Bond Behaviour of Lightweight Steel Fibre-Reinforced Concrete

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THESIS

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DECLARATION

I hereby declare that I have completed this research work without any improper help from any third part or using any aids other than those cited in this thesis. All the ideas derived directly or indirectly from any source are identified.

In the selection and use of the materials and in writing this thesis, I received support from the following persons:

Prof. Dr.-Ing. Klaus Holschemacher

Prof. Dr.-Ing. Thomas A. Bier

Persons other than those mentioned above did not contribute to writing of this thesis nor did I seek any help from professional doctorate consultant. No one has received financial payment in any form for any work done for me.

This thesis has not been submitted previously to any examination authority in the same or similar form inside or outside Germany.

MSc. Ahsan Ali

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DEDICATION

Thanks to God for enabling me to carry out this research work. I would like to dedicate my work to my father Dr. Umed Ali Buriro who throughout my life has been my guide and inspiration; to my mother Nazul Buriro whose patience and prayers were driving force in completing my work in time and to my wife Rashida Buriro who supported and looked after me and children during her difficult and stressful days.

ABSTRACT

With the availability of range of artificial lightweight aggregates and other mineral admixtures, lightweight aggregate concrete is getting momentum in construction industry. With such variety of materials, lightweight aggregate concrete can achieve compressive strength similar to that of normal weight concrete and thus be used in framed structures. Besides other qualities, like better fire resistance performance, better insulating properties, the most attractive part of lightweight concrete is the possibility of achieving greater reduction in the self-weight of the reinforced concrete member. On an average weight reduction of 20% or even higher can be achieved by replacing normal weight concrete with structural lightweight concrete. The effect of weight reduction on the whole structure is even more accomplishing, as not only can the size of supporting members, for example, columns at lower stories of a high rise buildings be reduced, creating more useable area, but also lower footing dimensions and savings in reinforcements are possible due to overall positive effect on foundations.

Like any other material, lightweight concrete also has its own demerits, such as higher initial cost, lower tensile strength, lower fracture energy, lower bond strength and higher shrinkage values. However some of these issues can be addressed with the use of discrete fibres. Another important issue in the wide acceptability of lightweight concrete whether or not reinforced with fibres is the unavailability of standard rules and regulations. This is particularly true in case of bond mechanism of this material with deformed reinforcing bars which directly affects the development and splice length of these bars.

There is plenty of literature available on the bond behaviour of normal weight concrete and most of the building code standards are based on the test results performed on conventional concrete. However, such rules or guidelines are not available for lightweight concrete, although, ACI-318 penalizes lightweight concrete by lowering its bond strength by 30% due to its lower tensile strength, but there is no reference of fibre reinforced concrete. Also, such reduction is not found in fib Model Code 2010, although, there is well defined bond stress-slip relationship, it is again specifically developed from bond test results of normal weight concrete and such a relationship is not available for LWFC.

Current research work was initiated to address these issues by performing bond tests on Normal Weight Fibre-reinforced Concrete (NWFC) and Lightweight Fibre-reinforced Concrete (LWFC) using pull-out test methodology. Besides significant improvement in flexural capacity, addition of fibres is also found to positively affect the bond strength of both the LWFC and NWFC. At highest fibre dosage (60 kg/m^3) the bond strength LWFC is either equal or higher than the conventional (NWC) concrete of comparable strength. Improvement in bond strength is found to be related with the fibre factor $\left(\frac{l_\mathrm{f} V_\mathrm{f}}{d_\mathrm{f}}\right)$ and is therefore recommended to be included in the fib Model Code 2010 bond equation. Modifications are also suggested in bond stress-slip relationship parameters to reflect the improvement in residual bond stress-slip profile of LWFC.

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α = Reinforcement location factor

β = Bar coating factor

δ = Mid span deflection of beams

γ_a = Bulk density of aggregates

γ_c = Density of concrete

λ = Lightweight aggregate concrete factor

τ, τ_b = Bond stress of concrete

A_s = Cross sectional area of reinforcing bar

A_{st} = Cross sectional area of transverse reinforcement

c_{clear} = Clear rib spacing of the deformed bar

c = Concrete cover

c_{max} = Maximum concrete cover

c_{min} = Minimum concrete cover

d_f = Fibre diameter

ϕ = Diameter of reinforcing bar

E_c = Modulus of elasticity of concrete

E_s = Modulus of elasticity of deformed steel bar

f_{ct} = Tensile strength of concrete

$f_{ct\text{-}sp}$ = Splitting tensile strength of concrete

f_c' = Compressive strength of concrete at 28th day

f_u = Ultimate tensile strength of deformed bar

f_v = Fibre volume in kg/m^3

f_y = Yield strength of deformed steel bar

h_r = Average height of deformations on reinforcing bar

k = Thermal conductivity

K_{tr} = Transverse reinforcement index

l_b = Bond length of deformed steel bar

l_f = Fibre length

n_1 = Number of legs of transverse reinforcement at a section

n_b = Number of anchored bars or pairs of lapped bars

P = Applied load

R_r = Relative rib area

s = Slip or displacement of pull-out bar

s_r = Average spacing of deformations on reinforcing bar

s_v = Spacing of transverse reinforcement

F = Tensile force in the reinforcing bar

F_b = Bond force

V_f = Fibre volume fraction, expressed as percentage of concrete volume

1 Introduction

1.1 General

As a structural concrete, lightweight aggregate concrete can be considered as a possible alternative to normal strength concrete in situations where dead load dictates the structural members' geometry, be it columns or foundations. However, lightweight aggregate concrete is not the first choice of engineers when it comes to the selection of structural material for construction due to its brittle nature and limited ability to absorb earthquake energy compared to the normal concrete.

Lightweight Fibre-reinforced Concrete (LWFC) on the other hand, which is made by adding discrete fibres to concrete has higher ductility, reduced crack width and also good impact resistance. Hence the limitations associated with lightweight concrete can be to some extent addressed by using different types of fibres. Acceptance of LWFC on a wide scale as a structural material for construction is subjected to detailed and thorough understanding of behaviour of this material. Current research work on bond performance of this material is a step in this regard.

1.2 Background

Normal weight and lightweight concretes are being used as structural concrete for quite some time. After many years of research, their properties and behaviour are well established and are entered in design codes. Developments in concrete technology and construction industry are still continuously pushing researchers and engineers for evaluation and standardization of modified properties and behaviour of these and other types of concretes – self-compacting concrete, fibre-reinforced concrete and self-healing concretes are the notable mentions.

Since its inception fibre-reinforced concrete has earned its place in construction industry, especially for projects dealing with impact and dynamic loadings. Subsequent advances in material technology, i.e. fibre types, mineral and chemical admixtures, artificial lightweight aggregates have made it possible to produce concrete that has favourable rheological behaviour.

Lightweight concrete due to lower density requires less effort and energy to handle and also has better thermal insulation when compared to normal weight concrete. Lower density of this material has been the most interesting point for design engineers as they are able to add more members or stories to a structure which otherwise may not be possible with normal weight concrete. This aspect is also attractive for design engineers who intend to use this concrete as strengthening material for rehabilitation of flexural members without adding too much dead load to them. Lightweight concrete however is more brittle in nature, besides this, difficulty associated with placing it in the form work meant for rehabilitation has also been the discouraging factor in its acceptance as the strengthening material. Researchers [1]–[3] have addressed these issues using fibres and superplasticizer, and successfully used lightweight concrete for repair and strengthening of reinforced concrete members.

Although, use of fibres has been found effective in enhancing the ductility of this concrete and thus addresses the issue of brittleness, another factor in low acceptance of lightweight concrete as a construction material has been higher initial investment cost. However, in long term, positive effects on foundations and savings from energy consumption because of better thermal insulation of structures can be more gainful. Based on these arguments, Lightweight Fibre-reinforced Concrete (LWFC) is expected to have good production and economic potential.

As a structural concrete, LWFC must first be extensively investigated for its application and design regulations and with years of experience, standards be developed. Properties of LWFC are being assessed for quite some time, however, because of the diversity of factors influencing, bond between the reinforcing bar and the concrete surrounding it is not fully understood or investigated. Current research work was carried out to investigate experimentally the interaction between deformed reinforcing bars and LWFC. Bond tests were also performed on Normal weight Fibre-reinforced Concrete (NWFC) of same strength class to better understand the performance difference between the two types of concretes. Within this research programme only hooked end steel fibres were used for making fibre-reinforced concrete. Other types of fibres, reinforcing bars (straight bars) and pre-stressing strands were not considered.

1.3 Scope of Research

There is plenty of literature available on bond behaviour due to two main reasons:

- diversity of parameters influencing it and
- recent developments in material technology.

Most of the literature is however focused on normal weight concrete or more recently on normal weight high strength concrete. After going through the reports on bond by ACI [4] and fib [5] and other research publications, it is realized that there is limited data available on bond behaviour of LWFC; furthermore, results reported in the literature were based on dosage and types of fibres which are not very common in concrete construction.

The research work presented here is aimed at serving additional database on bond behaviour of LWFC and its comparative performance with NWFC of same compressive strength class.

1.4 Objectives

The main objective of the present thesis is to investigate the bond behaviour of Lightweight Fibre-reinforced Concrete (LWFC) and how it performs in comparison to NWFC of same strength class. In order to achieve this aim, following tasks were undertaken:

- Review literature on bond performance of LWFC and its other mechanical properties. This would help in understanding the relation between bond and different concrete properties, besides this, influence of other structural and geometrical parameters on bond would be realized.
- Experimentally investigate the parameters that influence the bond performance: reinforcing bar diameter, fibre content and type of concrete.
- Evaluation of test results proposal of a new approach for description of bond in LWFC.

1.5 Thesis Outline

The whole document is composed of total seven chapters followed by the annexure of bond-slip profiles of test specimens. The details of the chapters are as under;

Chapter 2 is about the literature on LWFC, this includes history of lightweight concrete and fibres, subsequently leading towards the development of LWFC. It also includes literature on the effect of incorporation of fibres on different material properties of structural lightweight concrete.

In Chapter 3, previous literature on Bond is presented; it starts with explaining bond mechanism in a reinforced concrete member and its numerical interpretation, followed by discussion on factors influencing bond behaviour for both lightweight and normal weight concretes. However, due to certain limitations, which are discussed later in the chapter, only selective factors/parameters are considered.

Materials used and the concrete mix design approach adopted for current experimental program is presented in Chapter 4. Test parameters and the methodology adopted for the design of pull-out test specimens are also explained in this chapter.

Material properties test results for both LWFC and NWFC are presented in Chapter 5, these include fresh and hardened concrete test results, followed by bond test results in next chapter. It is done with a hope that these results will help in understanding the pull-out behaviour of specimens.

In Chapter 6, discussion and analysis of the bond test results is made. Results of the pull-out tests of NWFC and LWFC are presented along with discussion of effect of test parameters on bond behaviour. The analysis of results not only evaluates effectiveness of fibres in enhancing material and structural properties of LWFC and NWFC but also measures accuracy of code equations used for prediction of bond strength.

Chapter 7 summarizes the major outcomes of the research work and suggests potential future research work needed on current topic.

2 Literature Review on LWFC

2.1 About the Chapter

Chapter 2 covers brief history of development of Lightweight Fibre-reinforced Concrete (LWFC), its application and challenges in its acceptance on wide scale, which leads the focus to current research objectives that includes literature review on bond, which is done in chapter 3. It also includes different types of lightweight aggregates and fibres, along with their properties, used in production of LWFC. This is followed by the review of literature covering effect of fibres on fresh concrete properties and hardened concrete properties.

2.2 History of LWFC

2.2.1 Lightweight Concrete

LWFC is concrete made of hydraulic cement containing discontinuous fibres and either fine aggregates only or fine aggregates in combination with lightweight aggregates. It may also contain pozzolana and other admixtures commonly used in conventional concrete. It is therefore important to follow the development of lightweight concrete and fibre-reinforced concrete to understand the evolution of LWFC.

History of use of lightweight concrete in Europe dates back to Roman age when Grecian and Italian pumice were used as lightweight aggregate for its making. The Port of Cosa built in 273 B.C., Pantheon, built in 27 B.C., and famous amphitheatre Coliseum, built between 75 to 70 A.D. in Italy, are few of the known existing structures from past built using lightweight concrete [6]. In the construction of Pantheon dome (Figure 2.1) for example, pumice and tufa, both being lightweight aggregates are used in its top portion [7]. This dome still holds the credit of being largest unreinforced concrete dome with the of diameter 43.3 m [8]. It is believed that erection of such a structure would have not been possible, had the romans not opted for lightweight concrete over natural blocks. The structure is nowadays being used as a church.

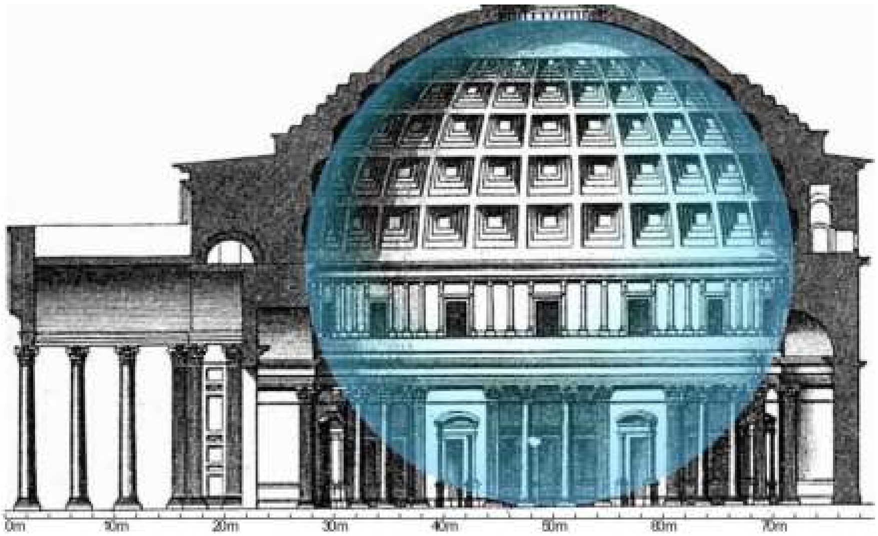


Figure 2.1 Cross-section of Pantheon - built in 27 B.C. [9]

Before Portland cement was conceived by Joseph Aspdin in 1824, roman cement was used as a binding medium for making concrete. This cement was so weak that the strengths of lightweight and normal weight concretes were almost comparable. But with the development of Portland cement, performance difference between the two concretes became wider. Various natural lightweight aggregates were tried to close this gap and take good advantage of greater strength of new cement but such attempts could not succeed. During 20th century, while addressing the bloating problem during brick making, Engr. Stephen J. Hayde, from Kansas, discovered that the bloated material had the potential of being used as artificial lightweight aggregates and if proper grading and size is used in making lightweight concrete then such a concrete can have mechanical properties similar to that of regular concrete. After years of experimentation, he was granted patent for the process of making artificial lightweight aggregates [10].

It was until the First World War when the full potential of lightweight aggregate concrete was realized. Although Lightweight Concrete (LWC) was being used for marine applications and in shipbuilding in U.S., its application on large scale was recognized during launch of 132.3 m long warship U.S.S. Selma (Figure 2.2). Successful operation of the Selma encouraged engineers and the production of concrete ships scaled from 14 to 104 by the end of World War II [10].

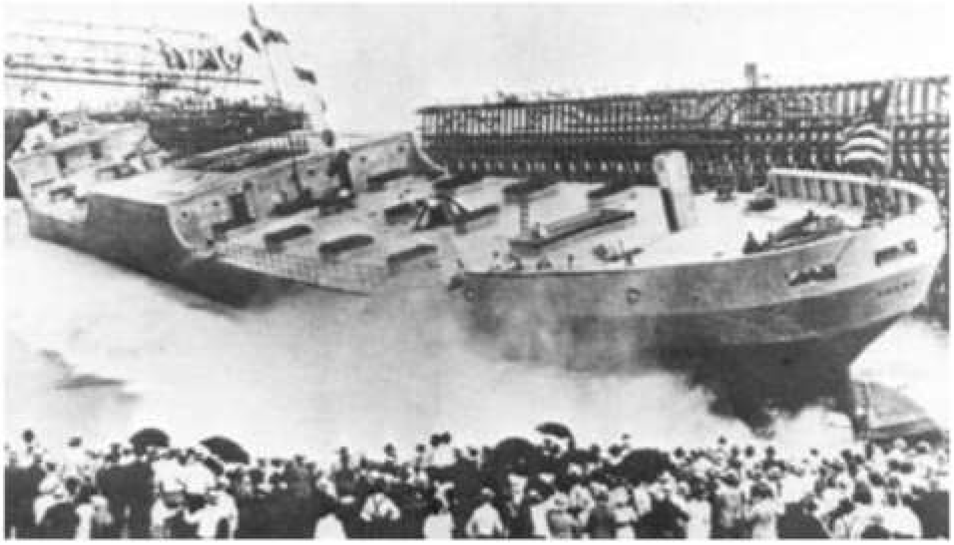


Figure 2.2 Launching of warship U.S.S Selma, June 1919 [10]

Figure 2.3 Southwestern Bell Telephone Office (Oak Tower) [11]

Although the first commercial plant started producing lightweight aggregates in 1920 in United States; in Germany production of artificial or synthetic aggregates started between 1935 to 1939 and Liapor which is produced by thermal treatment of expansive clay came to production in Germany in 1967 [12].

The first major project that involved use of structural lightweight concrete in construction of buildings was undertaken in 1928 in U.S. Engineers on the project wanted to add additional stories to the Southwestern Bell Telephone Company office (Figure 2.3) in Kansas City. Using LWC having compressive strength of 25 MPa, they were able to add 14 stories instead of eight (08) stories conceived with normal weight concrete [11].

2.2.2 Use of Fibres in Concrete

Like lightweight aggregates, use of fibres is also not new to mankind. According to the second book of Torah "The Book of Exodus", mud bricks reinforced with straw were in use by the Egyptians before Exodus that occurred in 1446 BC. Besides this use of asbestos is also reported for strengthening earthenware pots somewhat 4500 years ago [13].

Asbestos is a name given to a group of six fibrous minerals which occur naturally. Asbestos fibres are too small to be seen by the naked eye, these are also non-dissolvable in water, do not evaporate and resistant to fire and heat. Prolong exposure to asbestos can cause health problems like lung cancer and asbestosis. These health-related issues led to the option and introduction of alternate fibres. Although, products involving asbestos are still widely being used in world, use of asbestos in construction projects in developed countries including European Union has been banned [14].

Like mud bricks, brittleness of cementitious matrices and their weakness in tension was equally realized by researchers and is assumed to be the driving reason for using fibres to overcome these shortcomings. A good review of earlier patents on fibre reinforcement for concrete is presented by Naaman [15]. According to him development of fibre reinforcement for concrete can be divided into two phases/periods. He terms the first phase prior to 1960s as the pioneering phase during which different patents were registered throughout the world but saw almost no application and the second phase after 1960s as a phase of growth and application.

Different notable patents on fibre reinforced concrete include those filed by A. Berard from California in 1874, by R. Weakly from Missouri in 1912, H. Alfsen from France in 1918 and by A. Kleinogel from Germany in 1920 who suggested use of iron particles in concrete mix. In U.S.S.R. glass fibres were first used in concrete in the late 1950s, and then in early 1960s first major investigation was carried out to assess the prospective of steel fibres in concrete. This marked the point of second phase from where extensive research and development parallel with industrial application of fibre reinforced concrete has occurred [16]. Although century old patents exist for use of fibres in normal weight concrete, it is not clear exactly when the application of fibres in lightweight concrete begun.

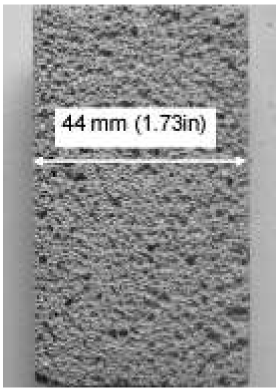
2.3 Structural Lightweight Fibre-Reinforced Concrete

2.3.1 General Remarks

There are different types of lightweight concretes used for various purposes, e.g. lightweight aggregate concrete, no-fines concrete and cellular concrete. As the name suggests in lightweight aggregate concrete, aggregate portion is lighter, generally done by replacing normal weight aggregates by lightweight aggregates. Concrete made by replacing only the denser coarse aggregates by lightweight aggregates is called either coarse lightweight aggregate concrete or sand-lightweight aggregate concrete; it is called all-lightweight aggregate concrete if both the fine and coarse aggregates (normal weight) are replaced by lightweight aggregates. Other details, the density, strength and applications about lightweight aggregate concrete are presented in subsequent sections.

No-fines concrete is produced by omitting the sand from concrete mix and therefore with larger voids is more porous than the conventional concrete but has the advantages of being lower in density (1600 – 2000 kg/m^3) with better thermal insulation property [17], [18]. The thermal conductivity (k) of no-fines concrete is 0.7 W/m-K compared to 2.0 W/m-K for dense concrete [19]. This concrete due to its lower strength (5 – 15 MPa) and higher brittleness is not used in load carrying members. Besides its use in non-load bearing walls and as large in situ panels, this concrete, due to its open texture and high permeability (see Figure 2.4 (a)) is also used as a drainage medium where it absorbs rainwater and facilitates the natural runoff [20].

(a)



(b)

Figure 2.4 Lightweight concrete (a) No-fines concrete [21] (b) Cellular concrete [22]

Cellular concrete (Figure 2.4(b)) is made from the mixture of cement, water and performed foam [23]. Cellular concretes may or may not include the sand aggregates and other admixtures and therefore covers wider range of density i.e. from 320 kg/m^3 to 1920 kg/m^3. In every case, these concretes contain stable air or gas cells resulting from the action of a foaming agent. Although there are many lightweight cementitious materials, the thing that differentiates cellular concrete from these cementitious materials is the use of a foaming agent to reduce the density [24]–[27]. Low density cellular concrete does not contain aggregates, this concrete therefore attains the density of 800 kg/m^3 or less and is used for geotechnical fills or on roof decks due to better thermal insulation. Typical thermal conductivity values for this concrete are presented in following table. Cellular concrete that has sand aggregate as one of the constituent materials, generally weight more than 800 kg/m^3 but have compressive strength less than 17 MPa. Concrete in this density range are used for cast-in-place, precast applications and non-structural floor fills [28]. Apart from better thermal insulation, other advantages of cellular concrete are better pump ability, ease of handling and better acoustical insulating properties.

Table 2.1 Thermal conductivity values for oven-dry cellular concrete [23]

Oven dry density (kg/m^3)	320	480	640	800
Thermal conductivity, k (W/m-K)	0.11	0.13	0.16	0.20

Like other building materials, performance expectations from lightweight concrete have also risen over the years and with the experience of every passing year, more predictable material behaviour and properties are being observed. With the possibility of achieving high performance lightweight concrete and the other reasons mentioned earlier, that, apart from other non-structural applications like lagging or sound-proofing, it is now being employed as a structural concrete in construction of precast elements and high-rise buildings.

Use of high strength lightweight concrete is particularly encouraging in situations where high strength with lower density is preferred. However, compared to the normal weight concrete, lightweight concrete demonstrates more brittle behaviour for the same strength level. By confining the LWC, using fibres, brittleness of LWC, lower residual tensile strength and lower fracture toughness can be overcome [29].

2.3.1.1 Types of aggregates

One of the important constituents of structural lightweight concrete is the lightweight aggregate. In general, lightweight aggregates have apparent specific gravity considerably lower than the normal weight aggregates. There are several types of lightweight aggregates that can be used in the production of variety of LWC mixes. For structural concrete, the lightweight fine aggregate and coarse aggregates should have maximum bulk density values up to 1120 kg/m^3 and 800 kg/m^3 respectively and are prepared either by processing natural materials (like scoria, tuff) or by sintering of blast furnace slag, clay, shale [30]. Compared to this, aggregates such as perlite and vermiculite, which are mainly used for making insulating concrete have bulk density values in the range of 120 to 160 kg/m^3 and are called non-structural aggregates [31]. Table 2.2 enlists different types of aggregates with their bulk density values, which can be used for production of structural and insulating lightweight concretes.

Table 2.2 Different lightweight aggregates used in the production of LWC

Types of lightweight aggregates	Aggregates' bulk density, γ_a (kg/m^3)	Used for making	f_c' (MPa)
Expanded slag	800 - 1040	Structural concrete	20 - 40
Expanded shale, clay, slate - sintered	300 - 800	Structural concrete	20 - 40
Expanded shale, clay, slate – rotary kiln	300 - 700	Structural concrete	17 - 38
Cinders	700 - 900	Structural concrete Fill concrete	12 - 40
Scoria	700 - 1000	Structural concrete, Fill concrete and insulating concrete	9 - 17
Pumice	500 - 700	Fill concrete and Insulating concrete	7 – 14
Expanded glass	220 - 2160	Structural and Insulating concrete	-
Perlite	120 - 192	Insulating concrete	2 - 7
Vermiculite	88 - 160	Insulating concrete	2-7

2.3.1.2 Range of properties of structural LWC

As stated earlier LWFC is a lightweight concrete reinforced with fibres; to fit in the definition domain of being called structural lightweight fibre-reinforced concrete, LWC, as per ACI building code (ACI 318) [32] guidelines should have a minimum of 28 days cylindrical compressive strength of 17 MPa and an equilibrium density between 1120 and 1840 kg/m^3. Model Code 2010 [33] on the basis of density classifies concrete into lightweight aggregate concrete ($\gamma_c \leq 2000$ kg/m^3), normal weight ($2000 < \gamma_c \leq 2600$ kg/m^3) and heavy weight concrete ($\gamma_c > 2600$ kg/m^3). As per ACI [32], [34] and Euro Code [35] lightweight concrete may be termed high strength if it attains compressive strength higher than or equal to 55 MPa.

Figure 2.5 shows full spectrum of lightweight concretes along with range of density values that can possibly be achieved by using different aggregate types. Low density concretes shown at the left side of the diagram are used for insulation purposes;

whereas those in the middle are used as fill concrete and insulating concrete. Compared to these two, structural lightweight concrete on the right side of the diagram has higher compressive strength and for this reason is primarily used in the framed structures. Table 2.2 not only lists bulk density of aggregates and applications of lightweight concrete, but also lists compressive strength values that are attainable with specified LWA types.

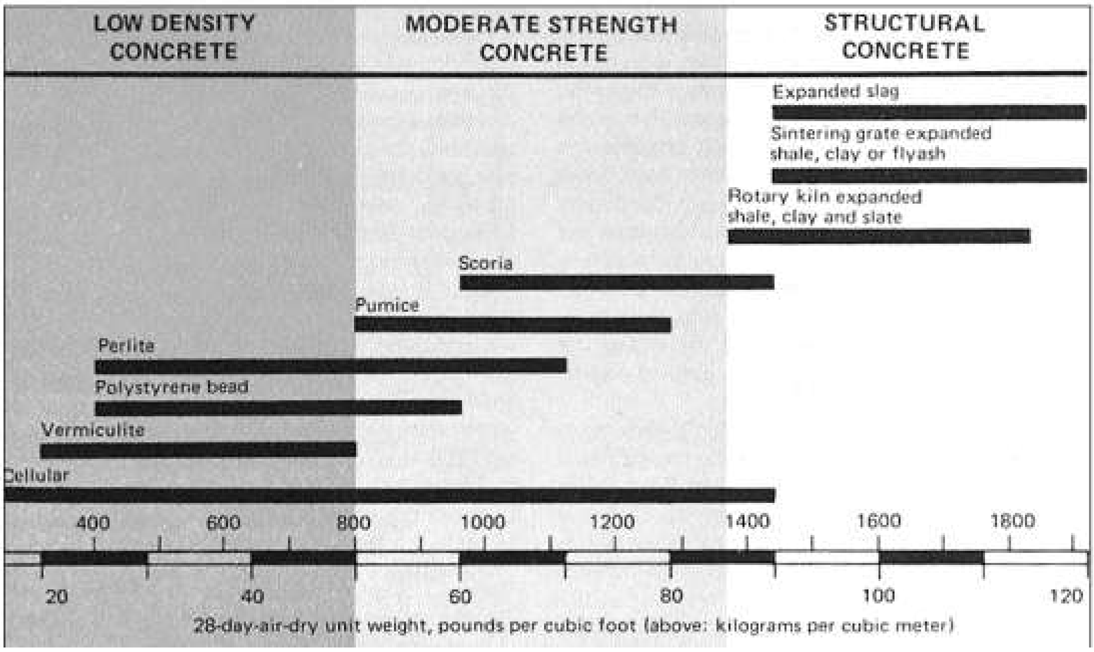


Figure 2.5 Lightweight concretes along with their typical range of 28 – day-air-dry unit weight (after [36])

2.3.1.3 Fibres used in concrete

Since 1960s development in fibre technology has given birth to numerous fibre types with different geometries. Fibres (Figure 2.6), are categorised as metallic fibres (e.g. steel fibres), synthetic fibres (glass fibres, polymeric fibres, carbon fibres), and natural fibres (cellulose fibres, basalt fibres, ceramic fibres) and most of the commercially available fibres used for concrete making fall into these main categories Fibres are produced and applied as continuous fibres (textile meshes) and as short discrete fibres. Selection of the specific fibre types or geometry is decided based on field/environmental conditions and the type of loading conditions that a structure incorporating fibre reinforced concrete has to sustain.

Natural fibres can be acquired from different sources such as jute, sisal, sugarcane, coconut etc. Although these fibres can be obtained at lower energy costs and their use has successfully been reported too by some researchers but durability issues have also been recorded mainly due to the expansion of these fibres in presence of moisture [16].

Wide variety of glass fibres with different chemical composition can be obtained for use in concrete. Initial research incorporating glass fibres used borosilicate and soda-lime-silica glass fibres, also known as E-glass and A-glass fibres respectively. However reaction of these glass fibres in an alkaline environment makes them unsuitable for use in concrete [37]. Nowadays many types of alkali resistant (AR) glass fibres are available commercially and almost all concretes made from glass fibres utilize this type of fibre. Other properties of glass fibres are tabulated in Table 2.3. More than 75% of the application of glass fibre-reinforced concrete involves making of exterior building façade panels [16].

Exposure of AR-glass fibre-reinforced concrete to weather conditions has shown too reduction in modulus of rupture strength, loss in strain capacity. Research findings of an experimental program [38] which assessed the effect of environment on glass fibre-reinforced concrete over 10 years reveal that modulus of rupture strength of concrete reduced to the strength level at proportional elastic limit

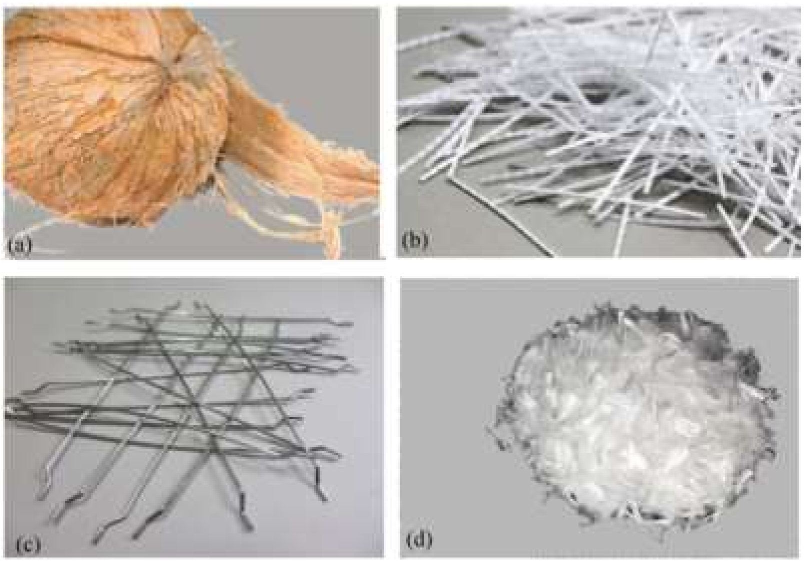


Figure 2.6 Different fibre types (a) coir – a natural fibre obtained from the husk of coconut (b) glass fibres (c) hooked-end steel fibres (d) polypropylene – synthetic fibres

Different types of fibres such as polypropylene, acrylic, carbon, polyethylene, nylon etc. categorized as synthetic fibres are used form making synthetic fibre-reinforced concrete (SNFC). Properties of some of these fibres commercially available are presented in Table 2.3. Earlier applications of synthetic fibres were in the projects of shotcrete concrete, as not only these were easy to handle but also unlike steel fibres were resistant to corrosion. At volume fraction of between 0.1% to 0.3% these fibres are primarily used for controlling shrinkage cracks [39].

