the left and right uplifts occurred alternately. As expected, more uplifts were observed in the wall without ropes compared to the wall with ropes. The maximum block uplift for both structures during considered earthquake loading is shown in Figure 7.67. The maximum block uplift is not proportional to any applied incremental earthquake loading. The reason for this could be the non-linear activation of inertial force within the wall due to the applied ground motions. The Kobe earthquake loading generated the largest uplift, followed by El Centro, Llolleo and Tabas earthquake loadings.

The rope tension due to 0.2 g El Centro 1940 earthquake loading is shown in Figure 7.68 for the wall with ropes. It can be observed that the left and right ropes were activated alternately. The maximum rope tension due to the considered earthquake loadings is shown in Figure 7.69. The maximum rope tension is also not proportional to any applied incremental earthquake loading. The Kobe earthquake loading generated higher tension, followed by El Centro, Llolleo and Tabas earthquake loadings.
7. Dynamic response of mortar-free structures

**Figure 7.66:** Uplift between blocks 1 and 2 due to 0.2 g El Centro 1940 earthquake loading for the (a) wall without and (b) wall with ropes.

**Figure 7.67:** Maximum uplift between block layers 1 and 2 due to the considered earthquake loadings for the (a) wall without ropes and (b) wall with ropes.
7. Dynamic response of mortar-free structures

7.5.4.3. Wall response due to the harmonic loading with constant frequency and varying amplitudes

As stated previously, the structures were tested under repeated loading, and any damage occurred at any stage was cumulative. There might be some internal degradation because of repeated dynamic loadings, but no damage was apparently observed until all the earthquake loadings had been applied. Finally, harmonic loadings with a constant frequency of 2 Hz and varying amplitudes were employed to introduce damage to the structure. As expected, the block uplifts and top relative displacements increased with increasing excitation amplitudes. The walls without and with ropes failed due to the harmonic loading of 60 mm and 70 mm, respectively. These ground displacements at 2 Hz caused peak table accelerations of 0.97 g and 1.13 g, respectively. Both walls failed in
bending in the out-of-plane direction because one of the bottom interlocking keys of the block in layer 2 cracked, creating unstable walls.

![Graph](image)

**Figure 7.70:** The response of the wall with rope at failure due to harmonic loading of 2 Hz and 70 mm amplitude; (a) top displacement - time history, and (b) top acceleration - time history.

![Images](image)

**Figure 7.71:** Typical wall failure; (a) isometric view, and (b) side view.
The response at wall top in terms of displacement–time history and acceleration–time history, during which the wall failed, are shown in Figure 7.70 for the wall without ropes. The wall failure is shown in Figure 7.71. A steel chain was put behind the wall for safety. From the experience of the column testing (section 7.4) and the block testing (Chapter 6), it was predicted that the main cause of wall failure would be damage in one of the bottom interlocking keys of the block.

### 7.6. Summary

A new approach to construct economical earthquake-resistant houses was initiated. As a pilot investigation, four mortar-free structures (two columns and two walls) were tested under different loadings to determine their dynamic characteristics and response. The considered structures consisted of innovative CFRC interlocking blocks. CFRC was prepared with 5 cm long coconut fibres and 1% content by mass of concrete materials. Columns and walls were tested without and with the rope reinforcement. The ropes were utilized as the vertical reinforcement to limit the lateral displacement of the considered structures. On the other hand, structures without ropes were expected to have higher damping due to greater relative movement of the blocks compared to the structures with ropes. There is a need to find a balance between the lateral displacement and damping so that the maximum benefit in terms of energy dissipation and avoiding structural collapse can be achieved. A mass of 150 kg was put at the top of the columns to simulate them as single degree of freedom structures, whereas no mass was put at the wall top because of the shake table pay load limitation. The testing of wall was performed to approximately assess the damping and their behaviour under dynamic loadings. The wall structures with a top mass would have different results (i.e. lower fundamental frequencies and lesser damping) compared to the structures without a top mass. Therefore, the presented results in this chapter can only be considered indicative, confirming that the proposed construction technology has the potential for earthquake prone regions.

Even though the considered structures were non-linear, linear methods were employed to estimate the damping and natural frequencies. The harmonic tests were more accurate in finding the natural frequencies of the considered structures compared to the push over and
7. Dynamic response of mortar-free structures

As expected, the natural frequencies and stiffnesses of the structures with ropes were higher than those of the structures without ropes.

The damping of the structures without ropes was higher than that of the structures with ropes because the relative movements of the blocks in the former structures were large compared to the latter structures.

The higher damping was observed for all structures due to harmonic loading of amplitude 15 mm compared to the low amplitude harmonic loadings.