For most structural and non-structural purposes, steel fibre is the most commonly used of all the fibres [40]. Steel fibres have length that range from 6.4 mm to 76 mm with an aspect ratio between 20 to 100 and diameter range of 0.25 mm to 1 mm. The maximum volume fraction of steel fibres in concrete is from 1.5% to 2%. Utilization of fibres above this limit tends to create issues like balling of concrete and other mixing and placement issues. Various shapes and cross sections of steel fibres are being used in research on steel fibre-reinforced concrete, some of which are shown in Figure 2.7.

Compared to most of the other fibre types, steel fibres have higher elastic modulus and specimens made from steel fibre-reinforced concrete have higher toughness values. Fibres are sometimes bent at ends or crimped at intervals along their length for better mechanical bond with matrix. Crimping process of fibres throughout the fibre length may cause loss in elastic modulus of fibres. Other way of increasing the mechanical bonding of fibres is by enlarging their surface area, however, this may result in difficulty of concrete mix. Major portion of the following write-up contains discussions focused on steel fibre-reinforced concrete

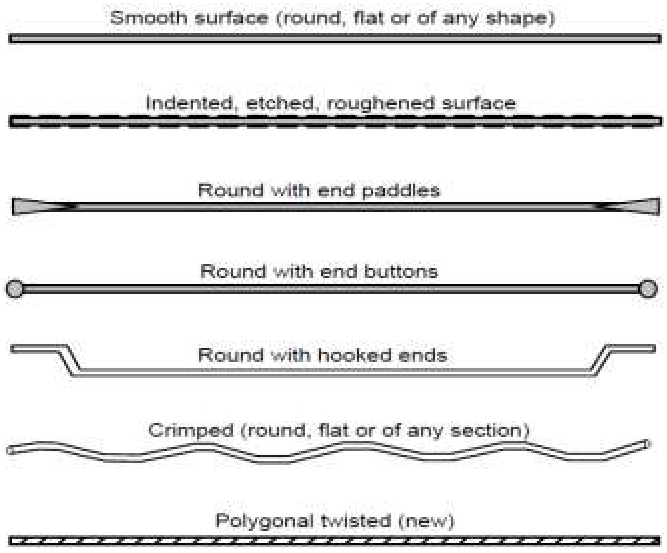


Figure 2.7 Various shapes of steel fibres used in FRC (after [41])

Table 2.3 Properties of different types of fibres

Fibre type	Tensile strength (MPa)	E (GPa)	Specific gravity	Strain at break %
Steel fibres	345 - 2600	200	7.8	3
Glass fibres				
A-glass	3100	65	2.46	4.7
E-glass	3450	72	2.54	4.8
AR-glass	2460	79	2.7-2.74	2.5-3.6
Synthetic fibres				
Acrylic	268-1000	14-19	1.16-1.18	7.5-50
Polypropylene	138-689	3-5	0.9-0.91	15
Carbon	482-4000	28-480	1.6-2.15	0.5-2.4
Natural fibres				
Coconut	120-200	19-26	1.12-1.15	10-25
Jute	248-350	26-32	1.02-1.04	1.5-1.9
Sisal	275-568	13-26	1.33-1.37	3-5

2.3.2 Effect of Fibres on Fresh Concrete Properties

The two constituting materials in lightweight fibre-reinforced concrete that distinct it from normal weight concrete are, the lightweight aggregates and the fibres. Subsequent sections therefore present the literature review on the effect of these materials on fresh and hardened properties of LWC and where ever possible performance comparison is made against normal weight concrete. It is done in a view of understanding the correlation, if any, of these properties with the bond of LWFC.

2.3.2.1 Workability

According to ACI Committee 544 [42], "workability of freshly mixed concrete is a measure of its ability to be mixed, handled, transported, and, most importantly, placed and consolidated with a minimal loss of homogeneity". It recommends measurement of workability of fibre-reinforced concrete by measuring time of flow through inverted slump cone (Figure 2.8). Compared to traditional slump cone test, this method involves use of vibrator to facilitate the flow of fibre-reinforced concrete and noting down the time it takes. The test method is standardized in ASTM C995 [43]; however the standard was withdrawn without replacement in 2008 due to complexities associated with this method like incompatibility with self-consolidating fibrous concretes, wrapping of long fibres along vibrator and filling the cone with concrete so that no concrete falls through the hole. Apart from this method, ACI Committee [16] also recommends use of Vebe test apparatus standardized in DIN EN 12350-3 [44] for the workability test.

German standard DIN EN 12350-5 [45] uses method of jolting the fibre-reinforced concrete rather than vibration. The method follows the procedure of filling the cone (Figure 2.9) in two layers and then letting the concrete spread by jolting the metal plate from specified height for 15 times and then measuring the spread in two perpendicular directions. Although the method does not account effectively the thixotropic nature of fibre-reinforced concrete compared to those involving vibration, but is advantageous in a way that similar test method/procedure is followed for concretes containing no fibres, making it easier to compare test results for evaluation and also it is easy to carry equipment and perform tests in field with this method.

The ease with which concrete is worked upon can be improved if the density/weight of concrete can be reduced. For such purpose, use of lightweight aggregates of low specific gravity becomes advantageous and results in good workable concrete even at lower slump values. In contrast higher slump values may cause separation and settling of heavier mortar away from lightweight aggregates [13], this phenomenon is opposite to that of segregation in normal weight concrete, where heavier aggregates settle leaving mortar at surface. ACI [6] therefore limits maximum slump value for lightweight aggregate concrete to 125 mm when determined using ASTM C-143 [46] procedure.

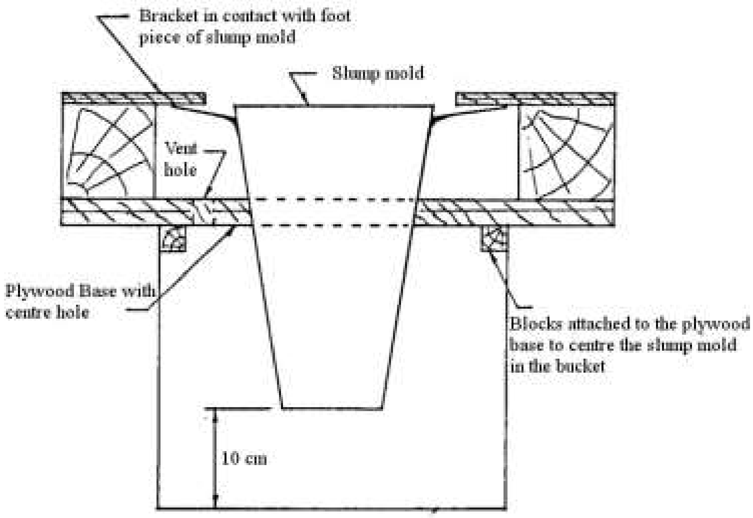


Figure 2.8 Inverted slump cone test apparatus [43]

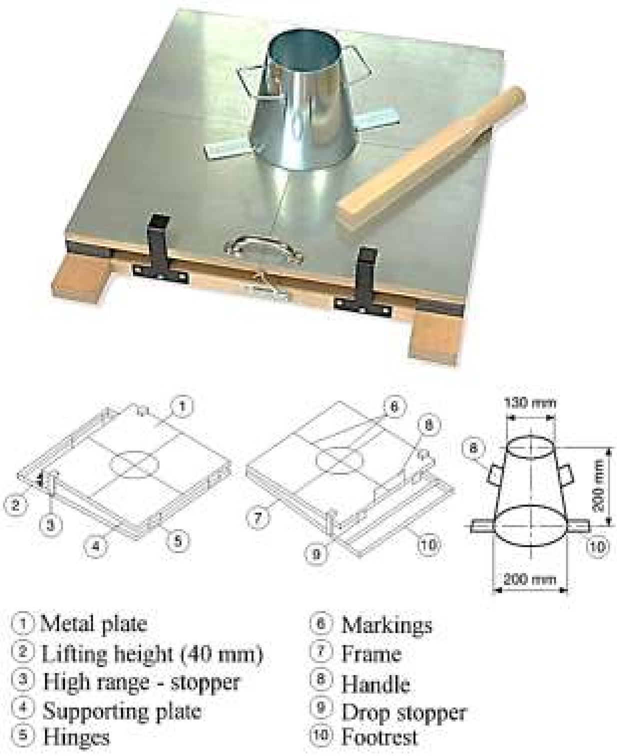


Figure 2.9 Flow table test equipment for measuring workability

Workability of fresh concrete is adversely affected by fibres, the magnitude of effect being directly proportional to the amount of fibres being used [47]–[49]. According to Bayasi and Soroushian [50], in a fibre-reinforced concrete, fibre aspect ratio l_f/d_f and fibre volume fraction (V_f) are the two main parameters that influence concrete workability. They investigated the effect of different steel fibre types (crimped, hooked end and straight fibres) on workability using inverted slump cone and Vebe test methods and found that inverted slump cone timings of hooked-end fibres were higher than the straight and crimped fibres. They also reported higher Vebe times for crimped fibres compared to straight fibres. Effect of fibres on workability also depends upon the fibre types being used as highlighted by Chen and Liu [51]. They observed that at 1% volume fraction, steel fibres reduced slump by about 54%, whereas this reduction was only 20.8% when propylene fibres were used in lightweight concrete. The effect of fibre aspect ratio and fibre volume fraction of steel fibres on workability is reported by Yazici et al. [52]. Hooked-end bundled fibres having aspect ratios of 45, 65 and 80 were used by the researchers in volume fractions of 0.5%, 1% and 1.5%. They observed that workability of the mix was adversely affected with increase in fibre volume fraction and fibre aspect ratio. Compared to control mix with no fibres, slump of the mix with fibres (0.5% V_f) reduced by only 4% at lower fibre aspect ratio ($l_f / d_f = 45$), for similar fibre content this reduction was however around 13% when fibres of higher aspect ratio ($l_f / d_f = 80$) were used. Authors further report that slump reduced by 13%, 34% and 37% when quantity of fibres was increased by 0.5%, 1% and 1.5% respectively for similar fibre aspect ratio ($l_f / d_f = 80$).

In another study [53] on the effects fibres on properties of lightweight self-compacting concrete, micro steel fibres were used. These straight fibres had length and diameter of 13 mm and 0.2 mm respectively. The authors report that when fibre volume fraction increased from 0.5% to 1.25%, the slump flow reduced from 725 mm to 630 mm – a reduction of about 13%.

Superplasticizers are therefore normally used in fibrous concretes to achieve good workability. In a study [29] that incorporated steel fibre dosages as high as 160 kg/m^3 (2% V_f) in lightweight concretes made from expanded clay and pumice stone, good workability was achieved by using 1.5% of superplasticizer by weight of cement. Iqbal et al. [54] increased quantity of fines in his mix design along with superplasticizer to achieve required workability for high strength self-consolidating lightweight fibrous concrete.

2.3.2.2 Density

Because of their higher specific gravity, steel fibres have tendency to increase the density of concrete [55]. A study [56], about steel fibres' effect on properties of lightweight pumice aggregate concrete reports an increase in unit weight of concrete by 3.1%, 6.5%, 8.5% for fibre volume fraction of 0.5%, 1% and 1.5% respectively. This factor of increase in weight of lightweight concrete therefore needs to be considered by design engineers especially for volume fraction of fibres greater than 1%. Use of fibres having lower specific gravity, like polypropylene or glass fibres or hybrid fibres (combination of steel and other fibre types) can be considered as an option in such scenario [57]. Silica fume, besides having good effect on interfacial transition zone (ITZ) is also found to reduce the concrete weight. Test results [58], on effect of silica fume and steel fibres on normal weight concrete show that when steel fibres having aspect ratio l_f/d_f of 65 were added at 1% V_f, unit weight of concrete increased by 1.5%, whereas the increment was only 0.4% when silica fume replaced cement by 15% at similar fibre dosage (Figure 2.10).

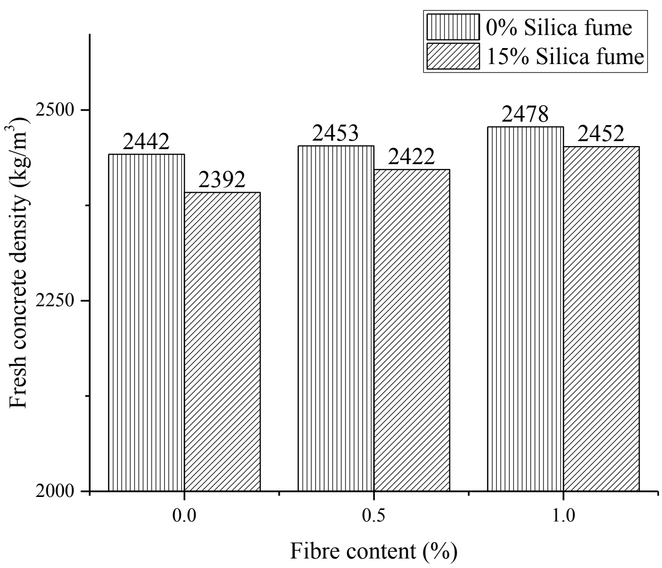


Figure 2.10 Effect of silica fume on fresh concrete density [58]

ASTM C-138 [59] method which is used for conventional concrete is equally applicable fibre-reinforced concrete, except that vibration instead of rodding is used for achieving compaction. Similarly German standard DIN 12350-6 [60] can also be used for both fibrous and non-fibrous mixes.

2.3.2.3 Pump ability

With the innovations of different materials for concrete making and to avoid city pollution, the consumption/requirement of ready mixed concrete is growing. This concrete in some situations is needed to be pumped to reduce construction period and therefore project cost as well [61]. During pumping controlling the consistency and quality of this concrete over long pumping distances like in multi-storey buildings can be a difficult task [62]. Basic requirement for efficient pumping of the concrete is that the mix should neither be too dry nor too wet.

Harsher mixes tend to exert pressure on pipe walls, whereas too wet mixes can be sticky. For lightweight concretes, extra care is required while pumping, as loss of slump at delivery point can be sometimes very high due to absorption of water by aggregates which could be as high as 15 to 20%. It is therefore generally standard practice that lightweight aggregates are soaked prior to concrete mixing to avoid slump loss. Pre-wetting of lightweight aggregates therefore not only improves pump ability but also maintains consistency at the end point [63]. ACI Committee 304 [64] limits the maximum coarse aggregate size to 1/3 and 2/5 of the smallest of the pump or pipe diameter for angular and rounded aggregates respectively. Besides this shape of coarse aggregates also affect pumping, e.g. angular aggregates require more mortar for effective pumping due to higher surface area compared to rounded aggregates [64].

Estimation of pumping pressure required for pumping fresh concrete is done till to date by the field tests in conjunction with the nomograms and guidelines generated from past experience [65]. For example, ACI Committee 304 [66] recommends production and pumping of trial mix before final mixing, for which the pumping pressure may be estimated from the Figure 2.11 shown below. The nomogram essentially estimates the pumping pressure based on the required flow rate, pipe length and the slump at delivery point. In recent years, researchers [61], [67], [68] have also attempted different ways of predicting pumping pressure and flow rate by estimating rheological properties of concrete and the lubricating layer, also called boundary layer that is formed between concrete and pipe wall during pumping.

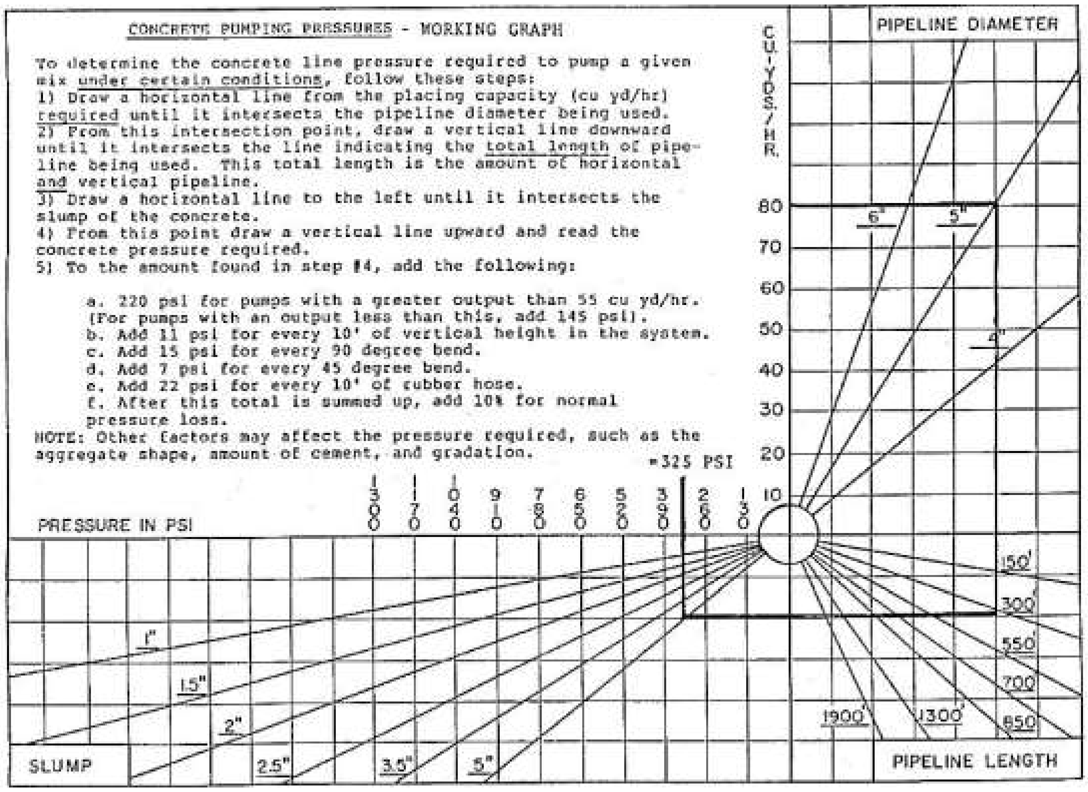


Figure 2.11 Estimation of concrete pumping pressure from the flow chart for required flow rate [66]

Note: 1″= 25.4 mm, 1′= 0.3048 m, 1 yd^3/hr = 0.76 m^3

Addition of steel fibres makes lightweight concrete harder to pump, therefore matrix must be rich enough to carry over the fibres and fibres need to be completely mixed with the mix. Fibres tend to increase the friction factor and modify viscosity of the mix [69]; however this could be compensated using higher HRWRA (High Range Water Reducing Admixture) dosage which will decrease viscosity and increase of pump ability [70]. Also, mixes rich in cement content or pumps with higher suction power or air entrainment can also be considered. Another option to improve the pump ability is to use mineral admixtures like fly ash and silica fume [71]. Spherical and smaller shape of fly ash particles make the mix more slippery and thus improve the pump ability, however its higher content could create sticky effect and reduce pump efficiency [72]. Jeong et al [70] investigated the effect of different mineral admixtures (fly ash, silica fume and zirconica silica fume) on pumpability and found that for similar pumping conditions (pumping pressure, pipe dimensions) flow rate of concrete doubled at 20% replacement of cement by zirconia silica fume, which was higher than the other types of admixtures.

According to ACI Committee 304, both synthetic and steel fibres can be pumped, but gives no guidelines on maximum fibre content that can be pumped for different conditions and recommends that manufacturer's manual be consulted in this regard. In a field and laboratory testing, however, concretes with fibre volumes up to 1.25 % V_f have been used without any difficulty [73], [74]. For pumping a slump of 50 mm to 150 mm are suitable, multiple factors such as setting of cement and absorption of water by aggregates may cause loss of slump while pumping and therefore should be considered while mix design. At higher slump values, aggregates might separate from rest of the matrix, in such a scenario use of superplasticizer can be effective [66].

2.3.2.4 Fibre agglomeration

Fibre agglomeration or fibre balling is the phenomenon where fibres interlock together during mixing of the concrete. It is especially undesirable, as such balls or bunches of fibres can create regions of unconsolidated concrete and make pumping of concrete difficult. An unconsolidated concrete when hardened would result in voids and therefore crack more easily under load. Apart from fibre balling, another aspect that needs to be looked upon is the settlement of fibres during mixing of lightweight aggregate concrete. This is common in pan type mixers, where steel fibres due to higher specific gravity travel to the bottom of mixer leaving the aggregates atop. This phenomenon can be avoided by selecting short fibres which result in lesser weight, opting for drum type mixer, and designing a mortar with enough viscosity to hold the discrete fibres in suspension.

Factors that contribute in fibre agglomeration are, shape of fibres, fibre aspect ratio, size and gradation of aggregates, fibre quantity and the way fibres are mixed to concrete. Fibres of specific shape, ball easily than others at similar aspect ratios and tendency of interlocking in steel fibres is higher if fibre aspect ratio is higher than 100 [75].

Certain measures, if taken, reduce the chances of balling effect. Attention shall be paid to the shape and aspect ratio while selecting the fibre type. Chances of balling are reduced if fibre volume fraction is kept lower than 2%, and may increase even at 1% if fibres having higher aspect ratio are used. Once type and fibre volume are selected then these should be fed into the concrete mixer in a rain type fashion this could be done by shaking the fibres through a screen before feeding them in the wet mix. Once fed clumped free, chances of fibre balling are minimal, but the mix must be fluid and rich enough to carry over the fibres, otherwise would continue to stack over each other forming the wet fibre balls. Use of auxiliary conveyor belt can be helpful for easy feeding of fibres as shown in Figure 2.12, the speed of mixer needs to be adjusted during this stage. Alternatively .on a batching plant site, fibres should be fed on a conveyor belt together with aggregates [76]. German standard, DIN EN 14721 method [77] can be used for checking balling effect in a mix by taking samples during casting stage. Another way of reducing the balling effect is by using bundled fibres glued together with the help of water soluble adhesive. Ramakrishnan et al. [78] in their work used glued hooked-end steel fibres and straight fibres which were not glued. They report that despite having hooked-ends and higher aspect ratio of 100 $l_f = 51mm, d_f = 0.5mm$ compared to that of straight fibres $l_f = 25.4mm, d_f = 0.5mm$, it was possible to produce tangle free concrete with such high aspect ratio due to bundling of fibres.

Figure 2.12 Adding fibres last to the transit mixer [76]

2.3.2.5 Air content

For non-air-entrained concrete, there is always some amount of air which is entrapped in concrete. The higher amount of entrapped air could be the result of poor mixing, consolidation and improper concrete placement. As a result unwanted and irregularly sized air pockets are formed which affect the concrete strength. In some cases, air is intentionally introduced in concrete to produce uniformly dispersed air bubbles for improving fresh and hardened concrete properties, such a concrete is called air-entrained concrete. In fresh state, air bubbles, normally created with the help of an air entraining admixtures, lubricate the mix and increase slump. In hardened state, durability of concrete is highly improved with air-entrainment. Its effectiveness is especially realized in areas with higher temperature fluctuations. Freezing and thawing cycles in such areas may cause water/ice pressure on cement matrix and its possible disintegration. In an air-entrained concrete, this pressure is taken by air bubbles which act as a relieve valve and enhance concrete durability. For lightweight aggregate concrete recommended and approximate air content values for air-entrained and non-air-entrained concretes respectively are presented in Table 2.4.

Table 2.4 Air content values for air-entrained and non-air-entrained lightweight aggregate concrete

Aggregate size (mm)	Approximate amount of entrapped air in non-air-entrained, %	Air content for air-entrained concrete for different levels of exposure, %		
		Mild	Moderate	Extreme
9.5	3	4.5	6	7.5
12.7	2.5	4	5.5	7
19	2	4	5	6

Source: ACI Committee 211 [79]

Like other constituent materials of mix e.g. cement, water and aggregates, variation in fibre volume, geometry and fibre aspect ratio also greatly affect the fresh properties of concrete [80]. The air content of fresh concrete mix is mostly reported to increase with subsequent increase in fibre volume and aspect ratio. Iqbal et al. [53] studied the effect of change in the volume micro steel fibres on fresh and hardened properties of high strength lightweight self-compacting concrete. He used four different volume fractions of fibres i.e. 0.5%, 0.75%, 1% and 1.25% in his work and recorded respectively the air content values as 3.63%, 4.17%, 5.25%, and 5.32% (see Figure 2.13).

In their experimental work Soulioti et al. [81] used two different types of steel fibres (hooked end & waved shape) in three different fibre volumes fractions. For both types of fibres, they observed increase in air content as the fibre volume increased. Although both the fibre types had different aspect ratios, 41 $l_f = 31mm, d_f = 0.75mm$ for hooked end and 33 $l_f = 25mm, d_f = 0.75mm$ for waved type, but both these resulted in similar increase in air content values. For both fibre types authors have reported 24% increase in air content from reference mix containing no fibres. According to Johnston [82] rise in the amount of entrapped air is due to the reduced workability of the fibre-reinforced concrete.

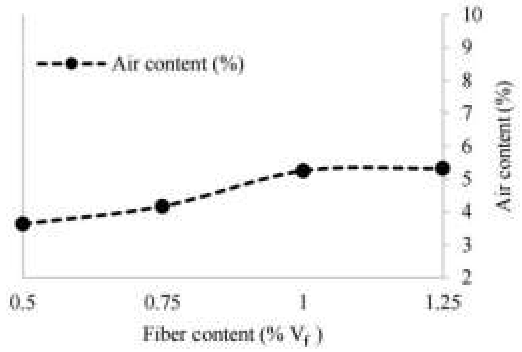


Figure 2.13 Effect of steel fibres on lightweight concrete [53]

2.3.3 Effect of Fibres on Hardened Concrete Properties

2.3.3.1 Compressive strength

Inherently all the concretes have micro cracks in them, which can increase in size for loads as below as 50% of ultimate load. Formation of network of these cracks causes concrete member to fail. Presence of discrete steel fibres resist formation of this network due to fibre-matrix bond and hence some improvement in strength and ductility can be achieved [83]. Lightweight aggregate concrete specimens under compression fail in a highly brittle way, this mode of failure changes to less brittle with successive addition of fibres. Irrespective of concrete type, fibres have variable effect on compressive strength of concrete; the effect generally being insignificant is dependent on the amount of fibres [42]. Studies of Johnston [82] and Williamson [83] report an increase of up to 20% in compressive strength test results for normal weight concrete after incorporation of steel fibres. In another study [84], increase in compressive strength values of just 3.5% and 5.95% was observed for fibre volume fractions of 0.5% and 1% respectively. Mahadik et al. [85] report that although compressive strength values of all fibrous mixes (V_f = 0.25%, 0.5%, 0.75% and 1%) were higher than the reference concrete mix containing no steel fibres, it essentially started decreasing after 0.75% V_f. The increase of 24% in strength value observed at this fibre content, decreased to 10% at 1% V_f. Recent experimental work by Hamzacebi and Sengul [86] on effect of using waste steel fibres in concrete suggest that fibres have variable influence on compressive strength. Their results indicated that for fibre volume of 20 kg/m^3, compressive strength of concrete reduced by 8% whereas an increase of up to 10% was observed for fibre dosage of 40 kg/m^3. Similar unclear trend is reported by Lee et al. [87], they used three different fibre aspect ratios of hooked-end steel fibres in their experimental work and observed variable profile of compressive test results for them (Figure 2.14).

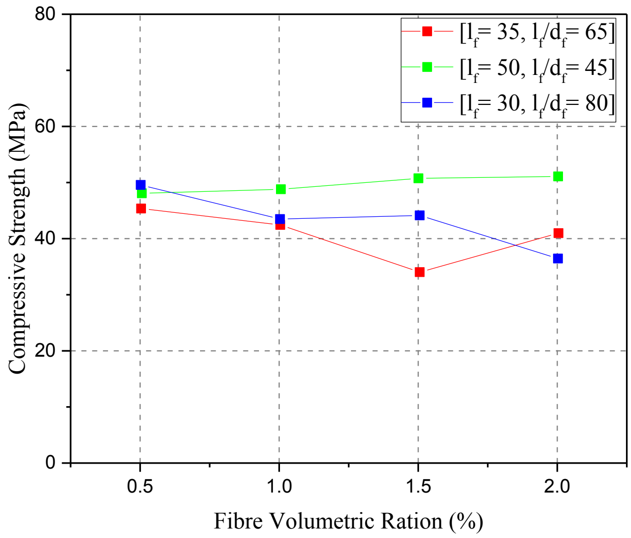


Figure 2.14 Effect of steel fibres of different aspect ratios on compressive strength of conventional concrete.(modiefied from [87])

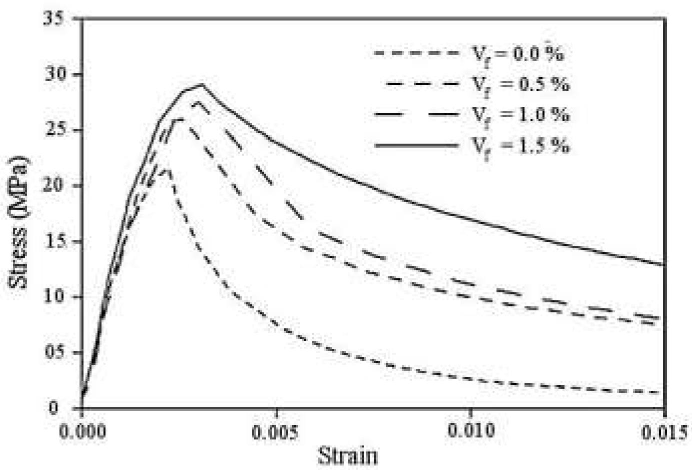


Figure 2.15 Stress-strain plot under compression for lightweight fibre-reinforced concrete [88]

An experimental work [88] on effect of steel fibres on bond between conventional reinforcement and lightweight concrete which also outlines compressive strength test results reports an increase in test results. Lightweight concrete in this study was made from expanded clay having maximum aggregate size of 17 mm and hooked-end steel fibres of 30 mm length and aspect ratio of 60 were added at dosages of 0.5%, 1% and 2% V_f. It was observed that compressive strength of these fibrous mixes increased (Figure 2.15) by 22%, 29% and 38% respectively. In another study [89] on effect of hybrid fibres on mechanical properties of lightweight concrete made from pumice aggregate, polypropylene fibres and hooked-end steel fibres $l_f = 35mm, d_f = 0.55mm$, it is reported that compared to polypropylene fibres, improvement in compressive strength of SFRC was more prominent. This improvement however reduced from 61.5% at fibre volume fraction of 0.5% to 54.5% for fibre volume fraction of 1%. Similarly, for propylene fibres compressive strength of concrete increased by 11.2% at V_f of 0.2% but a reduction of 8.56% (from reference mix) was observed at V_f = 0.4%.

Above studies have reported variable influence of fibres on compressive strength, and this unclear trend is considered to be result of lack of uniformity in material selection and specimen design. Nevertheless, majority of the past studies have reported that compressive strength can be increased from 0 to 15% by using steel fibres up to 1.5% [16]. This improvement is considered to be the result of arrest of micro cracks and delaying their formation into macro cracks under compressive loads.

Determination of compressive strength of LWFC can be made using similar testing equipment and testing procedure which are used for normal weight concrete. However size of molds used for casting specimens should be at least three times the fibre length and once filled, these moulds should preferably be vibrated externally to facilitate random fibre distribution and discourage preferential fibre alignment, which otherwise could yield unrealistic higher compressive strength test results [42] . These testing methods are reported by American Society for Testing Materials (ASTM) in ASTM-C39 [90] and by German Standard in DIN 12390-3 [91].

2.3.3.2 Splitting tensile strength

Multiple factors influence the tensile strength of concrete including concrete compressive strength, aggregates type and also type of test used for its determination.

Normally tensile strength of concrete ranges between 8 to 15 percent of the compressive strength and tensile strength of lightweight concrete can be as low as 30% than that of normal weight concrete [92]. Two types of tests, splitting tensile strength test and flexural tensile strength test (also called modulus of rupture test) are commonly used for determination of tensile strength of concrete. There is third type of test, called direct tensile strength test used for measuring concrete's tensile strength, but since, there is no international or European standard for the test [93], it is mostly used in research works, also among these tests splitting tension test is considered more practical and reliable [94].. American Society for Testing Materials [30] recommends that concrete made from sand-lightweight aggregate with a maximum density of 1140 kg/m^3 should have a minimum splitting tensile strength of 2.3 MPa when determined using ASTM C 496 [95] procedure.

Most of the previous studies [89], [96]–[99] have reported increase in splitting tensile strength of both lightweight and normal weight concretes. For steel fibre volume fraction of up to 1%, an increase in the splitting tensile strength of lightweight aggregate concrete can be as high as 116% [57]. Effect of steel fibres on split cylinder tensile strength of lightweight aggregate concrete made from expanded clay and pumice stone is reported by Campione et al.[29]. They performed split tension testes on cylinders of 100 x 200 mm size and used hooked-end steel fibres $l_f = 30mm, d_f = 0.5mm$ in four different volume percentages (0, 0.5, 1 and 2%). The expanded clay used in their experimental work had maximum aggregate size of 17 mm and bulk density of 650 kg/m^3 that produced the concrete of density 1640 kg/m^3. They observed that compared to pumice stone lightweight concrete, increase in splitting tensile strength was more in expanded clay concrete with addition of steel fibres. Results of their work show that for expanded clay lightweight concrete, splitting tensile strength was doubled as the fibre quantity increased from 0 to 160 kg/m^3. Furthermore, authors are of the view that strain hardening effect can be achieved for fibre dosages of 1% and 2% volume fractions (see Figure 2.16)

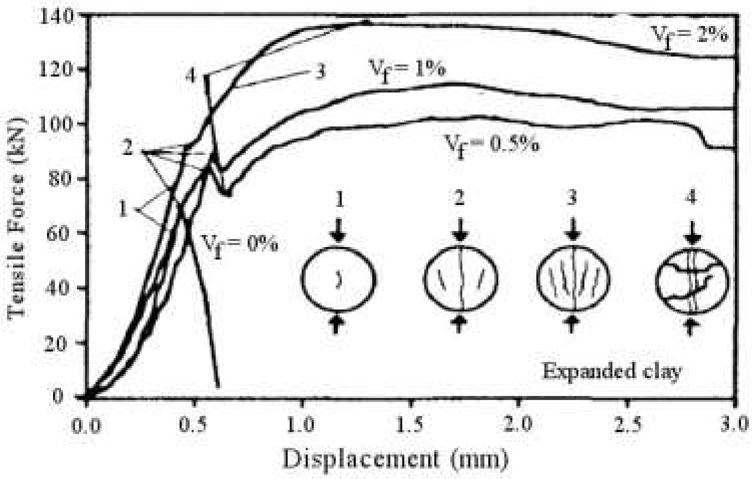


Figure 2.16 Effect of fibres on split cylinder tensile strength of lightweight concrete [29]

Study by Balendran et al. [96] in which they used 1% V_f of straight steel fibres $l_f = 15mm, d_f = 0.25mm$, concludes that splitting tensile strength of both normal weight and lightweight concretes increased after fibre addition and that for the same volume fraction of fibres this increase was more in lightweight concrete than normal weight concrete. Yao et al. [98] report an increase of 36.5% in splitting tensile strength of normal weight concrete after adding 0.5% V_f of hybrid fibres composed of carbon and steel fibres. Balaguru and Foden [99] observed that fibre length has insignificant effect on splitting tensile strength of expanded shale lightweight concrete but reported that the strength was more than doubled at maximum fibre content of 90 kg/m^3.

2.3.3.3 Modulus of elasticity

Lightweight concrete has lower elastic modulus than normal weight concrete [100] and fibres are found to have insignificant influence on modulus of elasticity of concrete [13], [97]. Results of elastic modulus tests performed on sintered fly ash Pollytag lightweight concrete by Domagala [101] show that the consistent increase of hooked end steel fibres from 0% to 0.8% V_f could not alter its elastic modulus (see Figure 2.17). Campione et al. [29] work shows that elastic modulus of expanded clay lightweight aggregate increased by 17.7% when fibre volume was raised from 0 to 2%, however for the same volume fraction it decreased by 12% for lightweight concrete made from pumice aggregate. Iqbal et al. [54] produced fibre reinforced high strength lightweight self-compacting concrete using expanded clay as coarse aggregate, fly ash and high strength micro steel fibres. Their test results on modulus of elasticity show slight decrease of 7% at maximum fibre content of 1.25%.

Compared to lightweight concrete, modulus of elasticity of normal weight concrete is typically 25% to 50% higher [102]. Nevertheless, effect of fibres is found to be similar on concrete's elastic modulus as observed in lightweight concrete. For example in a study [58] it was shown that fibres with different aspect ratios of 65 and 80 had identical effect on elastic modulus of normal weight high strength concretes and in both cases maximum change of negative 6.5% was observed at 1% V_f.

ASTM C469 [103] and DIN EN 12390-13 [104] are used by the respective American and German Standards for determination of modulus of elasticity of all types of concrete.

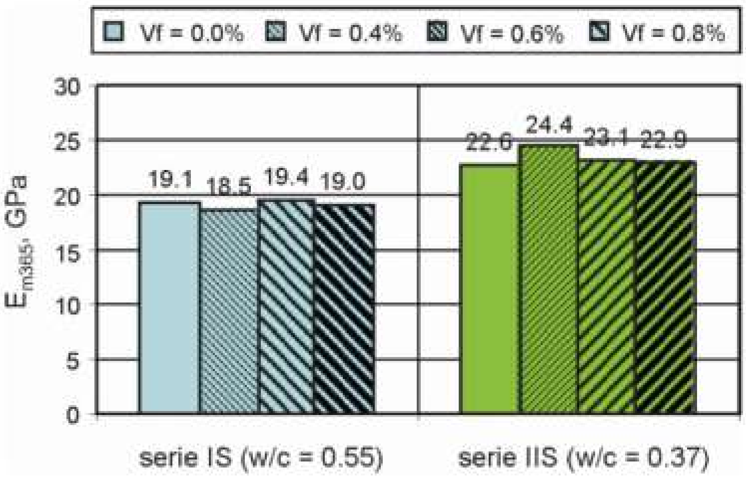


Figure 2.17 Effect of steel fibres on elastic modulus of lightweight concrete (after [101])

2.3.3.4 Flexural strength

Flexural strength of conventional concrete in laboratory under flexural loads is determined either by centre point loading test arrangements or by four-point loading test setup. These methods are detailed in ASTM C78 [105] and ASTM C293 [106] by American Society for Testing and Materials (ASTM) and in EN 12390-5 [107] as per European Standard. For concrete, reinforced with fibres, three different types of test arrangements are in practice. For example European Standard EN [108] utilizes the centre-point loading test setup in which specimen under flexure is notched at its bottom and the tensile behaviour is observed from the load-crack mouth opening displacement curve obtained from the test. German Committee for Structural Concrete (Deutscher Ausschuss für Stahlbeton - DAfStb) [109] uses four point bending test setup for flexural strength determination of fibre-reinforced concrete as shown in the Figure 2.18. This test setup is also documented in ASTM C1609/C1609M [110] with some changes, for example ASTM allows use of different specimens sizes, other than the two recommended sizes of 100 x 100 x 350 mm and 150 x 150 x 500 mm.

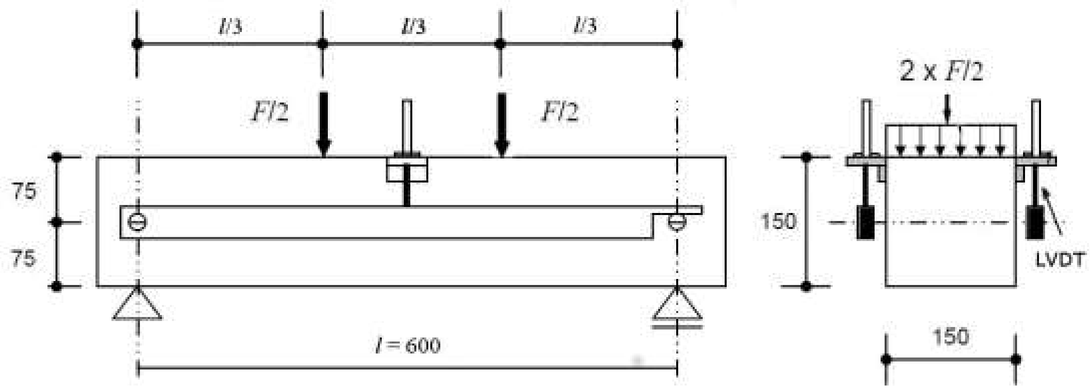


Figure 2.18 Four point bending test setup recommended by DAfStb [109]

Fibre addition imparts greater improvement in flexural behaviour of concrete than its other aforementioned mechanical properties. This improvement comes from the fact that once the beams under flexural loading start cracking, fibres start on taking tensile stresses, resist propagation of cracks toward compression zone and thus increase load carrying capacity of member. Using four point loading test setup, results show that flexural behaviour of unreinforced normal weight concrete beams could be improved by 50% to 70% with steel fibres at volume fraction of 1.5% to 2% [16]. Yoo et al. [111] used normal strength concrete, high strength concrete and ultra-high strength concrete beams reinforced with steel fibres of aspect ratio 60 $l_f = 30mm, d_f = 0.5mm$ for evaluation of their flexural behaviour. Their results show that for the same strength class of normal weight fibre reinforced concrete, increase in fibre content from 0.5% to 1%, flexural strength of concrete increased by 39%, whereas it was doubled when the fibre dosage was further increased to 2%.

Although LWAC has lower flexural strength than normal weight concrete, it has been reported by Balendran et al. [96] that improvement in flexural strength of LWAC is more than NWC after fibre addition (see Figure 2.19). They used 1% of steel fibres for both types of concretes and observed maximum improvement of 43% in flexural strength of normal weight concrete, whereas for lightweight concrete, strength increased by 91% with similar testing variables. Results of study by Düzgün et al [56] contradict with this statement, where flexural strength of concrete made from 75% of normal aggregates and 25% of Pumice aggregate increased by 79% at 1% V_f. At the same fibre content this improvement was 61%, when all normal weight coarse aggregate was replaced with lightweight aggregate.

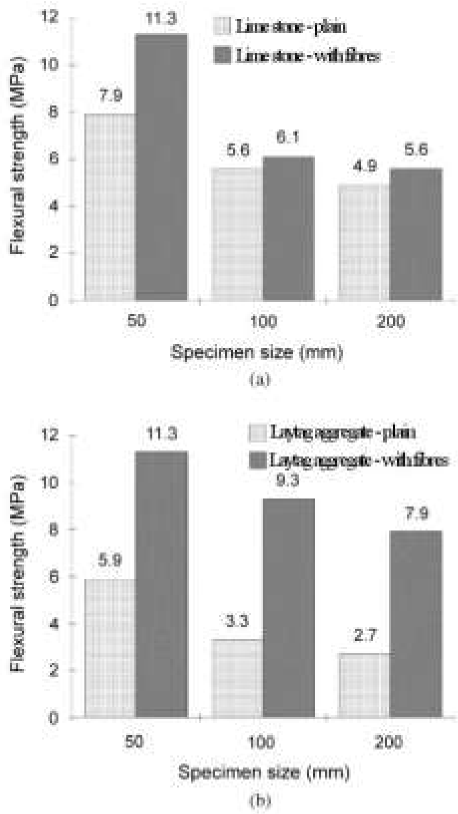


Figure 2.19 Flexural strength of plain and fibre-reinforced concretes for different beam specimen sizes (a) normal weight concrete (b) lightweight concrete (after [96])

General trend on the effect of fibres on tensile strength of concrete from both flexural and splitting tensile strength suggest that tensile strength of concrete is positively influenced. Tensile strength of test specimens also depends upon the type of test which is being used and also on the specimen size, loading conditions and other factors. For example, tensile strength decreases as the specimen size increases [112] and a four point loading test performed on a 150 mm square beam gives higher tensile strength (modulus of rupture) than a splitting tensile strength test, which is on an average 1.5 times higher. Similarly direct tensile strength as per Euro Code 2 is 10 % lower than the splitting tensile strength determined by EN 12390-6 [113]

2.3.3.5 Post cracking behaviour

The usefulness of fibres in enhancing the post cracking performance of fibre-reinforced concrete is undisputable. It is mainly due to this superior quality of arresting cracks that FRC (Fibre-reinforced concrete) is used in structures which are prone to crack under static, impact or seismic/environmental induced loadings [114]–[116].

Post cracking behaviour of FRC can be assessed using similar test setups mentioned in section 2.3.3.4, and is generally evaluated in terms of flexural toughness and the residual flexural tensile strength determined at specific deflection points as per test standards. ASTM 1609 [110] specifies two points, L/600 and L/150 (L = beam span length) for residual tensile strength calculations and determination of area of load deflection diagram up to net deflection of L/150 for flexural toughness. German Committee for Structural Concrete (Deutscher Ausschuss für Stahlbeton - DAfStb) [109] uses net deflection of 0.5 mm and 3.5 mm of load-displacement curve (see Figure 2.20) for residual flexural strength. The committee uses the mean values of these results with some additional calculations to determine the performance class of fibre-reinforced concrete. It recommends that apart from other classes such as compressive strength, exposure and humidity, the fibre performance class of FRC shall also be indicated as a ratio of L1/L2; where L1 and L2 here correspond to performance classes for minor and major deformations respectively. Classification of SFRC into different performance classes measures its ability to transfer tensile loads across a cracked section.

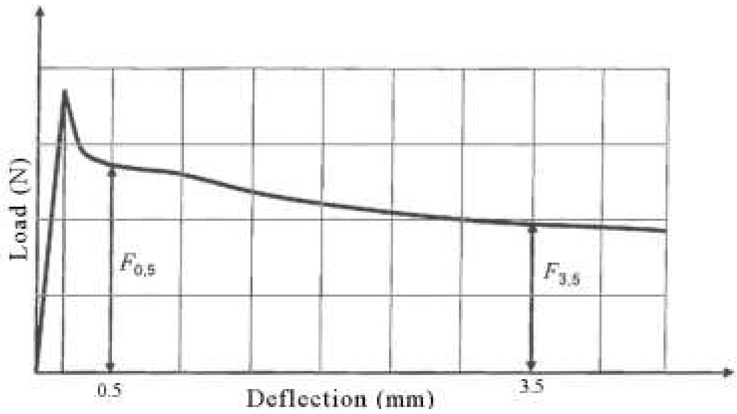


Figure 2.20 Specific deflection points on load-displacement curve for finding out residual tensile strength [109]

Performance improvement in post cracking region of FRC, for example improvement in flexural strength, toughness and residual load capacity are highly dependent on fibre aspect ratio and fibre volume fraction and fibre type [117].

Gao et al. [117], evaluated the effect of these two parameters i.e. fibre aspect ratio and fibre volume fraction) on the properties of lightweight concrete made from expanded clay. Four different volume fractions of steel fibres i.e. 0.6%, 1%, 1.5% and 2% were used. The fibre lengths (20 mm, 25 mm, 30 mm) used developed aspect ratios of 46, 58 and 70 respectively. Results of flexural test specimens from his work indicate that fibres play major role in increasing the flexural toughness or the area under load-deflection curve. The improvement in flexural toughness can be seen in Figure 2.21 as both the aspect ratio l_f/d_f and fibre volume fraction (V_f) increase.

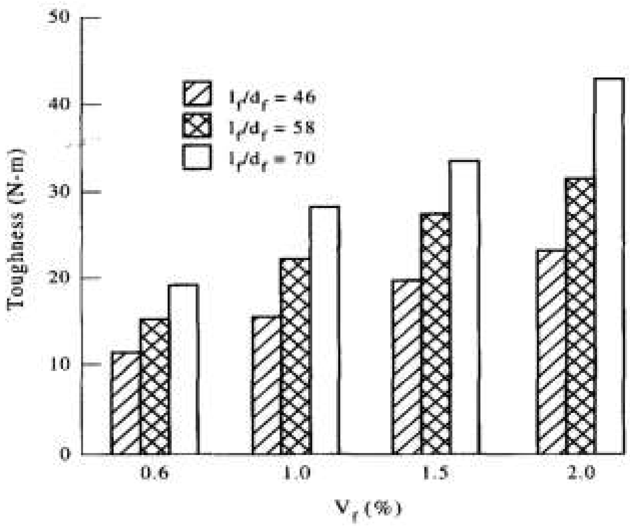


Figure 2.21 Effect of fibre volume and aspect ratio on flexural toughness (after [117])

Besides steel fibres researchers have also evaluated effect of other fibre types used individually and also in combination with steel fibres on post cracking behaviour of concrete. Corinaldesi and Moriconi [118] for example studied the effect of synthetic fibres on the properties of self-compacting lightweight aggregate concrete. Polypropylene micro fibres having length 19 mm with aspect ratio of 63 and macro fibres 50 mm long, generating aspect ratio of 110 were used in the concrete mixtures. Although authors did not observe any improvement in maximum tensile strength value, they however did observe significant improvement in post cracking behaviour of concrete having synthetic macro fibres.

Hybrid fibres composed of steel fibres and polypropylene fibres were used by Libre et al. [89] in their research work. The steel fibres were hooked-end shaped 35 mm in length and with a diameter of 0.55 mm; whereas synthetic fibres had length of 12 mm and diameter of 0.016 mm. Fibres were incorporated in both the conventional and lightweight concrete made from pumice aggregates. Authors observed higher improvement in flexural toughness values of LWAC compared to conventional concrete and suggest that this could be due to the higher brittleness of LWAC. They also observed that enhancement in toughness values was way higher than the improvement in flexural strength, for example, compared to 78 times improvement in toughness of concrete, flexural strength improved only by two times at steel fibre volume fraction of 1%. Similar observations have been reported by Kim et al. [119], who used oil palm shell as lightweight coarse aggregates and hooked end steel fibres for production of LWFC. Results of test specimens show that flexural toughness at 1% volume fraction of fibres was 25 times higher than the control specimens containing no fibres.

2.3.3.6 Thermal conductivity

Ability of a substance to conduct heat is called thermal conductivity, measured in watts per meter kelvin (W/m-K) and generally varies with the temperature. According to ASTM [120], thermal conductance is "time rate of steady state heat flow through a unit area of a material or construction induced by a unit temperature difference between the body surfaces" measured in W/m^2-K. Materials with lower thermal conductivity values, allow heat to transfer at lower rates than materials with higher conductivity values and it is essential that thermal properties of materials be known for energy calculations of any building. For concrete, guarded-hot-plate apparatus can be used for determination of thermal conductivity values. The methodology employing this apparatus is detailed in ASTM C177 [121] document.

Multiple factors such as environmental conditions, intrinsic characteristics of specimen for example size and distribution of pores and test conditions may affect the thermal conductivity [122]. Due to porous nature of most of the aggregates used in making LWAC, it has better insulating properties or in other words has lower conductance than conventional concrete. Typical values for NWC range between 1.2 to 1.8 W/m-K whereas for structural lightweight concrete the range is between 0.4 to 0.8 W/m-K [122], [123]. Thermal conductivity k test results performed on concretes having density ranging from 320 to 3200 kg/m^3 indicate strong relationship between concrete density γ_c and k values as shown in Figure 2.22 and expressed in the following equation.

$$k = 0.072e^{(0.00125\gamma_c)} \tag{2.1}$$

Where k and γ_c have units of W/m K and kg/m^3 respectively in above equation.

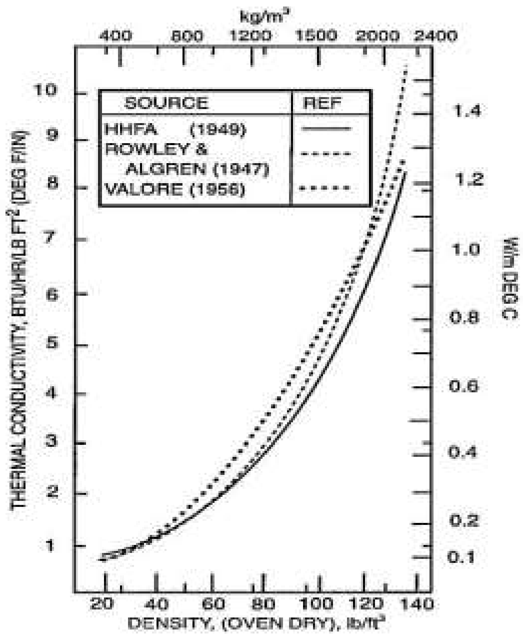


Figure 2.22 Plot of relationship between thermal conductivity and concrete density (after [124])

Small increase in k values has been reported by Cook and Uher [125] after increasing the steel fibre content in the concrete from 0.5% to 1.5%. Copper fibres were also used in their study work, for which they found higher thermal conductivity values compared to steel fibres. In another research work, Nagy et al. [126] studied the thermal properties of glass, steel and plastic fibre reinforced concretes. Authors used three different fibre contents i.e. 20, 27.5 and 35 kg/m^3 of steel fibres and reported lower k values for concrete mixes with higher fibre dosages. Higher porosity values of the samples are considered as the possible explanation for such behaviour as reported by the researchers.

2.3.3.7 Electrical resistivity

Use of fibre-reinforced concrete has increased over the years and lot of literature has been published focusing on its fresh and hardened properties; however little has been published about its electrical resistivity. From the available literature, it is generally agreed that the addition of steel fibres reduce the electrical resistivity of concrete and in doing so influence the corrosion rate of deformed bars [127]. For such a scenario a hybrid system, consisting of conductive and non-conductive fibres can be considered to limit the material degradation [128]. Similarly electrical resistivity of concrete reinforced with carbon fibres is discussed in other studies [129]–[131]. A case study on the corrosion of steel reinforcement by stray electric current and its effects is presented by Stanley [132]. In another research work, Amr S. El-Dieb [133] has reported an decrease in electrical resistivity after addition of steel fibres. Apart from the control specimens (without fibres), he prepared three other mixes with different amounts of fibres, the maximum volume fraction of steel fibres used in his work were 0.52%. He reports that on 28th day of testing, electrical resistivity of concrete containing highest amount of steel fibres was about 550% lesser than that of control specimens. Lower electrical resistance is recorded during early age of concrete, but increased with the time, up to 90 days as shown in Figure 2.23.

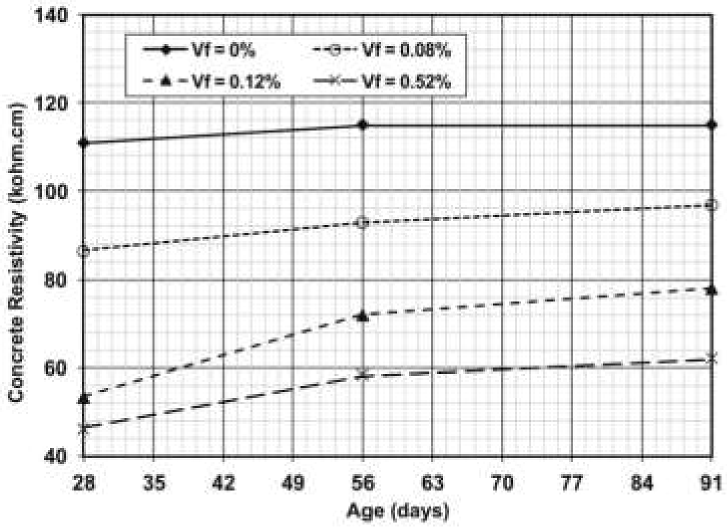


Figure 2.23 Effect of steel fibres on concrete's electrical resistivity (after [133])

3 Literature Review on Bond

3.1 Bond Mechanism

Concrete is a brittle material and has very low tensile strength, hence requires services of reinforcement to bear excessive stresses. Transfer of stresses between reinforcement and concrete is very significant for the response of reinforced concrete structures and is only possible if there is adequate bond between them, absence of which would means loss of strain compatibility between the two materials leading to excessive slip and thus to structural member' failure. A good bond between steel reinforcement and concrete is not only the prerequisite for any structural reinforced concrete member to function as a composite material but it also influences crack development and ductility of the reinforced concrete member [1]–[3].

Three different mechanisms are responsible for the transfer of forces between the two materials (reinforcing bar and concrete) namely adhesion, friction and mechanical anchorage (Figure 3. 1). Out of these three, surface adhesion is the first one to be lost as the reinforcing bar slips under loading. This is followed by reduction of frictional forces which act on barrel and ribs of bar. In the end, due to higher slipping and loss of adhesion and frictional forces, bearing of the ribs against surrounding concrete (mechanical anchorage) is left as the key mechanism for force transfer.

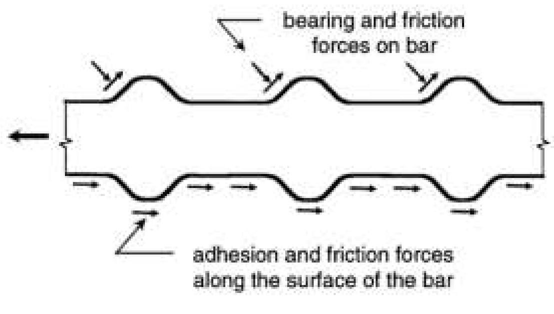


Figure 3.1 Bond force transfer mechanism (after [4])

According to Tepfers [134], the inclined forces on the bar are balanced by the ring tensile stresses in the surrounding concrete as shown in Figure 3.2. The inclined/bearing forces exerted by the ribs on the surrounding concrete can be split into horizontal and vertical components; the later component is responsible for the splitting mode of bond failure when it exceeds the tensile capacity of concrete. Another mode of failure i.e. pull-out failure occurs in more heavily confined concrete (with larger concrete cover and transverse reinforcement), when shear resistance of concrete between successive ribs is exceeded.

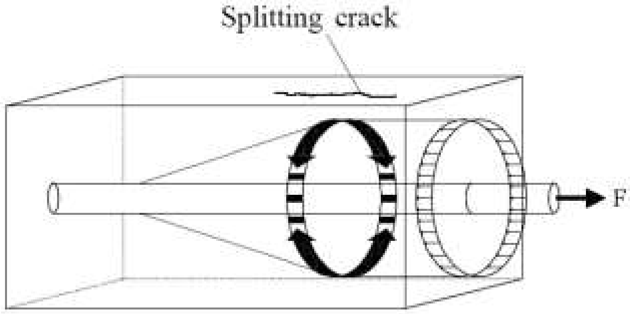


Figure 3.2 Tensile stresses in the form of ring band (after [134])

It would be interesting to see if splitting mode of bond failure could be delayed and bond capacity of the reinforced concrete member be increased by incorporation of fibres in lightweight concrete. In past, efforts have been made by researchers in this regard, but such efforts mostly involved conventional concrete. Harajili for example used hooked-end steel fibres in normal weight concrete and found that contribution of fibres in enhancing the bond strength was insignificant for all such cases where pull-out mode of failure occurred because the slipping of bar occurred prior to the activation of fibres through development of any splitting cracks. He also found that the ratio of product of fibre length and fibre volume to the fibre diameter (fibre index) had strong influence on bond-slip profile and recommended that this ratio should be considered in equations used for estimation of bond strength and proposed following equation.

Compared to the normal weight concrete, lightweight concrete perhaps is more deserving or worthy of fibre feeding for the reasons such as being more brittle in nature than normal weight concrete, achieves higher workability than convention concrete for same fibre content, and perhaps achieve higher percent of bond strength increase, like higher percentage of rise in compressive strength for similar fibre content than NWFC as reported earlier by and discussed in section 2.3.3.1. Despite all these LWFC has not received attention to the extent it deserves, especially when it comes to its bond behaviour. It is expected that arresting of splitting cracks by fibres as shown in Figure 3.3 would help in raising the bond strength and evaluate further the justification of penalty imposed by various building codes on lightweight concrete due to its brittle nature.

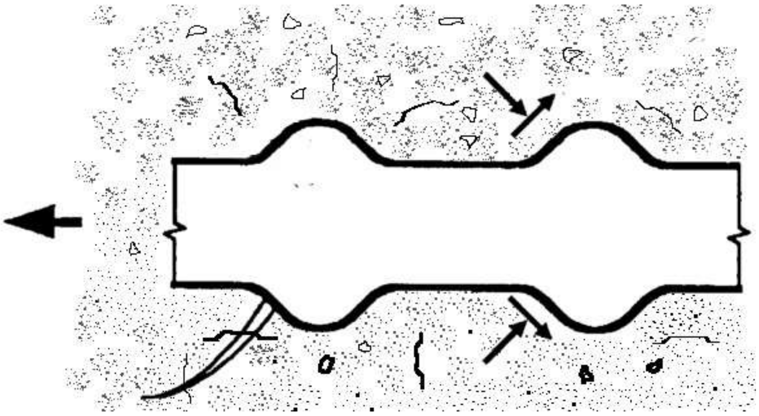


Figure 3.3 Mechanism of arrest of splitting cracks by fibres

3.2 Interpretation of Bond Stress

Consider Figure 3.4 (a) which shows the typical Free Body Diagram (FBD) of a beam and let there be the loading condition on this beam such that force system at section – 2 is greater than that of section – 1, which would mean that bar stress σ_{s2} would be greater than σ_{s1} by magnitude $\Delta\sigma_s$. This extra force/stress presence will try to dislocate the bar, therefor for the reinforcing bar to remain in equilibrium; bond stresses (τ_b) must be present on the surface of the bar as shown in Figure 3.4 (b). Fulfilment of force equilibrium condition leads to the bond stress expression as follows;

$$F_2 - F_1 - F_b = 0$$
$$\Rightarrow \sigma_{s2}A_s - \sigma_{s1}A_s - \tau_b\pi\phi l_b = 0$$
$$\Rightarrow \tau_b = \frac{\Delta\sigma_s A_s}{\pi l_b \phi}$$
$$\Rightarrow \tau_b = \frac{\Delta\sigma_s \phi}{4 l_b}$$

$$\Rightarrow \tau_b = \frac{F}{\pi l_b \phi} \tag{3.1}$$

Where; τ_b, F, l_b,ϕ are bond stress, tensile/pull-out force, bond length and diameter of pull-out bar respectively.

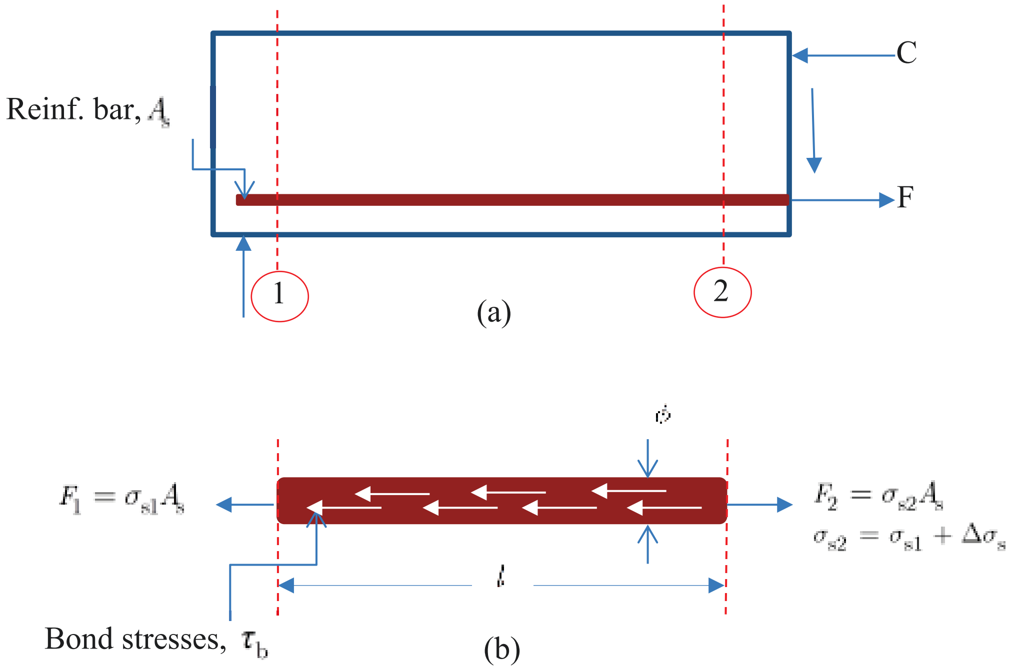


Figure 3.4 (a) Internal forces in a beam (b) relationship change in bar stress and bond stresses (modified from [100])

3.3 Factors Affecting Bond

3.3.1 General remarks

Multiple factors affect the bond property of a reinforced concrete member, it is this multiplicity and diversity of parameters that has resulted in ample literature on the subject, proof of which is evident from the fact that American Concrete Institute has set up a committee (ACI Committee 408 [4]) specifically for the Bond. Also, frequently international conferences on “Bond in Concrete” have been held in past in Paisley (1982), Riga (1992), Budapest (2002) and Brescia (2012); yet research on the subject continues as the new testing techniques and materials are regularly being introduced [135].