The energy dissipation in the considered structures was high at a frequency near to the resonant frequency compared to that at other considered frequencies. The same trend was observed for the maximum bending moment, uplift and rope tension.

The structures with ropes had smaller relative top displacements and block uplifts compared to the structures without ropes.

The base shear of the structures with ropes was smaller than that of structures without ropes. This is due to smaller top relative displacements.

As expected, the impact induced accelerations were highest at the impact locations; these were decreased along the height of the structures away from the impact location. During the harmonic and earthquake loading, the induced accelerations at the top of the structures were generally higher than the ground accelerations, while smaller accelerations were observed at the other considered locations.

Induced accelerations were correlated with the impact force which may be used as the approximation of the base shear.

The maximum rope tension recorded in any shake table test was significantly smaller than the force required to pull the ropes out from a 100 mm deep foundation.

The maximum rope tension generated due to any dynamic loading was also less than the tensile load of the rope.

This work was the first step towards exploring the behaviour of the deemed technology. Full-scale walls having top mass or even with a diaphragm should be investigated for
beter understanding of their seismic performance. The loss in post-tension stress of ropes over a long period should also be studied.
Chapter 8.

Conclusions and recommendations for future research

8.1. Conclusions

The objective of this doctoral study is to provide a strong base for developing a construction technology for cheap and safe earthquake-resistant housing. The suitability of natural materials as reinforcement for concrete under static and dynamic loading is considered. Coconut fibres have been selected on the basis of their highest toughness amongst all natural fibres as reported in literature. Coconut fibre reinforced concrete (CFRC) was studied in detail. Ropes made of coconut fibres were also used as vertical rebar reinforcement. The dynamic properties of CFRC members were determined at different considered damage stages. The basic static properties of CFRC were also determined with standard procedures used for plain concrete. The influence parameters include fibre length and content. The investigations on bond strength between (i) fibres and concrete (ii) rope and CFRC were also studied. An innovative interlocking block was invented for earthquake-resistant houses. The compressive and shear capacities of these blocks were determined experimentally. The mortar-free construction using these blocks and rope reinforcement can lead to the economical earthquake-resistant houses. The testing of mortar-free structures (column and wallette) on shake table under real earthquake loading was also conducted. The brief conclusions from the performed work during the doctoral research are explained in the following sub-sections.

8.1.1. Use of coconut fibres in cement composites

The versatility and applications of coconut fibres in cement composites was discussed in Chapter 2. Coconut fibres are reported as the toughest and most energy absorbent material
8. Conclusions and recommendations for future research

of all fibres. Coconut fibres are cheaper than artificial fibres and are renewable. It is concluded that coconut fibres have the potential to be used in composites for different purposes. Various aspects of many coconut fibre reinforced composites have already been investigated: the resulting products are economical and better than the commercially available products as reported by many researchers. The studies of John et al. (2005) also showed the durability of coconut fibre reinforced composites with no encountered problems over a time of 12. However, the investigations of Li et al. (2006) and Toledo et al. (2000) showed concerns regarding the durability of coconut fibre reinforced composites when subjected to severe ageing conditions (described in section 2.5). The prevailing ageing conditions in marine areas may put limitations for the use of coconut fibre reinforced composites. Toledo et al. (2003) also recommended some solutions for increasing its durability. In civil engineering, coconut fibres have been used as reinforcement in cement composites, mainly for nonstructural components. There is a need for investigating the static and dynamic behaviours of coconut fibre reinforced concrete for its application as a structural material in beams, columns, shear walls and blocks.

8.1.2. Properties of CFRC

Experiments were performed to investigate the static and dynamic properties of CFRC. The mechanical properties investigated were the static modulus of elasticity $E_{static}$, compressive strength $\sigma$, compressive toughness $T_c$, splitting tensile strength $STS$, modulus of rupture $MOR$, total toughness index $TTI$ and density. These properties were also compared with those of plain concrete. The dynamic properties investigated were damping ratio $\xi$, fundamental frequency $f$, and dynamic modulus of elasticity $E_{dynamic}$ of CFRC beams. The considered fibre lengths were 2.5, 5 and 7.5 cm and the fibre contents were 1%, 2% and 3% for all fibre lengths, and 5% for 2.5 and 5 cm long fibres. Three specimens of CFRC were tested for each combination of fibres to obtain reliable average results.