The state of the art report on bond of reinforcement in concrete by fib [5] discusses not only the factors influencing bond conventional reinforcement but also cover bond properties of corroded, non-metallic and pre-stressing tendons. ACI 408 [4] has grouped all the parameters that affect bond behaviour of concrete into three main categories as shown in Figure 3.5. Parameters / factors filled in grey colour in the figure are covered in this literature review in detail, as covering all the parameters would not only be difficult but would also shift the focus of the research work away from main theme.

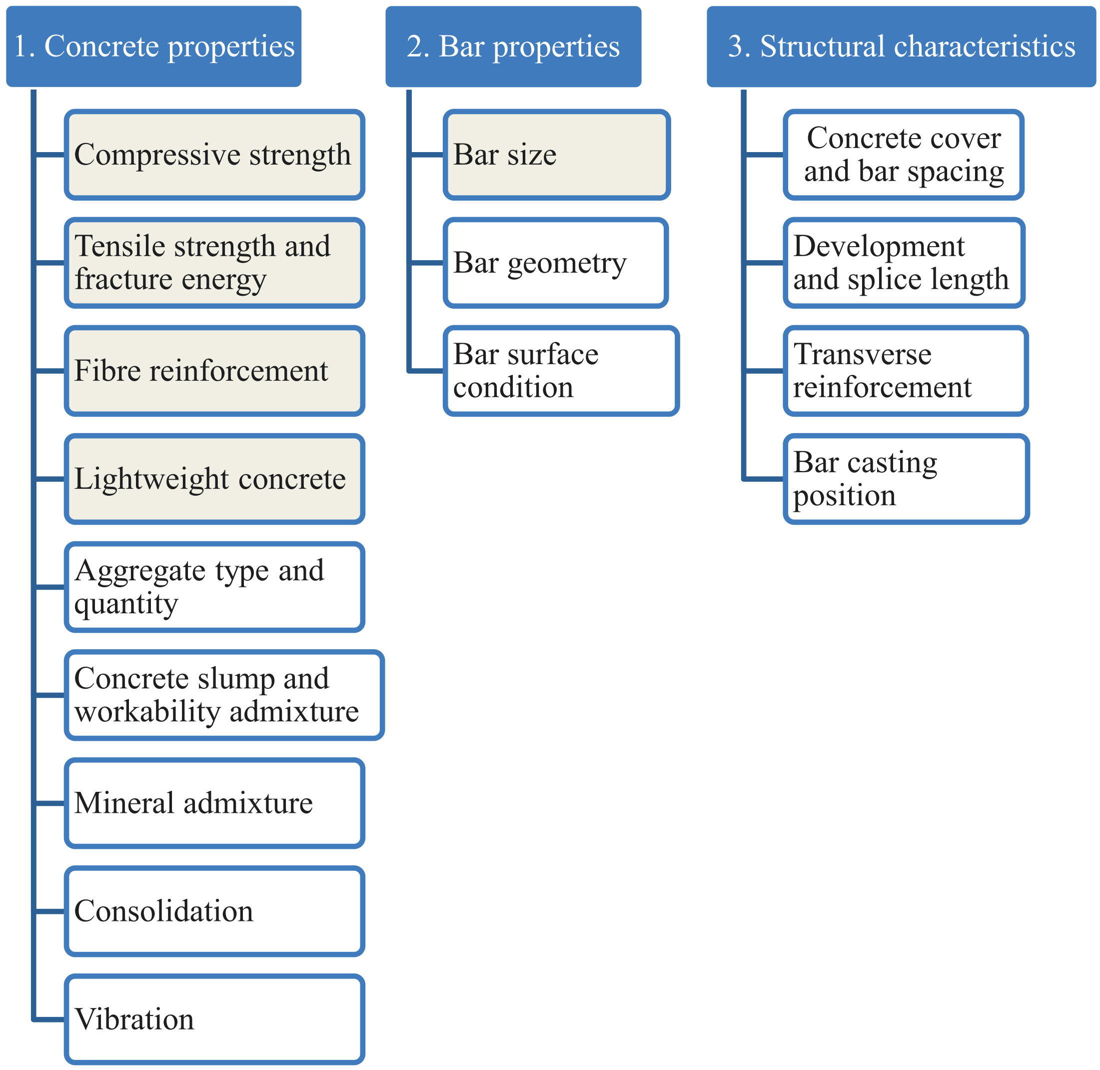


Figure 3.5 Factors affecting bond property of reinforced concrete members

3.3.2 Lightweight concrete

Compared to normal weight concrete, limited and in some cases lacking information is available about the bond performance of lightweight concrete. Although there are numerous advantages of using lightweight aggregate concrete, this lack of information about bond properties could result in non-acceptance of this material for structural applications. The ever-increasing use of lightweight aggregate concrete due to enhancements in its properties because of addition of some ingredients have made this concrete a common construction material and pushed researchers time and again to evaluate its structural and mechanical properties.

Since lightweight aggregates have lower strength, therefore concrete made from it also possesses lower mechanical properties like tensile strength and fracture energy and also the bond strength when compared with the normal weight concrete of same strength class [4]. There are several studies that support these findings that lightweight aggregate concrete has lower bond strength and should have larger development/bond length than the normal weight concrete. Robins and Standish [136] for example compared the bond behaviour of lightweight aggregate concrete and normal weight concrete in presence of lateral pressure. The lightweight concrete was made from Lytag and pull-out specimens were used in the experimental work. Their study concluded that in splitting mode of failure, pull-out specimens made from normal weight concrete had 10 to 15% higher bond strength than the lightweight concrete. They observed that with increase in lateral pressure the difference in bond strength between the two concretes also increased and that nature of failure changed from splitting to pull-out. Test results showed that normal weight concrete had more than 40% bond strength than the lightweight concrete at higher lateral pressure level.

For concretes having compressive strength lower than 40 MPa, Chen et al. [137] observed that bond strength of lightweight concrete was lower than the normal weight concrete. They used expanded clay in their experimental program for making lightweight aggregate concrete. The tested pull-out specimens had dimensions of 150 x 150 x 150 mm and were reinforced with 19 mm pull-out bar.

Results of the 24 pull-out tests conducted by Lachemi et al. [138] show that the two different lightweight self-compacting concretes, one made from expanded shale and the other from blast furnace slag had 16 to 38% lower bond strength than the self-compacting concrete made from normal weight aggregates. They used two different bond/embedment lengths i.e. 100 mm and 200 mm in their study, and for both these bond lengths normal weight concrete had better bond resistance than all other concretes, followed by lightweight aggregate concrete made from expanded shale. The lowest bond strength was observed to be that of all-lightweight concrete made from blast furnace slag aggregates. Figure 3.6 shows the comparison of normalized bond strength (ratio of bond strength to the square root of compressive strength) of normal and lightweight self-compacting concretes.

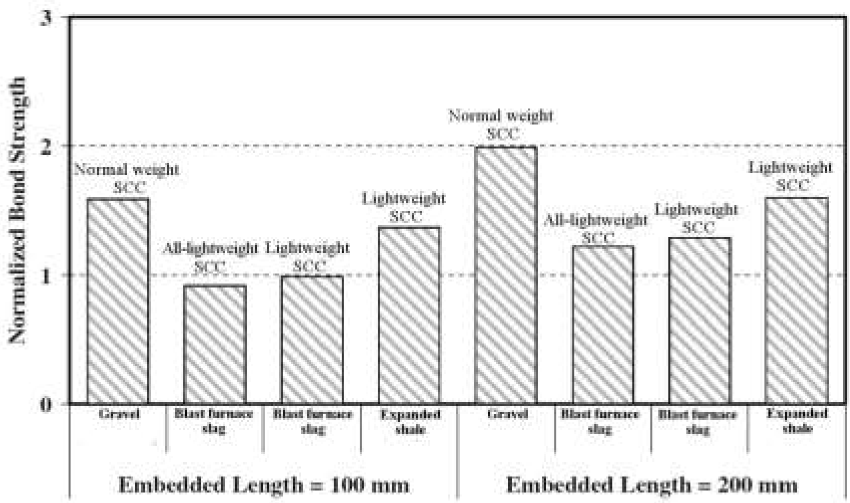


Figure 3.6 Comparison of bond strength of lightweight and normal weight self-compacting concretes (after [138])

Collepardi and Colladi [139] investigated different properties including bond strength of lightweight aggregate concrete and of normal weight concrete. The lightweight aggregate concrete was made from expanded clay. They found that it is possible to achieve higher bond strength for lightweight concrete than the normal weight concrete by using chloride free superplasticizing admixture (Rheomac 877) based on sulphonated nepthalene formaldehyde polymer and higher amount of cement content. However, without admixture, their results showed steel-concrete bond value of 9 MPa for lightweight concrete which is 40% lower than the bond strength value of 15 MPa for conventional concrete at 28 days of testing.

Present-day studies [140], [141] also support these results. More recently 72 pull-out tests were conducted by Kaffetzakis and Papanicolaou [140] for understanding the bond behaviour of lightweight self-compacting concrete. They have reported an increase in the bond strength of test specimens (containing 12 mm pull-out bar) from 15.47 MPa to 22.46 MPa (an increase of 45%) when only pumice fine sand was replaced with the normal river sand. This increase was more pronounced - up to 70%, when both coarse and fine aggregates (both pumice aggregates) were replaced by normal weight aggregates. Besides the above mentioned literature, similar findings have been reported by other researchers [142]–[144] in past.

There are also several studies which have reported different results from those mentioned earlier. According to Chen et al. [137] for concrete with compressive strength above 40 MPa, lightweight concrete performed better in bond test results than the conventional concrete. They attribute this to the higher mortar strength, whereas lower aggregate strength governed the bond strength of concretes having compressive strength lower than 40 MPa. However results of Mor [144] contradict with this observation as his test results show that even for concretes having compressive strength as high as 70 MPa, bond strength of lightweight aggregate concrete was lower (65%) than conventional concrete. His findings were based on test results of an experimental program intended for investigating the effect of condensed silica fume on bond strength. He used pull-out specimens 76 x 76 x 508 mm in size, embedded with 19 mm bar in his work. Also, no any significant influence of silica fume on bond is reported by him.

Mitchell and Marzouk [145] performed bond tests on 72 pull-out and push-in specimens made from high strength lightweight aggregate concrete with average compressive strength of 83 MPa. They compared their results with the results of a previous study [146] conducted with similar testing conditions on high strength normal weight concrete and observed that high strength lightweight concrete had similar or slightly higher bond strength than the high strength normal weight concrete. Therefore they are of the opinion that the use of factor 1.3 by ACI-318 [32] in design equation for development length is not justified for lightweight concrete. Studies by Clarke and Birjandi [147], Martin [148], and Shideler [149] also reported either comparable or higher bond strength values for lightweight concrete than the normal weight concrete.

Observations made in this section are difficult to ascertain that bond strength of lightweight concrete is lower, comparable or better than normal weight concrete because of the different testing parameters and characteristics associated with mix designs.

3.3.3 Fibre reinforcement

As mentioned earlier in Section 2.1.1, use of fibres is not new to mankind and since 1960s their use as an ingredient in concrete has seen remarkable growth. Among the many commonly used fibre-reinforced concrete (FRC) types, like, glass fibre-reinforced concrete, synthetic fibre-reinforced concrete etc. steel fibre-reinforced concrete (SFRC) is used widely due to better performance of steel fibres than other fibre types in enhancing tensile strength, resistance to splitting cracks, shear, toughness and flexural strength of concrete [150]–[153].

Since code requirements for bond length are governed by splitting failure of concrete, therefore, it is believed that by enhancing the tensile strength of concrete using fibres which should improve the splitting failure of concrete, bond strength of the concrete can also be improved. According to Cairns and Plizzari [154], even at lower fibre volume, fibres can improve bond capacity. This improvement is also the result of better confinement condition of FRC; when distributed randomly in sufficient volume, the discrete fibres not only function as longitudinal but also as a transverse reinforcement thus effectively confining the concrete and would thus require higher force to pull-out reinforcing bars from such a concrete [4], [154]–[156].

Campione et al. [88] explored the bond strength of LWFC made from expanded clay aggregates and hooked-end steel fibres. In his study fibres were added to concrete in quantities of 0.5, 1 and 2% V_f and a 12 mm diameter bar with two different bond lengths i.e. 60 mm and 96 mm was used as a pull-out bar. The pull-out test results of specimens having squat geometry show that addition of 1% V_f of steel fibres resulted in an increase in bond strength from 15.53 MPa to 19 MPa – an increase of about 22%. Whereas increase in bond strength was about 39% when volume of fibres added was increased up to 2%. He concluded that using steel fibres in sufficient quantity, a better post cracking behaviour for lightweight concrete (see Figure 3.7) can be obtained which is otherwise characterized by a brittle failure in absence of any such reinforcement.

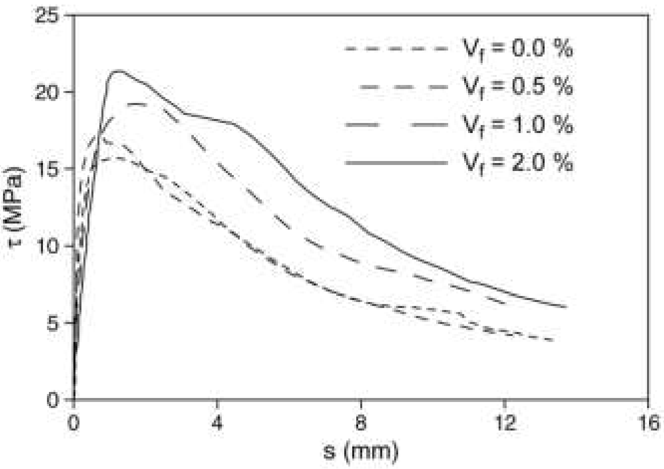


Figure 3.7 Effect of hooked-end steel fibres on bond-slip behaviour of LWFC (after [88])

Besides various other parameters, Garcia-Taengua et al. [157] also studied the effect of quantity of steel fibres, fibre length and fibre slenderness on the bond capacity of NWFC. In their study, pull-out specimens with four different bar sizes (8, 12, 16 and 20 mm) and concrete compressive strengths in the range of 32 to 48 MPa were used. They observed only a limited improvement in bond strength after fibre addition of up to 70 kg/m^3 and found that shorter fibres were more effective in improving the bond strength than the longer fibres (Figure 3.8). Their reasoning for this is that upon loading, micro cracking around the bar is so progressed and developed that bond strength of the specimen was reached, but the long fibres were still not activated. In another study by Ezeldin and Balaguru [158], decrease in bond strength was recorded when steel fibres were used in quantity of 0.25% by volume, however at higher fibre contents of 0.5 and 0.75% an improvement of 18% was observed.

Harajli et al. [159] obtained 26% and 33% improvement in the splitting bond strength of NWFC at fibre dosage of 1% and 2% by volume fraction respectively. In their study hooked-end steel fibres having aspect ratio of 60 (l_f = 30 mm, d_f = 0.5 mm) and bond length equal to 5 times the diameter of pull-out bar was used. The pull-out bars had diameter of 16, 20, 25 and 32 mm. They tested 32 small scale beam specimens to study the local bond stress-slip behaviour of plain and fibre-reinforced concrete. They observed that addition of fibres also improved the ductility of bond failure and concluded that compared to the beam specimens, pull-out testing method underestimates the bond strength of deformed bars in tension.

Guneyisi et al. [160] tested the bond strength of fly ash lightweight aggregate concrete using pull-out specimens with dimensions of 150 x 150 x 150 mm . They studied the effect of fibre volume fraction and aspect ratio of steel fibres on bond strength. Hooked-end steel fibres with four different fibre volume fractions (0.35, 0.7, 1 and 1.5%) and three different aspect ratios (55, 65 and 80) were used in their work. They observed that, although specimens casted from fibres with higher aspect ratios had better bond strength, but compared to the parameter of fibre volume fraction, effect of aspect ratio was less significant. At maximum fibre dosage of 1.5% bond strength of specimens improved by 67.7, 73.5 and 78.6% for fibre aspect ratios of 55, 65 and 80 respectively.

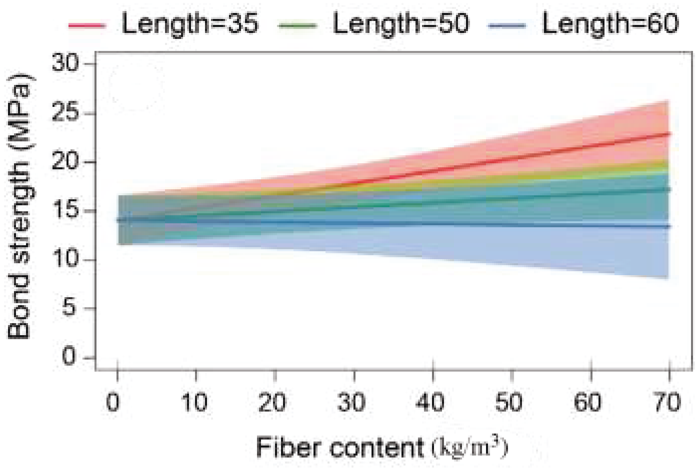


Figure 3.8 Effect of fibre length on bond strength (after [157])

Effect of different types of fibres on bond strength has also been reported by various researchers [161]–[163]. According to these studies, although, in most cases, the bond strength was found to increase with other types of fibres as well, these fibres were however less effective than steel fibres. Türker et al. [161] for example studied the effect of three fibre types (steel, polypropylene and polyvinyl alcohol fibres) at elevated temperatures on the bond characteristic of fibre-reinforced concretes. Pull-out test results from their work show that bond strength of steel fibre reinforced concrete at elevated temperature of 800 ^{o}C decreased drastically from 11.6 MPa to 5 MPa. However, at both the room temperature and elevated temperature it was higher than the bond strength of specimens made from propylene and polyvinyl alcohol fibres.

In an experimental study conducted by Yerex et al. [163], effect of polypropylene fibres on bond strength was studied. Test results from their work show that bond strength of polypropylene fibre-reinforced concrete was similar to the normal weight concrete. Harajli, and Salloukh [164] also investigated the effect of multiple parameters (including type of fibres) on development strength of reinforcing bars in tension by testing 15 full scale beams. They observed that addition of 0.6% V_f of polypropylene fibres improved the ductility of bond failure but compared to steel fibres, these were less effective in enhancing the development strength of the deformed bars. They also observed that in comparison to plain unconfined concrete, bond strength of concrete reinforced with 2% by volume of hooked-end steel fibres was 55% higher.

Summarizing the test results of different researchers, Kim et al. [165] observed that irrespective of type of failure, whether splitting or pull-out, bond strength of the specimens increased as the fibre content increased for LWC (Figure 3.9). Study by Harajli et al. [166] however reported no significant improvement in bond strength of specimens that had pull-out failure.

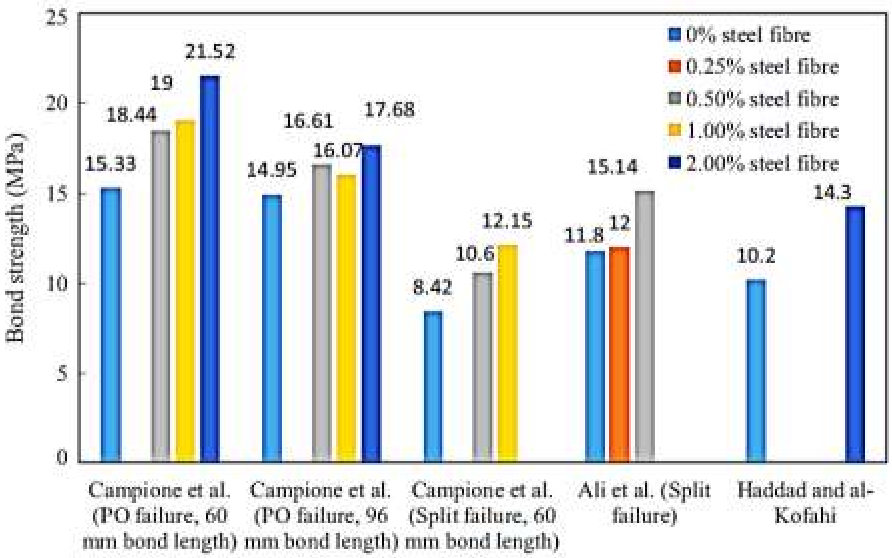


Figure 3.9 Effect of steel fibres on bond strength of lightweight concrete (Kim et al. [165])

3.3.4 Compressive strength

It is acknowledged in many studies [138], [167] that bond strength increases with increase in compressive strength. Kim et al. [167] used compressive strength and bond length as test parameters in their experimental programme for studying bond properties of artificial lightweight aggregate concrete. They tested 144 pull-out specimens made from bottom ash (by product of coal combustion) lightweight aggregate concrete to measure the bond strength. Their test results performed on 16 mm diameter bar are presented in Figure 3.10 which shows increase in bond strength as the compressive strength increases. According to the authors, increases in compressive strength induces more confinement around the bar, resulting in improved bond strength.

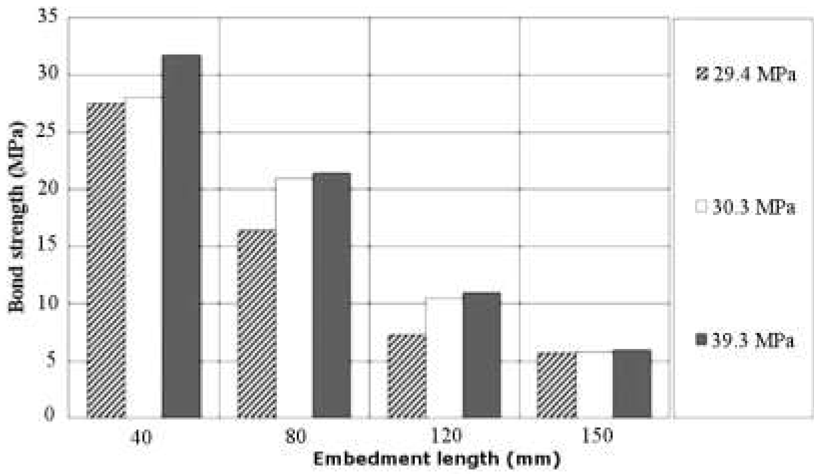


Figure 3.10 Effect of compressive strength on bond strength of lightweight concrete (after [167])

Almost all known bond expressions proposed by researchers contain compressive strength parameter. Esfahani et al. [168] for example, after some modification to the Tepfer's [134] partly cracked thick cylinder theory proposed an expression (3.2) for estimating bond stress where it is proportional to the square root of compressive strength of concrete.

$$\tau_{\mathrm{b}} = 2.7\sqrt{f_c'}\left[\frac{\dfrac{c}{\phi}+0.5}{\dfrac{c}{\phi}+3.6}\right] \text{MPa} \tag{3.2}$$

Representation of effect of concrete properties on bond strength by $\sqrt{f_c'}$ is also existent in expressions proposed by Harajli [169], Orangun et al. [170].

The design expression of ACI-318 (see Eqn. (3.8)) and also that of fib Model Code 2010 (see Eqn. (3.11)) indicate that bond strength increase in proportion to the square root of compressive strength. However, according to ACI committee on bond (ACI-408) [4], this representation for concretes having strengths higher than 55 MPa is not adequate and based on the database of 171 beam test results finds out that concrete's contribution to bond is best represented by $\sqrt[4]{f_c'}$.

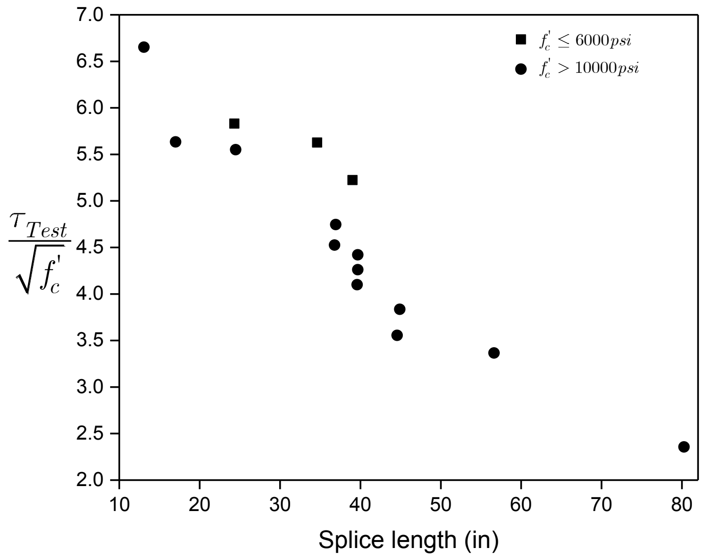


Figure 3.11 Effect of compressive strength and splice length on normalized bond strength (after [171])

Note. 1 in = 25.4 mm, 1 MPa = 145 psi

This phenomenon of lower normalized bond strength for higher strength concrete is best explained in the work by Azizinamini et al. [171], where authors are of the view that since tensile strength of concrete does not increase in proportion to its bearing strength, therefore, concrete surrounding the bar is more prone to splitting than to crushing. This leads to bond failure in splitting by first few lugs, before all lugs could be activated resulting in lower normalized bond strength for higher concrete strength as shown in Figure 3.11.

3.3.5 Fracture energy

It is now generally agreed that fibres are activated once the cracks have developed in concrete matrix and are effective way of improving fracture energy; in other words fibres are ineffective in enhancing the tensile strength of concrete before cracking or the first-cracking stress [162]. Shah and Naaman [172] used two different types of fibres in their work to see their effect on mechanical properties of reinforced mortar and observed no improvement in first cracking stress of fibrous composites as shown in Figure 3.12. Similar observations have also been reported in other studies by Nanni [173] and Naaman [174]. Also ACI Committee 408 on Bond [4] is of the view that fracture energy rather than the tensile strength has more effect on bond strength. Findings of the committee suggest that since bond strength is represented better by $\sqrt[4]{f_c'}$ rather than square root of compressive strength (tensile strength); therefore, tensile strength alone is not the significant parameter in improving bond strength. And since bond strength increases at slower rate during increase in the compressive strength, it is therefore better to enhance the fracture energy using fibres to increase the bond strength.

3.3.6 Bar size

Smaller bars perform better than the larger diameter bars when their performance is measured in terms of bond stresses [4]. Ezeldin and Balaguru [158] in their experimental work used three different bar sizes (9, 16, 19 and 25 mm) as pull-out bars. They studied the bond behaviour through modified pull-out specimens made from normal and high-strength fibrous concretes mixes. For both the plain and fibrous concrete mixes, they observed higher bond strengths as the bar size reduced (see Figure 3.13). They also found that fibres were slightly more effective in enhancing bond capacity for larger bar sizes. Authors attribute this phenomenon to the fact that in case of larger bar sizes bond failure is dominated more by splitting than pull-out and therefore fibres can play their part of arresting these splitting cracks and thus raise the bond capacity.

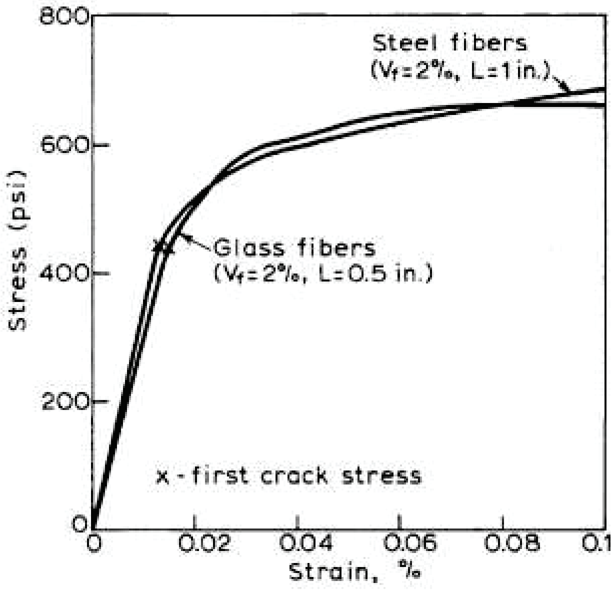


Figure 3.12 Tensile stress-strain curve for fibre reinforced concrete (after [172])

Note: 1 in = 25.4 mm, 1 MPa = 145 psi

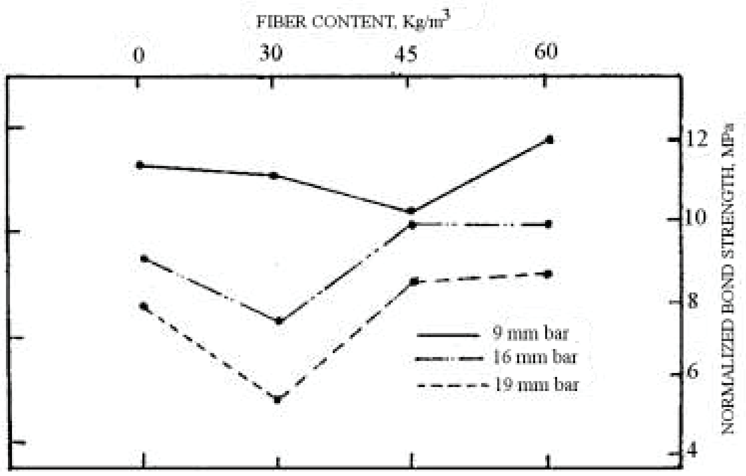


Figure 3.13 Bond strength of different bars against fibre content (after [158])

Besides lower bond strength compared to the smaller bars, larger diameter bars also attain higher slip values. According to Barbosa et al. [175], since larger bars have bigger transition zone around them which is considered to be the weaker link between reinforcement and concrete, therefore concrete around these bars gets crushed more easily allowing higher slippage at lower stress values (see Figure 3.14). For a similar development length larger diameter bars attain higher bond force due to bigger cover and bar spacing requirement for such bars, however since bond stress is also function of bar area, therefore increase in bond force at failure is slow as the bar size increases and for this reason larger development lengths are needed for bigger diameter bars [4].

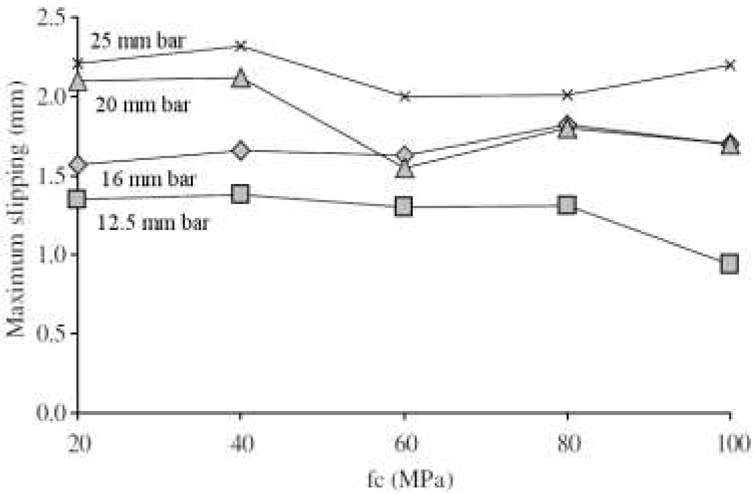


Figure 3.14 Maximum slip values for various bars against concrete strength (after [175])

3.4 Consideration of Bond in Normative Rules

3.4.1 ACI-318

Present design provisions of ACI Code [32] for the anchorage length or development length of straight reinforcing bars in tension are derived from the work of Orangun, Jirsa and Breen [170], who using statistical techniques developed following expressions.

$$\frac{\tau_{\mathrm{b}}}{\sqrt{f_c'}} = 0.10 + 0.25\frac{c_{\min}}{\phi} + 4.15\frac{\phi}{l_{\mathrm{b}}} \tag{3.3}$$

$$\frac{\tau_{\mathrm{b}}}{\sqrt{f_c'}} = 0.10 + 0.25\frac{c_{\min}}{\phi} + 4.15\frac{\phi}{l_{\mathrm{b}}} + \frac{A_{\mathrm{tr}}f_{\mathrm{yt}}}{41.5sn\phi} \tag{3.4}$$

Equation (3.3) is used to describe the bond strength of bars not confined with transverse reinforcement, whereas, equation (3.4) is used for bars confined by transverse reinforcement

ACI committee 408 [4], finds the representation of effect of concrete strength by $\sqrt{f_c'}$ inadequate for concrete strengths greater than 55 MPa and observes that concrete contribution to bond strength is best characterized by $\sqrt[4]{f_c'}$ as shown in following expressions.

$$\frac{\tau_b}{f_c'^{1/4}} = \frac{\left[1.43 l_b \;\; C_{min} + 0.5\phi \;\; + 57.4 A_s\right]\left(0.1\frac{C_{max}}{C_{min}} + 0.9\right)}{\pi l_b \phi} \tag{3.5}$$

$$\frac{\tau_b}{f_c'^{1/4}} = \frac{\left[1.43 l_b \;\; C_{min} + 0.5\phi \;\; + 57.4 A_s\right]\left(0.1\frac{C_{max}}{C_{min}} + 0.9\right) + \left(8.9 t_r t_d \frac{N A_{tr}}{n} + 558\right) f_c'^{1/2}}{\pi l_b \phi} \tag{3.6}$$

The design expression for anchorage length in ACI-318, described here as equation (3.7) is obtained by solving above equation (3.4). The detailed transformation of Eqn. (3.4) to Eqn. (3.7) and the modification of constants can be found in the report of ACI-408 Committee [4] on bond and development of straight reinforcing bars in tension.

$$\frac{l_b}{\phi} = 0.9 \frac{\psi_t \psi_e \psi_s f_y}{\lambda \sqrt{f_c'} \left(\frac{c + K_{tr}}{\phi}\right)} \tag{3.7}$$

Where, l_b, c, and K_{tr} are bond length, concrete cover and factor representing contribution of transverse reinforcement. In above equation ψ_t, ψ_e, ψ_s and λ represent the effects of bar location, epoxy coating, bar size factor and factor for lightweight concrete respectively.

Because of this lower bond strength of lightweight concrete the design provisions require for larger development length for lightweight concrete to compensate for the lower tensile strength of aggregates. ACI 318 for example incorporates a factor λ (λ = 0.75) in its design equation for bond length. If the reinforcement to be anchored is so placed that there is more than 300 mm of fresh concrete below it than bar location factor (ψ_t) shall be taken as 1.3, whereas for all other cases its value shall be 1. Similarly for concrete reinforced with epoxy coated reinforcement having size smaller than or equal to 19 mm diameter, values of ψ_e and ψ_s are, 1.5 and 0.8 respectively. For conventional concrete, reinforced with uncoated bars, having diameter greater than 19 mm, all such values shall be taken as 1.

Manipulation of equations (3.1) and (3.7) yields the ultimate design bond stress as;

$$\tau_{\mathrm{bd}} = \frac{1.11\lambda\sqrt{f_c'}\left(\dfrac{c+K_{\mathrm{tr}}}{\phi}\right)}{4\psi_{\mathrm{t}}\psi_{\mathrm{e}}\psi_{\mathrm{s}}} \tag{3.8}$$

3.4.2 fib Model Code

For Model Code 2010 [33], design ultimate bond strength equation for ribbed bars (Eqn. (3.10)) is derived from the semi-empirical relationship (Eqn. (3.9)) for reinforcement stress (f_{stm}), obtained from the calibration of over 800 test results.

$$f_{\mathrm{stm}} = 54\left(\frac{f_c}{20}\right)^{0.25}\left(\frac{20}{\phi}\right)^{0.2}\left(\frac{l_{\mathrm{b}}}{\phi}\right)^{0.55}\left[\left(\frac{c_{\mathrm{min}}}{\phi}\right)^{0.33}\left(\frac{c_{\mathrm{max}}}{c_{\mathrm{min}}}\right)^{0.1} + 8K_{\mathrm{tr}}\right] \tag{3.9}$$

$$\tau_{\mathrm{bd}} = \tau_{\mathrm{b,0}}\left[\left(\frac{c_{\mathrm{min}}}{\phi}\right)^{0.5}\left(\frac{c_{\mathrm{max}}}{c_{\mathrm{min}}}\right)^{0.15} + kK_{\mathrm{tr}}\right] \tag{3.10}$$

Where, $\tau_{b,0}$ is basic bond strength considered as an average stress over the bond length of reinforcing bar and is given by following equation:

$$\tau_{\mathrm{b,0}} = \frac{\eta_1\eta_2\eta_3\eta_4}{\gamma_{\mathrm{c}}}\left(\frac{f_{\mathrm{ck}}}{20}\right)^{0.5} \tag{3.11}$$

Coefficients η_1, η_2, η_3, and η_4 represent the effects of rebar geometry, bond condition, bar size and characteristic strength of reinforcement being anchored respectively. For ribbed bars η_1 = 1, whereas for plain bars this value is 0.9. Values of other coefficients shall be taken as unity as well if their size is less than or equal to 20 mm, have characteristic yield strength of 500 MPa and are laid in such a way that good bond conditions are assumed. Values for all other cases can be found in the Model Code 2010 [33].

In equation (3.10), k is effectiveness factor, whose value depends on the reinforcement details; $K_{\mathrm{tr}} = \dfrac{n_1 A_{\mathrm{st}}}{n_{\mathrm{b}}\phi s_{\mathrm{v}}}$ is the transvers reinforcement factor. From equation (3.10), the design anchorage length may be calculated as follows:

$$l_{\mathrm{b}} = \alpha_4 \frac{\phi \sigma_{\mathrm{sd}}}{4 f_{\mathrm{bd}}} \geq l_{\mathrm{b,min}} \tag{3.12}$$

For design bond length, it shall be ensured that it is not less than the minimum required length, such requirements are highlighted in the Model Code, one such condition is that $l_{\mathrm{b,min}}$ > 200 mm. The value of coefficient α_4 depends upon the percentage of reinforcement lapped or anchored within 0.65 l_{b}.

3.4.3 Eurocode 2

The ultimate design bond stress in Eurocode 2 [176] is given by following expression;

$$\tau_{\mathrm{bd}} = 2.25 \eta_1 \eta_2 f_{\mathrm{ctd}}$$

Where; τ_{bd} is ultimate bond stress (design) and f_{ctd} is design value for concrete tensile strength. η_1 and η_2 are the coefficients which depend upon bond conditions and bar diameter respectively. For good bond conditions η_1 is 1 and 0.7 for all other cases. For bar sizes, having diameter equal or smaller than 32 mm, $\eta_2 = 1$ and for higher bar sizes, $\eta_2 = (132 - \phi)/100$. The value of f_{ctd} is calculated using formula;

$$f_{\mathrm{ctd}} = \alpha_{\mathrm{ct}} f_{\mathrm{ctk,0.05}} / \gamma_{\mathrm{c}}$$

Where; γ_{c} is partial safety factor for concrete equal to 1.5 and α_{ct} is the factor for consideration of long term effects on tensile strength and its recommended value is 1.

According to Eurocode 2, assuming constant bond stress (τ_{bd}), the basic anchorage length or development length required $(l_{\mathrm{b,req}})$ to anchor the force $(\sigma_{\mathrm{sd}} A_s)$ shall be calculated as follows;

$$l_{\mathrm{b,req}} = \frac{\phi \sigma_{\mathrm{sd}}}{4 f_{\mathrm{bd}}}$$

3.5 Methods for Experimental Investigation of Bond

Different test arrangements are in practice for studying the bond behaviour (see Figure 3.15), among these pull-out test setup is widely used due to ease and simplicity in making and testing of specimens [4]. Because of these reasons, majority of database on bond is available perhaps from these. Both the ASTM [178] and RILEM [177] test procedure do not mention the use of transverse reinforcement in pull-out tests, nevertheless the confining effect by such reinforcement has been studied by some researchers [145], [179], [180]. Since the concrete surrounding the pull-out bar during testing is under compression as against of flexural tension in real structures; the testing standard was therefore withdrawn by ASTM in year 2000. It is however, an effective method for comparison of bond of different concretes of similar strength, or of reinforcing bars with different rib geometry, reinforced in concrete of similar mix design [177].

Beam specimens and beam-end specimens for bond test can be prepared following the guidelines of RILEM [181] and ASTM [182] respectively. These methods produce realistic test results as the stress state in the concrete and the pull-out bar depicts closely the conditions or concrete members. Attention shall be paid in placing the compressive force away from the test bar, to avoid formation of compression struts, which may alter the actual state stress around bar. ACI [4] suggests that minimum distance between test bar and compressive force should not be less than bond length.

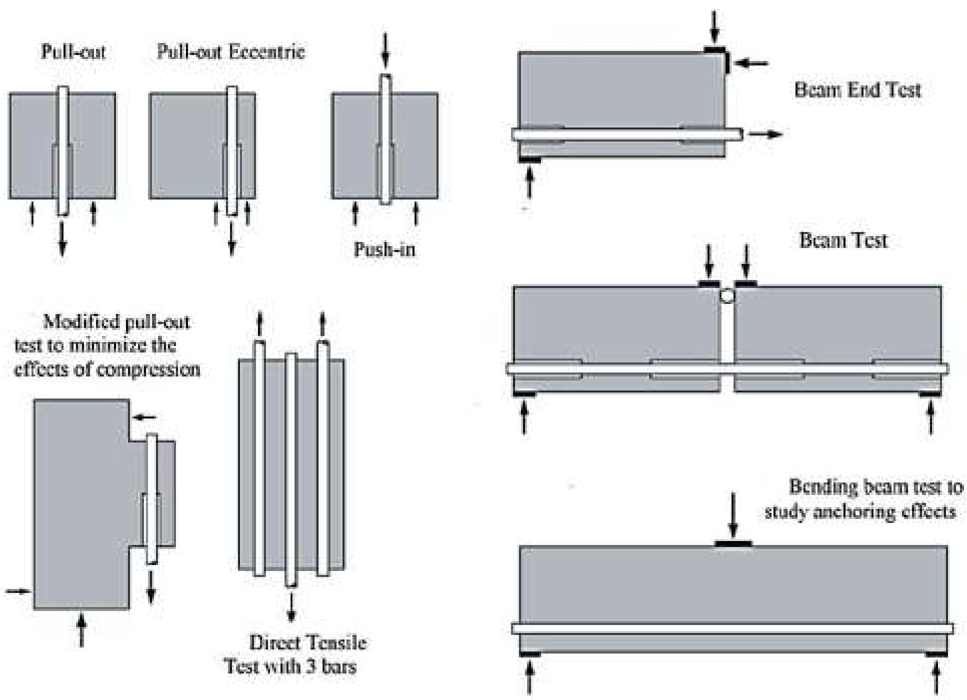


Figure 3.15 Different test setups used for the study of bond between rebar and concrete (modified from [183])

4 Experimental Program

4.1 General Aspects

Main objective of the research program was to evaluate the bond behaviour of Lightweight Fibre-reinforced Concrete (LWFC). However, since design expression for development length for bars in building codes, for example in ACI – 318 are based on the work of bond tests on normal weight concrete; therefore, for analysis and comparison purpose, test specimens of Normal Weight Fibre-reinforced Concrete (NWFC) were also included in experimental program. It would have been unfair, if this comparison between two different types of concretes was made at different compressive strength levels, and for this reason the experimental work included following main tasks;

- Development of concrete mixes for both types of concretes i. e. lightweight and normal weight, which would yield similar strength class at various fibre dosages.
- Investigation of the influence of fibre volume, concrete type and bar size on bond performance of LWFC and NWFC. These parameters are further elaborated in Table 4.1
- Comparison and analysis of test results for both types of concretes with different bond strength expressions of fib-2010 and ACI standards.

For bond strength evaluation, pull-out specimens with varying bar sizes, fibre content were tested and all these bond tests were grouped under LWFC and NWFC series. Also, tests were performed for determination of mechanical properties of concrete, such as modulus of rupture test, splitting tensile strength test, compressive strength and elastic modulus test. Total 72 pull-out tests were performed, these included 36 tests for series of LWFC and 36 for NWFC, other than these, 48 tests for each series were performed for hardened properties. All the pull-out tests and other hardened concrete tests were performed at concrete's age of 28 days. It shall be noted that most of the tests performed for hardened and fresh concrete properties followed the German DIN standards, whereas for bond behaviour, most of the guidelines of pull-out test which are set by RILEM (Réunion Internationale des Laboratoires et Experts des Matériaux, systèmes de construction et ouvrages), the International Union of Laboratories and Experts in Construction Materials were followed.

4.2 Scope of Test Program

Scope of the experimental work included understanding the bond behaviour of lightweight concrete after addition of steel fibres. One of the main question to answer through this research was whether fibres affect the bond strength of concrete or not and if they do then what changes need to be made in expressions on which design equations for bar development length are based. To answer this first question, it was decided to incorporate steel fibres to the reference mix in dosages of 20, 40 and 60 kg/m^3 which corresponds to fibre volume fraction of 0.25%, 0.5% and 0.75% respectively. Secondly, to what extent the difference between the bond performance of LWFC and NWFC exist at each fibre content level, if they have similar compressive strength class at those levels.

Since there are only few experimental investigations about the bond behaviour of reinforcement in LWC, especially for bigger bar sizes [33], therefore, the program included range of bar sizes (10, 16 and 20 mm) to generate database of ultimate bond stresses test results for future codes.

Scope of the work was further expanded to cover the behaviour of normal weight fibre-reinforced concrete for the reasons mentioned in previous section. ACI – 408 [4] recommends decrease in bond strength for LWC by factor λ. Evaluation of bond behaviour of NWFC and its comparison with the results of pull-out tests of LWFC having similar strength class would enable to see if this factor could be adjusted for fibre-reinforced concretes.

Additional tests for hardened concrete properties, apart from main bond tests were included in the program scope to support test findings. These included compression tests, split tensile strength test, flexural beam tests and elastic modulus tests.

4.3 Materials

In following sections, materials used in design mix of both LWFC and NWFC and the reinforcement are described

4.3.1 Cement

Since the major objective was to achieve a comparable strength for both LWFC and NWFC and database for concretes having normal strength, therefore, for both types of concretes (LWFC & NWFC) Ordinary Portland Cement with a strength class of 42.5 (CEM-1/42.5 N) was used. It was supplied by Lafarge Cement GmbH.

4.3.2 Fine aggregate

Natural sand having particle size in the range of 0 – 2 mm and particle density of 2570 kg/m^3 was used as a fine aggregate. The moisture content of the aggregate was determined using the ASTM procedure [184] and was adjusted during mix design stage.

4.3.3 Coarse aggregate

Initial approach while making selection of coarse aggregates for production of LWFC was to opt for a type that had specific gravity closer but slightly higher than that of water for avoiding aggregates' floating issue, and the one which has regular shape. Rationale behind this approach was to be able to produce concrete which is sufficiently lighter so that addition of maximum steel fibre dosage does not increase concrete's density beyond set limit of 1850 kg/m^3, and because of regular shape ease in handling and working with concrete was expected.

For the reasons mentioned above, expanded clay (Commercial name Liapor 6.5), round in shape (Fig. 4.1), having particle density of 1190 kg/m^3 and particle size ranging from 2 to 10 mm was used as a coarse aggregate for production of LWFC. Aggregates had water absorption of 14% and were found within the specified range provided by the supplier of material. For NWFC, gravel having grain size range of 2 – 8 mm was used as coarse aggregate. Other details of the aggregates are given in Table 4.2.

Figure 4.1 Aggregates used in experimental program

4.3.4 Superplasticizer

Polycarboxylate ether based superplasticizer (Glenium ACE 391 (FM)) provided by BASF Construction Solutions GmbH was incorporated as high range water reducing agent.

4.3.5 Fibres

The steel fibres have a length, (l_{f}), usually ranging from 6 mm to 70 mm and an equivalent diameter, (d_{f}), ranging from 0.15 mm to 1.20 mm [185]. Due to their better performance under tensile loading and other advantages over other fibres mentioned in section 2.3.1.3, hooked-end steel fibres were chosen as reinforcement for fibrous mixes and were provided by the company ArcelorMittal. These fibres (see Figure 4.2) were 35 mm long and had diameter of 0.55 mm; developing an aspect ratio $\left(\frac{l_{\mathrm{f}}}{d_{\mathrm{f}}}\right)$ of 0.64 and tensile strength of 1200 MPa.

Figure 4.2 Hooked-end steel fibres used in experimental work

4.3.6 Reinforcement

All three bar sizes (10, 16, 20 mm) used in the research work were tested in laboratory to verify their strength and geometric profiles and results of these tests that include yield stress, ultimate strength, bar diameter and relative rib area (ratio of bearing area to shearing area of bar) are presented in Table 4.4 and were found in accordance to the specifications provided by supplier, the Stahl Center Leipzig, GmbH. Figure 4.3 not only shows the rib pattern of these bars but also their load-displacement profiles, obtained after testing three specimens for each bar.

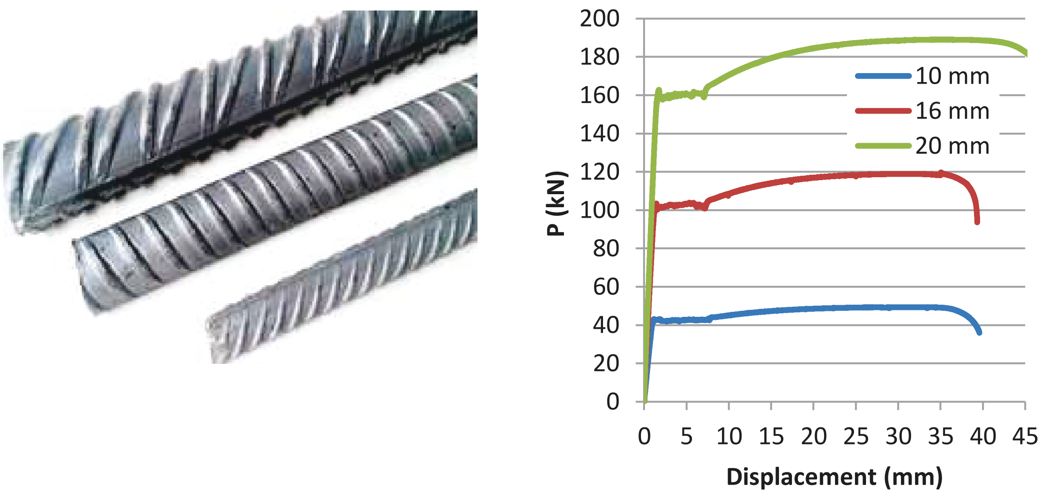


Figure 4.3 Pull-out bars along with their load-displacement profile

4.4 Mix Design Approach

To cover the test parameters, there were four different types of concretes used i.e lightweight concrete (LWC), normal weight concrete (NWC), lightweight fibre-reinforced concrete (LWFC) and normal weight fibre-reinforced concrete (NWFC). But for ease, LWC and NWC were grouped under series LWFC and NWFC respectively with suitable notations indicating that theses had zero fibre content.

In order to be able to compare these concrete types, the strength class (compressive strength) of these should be similar. It would also give fair idea as to which fibre content in LWFC yields bond strength values that are comparable / equal to NWC. Therefore, one of the main challenges was to design mix for LWFC and also for NWFC (both having coarse aggregates different from the other), in a way that at every subsequent fibre content level, both these mixes have similar compressive strength class.

Initial quantities for trial mixes were calculated ACI [186] procedure. The approach adopted for bringing the difference in compressive strength of LWFC and NWFC to less than 5 MPa was to use wherever possible similar material quantities and material sizes in order to lessen the number of factors influencing concrete strength. For example, same type of cement, fine aggregate, fibres and superplasticizer were used for both mixes, moreover, there were marginal differences between the quantities of cement and sizes of coarse aggregates used for concrete mix of LWFC and NWFC. Quantity of cement used for LWFC was 360 kg/m^3 and for NWFC it was 350 kg/m^3, whereas lightweight aggregate had particle size range of 2-10 mm and for gravel which was used for producing NWFC, this range was from 2 – 8 mm. The higher water absorption percentage of lightweight aggregate (14.36%) influenced the water demand; therefore, the total water demand for LWFC was 25 kg/m^3 higher than NWFC. However, the effective water (water needed for hydration) was higher for NWFC with effective w/c of 0.45 compared to 0.35 for LWFC. Since lower w/c ratio results in higher compressive strength of concrete, therefore it was expected that this factor would bring the compressive strength of LWFC closer to NWFC which otherwise would not be possible without increasing cement content.

During trial mix stage, it was realized that fibres at maximum dosage seriously affected workability of concrete, especially handling of NWFC became difficult. For this reason, superplasticizer in a small amount of 0.5% was introduced in every mix. Details of the mixes for both types of concretes are listed in Table 4.3.

4.5 Mixing and Placing of Concrete

A rotating type drum mixer having capacity of 500 litres (0.5 m^3) was used for production of all batches of concrete. After batching of the constituting materials, they were added in the mixing machine in following order;

Coarse aggregate was added first, followed by the fine aggregates and the water calculated based on aggregates' (both fine and coarse aggregates) water absorption capacity, these were then allowed to get uniformly mixed during allocated time of one minute. Later cement was added and immediately after that remaining water which also contained superplasticizer was poured, making sure that it covered whole mix in the drum. The mix was given one minute to mix properly and then steel fibres were added. To avoid balling of fibres in concrete mix, they were fed in a rain type fashion, this technique although costs some time at maximum fibre content level, but was necessary to avoid balling of fibres. Mix was then allowed to rest for 3 minutes, followed by additional 2 minutes of mixing before finally being discharged. The whole procedure on an average after addition cement and water took about 9 minutes.

Fresh concrete tests were performed using ASTM method [59] before placing it into the formwork. Fresh concrete density for all mixes was determined using 0.003 m^3 cylindrical mould as shown in Figure 4.4. The mould was filled in a single layer and was then externally vibrated at frequency of 115 Hz In current experimental work guidelines of German standard DIN EN 12350-5 [45] were followed to determine the workability of both types of concrete i.e. NWFC and LWFC (Figure 4.5). This test method was preferred over other methods due to its ease, simplicity and application. For example, compared to the ASTM standard, German DIN standard uses same testing method for workability measurement of concretes with or without fibres, making it suitable to quantify the effect of fibres.

All the pull-out specimens (09 specimens) and specimens for hardened concrete properties (12 specimens) for single fibre dosage were casted on the same day. Scoops were used to place the concrete into the formwork after which they were vibrated externally on a vibration table at a frequency of 115 Hz. External vibration (through vibration table) was used for compaction during concrete placement to avoid fibre alignment, also with internal vibrator, disturbance of pull-out bar in smallest pull-out specimen could not have been avoided.

Before placing the concrete in formwork, it was properly cleaned and lubricated with oil (Figure 4.6); extra care was taken to avoid any contact of deformed bars with oil during lubrication process. All these specimens were then covered with plastic sheets and allowed to harden. On the next (after 24 hours) they were demoulded and were wrapped with 3 layers of plastic foils and then placed in a climate room (see Figure 4.7) where continuous humidity and temperature levels were maintained at 65% and 20 ^{o}C.

Figure 4.4 Fresh concrete density measurement

(a) (b)

Figure 4.5 Measurement of slump flow (a) NWFC (b) LWFC

Figure 4.6 Formwork ready before concrete placement

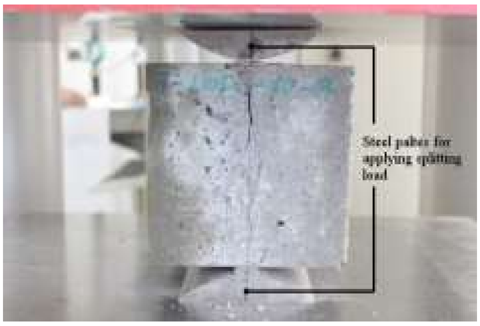
Figure 4.7 Specimens placed in climate control room

4.6 Testing for Material Properties

Compressive strength, splitting tensile strength test and elastic modulus tests were performed in the similar testing machine with some minor setup arrangements. The machine had capacity of applying load maximum up to of 5000 kN. In case of splitting tensile strength test, additional steel plates designed for directing the splitting load at the centre and across the specimen length were placed on both of its sides (top and bottom side) as shown in Figure 4.8(b). Measurement of elastic modulus in this machine was possible with the help of instrument supported by jig attached to the specimens. The vertical displacement during load application was recorded by this instrument and transferred to processing unit via data cable (Figure 4.8 (c)). Cylinders having dimensions of 100 x 200 mm were used for elastic modulus tests, whereas for splitting tensile strength and compressive strength tests, cubes of dimensions 150 mm x 150 mm x 150 mm were used. True dimensions of these specimens were also measured with the help of scale (see Table 5.4 to Table 5.9) and thus machine readings for these tests were corrected.

Few of the issues while comparing the flexural performance of the LWFC and NWFC have been the norms of using different testing standards, specimen sizes and loading rates. All of these factors affect the results and therefore question on the validity of test results may arise if these are ignored. Results of a study [187] on effect of loading conditions and specimen size showed different behaviour of fibrous specimens under flexure for different loading environment and concluded that third-point loading arrangement is more appropriate for assessing flexural and toughness capacity compared to the centre-point test setup. For the current experimental program, therefore, similar testing environment, specimen size, and material selection (except coarse aggregates) were ensured. Experimental program involved testing of 24 beams (150 x 150 x 700 mm) under flexure using four-point loading test setup. The test setup is preferred over centre point loading method [188], as the failure crack is not forced to initiate at the centre of the beam where the matrix can have higher strength, rather it is allowed to arise where ever there is weak zone in maximum moment area of the test beam. Beams were tested using deflection controlled machine with four-point loading arrangement as shown in Figure 4.9. Loading rate was increased in steps with increasing deflection as per ASTM [110] guidelines. Two Linear Variable Displacement Transducers (LVDTs) were used – each one on the longitudinal side of the beam for recording centre point deflection and corresponding loading. Loading of specimens continued until they broke or achieved net deflection of 4 mm.

(a)



(b) (c)

Figure 4.8 Test arrangements for (a) compression test (b) splitting tensile strength test (c) elastic modulus test

Figure 4.9 Flexure test setup

Effect of addition of fibres, in quantities of low volume fraction (V_f < 1%), and lower to medium volume fraction (V_f < 2%) on mechanical properties of concretes has been extensively investigated, and in most cases specific concrete with different fibre types or geometry was the focus. Iqbal et al. [54] for example used micro steel fibres in high strength lightweight self-compacted concrete for evaluation of mechanical properties. His results of flexural beam tests show more than 65% improvement in flexural strength at 0.75% volume fraction of fibres. Kim et al. [189] observed deflection hardening behaviour for specimens under flexure at fibre volume as low as 0.4%. He achieved this behaviour by incorporating chemical admixtures and using high strength concrete. Test results of experimental work by Adyin [190] also reported similar behaviour for normal strength concrete beam specimens that were made without using any chemical admixtures at 0.75% fibre volume fraction. For low volume fraction of fibres Soutsos et al. [190] reported deflection softening behaviour for concrete having compressive strength of 32 MPa and observed higher flexural toughness values when steel fibres were used in combination with synthetic fibres.

All of these studies reported results for specific types of concrete with different fibre geometry and type and suggest that it is possible to obtain better flexural behaviour at lower fibre content. Under similar testing environment, at low volume fraction of fibres and for the same concrete strength class, what performance difference exists between LWFC and NWFC, when tested in flexure, however is not extensively reported.

4.7 Pull-out Test Parameters

4.7.1 Concrete type

Current descriptive and design expression for bond strength in ACI and fib codes are based on the test results carried on normal weight concretes. These are then adjusted for lightweight concrete with some factors, for example, ACI – 408 [2] recommends decrease in bond strength for LWC by factor λ. By including LWFC and NWFC as test parameters, it was aimed at to find out how these expressions perform for these types of concrete and whether the λ factor for lightweight concrete could further be adjusted.

4.7.2 Fibre content

There are various studies which have highlighted the effect of fibres on bond properties of different types of concretes and in most cases normal weight concrete has been the subject. Also, these studies either included a specific type of fibre or had fibre dosages above the practical range used in construction. Considering these, hooked end steel fibres' dosage was chosen as a parameter in experimental work.

4.7.3 Bar/Specimen size

As mentioned earlier in section 4.6 that the size of the specimens used in current experimental work was chosen to vary in proportion to the size of reinforcing bars used. Unlike RILEM's [191] specimen design approach, variable concrete cover was chosen for test specimens depending upon bar diameter. Dimensions of the specimens were multiple of 10 of bar diameter, for example, for a 10 mm bar; dimensions of bond specimen were 100 x 100 x 100 mm. Similarly, for 16 mm and 20 mm bar sizes, specimens' dimensions were 160 x 160 x 160 mm and 200 x 200 x 200 mm respectively. Advantage of this approach is that it ensures similar cover to bar size ratio (c/d_b) or concrete confinement for all bar sizes, thus enabling to specifically monitor the effect of fibres on bond behaviour of concrete. Also, for smaller bar sizes, for example 10 mm bar size, concrete cover around the bar as per RILEM guidelines is as high as 95 mm which is also very high.

4.8 Pull-out Specimen Design

Pull-out test method is one of the easiest way for bond strength determination, there are also various test arrangements for evaluation of bond strength that closely relate with stress conditions in actual scenario [192], details of these methods can be found in reports of ACI 408 [4] and fib-2010 [5]. Diagrammatic details of one of these test methods used by Desnerck et al. [193], where the concrete around reinforcing bar is under tension is shown in Figure 4.10. However, production and handling of specimens used in such test arrangements become an issue, especially in conditions where test parameters demand for higher number of specimens. Apart from these reasons, production of specimens at higher quantity would also be not economically viable.

Compared to these tests, concrete surrounding the bar in pull-out test is under compression beside this other issue is of presence of frictional forces which are generated due to bearing stresses of concrete against steel plate supporting the specimen and are responsible for resisting concrete expansion in transverse direction as shown in Figure 4.11. It also shows the bar stress and bond stress profiles, it can be seen that since unlike real scenario concrete is not cracked, therefore bond stresses do not fluctuate i.e. there are no in and out bond stresses [92]

There have been some efforts in past by researchers to overcome these shortcomings of pull-out tests. Aiello et al [194] for example embedded four steel bars in concrete of the specimen and then fixed those bars with a steel plate (see Figure 4.12). By this way the authors avoided the direct contact of specimen with steel plate which otherwise would have induced compressive stresses. However, there is not ample literature/database available with these modified test setups for comparison. For these reasons traditional pull-out test method standardized by RILEM [191] with some minor modification was chosen for current experimental work.

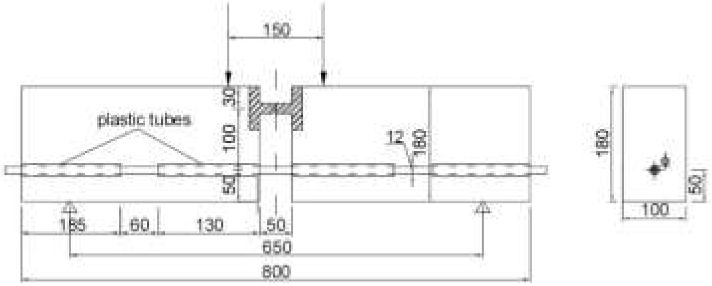


Figure 4.10 Beam test method for bond strength evaluation (after [193])

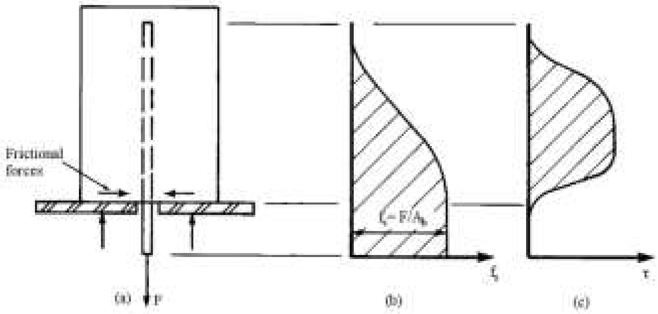


Figure 4.11 (a) Typical pull-out test method (b) Pull-out bar stress profile (c) bond stress profile (after [92])

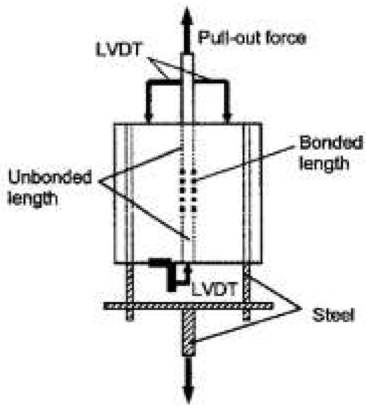


Figure 4.12 Modified pull-out test setup used by Aiello et al. [194]

Experimental program included testing of 72 Pull-out specimens; these included 36 specimens for LWFC and 36 for NWFC. Compared to the standard RILEM pull-out specimens, size of the test specimens was altered in proportion to the bar size. Detailed discussion on this aspect is made in section 4.7.3. Following the RILEM guidelines bond length (l_b) and the bond free portion have to be the multiples of bar diameter. Therefore, this length for 10, 16 and 20 mm bar sizes is 50, 80 and 100 mm respectively. Contact of concrete with reinforcement in bond free length was avoided by using PVC tube, whose ends were sealed using elastic silicon material.