The static investigation revealed:

- The properties can improve or worsen depending on fibre length and content, and CFRC strengths can be greater or smaller than that of plain concrete.
8. Conclusions and recommendations for future research

- The testing confirmed that the inclusion of coconut fibres in concrete can improve the flexural toughness of concrete considerably for all cases considered.
- The CFRC with 5 cm long fibres and 5% fibre content had an increased $\sigma$, $T_c$, $MOR$ and $TTI$ up to 4%, 21%, 2% and 910%, respectively, and decreased $E_{static}$, $STS$ and density up to 6%, 2% and 3%, respectively, when compared to that of plain concrete.

The dynamic tests revealed:

- Damping of CFRC beams had, as expected, a growing trend and the fundamental frequency had a declining trend with the increasing damage.
- Increasing fibre content resulted in a higher damping ratio and a lower fundamental frequency.
- CFRC beams with 5 cm long fibres had higher damping when compared to those with other fibre lengths.
- The static and dynamic modulii of elasticity generally decreased with an increase in fibre content or fibre length.
- Of the considered cases, CFRC with 5 cm long fibres and 5% fibre content had the best overall static and dynamic properties.
- Only 7% difference was observed in static and dynamic modulus of elasticity. This small difference indicated that non-destructive modal testing may be used to determine the modulus of elasticity of built members.

8.1.3. Bond strength between fibre and concrete

The effects of fibre embedment length, diameter, pre-treatment condition and mix design ratio on the bond strength between coconut fibres and concrete were investigated experimentally. The energy required for fibre pullout from concrete was also taken into account. Based on the conducted investigation, empirical equations are developed to determine the bond strength between coconut fibre and concrete and the energy required for fibre pullout. The properties of concrete (compressive strength, modulus of elasticity, compressive toughness, splitting tensile strength and modulus of rupture) and fibres (tensile load, stress and strain at break, modulus of elasticity and toughness) were determined using standard procedures. The simplified equations are developed for
estimating fibre tensile strength, elastic modulus and toughness. All concrete properties improved as expected by increasing the cement content. Single fibre pullout tests were performed to determine the bond strength and the energy required for pullout. The experiment on fibre tensile and fibre-concrete bond strength revealed that:

- thick (0.30 ~ 0.35 mm diameter) and boiled fibres had higher tensile strengths compared to thin (0.15 ~ 0.20 mm diameter), medium (0.20 ~ 0.30 mm diameter), soaked and chemically treated fibres.
- the bond strength increased with embedment length and had the highest value with a 30 mm embedment.
- the pullout energy increased with an increase in embedment length from 10 to 40 mm.
- all thin fibres in concrete with a 1:3:3 mix design ratio and medium fibres in concrete with a 1:2:2 mix design ratio were broken because the pullout load was higher for 20 mm embedment than the fibre tensile load.
- the thick fibres had bond strength and tensile strength of 0.37 and 82 MPa, respectively.
- the fibre tensile strength, fibre toughness and fibre-concrete bond strength increased by 34%, 55% and 184%, respectively, when fibres were boiled and washed.
- chemical pre-treatment caused a decrease of bond strength and tensile strength by 25% and 23%, respectively.

8.1.4. Bond strength between rope and CFRC

The effects of rope embedment length, diameter, pre-treatment condition, concrete mix design ratio, fibre content and knott on the bond strength between coconut-fibre rope and coir fibre reinforced concrete (CFRC) were experimentally investigated. The energy required to pull out rope from CFRC was also taken into account. Rope pullout tests were performed. Based on the conducted study, empirical equations were developed to determine the bond strength between rope and CFRC and the energy required for rope pullout. The higher pullout load (i.e. rope tension) generated due to earthquake loading may govern the foundation depth. The rope tension should be less than the rope tensile capacity and the pullout load for avoiding structure collapse. Thus, the tensile strength
and elongation of coconut fibre rope were also determined considering parameters like rope diameter and pre-treatment. All concrete properties improved, as expected, by increasing the cement content.

The experiments on rope tensile and rope-CFRC bond strength revealed that:

- thin and boiled ropes had higher tensile strength and elongation than that obtained with thick, soaked and chemically treated ropes. However, higher tensile load was required for thick ropes compared to thin ropes.
- the bond strength decreased by 11\% and 27\%, and pullout energy increased by 44\% and 28\% with an increase in embedment length from 100 to 200 mm and rope diameter from \(~18\) to \(~36\) mm, respectively.
- the bond strength increased by 41\% and 90\% and pullout energy enhanced by 35\% and 52\% with an increase in fibre from 0\% to 1\% and cement content in the matrix from 1:4:4 to 1:2:2, respectively.
- the bond strength and pullout energy reduced by 24\% and 25\%, respectively, with chemical treatment and increased by 15\% and 13\%, respectively, with boiled ropes compared to soaked ropes.
- the ropes broke during rope pullout when a knot was provided in the matrix.