There were three (03) specimens for each bar size in every concrete mix i.e. nine (09) specimens for every mix and total 36 specimens for all the four concrete mixes of LWFC, similarly other 36 specimens for NWFC. Specimens were labelled as LWFC/NWFC-N1-N2-N3, where NI, N2, N3 refer to fibre content in kg/m^3, bar diameter, and specimen number respectively (see Figure 4.13). Figure 4.14 highlights detials of the different sizes of specimens used in present research work.

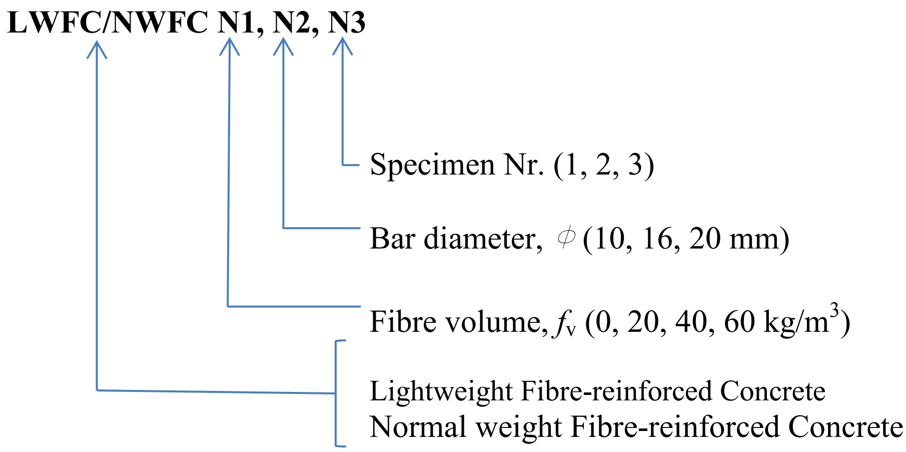


Figure 4.13 Notations indicating type of specimen used in current experimental work

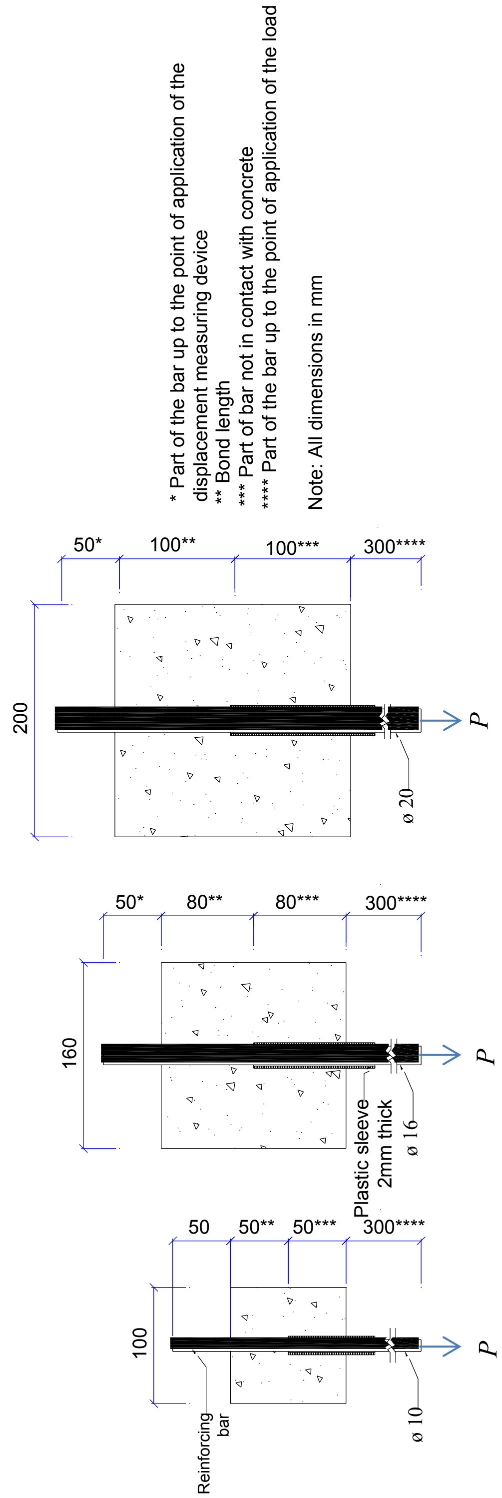


Figure 4.14 Pull-out specimen details used in current experimental work

4.9 Test Setup and Procedure

Specimens were tested in 600 kN displacement controlled machine. The bar from the specimen was pulled out at the rate of 0.005 mm/s, this displacement rate was chosen based on the literature review and experience of previous series of experiments done at HTWK Leipzig, Germany [195]–[198]. This loading rate was held constant for all series of specimens. The relative slip of bar against concrete was measured with the help of six Linear Variable Displacement Transducers (LVDTs) having precision of 1/1000 mm. These were placed on both loaded and free end (3 LVDTs on each side) of specimens, however due to local disturbance, test readings from free end only are used for analysis purpose. Figure 4.15 shows rest of the features of test setup.

Pull-out specimens before start of test were placed concentrically on a steel plate measuring 300 x 200 x 40 mm in dimensions; this plate was supported with the help of screws hanging from another steel plate at top having similar dimensions (Figure 4.15). The upper steel plate was then attached to the top clamp of loading frame with the help of high strength steel bar. After attaching the LVDTs at the mentioned positions and verification of connection and functioning of these, the whole frame including specimen were slowly moved until pull-out bar was correctly fixed within the bottom clamp of loading frame.

Before the start of test, unevenness of contact between specimen and supporting steel plate was removed by using bubble-level and adjusting the screw nuts. The test then begun with a pre-load of 0.5 kN, after which it continued with the selected loading rate. The test continued until the specimen fractured which was expected from brittle nature of concrete or it attained a minimum displacement of 4 mm in case of fibre-reinforced concrete. On an average specimen from beginning of test, till a slip of pull-out bar to 4 mm took 15 minutes, whereas for placement and dismantling of specimens and accessories required approximately additional 20 minutes. Thus, for all the 9 specimens from single batch on a single testing day consumed 5.25 hours. Additional 4 hours were utilized in testing the specimens made for evaluation of mechanical properties of concrete.

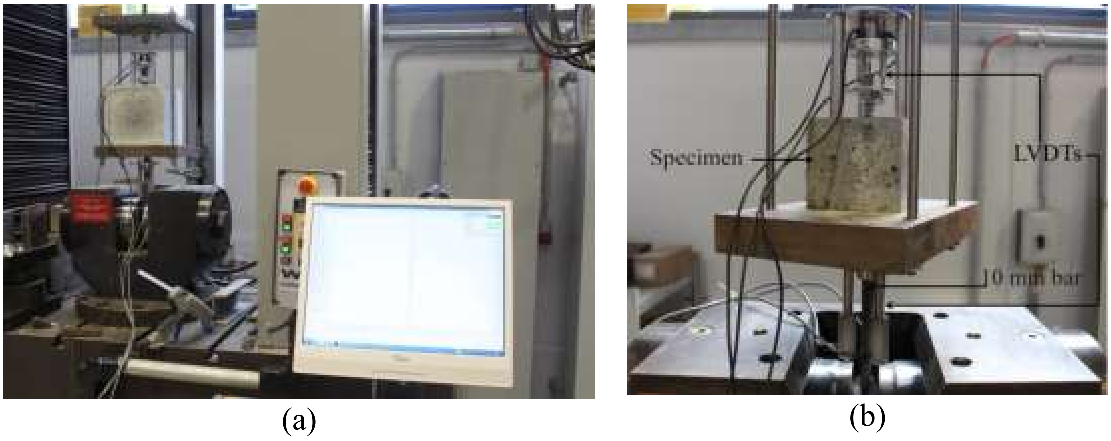


Figure 4.15 (a) Pull-out test setup (b) close-up of specimen

Table 4.1 Pull-out test program

Concrete type	Fibre volum f_v (kg/m^3)	Pull-out bar diameter ϕ (mm)	Casting date	Nr. of specimens
Lightweight Fibre-reinforced Concrete (LWFC)	0	10	07-04-15	03
		16		03
		20		03
	20	10	14-04-15	03
		16		03
		20		03
	40	10	30-06-15	03
		16		03
		20		03
	60	10	28-04-15	03
		16		03
		20		03
Normal weight Fibre-reinforced Concrete (NWFC)	0	10	13-05-15	03
		16		03
		20		03
	20	10	20-05-15	03
		16		03
		20		03
	40	10	27-05-15	03
		16		03
		20		03
	60	10	03-06-15	03
		16		03
		20		03
Total				72

Note: f_v = 0, 20, 40, 60 kg/m^3 = 0, 0.25, 0.5, 0.75% V_f

Table 4.2 Material properties

Aggregates				
Type	Particle size [mm]	Bulk density [kg/m^3]	Particle density [kg/m^3]	24H water absorption %
Expanded clay	2-10	650	1190	14.36
Gravel	2-8	1474	2520	1.48
Sand	0-2	1604	2573	1.02
Fibres				
Shape	l_f [mm]	d_f [mm]	Aspect ratio l_f/d_f	Tensile strength [MPa]
Hooked-end	35	0.55	0.64	1100

Table 4.3 Mix design for LWFC and NWFC

Material	Unit	LWFC	NWFC
Cement	[kg/m^3]	360	350
Coarse aggregate	[kg/m^3]	472	884
Fine aggregate	[kg/m^3]	772	955
Total water	[kg/m^3]	205	180
Superplasticizer	[% weight of cement]	0.5	0.5
Fibre volume, f_v	[kg/m^3]	0, 20, 40, 60	0, 20, 40, 60
Effective w/c		0.35	0.45

Note f_v = 0, 20, 40, 60 kg/m^3 = 0, 0.25, 0.5, 0.75% V_f

Table 4.4 Mechanical & geometrical properties of pull-out bars

ϕ (mm)	A_s (mm^2)	Weight (kg/m)	f_y (MPa)	f_u (MPa)	Rib height (mm)	Relative rib area $R_r = \frac{h_r}{s_r}$	Clear rib spacing C_{clear}
10	78.5	0.61	509	623	0.58	0.116	2.64
16	201	1.58	497	591	1.02	0.204	5
20	314	2.47	509	601	1.24	0.231	5.325

5 Material Properties

5.1 General

Results of tests performed for fresh and hardened concrete properties of Lightweight Fibre-reinforced Concrete (LWFC) and Normal weight Fibre-reinforced Concrete (NWFC) are presented in this chapter. Most of these properties also have direct effect on the bond behaviour of concrete. This effect, however, will be discussed in detail in following chapter.

5.2 Fresh Concrete Properties

5.2.1 Workability

Effort required for working with fibre reinforced concrete is reduced when lightweight aggregate is used instead of normal weight aggregate. Due to their lower density, LWFC having even low slump values can attain sufficient workability, which is comparable to high slump normal weight fibre reinforced concrete. It is now established that addition of fibres in concrete has adverse effect on the slump of fresh concrete [57], [199], [200] and becomes more pronounced as the length-diameter ratio of fibre gets increased [201]. It was observed that, compared to the normal weight concrete, fibres influenced the workability of lightweight concrete significantly (see Figure 5.1). However, even for lower slump values, LWFC was found easy to handle in laboratory than NWFC because of round shape of coarse aggregate and also due to their lower density. Test results presented in Table 5.1 indicate that workability of LWFC when measured in terms of slump reduced by 12.5% when fibre dosage was raised from 0 to 0.75% by volume, whereas for NWFC this decrease was only about 6.4%. It can also be seen that up to the fibre dosage level of 40 kg/m^3 fibres did not affect the slump flow of NWFC. This is mainly due to the higher density of normal weight aggregates which under jolting of slump flow table overcame the resistance to flow offered by steel fibres. Depending upon the spread achieved during flow table test, consistency of the concrete can be stiff, plastic, soft or flow-able as shown in Table 5.2 below. All the mixes fell into F2 to F3 consistency class, i.e. they were plastic or soft in fresh state.

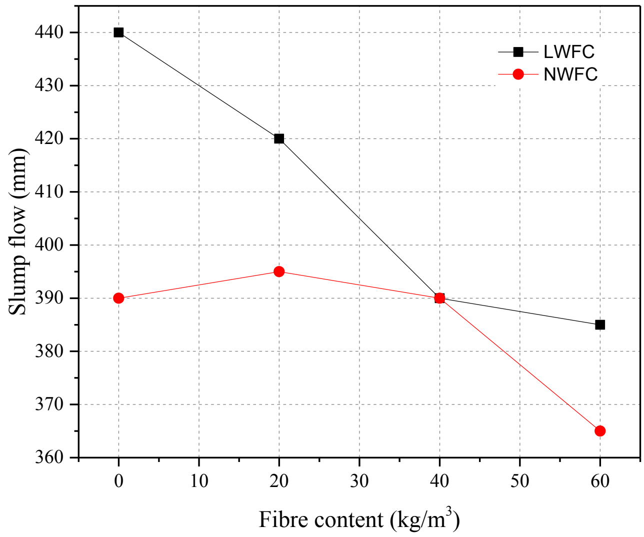


Figure 5.1 Effect of fibres on slump flow (workability) of LWFC & NWFC

Table 5.1 Slump values of LWFC and NWFC

Slump flow (mm)				
Fibre content (kg/m^3)	LWFC	% increase/decrease	NWFC	% increase/decrease
0	440	-	390	-
20	420	-4.5	395	+1.2
40	390	-11.36	390	0
60	385	-12.5	365	-6.4

Table 5.2 Consistency classes of concrete

Consistency class	F1	F2	F3	F4	F5	F6
Consistency description	Stiff	Plastic	Soft	Very soft	Flow-able	Highly flow-able
Flow spread (mm)	≤ 340	350 - 410	420 - 480	490 - 550	560 - 620	≥ 630

5.2.2 Density

For structural lightweight concrete, oven dry density should not be more than 1840 kg/m^3 and 2000 kg/m^3 as per ACI [32] and EN [202] standards respectively. Apart from reducing the workability another disadvantageous effect of fibre addition on concrete is that they increase its weight. From the results, it can be seen that highest density was recorded as 2293.77 kg/m^3 for NWFC and 1831 kg/m^3 for LWFC at maximum fibre content. All these results of fresh concrete density for both LWFC and NWFC are presented in Table 5.3 alongside percentage variation in density due to fibre addition and air content values. General trend of the results shows that fibres due to their higher specific gravity tend to raise the density of both the concretes (LWC and NWC) at all volume fraction of fibres. The only exceptional case where NWFC had reduced density was at 20 kg/m^3 fibre content level, which can only be justified by the higher air content level in the mix due to possible insufficient compaction. Similarly, like workability test results, fibres are also found to affect the density of LWC more than NWC. An increase of 2.2% is recorded in in the density of NWC whereas for LWC this increase was 6.7% when fibre volume fraction was increased from 0 to 60 kg/m^3 (0 to 0.75% V_f) as shown in Figure 5.2.

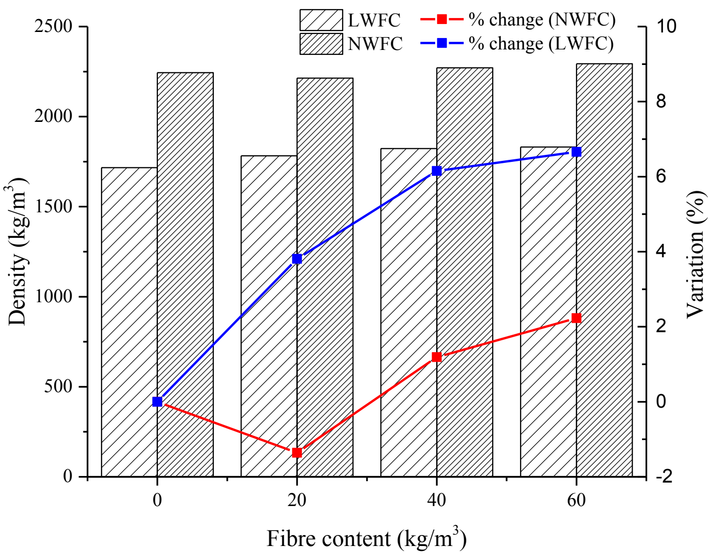


Figure 5.2 Effect of fibres on fresh concrete density of LWFC & NWFC

Table 5.3 Fresh concrete density and air content values of LWFC and NWFC

Fresh concrete density & air content values						
	LWFC			NWFC		
Fibre content (kg/m^3)	A (%)	γ (kg/m^3)	% variation	A (%)	γ (kg/m^3)	% variation
0	5.09	1716.73	-	5.30	2243.83	-
20	2.31	1782.10	+3.8	7.37	2213.33	-1.3
40	0.99	1822.33	+6.2	5.76	2270.60	+1.2
60	2.02	1831	+6.7	5.58	2293.77	+2.2

5.3 Hardened Concrete Properties

5.3.1 Compressive strength

Table 5.4 and Table 5.5 present the test results for all the 24 cube specimens of LWFC and NWFC tested under compression. Besides these cylindrical compressive strength test results determined by testing the cylinders having dimensions of 100 mm x 200 mm are also presented in Table 5.6 and Table 5.7 for LWFC and NWFC respectively. Determination of compressive strength through testing of cylinders was necessary as fib-2010 [33] and ACI-318 [32] utilize 28-day cylindrical compressive strength test values in their bond expressions. From these results, it can be seen that for every respective fibre volume, the difference between the compressive strength of LWFC and NWFC is less than 5 MPa – a prerequisite for both the concretes to fall in similar strength class. This task was achieved through extensive trial mixes, altogether 18 different combinations were tried before final selection of mixes for both concretes.

Table 5.4 Compressive strength test results of LWFC cubes

Specimen ID	Dimensions (mm)			Testing date	Age	Max. load at failure	Compressive Strength
	L	W	D		(days)	(kN)	(MPa)
C-LWFC-00-01	150	150	146			821.67	37.52
C-LWFC-00-02	150	150	147	5-05-15	28	857.9	38.91
C-LWFC-00-03	150	150	148			832.83	37.51
Average							37.98
C-LWFC-20-01	150	150	148			843.5	38.00
C-LWFC-20-02	150	150	148	12-05-15	28	881.55	39.71
C-LWFC-20-03	150	150	148			937.76	42.24
Average							39.98
C-LWFC-40-01	150	150	150			1003.76	44.61
C-LWFC-40-02	150	150	149	19-05-15	28	1023.82	45.81
C-LWFC-40-03	150	150	148.5			1058.48	47.52
Average							45.98
C-LWFC-60-01	150	150	148			877.1	39.51
C-LWFC-60-02	150	150	148	26-05-15	28	994.84	44.81
C-LWFC-60-03	150	150	148			883.46	39.80
Average							41.37

Table 5.5 Compressive strength test results of NWFC cubes

Specimen ID	Dimensions (mm)			Testing date	Age	Max. load at failure	Compressive Strength
	L	W	D		(days)	(kN)	(MPa)
C-NWFC-00-01	150	150	149			972.6	43.52
C-NWFC-00-02	150	150	149	10-06-15	28	948.56	42.44
C-NWFC-00-03	150	150	150			961.68	42.74
Average							42.90
C-NWFC-20-01	150	150	150.5			871.39	38.60
C-NWFC-20-02	150	150	150	17-06-15	28	861.87	38.31
C-NWFC-20-03	150	150	149			904.99	40.49
Average							39.13
C-NWFC-40-01	150	150	150			1045.34	46.46
C-NWFC-40-02	150	150	150	24-06-15	28	1044.53	46.42
C-NWFC-40-03	150	150	149			1021.78	45.72
Average							46.20
C-NWFC-60-01	150	150	150			886	39.38
C-NWFC-60-02	150	150	150	1-07-15	28	912.37	40.55
C-NWFC-60-03	150	150	148			973.74	43.86
Average							41.26

Table 5.6 Cylindrical compressive strength test results of LWFC

Specimen ID	Dimensions (mm)		Testing date	Age	Max. load at failure	Compressive Strength
	L	Diameter		(days)	(kN)	(MPa)
C-LWFC-00-01	200	100			289.38	36.85
C-LWFC-00-02	200	100	5-05-15	28	313.95	39.97
C-LWFC-00-03	200	100			280.63	35.73
Average						37.52
C-LWFC-20-01	200	100			290.72	37.02
C-LWFC-20-02	200	100	12-05-15	28	311.26	39.63
C-LWFC-20-03	200	100			334.99	42.65
Average						39.77
C-LWFC-40-01	200	100			375.39	47.80
C-LWFC-40-02	200	100	19-05-15	28	336.21	42.81
C-LWFC-40-03	200	100			342.9	43.66
Average						44.75
C-LWFC-60-01	200	100			254.69	32.43
C-LWFC-60-02	200	100	26-05-15	28	296.18	37.71
C-LWFC-60-03	200	100			283.44	36.09
Average						35.41

Table 5.7 Cylindrical compressive strength test results of NWFC

Specimen ID	Dimensions (mm) L	Dimensions (mm) Diameter	Testing date	Age (days)	Max. load at failure (kN)	Compressive Strength (MPa)
C-NWFC-00-01	200	100	10-06-15	28	291.36	37.10
C-NWFC-00-02	200	100			278.21	35.42
C-NWFC-00-03	200	100			308.1	39.23
Average						37.25
C-NWFC-20-01	200	100	17-06-15	28	269.93	34.37
C-NWFC-20-02	200	100			287.3	36.58
C-NWFC-20-03	200	100			244.04	31.07
Average						34.01
C-NWFC-40-01	200	100	24-06-15	28	342.16	43.57
C-NWFC-40-02	200	100			307.55	39.16
C-NWFC-40-03	200	100			339.36	43.21
Average						41.98
C-NWFC-60-01	200	100	1-07-15	28	273.51	34.82
C-NWFC-60-02	200	100			265.93	33.86
C-NWFC-60-03	200	100			286.91	36.53
Average						35.07

Specimens without fibres failed in more brittle fashion compared to those with fibres. Also, number of cracks at failure increased with increasing fibre content, and the width of cracks is observed to be decreasing as shown in Figure 5.3. Test results performed on both the 150 mm cubes and on cylinders (100 mm x 200mm) showed no clear effect of fibres on compressive strength. For both types of specimens, maximum compressive strength value was observed at 0.5% (V_f = 40 kg/m^3) volume fraction of fibres as shown in Figure 5.4. This increase was about 21.1% for specimens of LWFC and only 7.7% for cylindrical specimens. It can be assumed that beyond this point further addition of fibres started creating cavities and voids in the mix and therefore specimens crushed at lower compressive loads. Values in the Figure 5.4 represent the mean values alongside error bars of standard deviation. In general, compressive strength of cylindrical specimens was 80 to 86 % lower than the cubes. Similar observations have been reported by Topçu and Canbaz [203] that compressive strength values determined by cylindrical specimens are lower. Contrary to this, results of work of Domagala [101] show that for 0.6% fibre volume fraction compressive strength of LWFC increased by 4.6% when determined by cubes and by 7% when determined by cylinders of 150 mm x 300 mm.

Fibre content (kg/m^3)	LWFC	NWFC
0		
20		
40		
60		

Figure 5.3 Overview of specimens tested under compression at different fibre content levels

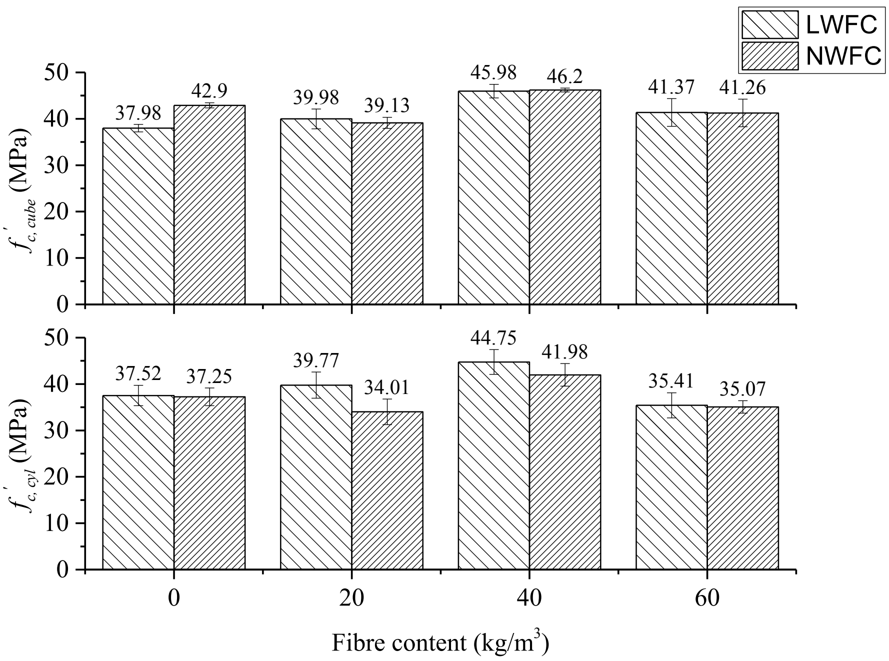


Figure 5.4 Effect of fibres on compressive strength of LWFC & NWFC

5.3.2 Tensile strength

Splitting tensile strength of the specimens was calculated using following equation and German DIN standard DIN EN 12390-6 [113].

$$f_{\text{ct-sp}} = \frac{2F}{\pi l d} \tag{5.1}$$

Where $f_{\text{ct-sp}}$ is splitting tensile strength in MPa, F is applied force in N and l, d are length and depth of specimen in mm.

For all mixes of LWFC, tensile strength is found to be, on an average 6.59 percent of the compressive strength, whereas for NWFC this percentage ranges from 6.41 to 6.88%. All test results determined on 150 mm cube specimens are presented in Table 5.8 and Table 5.9 for LWFC and NWFC respectively. The maximum difference of 0.247 MPa in the splitting tensile strength of both concretes was observed at 0% volume fraction of fibres, just like compressive strength test results. At zero fibre content level, specimens failed with single major crack in two halves, whereas, multiple cracking at centre due to uniform tensile stresses and crushing at ends due to transverse compression stresses was observed at highest fibre dosage (see Figure 5.5)

Table 5.8 Splitting tensile strength test results of LWFC

Specimen ID	Dimensions (mm)			Testing date	Age	Max. load at failure	Tensile Strength
	L	W	D		(days)	(kN)	(MPa)
LWFC-00-01	150	150	148	05-05-15	28	92.24	2.61
LWFC-00-02	150	150	148			91.93	2.60
LWFC-00-03	150	150	148			81.38	2.30
Average							2.50
LWFC-20-01	150	150	148	12-05-15	28	94.39	2.67
LWFC-20-02	150	150	149			98.85	2.80
LWFC-20-03	150	150	149			98.25	2.78
Average							2.75
LWFC-40-01	150	150	149	19-05-15	28	100.68	2.85
LWFC-40-02	150	150	149			100.94	2.86
LWFC-40-03	150	150	149.5			102.62	2.90
Average							2.87
LWFC-60-01	150	150	148	26-05-15	28	95.85	2.71
LWFC-60-02	150	150	148			92.64	2.62
LWFC-60-03	150	150	148			101.04	2.86
Average							2.73

Table 5.9 Splitting tensile strength test results of NWFC

Specimen ID	Dimensions (mm)			Testing date	Age	Max. load at failure	Tensile Strength
	L	W	D		(days)	(kN)	(MPa)
NWFC-00-01	150	150	150	10-06-15	28	98.9	2.80
NWFC-00-02	150	150	150			101.82	2.88
NWFC-00-03	150	150	149			91.01	2.58
Average							2.75
NWFC-20-01	150	150	149.5	17-06-15	28	94.01	2.66
NWFC-20-02	150	150	149.5			90.27	2.55
NWFC-20-03	150	150	148.5			100.23	2.84
Average							2.68
NWFC-40-01	150	150	150	24-06-15	28	111.1	3.14
NWFC-40-02	150	150	150			106.72	3.02
NWFC-40-03	150	150	150			99.6	2.82
Average							2.99
NWFC-60-01	150	150	150	01-07-15	28	98.93	2.80
NWFC-60-02	150	150	148			102.44	2.90
NWFC-60-03	150	150	150			99.61	2.82
Average							2.84

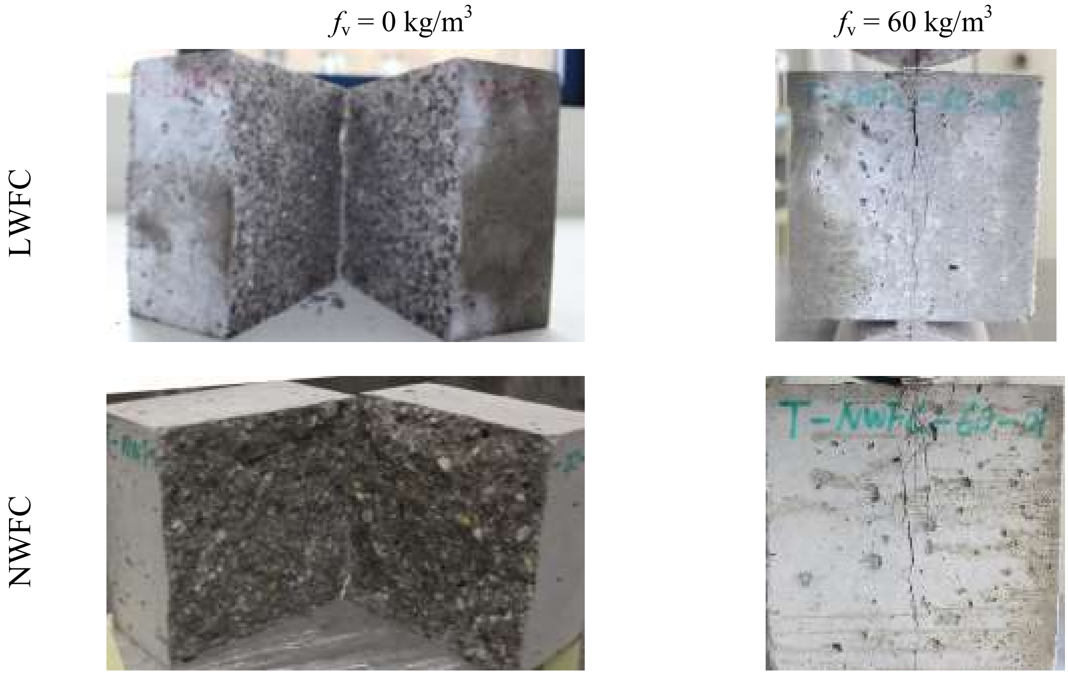


Figure 5.5 Overview of specimens after split tensile strength test

Also, variation of tensile strength strongly follows the compressive test results, as shown in Figure 5.6. These observations suggest that compressive strength has strong influence on splitting tensile strength of concrete and improvement in first cracking tensile stress through addition of steel fibres may not be a viable option. This is also evident from Table Table 5.10 which shows that increase in fibre volume does not affect the ratios of splitting tensile strength to $\sqrt{f_c}$ and to $f_c^{2/3}$ for both conventional and lightweight concretes and is almost constant. For ACI Code [32], the ratio of splitting tensile strength to the square root of compressive strength of concrete is 0.56 and 0.476 for conventional and lightweight concretes respectively. Although, some improvement in tensile strength was observed but it was of no appreciable amount; for example, at maximum fibre content level an improvement of only 9% is recorded in case of LWFC, whereas for NWFC it was merely of 3%. Maximum improvement of 8.8% and 14.6% were recorded at 0.5% volume fraction of fibres. At this volume fraction i.e. 0.5%, Song and Hwang recorded 19% improvement in splitting tensile strength of high-strength fibre-reinforced concrete, whereas 11% increase is reported by Yazici et al. [52] for normal strength fibre-reinforced concrete. Nevertheless, for current experimental program, improvement in splitting tensile strength due to fibre addition was more pronounced in LWFC then NWFC. Similar observation is noted by Balendran et al. [96] in their work.

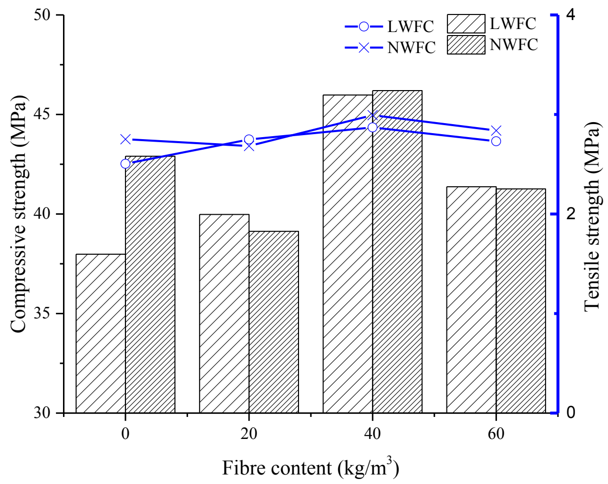


Figure 5.6 Variation of splitting tensile strength with respect to compressive strength for LWFC & NWFC

Table 5.10 Effect of fibre volume on the ratios of splitting tensile strength to $\sqrt{f_c}$ and to $f_c^{2/3}$

Fibre content (kg/m^3)	LWFC		NWFC	
	$\frac{f_{ct}}{f_c^{1/2}}$	$\frac{f_{ct}}{f_c^{2/3}}$	$\frac{f_{ct}}{f_c^{1/2}}$	$\frac{f_{ct}}{f_c^{2/3}}$
0	0.408	0.223	0.451	0.247
20	0.436	0.236	0.460	0.255
40	0.429	0.228	0.461	0.248
60	0.459	0.253	0.480	0.265

5.3.3 Flexural performance

In current experimental work, performance of LWFC and NWFC beams under flexure is evaluated by first peak strength, energy absorption capacity (toughness), and residual capacity. Fibre reinforced concrete specimens can attain multiple peak load values under flexure depending upon the quantity of fibres added to the matrix. The first peak strength f_1 is the flexural strength of specimens calculated at first peak load F_1 using equation (5.2). It is also termed as modulus of rupture as it characterizes the point of onset of the first crack where load-deflection curve has zero slope.

$$f_1 = \frac{F_1 l}{bd^2} \tag{5.2}$$

For above equation, F_1 is flexural load at first peak, f_1, the flexural capacity corresponding to this load, and *b*, *d*, *l* are the width, depth and clear span of beam specimens.

Flexural toughness is the measure of the energy absorption capacity of specimens and can be determined by finding out the area under the load-deflection curve. ASTM [110] suggests determination of this area up to deflection of 1/150 of the clear span of beams. Residual capacity is the flexural strength or stress value of the damaged concrete in post cracking region, and here it is determined at deflection of *l*/600 and *l*/150.

5.3.3.1 First peak strength

Fibres can impart greater ductility to the concrete and such reinforced concrete beams under flexural loading can attain multiple peak loads with progressive deflection. First peak strength f_1 , calculated at first peak load, F_1 (see Figure 5.7.), characterizes the point of onset of the first crack; hence stress at this level is also called first cracking stress. Results of both the splitting tensile strength and flexural tensile strength test f_1 , show similar trends. Higher values were recorded in case of beam tests compared to splitting tensile strength tests, possible reasons for this difference as per Nilson are the localization of the bending stresses at the outermost beam surface and the application of Hook's law for a material with non-linear behaviour. The ratio of mean value of flexural tensile strength to the square root of compressive strength for conventional and LWC is 0.69 and 0.58 respectively [92]. With the addition of steel fibres, these values were however, found to be higher compared to the control specimens without fibres (see Table 5.11), in addition, it shall also be noted that results of tensile tests tend to have more scatter due to its dependence on multiple factors, such as, aggregate strength, bond between aggregate and cement paste etc.

Table 5.11 Effect of fibre volume on the ratios of flexural tensile strength to $\sqrt{f_c}$ and to $f_c^{2/3}$

Fibre content (kg/m^3)	LWFC		NWFC	
	$\frac{f_1}{f_c^{1/2}}$	$\frac{f_1}{f_c^{2/3}}$	$\frac{f_1}{f_c^{1/2}}$	$\frac{f_1}{f_c^{2/3}}$
0	0.568	0.311	0.678	0.371
20	0.614	0.332	0.806	0.448
40	0.700	0.371	0.795	0.426
60	0.660	0.364	0.851	0.470

Both fib Model Code 2010 and Eurocode 2, provide the relationship between direct axial tensile strength and modulus of rupture presented here as equation (5.3) and (5.4) respectively.

$$f_{\mathrm{ctm}} = A_{\mathrm{fl}} f_1 \tag{5.3}$$

where $A_{\mathrm{fl}} = \dfrac{0.06 h_{\mathrm{b}}}{1 + 0.06 h_{\mathrm{b}}^{0.7}}$

$$f_{\mathrm{ctm}} = \frac{f_1}{\left(1.6 - \dfrac{h_{\mathrm{b}}}{1000}\right)} \tag{5.4}$$

Where h_{b} represents beam height, and f_{ctm} is mean axial tensile strength.

From these relationships one can establish a relationship between splitting tensile strength, $f_{\text{ct-sp}}$ and modulus of rupture f_1, as according to Eurocode 2 $f_{\text{ct-sp}} = f_{\mathrm{ctm}}/0.9$. Thus for the size of beams (150 x 150 x 700) used in current work h_{b} is 150 and the relationship between $f_{\text{ct-sp}}$ and f_1 as per Model Code 2010 and Eurocode 2 are as $f_{\text{ct-sp}}/f_1 = 0.67$and $f_{\text{ct-sp}}/f_1 = 0.77$ respectively. Compared to this the calculated values for LWFC and NWFC are tabulated as under (see Table 5.12), which indicates that the modulus of rupture value are 1.3 to 1.5 times higher than the splitting tensile strength test results.

Table 5.12 Relationship between splitting tensile strength and flexural tensile strength (modulus of rupture)

Fibre content (kg/m^3)	$f_{ct\text{-}sp}/f_1$	
	LWFC	NWFC
00	0.718	0.664
20	0.710	0.570
40	0.613	0.5805
60	0.694	0.563

Flexural strength values at first peak loads (F_1) for specimens of both LWFC and NWFC are tabulated in Table 5.13. Although, it appears as if fibres have increased the flexural strength at this load value, fluctuation of values with compressive strength suggests that, not fibres, rather compressive strength is the reason for this increase. This observation is in agreement with the findings of Shah [204], who observed no improvement in first cracking stress for the amount of fibres used in current experimental work. Comparison of the results of LWFC and NWFC (see Figure 5.8) shows that maximum and the minimum difference of first peak strength is at fibre dosages of 60 and 40 kg/m^3 where LWFC achieved 28.24% and 10.04% respectively lesser strength than NWFC.

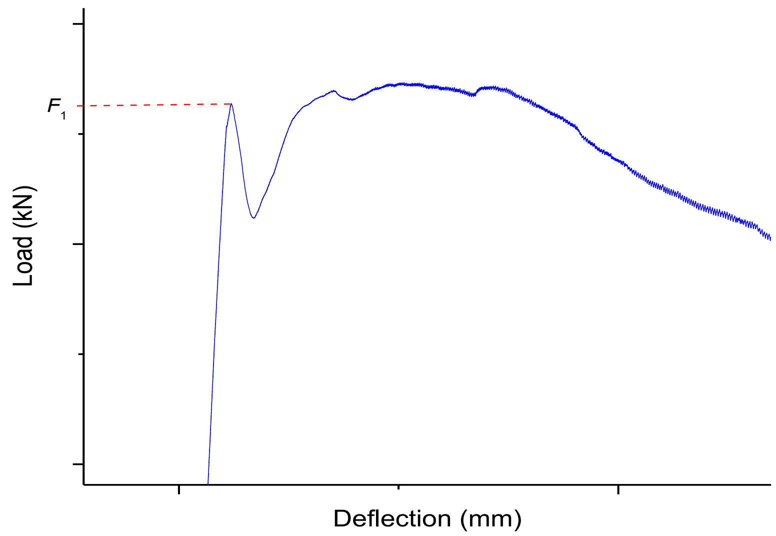


Figure 5.7 Definition of first peak load on a typical load-deflection curve of FRC beam

Table 5.13 Flexural performance indicators

Specimen	Fibre content [kg/m^3]	l [mm]	F_1 [kN]	f_1 [MPa]	% increase	δ_1 [mm]	T [Joules]
LWFC	0	600	19.31	3.48	0.00	0.098	1.21
	20		21.55	3.87	11.21	0.104	41.26
	40		25.97	4.68	34.48	0.33	71.58
	60		22.09	3.93	12.93	0.108	67.01
NWFC	0	600	23.04	4.14	0.00	0.098	1.28
	20		26.42	4.70	13.53	0.104	52.69
	40		28.60	5.15	24.4	0.08	86.25
	60		28.26	5.04	21.74	0.12	93.94

Note: Values are average of 3 specimens
δ_1 = Beam deflection at first peak load

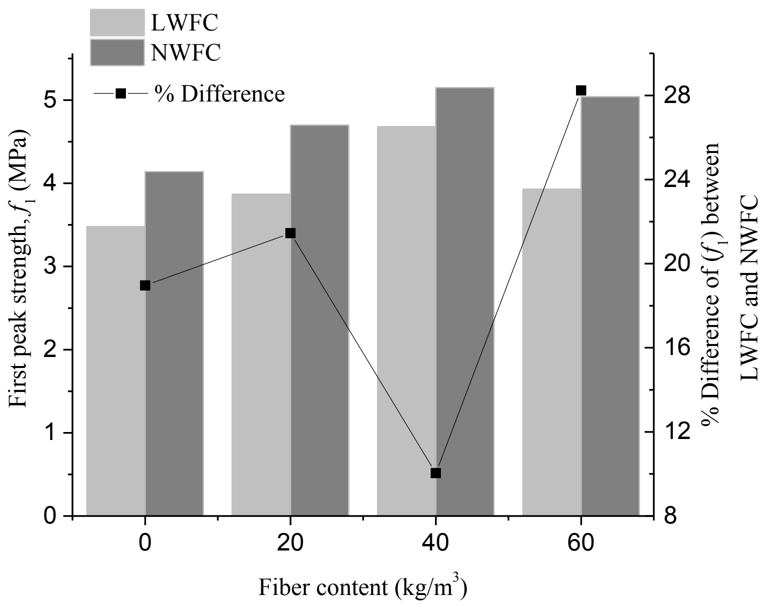


Figure 5.8 Flexural strength of beam specimens at first peak/crack point

5.3.3.2 Flexural toughness

In recent years with growing security concerns, toughness T is expected to become the most desired property of the concrete, especially in situations where structural elements have to withstand impact from shocks and blasts. Toughness is also highly influenced property of concrete when fibres are added to it. It characterizes material's capacity to energy absorption and is evaluated by determining the area under the load-deflection curve up to specific deflection point. Figure 5.9 shows area under load deflection curves up to deflection of 4 mm (*l*/150) along with toughness values in joules. These curves are the average of load-deflection curves of three beams specimens, and the plots (load-deflection curves) of all those individual specimens are presented in Figure 5.10. Although LWFC achieved lower toughness values, pattern of the curves suggests that fibres had similar effect on both concretes. Poor bond of lightweight aggregates with the mortar and their lower strength under tensile loading are assumed to be the reason due to which fibres kept on slipping at reduced loading. As a result of this slippage, contribution of fibres in bridging the cracks is also affected. Failure of all the specimens occurred in the maximum moment region of beams.

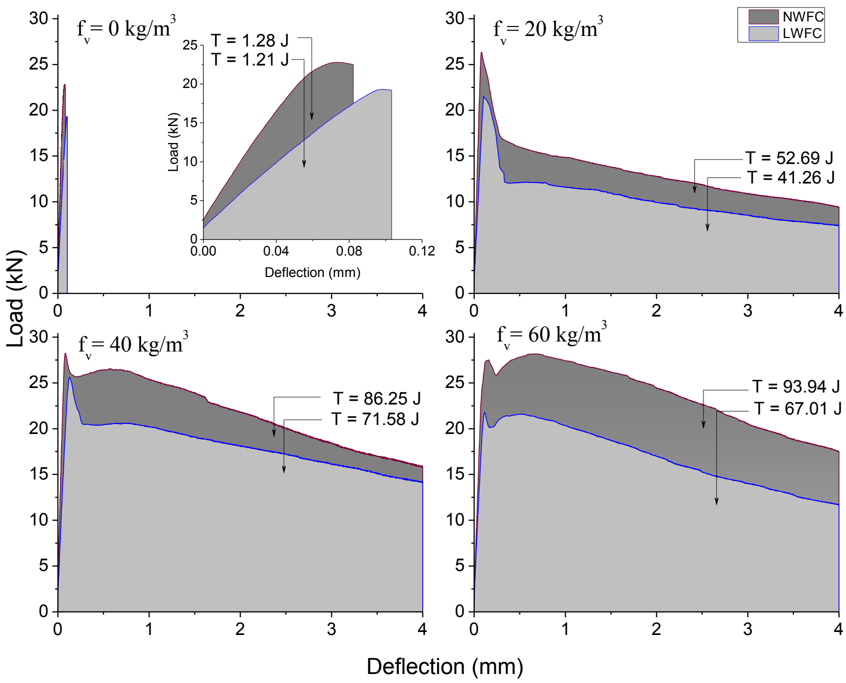


Figure 5.9 Evaluation of area under load-deflection curves

Note: [f_v 20 kg/m^3 = 0.25% V_f], [f_v 40 kg/m^3 = 0.5% V_f], [f_v 60 kg/m^3 = 0.5% V_f]

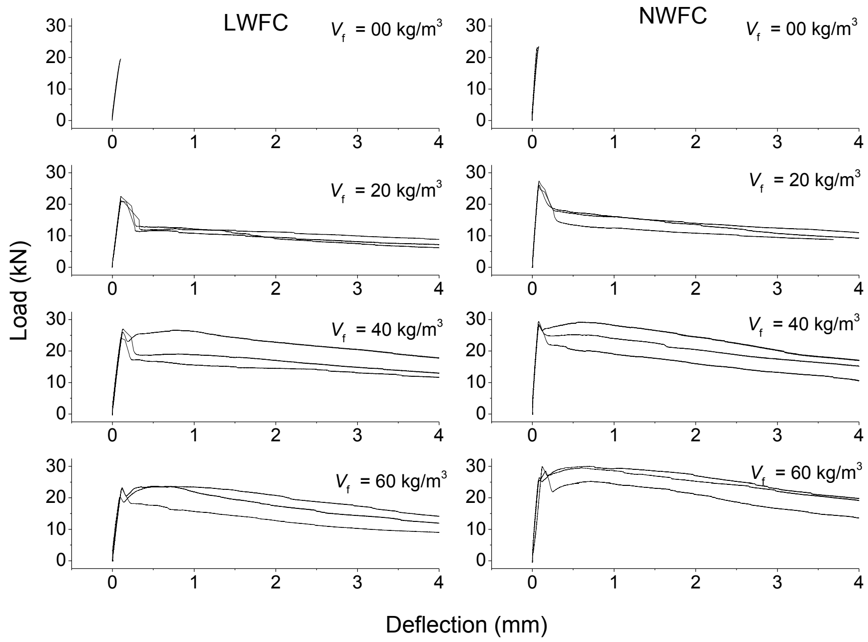
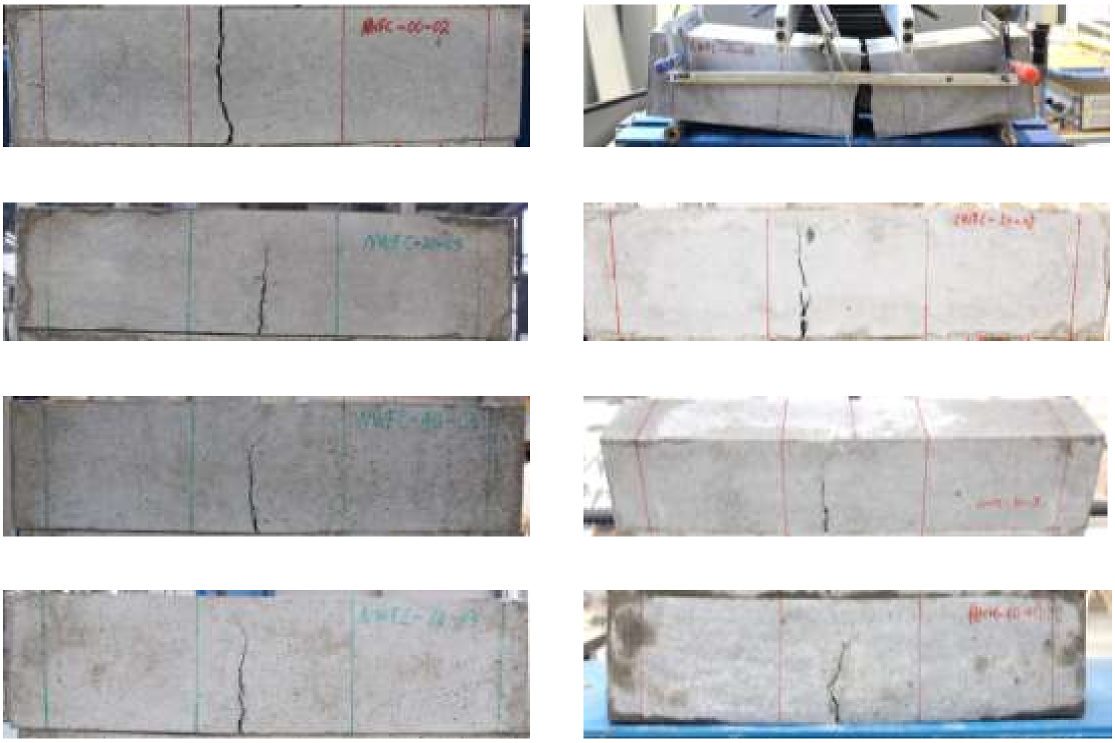


Figure 5.10 Load-deflection diagrams of all the tested beam specimens



(a) (b)

Figure 5.11 Overview of beam specimens after flexural test (a) NWFC (b) LWFC

As expected, presence of multiple cracks at the tensile surface was not observed at such low percentage of fibres. Although few small cracks started appearing at higher volume fraction (40 and 60 kg/m^3) of fibres, these could not progress further to compression zone, and failure occurred due to single major crack (see Figure 5.11). At maximum fibre content, LWFC and NWFC had toughness values 55 times and 73 times higher than their respective control specimens without fibres. Toughness values of LWFC were lower than NWFC by 5.8%, 27.8%, 20.5% and 40% at 0, 20, 40 and 60 kg/m^3 fibre contents respectively.

5.3.3.3 Residual capacity

Post first peak performance of cracked concrete is highly dependent on the amount of fibres added to it [205]. Multiple peaks on load deflection curves started becoming prominent for fibre volume fraction higher than 0.5% (f_v = 40 kg/m^3). For these specimens, after the first crack point, load started decreasing rapidly with increasing deflection for a while but later started rising as the tensile load shifted to fibres. From this point onwards, beams of NWFC were able to regain flexural strength higher than first peak strength at higher deflection values (0.5 mm), whereas, for lightweight concrete, the residual strength at maximum peak strength (at around 0.4 mm for all specimens) and the first peak strength remained almost at the same level. Residual strength of LWFC also remained lower than NWFC at all stages of flexural tests of all specimens. Figure 5.12 shows residual flexural strength values calculated at 1 mm $f_{l/600}$ and 4 mm $f_{l/150}$ for fibre volume of 20, 40 and 60 kg/m^3 (0.25, 0.5 and 0.75% V_f). These calculated values are also tabulated in Table 5.14. Specimens of 0% V_f could not achieve deflections higher than 0.15 mm and showed no residual capacity, which is typical of unreinforced concrete beams. At volume fraction of 0.25%, NWFC achieved 27% higher residual capacity than LWFC measured at deflection of 1 mm and 4 mm. At similar deflection levels, difference between the two concrete became wider at maximum volume fraction (0.75%) as the NWFC achieved residual capacity 37% and 49% higher than LWFC. Reason for the widening of this performance gap, with increasing fibre quantity, is considered to be the better synergy between stiff concrete matrix resulting from irregular, rough textured coarse aggregate and the fibres.

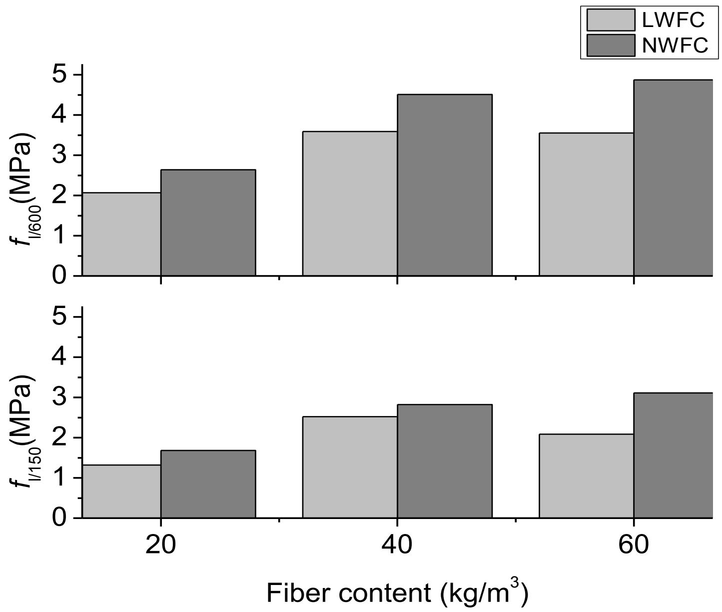


Figure 5.12 Residual flexural capacity of beam specimens

Table 5.14 Residual capacity of beam specimens at specific deflection points

Fibre content (kg/m^3)	LWFC		NWFC	
	$f_{l/600} = 1\text{mm}$	$f_{l/150} = 4\text{mm}$	$f_{l/600} = 1\text{mm}$	$f_{l/150} = 4\text{mm}$
20	2.069	1.318	2.639	1.680
40	3.589	2.525	4.509	2.821
60	3.555	2.085	4.876	3.113

5.3.4 Modulus of elasticity

Test results for modulus of elasticity (E) are presented in Table 5.15 and illustrated in Figure 5.13 as a function of fibre content. Table 5.15 also contains values calculated from ACI-318 and Eurocode-2 equations presented below as equations (5.5) and (5.6) respectively. For lightweight concrete ACI-318 recommends that results be multiplied by with 0.85 for sand lightweight concrete and with 0.75 for all-lightweight concrete.

Although Eurocode-2 specifies the changes required in calculation of modulus of elasticity for different types of aggregates, however, author could not find guidelines for lightweight concrete. Therefore tabulated values for LWFC as per Eurocode-2 could not be estimated.

$$E_c = 4.7\sqrt{f_c'} \quad \text{(GPa)} \tag{5.5}$$

$$E_c = 22\left[\frac{f_{\text{cm}}}{10}\right]^{0.3} \quad \text{(GPa)} \tag{5.6}$$

For lightweight concrete, results' trend clearly indicates that fibre addition causes a slight reduction in elastic modulus of the concrete. This decrease was on an average 5.86% at the highest fibre volume. Mansur et al. [206] and Neves & Almeida [207] have reported similar behaviour and can be explained because addition of fibres results in not so compacted concrete as it would be without fibres. Compared to this, test values of NWFC follow trend of compressive strength results, suggesting well established relation between elastic modulus and compressive strength i.e. any change in compressive strength will affect the elastic modulus exponentially (see Figure 5.14). Ineffectiveness of steel fibres on modulus of elasticity of NWFC can only be explained by the fact that due to higher specific gravity of coarse aggregates, NWC specimens were able to achieve enough compaction under vibration which counteracted the effect of fibre presence. Since modulus of elasticity of concrete is highly dependent on specific gravity of coarse aggregate as well, for this reason, results show that despite having similar strength class, LWFC had on an average 14 GPa lower modulus of elasticity values than NWFC.

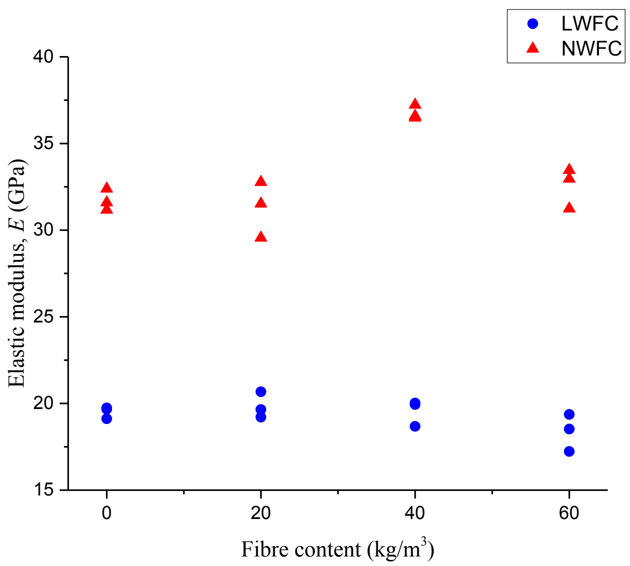


Figure 5.13 Elastic modulus as a function of fibre content for LWFC & NWFC

Table 5.15 Modulus of elasticity test results for LWFC and NWFC

Concrete type	f_v (kg/m^3)	Elastic modulus, E (Gpa)			Average (GPa)	ACI-318	Eurocode-2
LWFC	0	19.67	19.12	19.74	19.51	24.47	
	20	20.67	19.21	19.65	19.85	25.19	
	40	19.94	18.68	20.02	19.55	26.72	
	60	18.53	17.23	19.37	18.37	23.77	
NWFC	0	31.17	32.39	31.59	31.71	28.69	32.64
	20	32.77	31.52	29.56	31.28	27.41	31.76
	40	36.58	37.23	36.49	36.77	30.45	33.83
	60	33.46	31.24	32.96	32.55	27.83	32.06

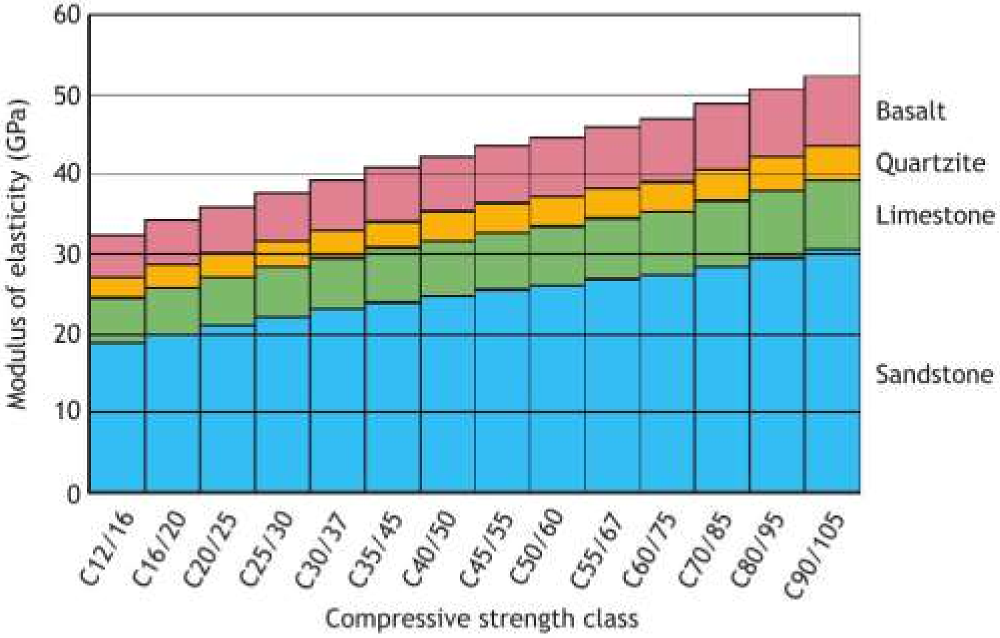


Figure 5.14 Effect of compressive strength of concrete made from different types of aggregates on modulus of elasticity [93]

6 Results & Analysis of Bond Test

6.1 General

This chapter presents results of all the pull-out tests performed on Lightweight Fibre-reinforced Concrete (LWFC) and Normal weight Fibre-reinforced Concrete (NWFC). The effect of test parameters on bond behaviour is also discussed, followed by influence of selected material properties – the results of which are presented in previous chapter. It has been made sure in this chapter to present all the test results in tabular form; however, the plots of pull-out tests include the average of three test results. Reader can access any specific pull-out test plot/curve for any type of concrete in Annex – A for LWFC and Annex – B for NWFC

6.2 Pull-out Test Results

No pull-out failure was observed, and failure of all the specimens took place by splitting of concrete. The maximum pull-out force and the corresponding bond strength for all the specimens of LWFC and NWFC are presented in Table 6.2 and Table 6.3 respectively.

Cracks became visible at the outer surface of all the specimens at the maximum pull-out force radiating outward from pull-out bar along its bonded length. Wider cracks are observed in both the concrete types (LWC and NWC) with increasing bar or specimen size, and compared to NWC, cracks in LWC specimens were wider and these specimens at failure fractured completely in absence of fibres as shown in Figure 6.1. Besides enhancing the post cracking performance, another advantage for which steel fibres are well known, is their ability in to delay crack formation and reducing the crack width [208], [209] . This phenomenon is observed during pull-out tests as well. The crack width is function of concrete's tensile strength, which means lower the tensile strength higher the crack width. Since residual tensile capacity is found to increase with increasing fibre dosage as noted in section 0 therefore crack widths noted at higher fibre dosages and higher displacements are also lower in pull-out tests.

Increased interest by the building codes in improving durability and appearance of structure by limiting crack width and propagation is also an important factor in widespread use of fibres in concrete. Structures with larger crack widths not only give dull view to the observer but also lose some part of their service life because of corrosion of reinforcement that is exposed to outside environment due to larger crack widths [210].

Figure 6.2 shows the effect of fibres on crack width on the pull-out tests of 20 mm bar size specimens of LWFC. Crack width measured at slip of 4 mm is 1.2 mm at fibre content of 20 kg/m^3, which can be seen to reduce to 0.15 mm at highest fibre dosage. Figure 6.2 (b) presents the magnified view of these cracks.