8.1.5. Interlocking block and block capacities under monotonic loading

A novel interlocking block was presented. Different block shapes were developed in order to facilitate the wall construction. These blocks are named as standard, bottom, top and half blocks. Coconut fibre reinforced concrete (CFRC) was used in the manufacture of these blocks. The properties of CFRC were determined for different mix design ratios (i.e. cement:sand:aggregates) and fibre contents. The first case was 1:2:2, 1:3:3, and 1:4:4 with a constant fibre content of 0.5\% by mass of concrete materials, the second case was 1:3:3 with fibre contents of 0 and 1\%, and the third case was 1:4:2 with a fibre content of 1\%. The 20 MPa strength of coconut fibre reinforced concrete (CFRC) may be sufficient for single-storey earthquake-resistant houses. Higher strength (by using higher cement content) will increase the construction cost.
The compressive capacities of single (standard, bottom, top and half blocks) and three blocks (standard as well as combination of top, standard and bottom blocks) were determined experimentally. The in-plane and out-of-plane shear capacities of the blocks were investigated. An average of three readings was taken to represent a particular property. The shear toughness of the blocks is measured by the shear index, defined as the ratio of the total area under the load-displacement curve to the area under the curve up to maximum load. The following conclusions are drawn:

- CFRC with a mix design of 1:4:2 (cement:sand:aggregates), a W/C ratio of 0.64 and a fibre content of 1% by mass of concrete materials is recommended for manufacturing blocks, because of its better properties compared to those of CFRC with a mix design of 1:3:3 with 0, 0.5 or 1% fibre content.
- The compressive strength and total compressive toughness of the bottom block were higher than that of other blocks (standard, top and half blocks).
- The compressive strength of multiple standard blocks was only slightly less than that of the single standard block because of the slenderness effect, i.e. for standard blocks, \( f_{cm} = 0.96 f_{cs} \), where \( f_{cm} \) and \( f_{cs} \) stand for compressive strength of multiple and single blocks, respectively.
- Compressive toughness was the same i.e. 0.56 MPa for single and multiple standard blocks.
- The average of compressive peak load for multiple blocks with top, standard and bottom blocks was 84% larger than that for multiple standard blocks, but the compressive strength of the latter was 70% higher.
- The out-of-plane shear strength was 25% larger than the in-plane strength.
- More energy was required to cause an in-plane shear failure than the out-of-plane shear failure.
- The average in-plane shear index was 7% higher than the out-of-plane shear index.
- During manual casting, the compaction of CFRC in interlocking keys of the developed block should be done with great care to ensure uniform compaction.
8. Conclusions and recommendations for future research

8.1.6. Dynamic characteristics and response of mortar-free structure

A new approach to construct economical earthquake-resistant houses was initiated. As a pilot investigation, four mortar-free structures (two columns and two walls) were tested under different loadings to determine their dynamic characteristics and to understand their response. The considered structures consisted of innovative CFRC interlocking blocks. CFRC was prepared with 5 cm long coconut fibres and 1% content by mass of concrete materials. Columns and walls were tested without and with ropes reinforcement. The ~36 mm diameter rope reinforcement was utilized to limit the lateral displacement of the structure to some extent. On the other hand, structures without ropes were expected to have higher damping due to greater relative movement of the blocks compared to the structures with ropes. There is a need to find a balance between excessive lateral displacement and damping of such structures so that the maximum benefit in terms of reduction of earthquake impact can be achieved. A mass of 150 kg was placed at the top of the columns to simulate the mass of diaphragm, whereas no mass was put at the wall top because of the shake table limitation. It is important to mention that the wall structures with a top mass would have different results (i.e. higher fundamental frequencies and lesser damping) compared to the structures without a top mass. Therefore, the presented results in this chapter were just indicative, confirming that the proposed construction technology has the potential for earthquake prone regions.

Even though the considered structures were non-linear, linear methods were employed to estimate their damping and fundamental frequencies. The harmonic tests were more accurate in finding the fundamental frequencies of the considered structures as compared to the static and snap back tests. The following conclusions can be drawn from the investigation of the mortar-free column and wall structures:

- As expected, the natural frequencies and stiffness of the structures with ropes were higher than that of the structures without ropes.
- Damping of the structures without ropes was higher than that of the structures with ropes because of more relative movements of the blocks in the former structures.
- The energy dissipation in the considered structures was high at a frequency near to resonant frequency compared to that at other excitation frequencies considered.
The same trend was observed for the maximum bending moment, uplift and rope tension.

- Structures with ropes had smaller relative top displacements and block uplifts than that of structures without ropes.
- The base shear of structures with ropes was smaller than that of structures without ropes. This is due to smaller top relative displacements.
- As expected, the impact induced accelerations were highest at the impact locations; these were decreased along the height of the structures away from the impact location. During the harmonic and earthquake loading, the induced accelerations at the top of the structures were generally higher than the ground accelerations, while smaller accelerations were observed at the other considered locations.
- Induced accelerations were correlated with the impact force which may be taken as approximation of the base shear.
- The maximum rope tension recorded during any shake table test was significantly smaller than the force required to pull the ropes out from a 100 mm deep foundation.
- The maximum rope tension generated during the considered harmonic and earthquake loadings was also less than the tensile load of the rope.

**8.1.7. Conclusive remarks**

Construction of earthquake-resistant structures using local materials like coconut fibres and ropes is more economical than the construction of such houses with steel reinforcement. The fibre lengths, contents, ratios of cement, sand and aggregates have been optimized. The innovative interlocking blocks were invented for mortar-free construction, as these blocks can facilitate energy dissipation during ground excitations. Although the full-scale mortar-free structures with diaphragm need to be tested for better understanding of their behaviour before implementation, it is evidence from the results of this doctoral study that the concept of construction presented can surely help in the development of cheap and safe houses in seismic prone regions.
8. Conclusions and recommendations for future research

8.2. Recommendations for future research

Since research is a never ending process, there are always many horizons to explore at the end of a particular research step. The same is the case with this doctoral study. There are many things which can be done for improving the presented concept for earthquake-resistant houses. It can be broadly divided into two main streams: one relates to the materials and the other to the mortar-free construction. These are explained in the following sub-sections.

8.2.1. Future work for materials to be used

In this thesis, coconut fibre reinforced concrete (CFRC) was studied in detail for its static and dynamic properties. Ordinary Portland cement, sand, aggregates, water and imported brown coconut fibres were used for preparation of CFRC. The maximum size of aggregates was 12 mm (passing through 12 mm sieve and retained at 10 mm sieve). The mean diameter of coconut fibres was 0.25 mm. Alternative materials can also be explored for their potential. Following are the recommendations which can be looked at depending upon their availability to make CFRC more affordable:

- the use of fly ash, rice husk ash and palm oil fuel ash as a partial replacement of cement as a binding agent in CFRC. Sujivorakul et al. (2011) recommended up to 20% replacement without affecting the properties of glass fibre reinforced concrete.
- the use of coconut shell and / or palm kernel shell as a replacement of aggregates in CFRC. The previous studies of Olanipekun et al. (2006) and Gunasekaran et al. (2011) have shown the potential of natural shells in cementitious matrix.
- the use of pumice in CFRC is also one option. Hossain and Ahmed (2011) and Parhizkar et al. (2012) suggested lightweight concretes incorporating pumice for structural applications.
- the use of recycled concrete aggregates (RCA) as a replacement for normal aggregates in CFRC. There is a prospect for using RCA in concrete as reported by Lee et al. (2012) and Kwan et al. (2012).
- the combined use of soil, cement, fly ash, rice husk ash and / or palm oil fuel ash as a matrix.
• utilizing other natural fibres in concrete for structural purposes. Different pre-treatments of fibres can also help in achieving the optimum properties. There are many natural fibres which have been used in cement composites. These fibres include sisal, jute, hibiscus cannabinus, eucalyptus grandis pulp, malva, ramie bast, pineapple leaf, kenaf bast, sansevieria leaf, abaca leaf, vakka, date, bamboo, palm, banana, hemp, flax, cotton and sugarcane.
• mechanical processes should be employed for the preparation of fibres and composites for large-scale productions.
• the combination of different natural fibres in a matrix would be interesting.
• numerical investigations for predicting the behaviour of fibre reinforced composites.

8.2.2. Future work for mortar-free construction in seismic-resistant housing

In this thesis, mortar-free construction comprised of an invented block and rope reinforcement that was studied in detail. Mortar-free structures were tested under earthquake loading with the limitation of available resources (i.e. the current constraints of shake table). Following are the recommendations for the future:
• in-plane and out-of-plane testing of full-scale wall with diaphragm and top mass under harmonic and seismic loading
• in-plane and out-of-plane testing of required connections under earthquake loading
• testing of wall with openings
• full-scale testing of house under 3D seismic loading
• long-term durability of rope reinforcement
• loss in post-tension stress of rope reinforcement over a long period
• numerical modelling can help in better understanding the behaviour of mortar-free structures
• mechanical production of blocks
• preparing simple and easy-to-follow construction and design guidelines for mortar-free structures.
References


References


References