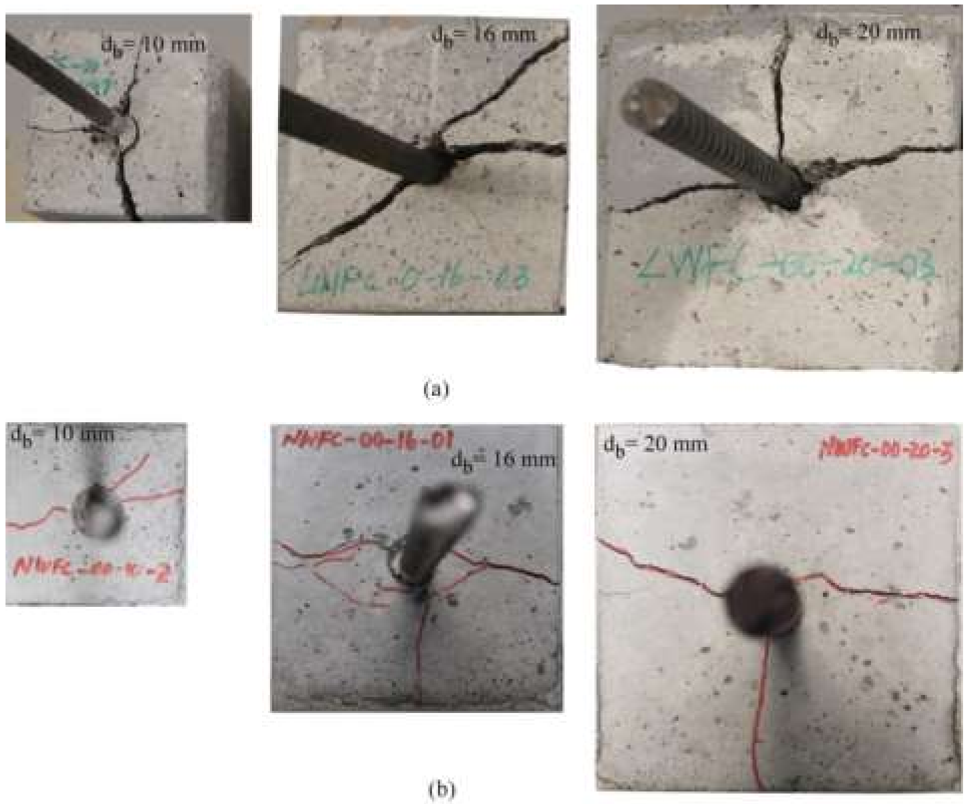


Figure 6.1 Overview of specimens of different sizes at the end of bond test (a) LWFC (b) NWFC

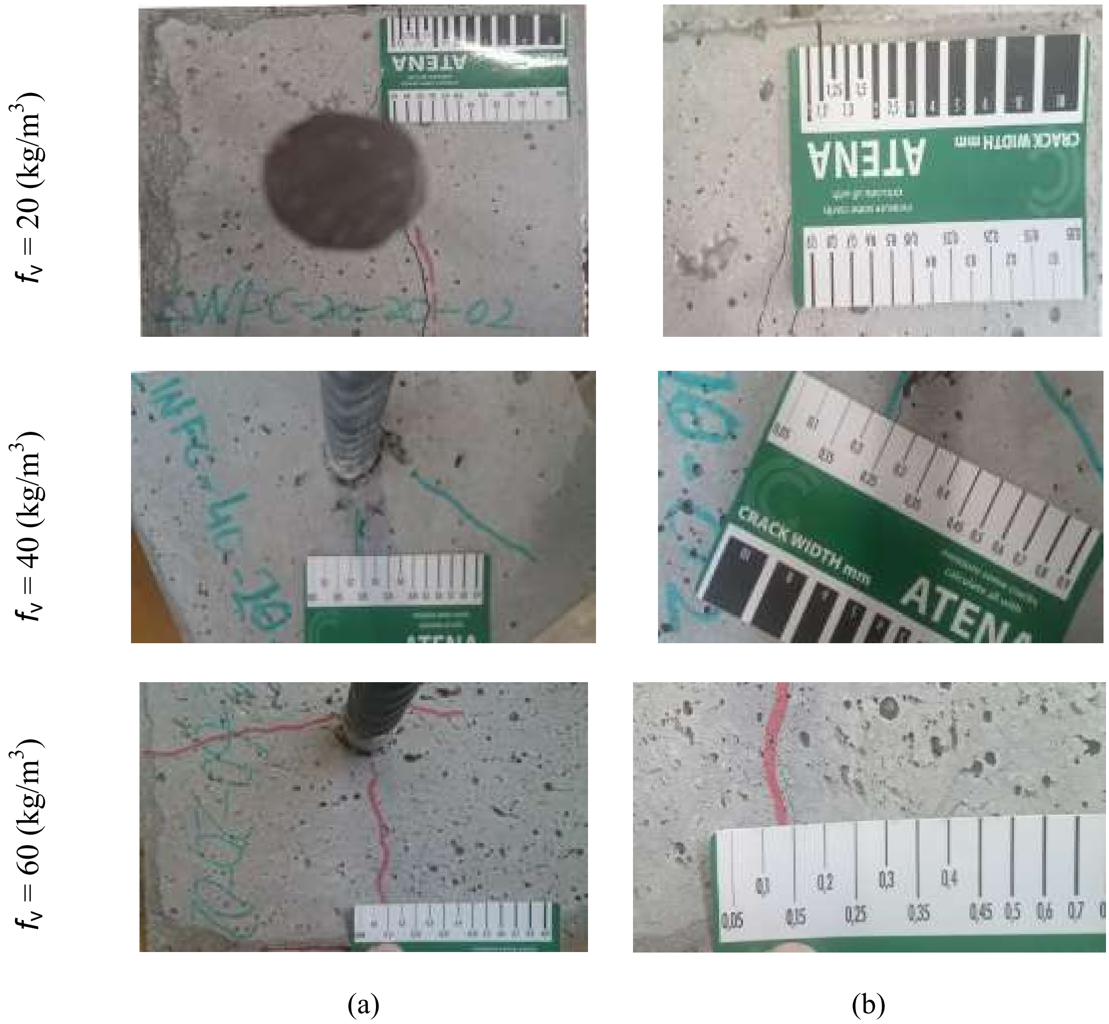


Figure 6.2 (a) Effect of fibres on cracks measured at slip of 4 mm for 20 mm bar size specimen of LWFC (b) magnified view

6.2.1 Effect of bar/specimen size

Bond stress at failure was observed to decrease with development of larger crack widths as the bar/specimen size increased, because in larger specimens, due to distributed cracking, non-simultaneous nature of failure in different zones is more pronounced which help to cause the final failure resulting in larger cracks at same slip values (see Figure 6.1). Also, compared to conventional concrete, brittleness of lightweight concrete was evident, as specimens of this mix completely split at the end of test. This is because, normal weight aggregates have higher values of density, particle strength and elastic modulus, therefore these act as an obstacle to further propagation of cracks, whereas lightweight aggregate being lighter and porous offer less resistance.

Effect of bar size and specimen size in different concretes on bond has been reported in earlier literature [211], [212]. These reports suggest decrease in bond strength as the bar size/specimen size increase. This decrease is attributed to the fact that there is increase in circumferential shearing area as the bar diameter increases. Even with same cover to bar size $\left(\frac{c}{\phi}\right)$ ratio, results for all the mixes of current experimental work show that 10 mm bar size attained highest bond strength. For NWFC, on an average bond strength of specimens with 20 mm bar and 16 mm bar were found to be 82.5% and 86.4% respectively of the bond strength of 10 mm bar. Whereas for LWFC bond strength of 16 mm and 20 mm bars were 83.9% and 69.76% respectively of the bond strength of 10 mm bar. Higher difference in percentage of bond strength of 20 mm and 10 mm bar size specimens in case of LWFC reflects here two things, one brittleness of lightweight aggregate concrete against the higher pull-out loads achieved in case of larger specimens and second the fact that in larger specimens, non-simultaneous nature of failure is more pronounced as explained earlier.

Lightweight concrete has lower bond strength than normal weight concrete due to the lower particle strength [213]. This trend was also observed in current experimental work as shown in Figure 6.3. Results show that, except 10 mm bar size specimens at 0% fibre volume fraction, bond strength of LWFC was lower than that of NWFC at every other fibre content level. Figure 6.3 presents average results of ultimate bond strength for both concrete types (LWFC & NWFC) including all bar size specimens. On an average for 10 mm, 16 mm and 20 mm bar size specimens, bond strength of NWFC was higher than LWFC by 1.396, 1.18 and 1.35 times respectively.

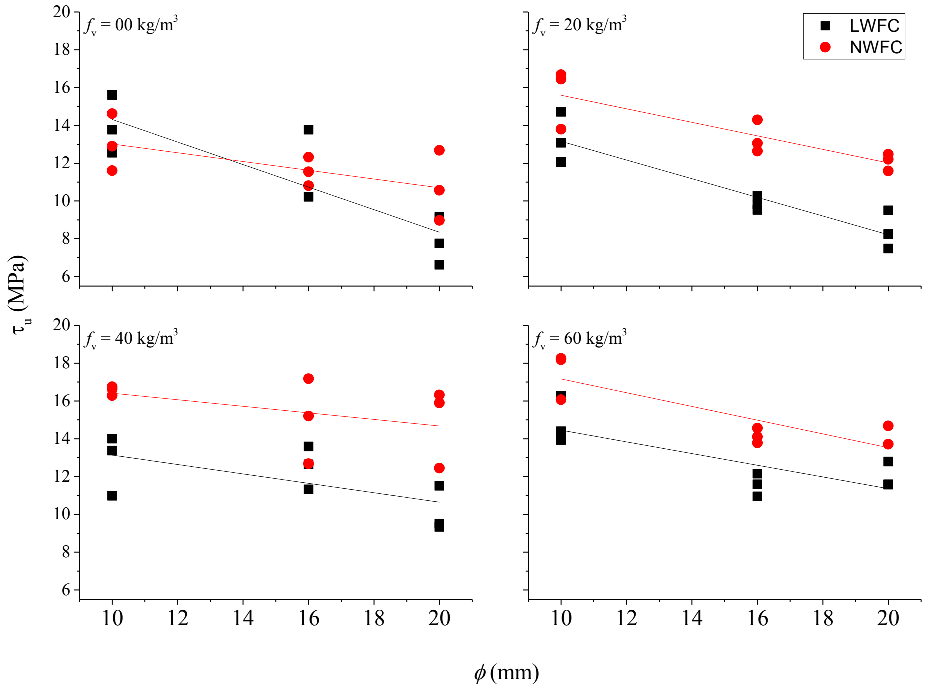


Figure 6.3 Ultimate bond strength as a function of bar size for all concrete types

6.2.2 Effect of fibres

6.2.2.1 Effect on bond stress-slip profile

In non-fibrous mixes of lightweight concrete, the descending branch of bonds stress-slip profile is very steep and short when compared with the NWC as can be seen (see Figure 6.4 (a)) in the bond stress-slip plot of 10 mm bar size specimens of both the Lightweight and normal weight concretes. It can be inferred from this behaviour that failure occurred suddenly in the aggregates due to their lower strength. This kind of behaviour started to change with the subsequent addition of steel fibres and more consistent softening branch was observed in the post splitting region of bond slip plots i.e. compared to the abrupt dipping of bond force the descent was more gradual and smooth as the fibre quantity in the mix increased, indicating the effectiveness of fibres in trapping the progressing cracks (see Figure 6.4 (b)).

Almost similar behaviour is observed in higher diameter bar specimens of non-fibrous mixes of LWC; i.e. failure also occurred in aggregates with sharp dipping of bond stress. However, because of better confinement due to larger concrete cover, resistance to slip was noted with higher bond stresses, whereas profiles of NWC have well defined descending branch without any sharp peaks (see Figure 6.4 (c)), which suggests that in case of normal weight concrete, aggregates did not fail, but the failure initiated in the cement paste. Overview of non-fibrous bond specimens after testing confirms this observation, compared to normal weight aggregates, lightweight aggregates particles can be seen completely fractured in Figure 6.5 (a).

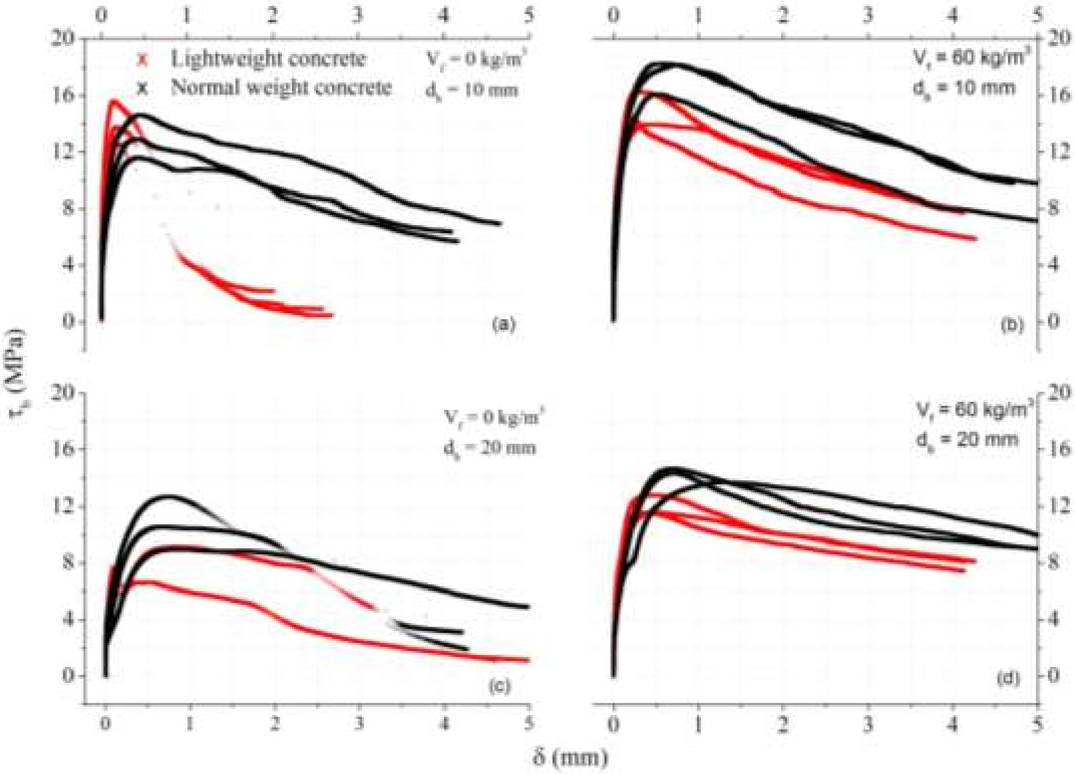


Figure 6.4 Bond stress-slip profiles of lightweight and normal weight concretes for 10 mm and 20 mm bar size specimens

(a) (b)

Figure 6.5 Overview of specimens after bond test (a) LWC, (b) NWC

Another interesting aspect observed in the bond behaviour of both the concretes was the improvement in the residual bond strength values. In current experimental work, it is observed that, there is significant improvement in the residual bond strength of LWFC and NWFC with addition of steel fibres. It is primarily because immediately after reaching to maximum bond stress values in non-fibrous mixes cracks becomes so wide that reinforced bar is easily pulled out without causing much damage to the concrete present between the bar ribs (see Figure 6.6 (a)), thus facing virtually no further resistance as reflected by the lower residual bond strength of the specimens (see Figure 6.6.(b)) Contrary to this behaviour, more of the concrete before lugs of the pull-out bar is found to be damaged (Figure 6.6 (b)) in fibrous mixes due to slip of bar due to increasing confining pressure with increasing fibre dosage, raising significantly the residual bond strength (Figure 6.6 (d)).

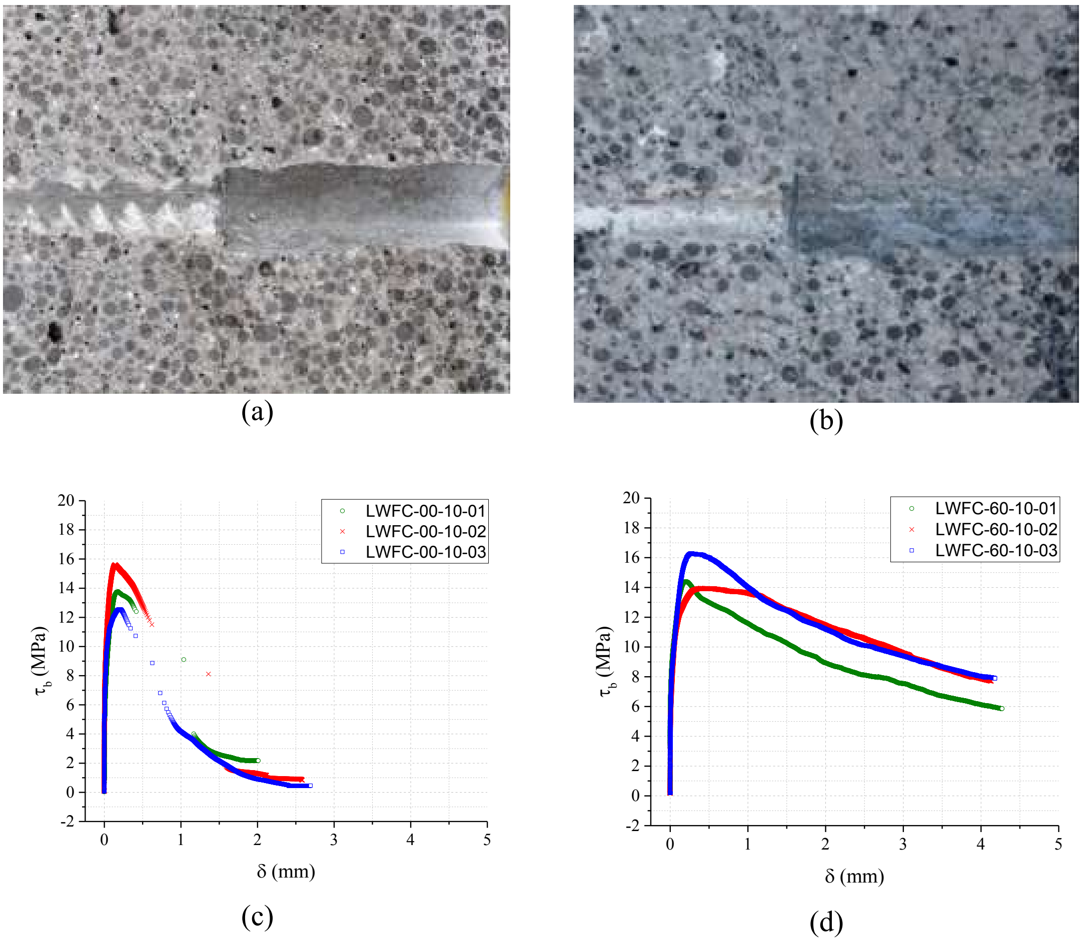


Figure 6.6 Overview of the bond specimens after test (a) without fibres (b) with fibres, along with their corresponding stress slip profiles (c) without fibres (d) with fibres

This improvement is especially higher in case of LWC (see Figure 6.4 (b)) and also for larger sized pull-out specimens (Figure 6.4 (d)). In case of NWC, normal weight aggregates also contribute in resisting crack propagation, for this reason, not so much improvement could be achieved by fibre addition compared to LWFC. Since, larger specimens, attain higher pull-out loads and hence dissipate more energy with more brittle nature of failure than smaller specimens, therefore better improvement in residual bond strength can be observed due to fibre incorporation. For example, when the fibre volume is increased from 20 kg/m^3 to 60 kg/m^3, residual bond strength of bond specimens of LWFC containing 16 mm pull-out bar increased by 60%, whereas, for specimens having 20 mm diameter bar this increase was about 95%. Similarly, for NWFC, these percentages are 53% and 65% for 16 mm and 20 mm pull-out bar specimens respectively.

Residual bond stress values determined at slip of 1, 2 and 4 mm are presented in Table 6.5 for LWFC and in Table 6.6 for NWFC. Significant improvement in the stress values recorded at bar displacement of 4 mm, with subsequent increase in fibre quantity, can be seen in the Figure 6.7. Due to lower fracture energy and high brittleness, almost all LWC specimens have zero bond strength at displacements greater than 3 mm. For this reason, percentage improvement in residual bond strength due to fibre addition is higher in LWFC than NWFC bond specimens; nevertheless, NWFC attained higher bond stress values at all fibre content levels. For example, at the slip value of 4 mm and for fibre dosage of 60 kg/m^3, the average of three test results shows that bond strength of LWFC is 73.77%, 78.18% and 78.02% of the bond strength of NWFC for 10 mm, 16 mm and 20 mm pull-out bars specimens respectively.

Fib – 2010 [33] for typical splitting mode of failures for all types of concretes, whether confined or unconfined, considers no contribution of frictional resistance and hence the residual bond strength values are zero as shown in its typical bond stress-slip law for monotonic loading (see Figure 6.8). In case of pull-out type of failures, code considers further resistance to slip offered by friction and its magnitude is taken equal to 40% of maximum bond stress. Also, in both the pull-out and splitting modes (for concrete confined with stirrups) of failure, bond law, after maximum bond stress, has the bi-linear stress-slip relationship. Although, fibre reinforcement depicts to some extent the confinement effect similar to that of stirrups, experimental bond stress-slip laws did not show any bi-linear relationship in post peak splitting region at maximum fibre content level and up to maximum slip value that could possibly be recorded with current experimental setup.

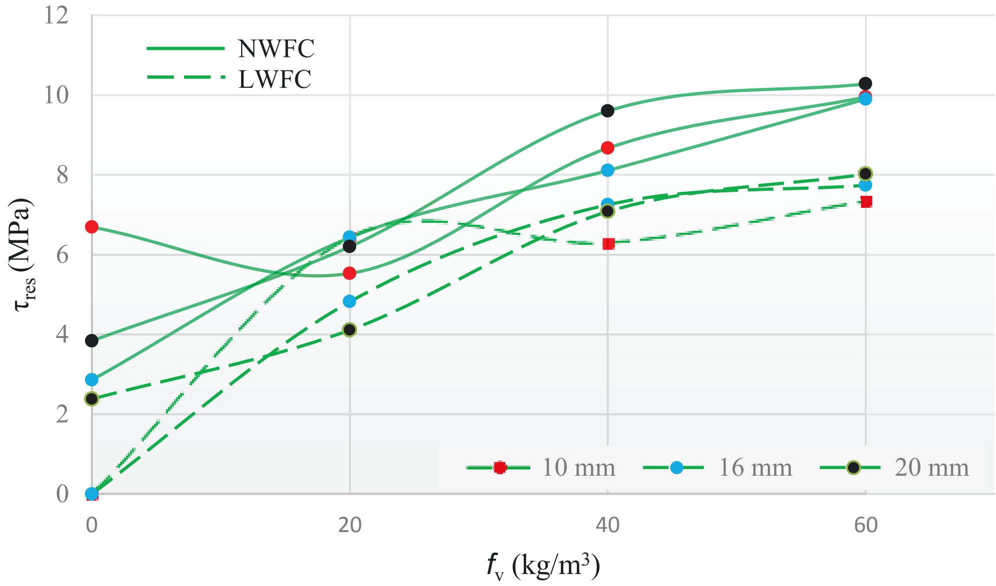


Figure 6.7 Residual bond stress values recorded at slip of 4 mm

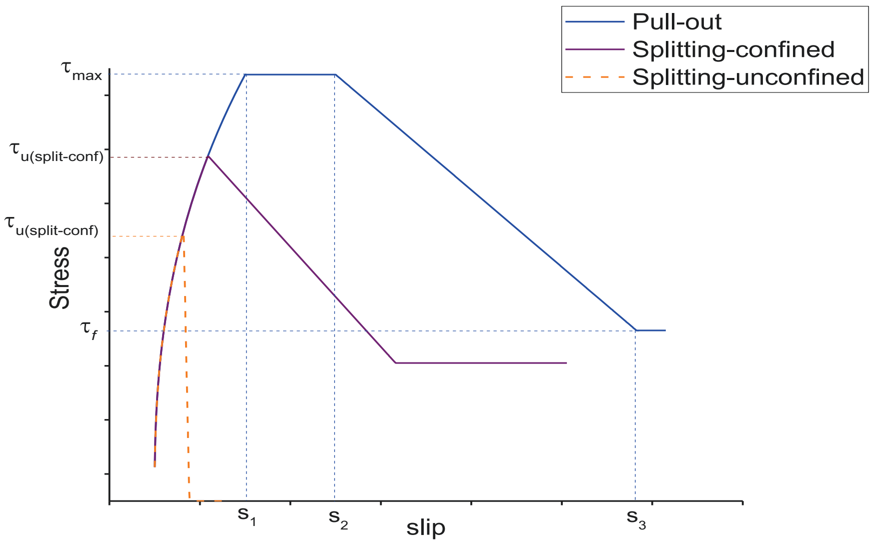


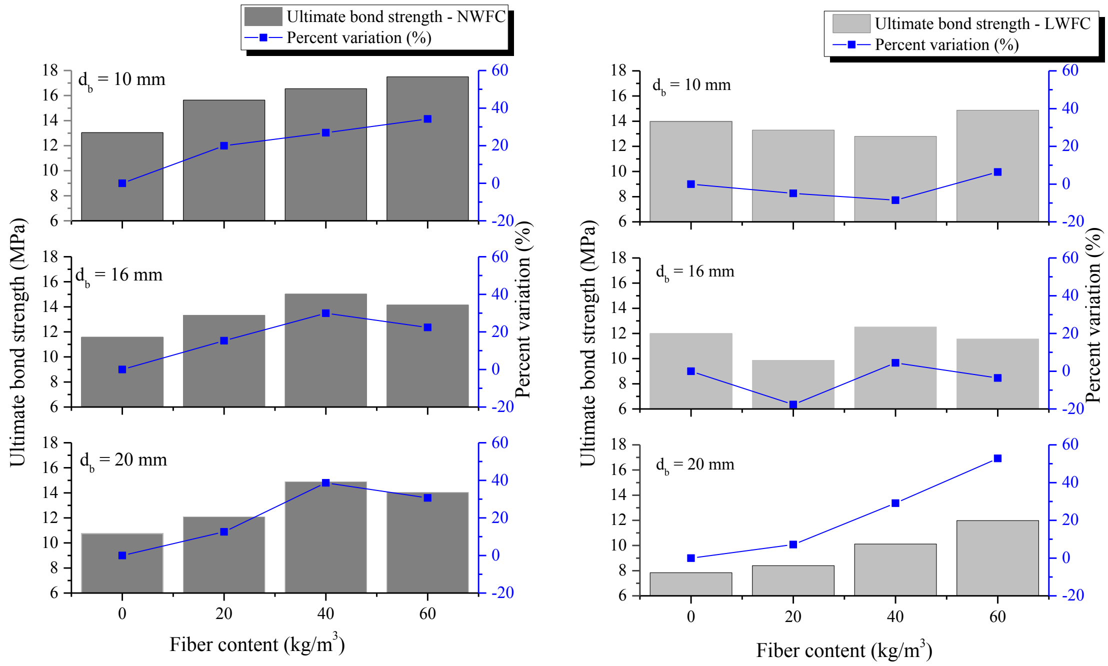
Figure 6.8 Analytical bond stress – slip law for confined and unconfined concretes proposed in fib-2010 code [33]

6.2.2.2 Effect on ultimate bond strength

Clear effect of fibres in improving the ultimate bond strength of NWFC is observed, where, on average bond strength of pull-out specimens, for all bar sizes, increased by 29% at the maximum fibre content. Improvement in the bond strength was more pronounced in higher diameter bars at fibre dosage of 40 kg/m^3 with an increment of 38% for 20 mm bar size and about 30 % for 16 mm bar shown in Figure 6.9 (a). This enhancement with increasing fibre content could be due to better bond between matrix and fibres. Disturbance in packing/density of matrix at maximum fibre content is believed to be the reason for reduction in compressive strength of concrete and thus the ultimate bond strength of 16 mm and 20 mm bars. Although, density is higher at this fibre volume (see Table 5.3), but, this rise in density is believed to be due to weight of fibres. This highlights the fact that compressive strength has strong influence on bond. Contrary to this 10 mm bar has improved bond strength at this fibre content (60 kg/m^3) which could be due to the presence of fibres near crack region or/and that length of fibres was adequate enough for trapping multiple cracks due to smaller size of specimens, thus delaying crack propagation and increasing bond strength, however this aspect needs further investigation.

For both 10 mm and 16 mm bar size specimens of LWFC, no distinct effect of fibres on the ultimate bond strength could be observed, and the bond strength for these bar sizes remains almost unchanged even at the highest fibre content level. However, progressive improvement in the ultimate bond strength of 20 mm bar size specimens is recorded and specimens of this series attained the highest percent increase among all the pull-out specimens of both the LWFC and NWFC series (see Figure 6.9 (b)). Lower relative density/strength of LWA particles may be considered as the factor for ineffectiveness of fibres in improving the ultimate bond strength of 10 mm and 16 mm bar size specimens, as fibres may not have been able to cover cracks initiated from both the cement and weaker aggregates, whereas for 20 mm bar size specimens additional concrete cover helped in raising the bond strength. Nevertheless, the ultimate bond strength of LWFC specimens for all bar sizes at maximum fibre dosage is found either equal to or higher than the conventional normal weight concrete having no fibres as shown in Figure 6.10. At this fibre content i.e. 60 kg/m^3, the ultimate bond strength of LWFC-60-10 is higher than NWFC 00-10 by 14% and similarly for

20 mm bar size specimens it is higher by 11.5 %. This suggests that using adequate quantity of fibres in LWC, bond strength, similar to that of NWC can be achieved.



(a) (b)

Figure 6.9 Effect of fibres on ultimate bond strength (a) NWFC (b) LWFC

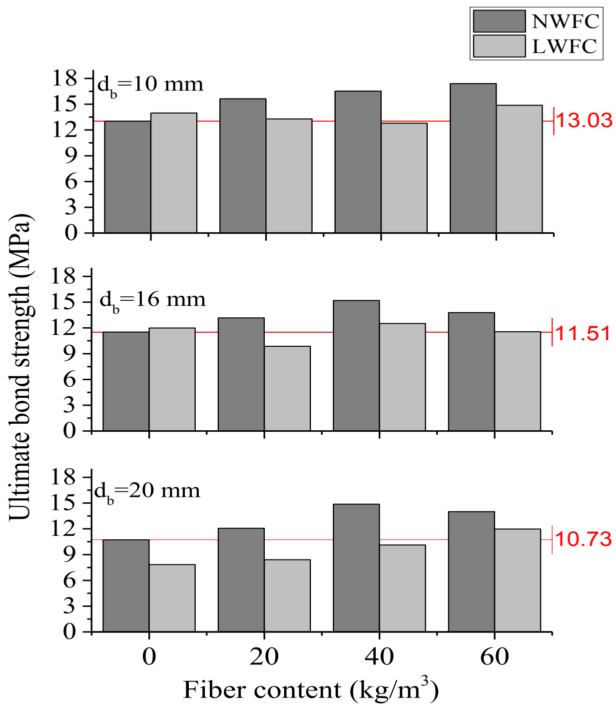


Figure 6.10 Ultimate bond strength of LWFC and NWFC in relation with the bond strength of NWC shown by red line.

6.3 Estimation of Bond Strength

Since most of the bond expressions are based on the experimental results of NWC and that too without fibres. It is therefore reasonable to see first, how closely these equations predict bond strength of conventional concrete reinforced with steel fibres. Pull-out test results of NWFC presented in Table 6.3 are compared with the equations of ACI 408 [4], fib-2010 [33] and equation given by Orangun et al. [170] on which the famous design equation of ACI-318 [32] for development length is based. Table 6.7 presents test results and prediction by these equations ((6.1) to (6.3)).

$$\frac{T_b}{f_c'^{1/4}} = \frac{A_s f_s}{f_c'^{1/4}} = \left[1.43 l_b \ C_{min} + 0.5\phi \ + 57.4 A_s\right]\left(0.1\frac{C_{max}}{C_{min}} + 0.9\right) \tag{6.1}$$

$$\tau_{u,split} = \eta_2 .6.54.\left(\frac{f_{ck}}{20}\right)^{0.25}.\left(\frac{20}{\phi}\right)^{0.2}.\left[\left(\frac{c_{min}}{\phi}\right)^{0.33}.\left(\frac{c_{max}}{c_{min}}\right)^{0.1} + 8K_{tr}\right] \tag{6.2}$$

$$\frac{\tau_b}{\sqrt{f_c'}} = 0.10 + 0.25\frac{c_{min}}{\phi} + 4.15\frac{\phi}{l_b} \tag{6.3}$$

Origin of the above equations is presented in detail in section 3.4. (6.1)) is expressed in terms of bond force and must be converted to bond stress by dividing it with circumferential area $(\pi l_b \phi)$.

Because of the similar cover to bar diameter ratio $\left(\frac{c}{\phi}\right)$ and also similar bar diameter to development length ratio $\left(\frac{\phi}{l_b}\right)$, (6.1)) and (6.3)) yield identical bond strength results for all specimen sizes of any fibre content series. (6.2)), however is independent of bar size to development length ratio but considers the effect of bar size using factor $\left(\frac{20}{\phi}\right)^{0.2}$ in the equation. It is for this reason that it shows variation of bond strength with good prediction results as shown in Figure 6.11. The parameter η_2 in (6.2) is taken equal to 1 for calculations, assuming that good bond conditions exist, for other bond conditions code suggests value of 0.7.

It can also be observed that as the fibre content increases, prediction of ultimate bond strength by all these bond expressions start to become conservative. Many of the research works [173], [169], [214] show that improvement in the material and structural properties of concrete as a result of steel fibre addition could be the function of fibre aspect ratio multiplied by the fibre volume fraction, the product is also known as fibre factor $\left(\frac{l_\mathrm{f} V_\mathrm{f}}{d_\mathrm{f}}\right)$.

The effect of confinement, provided by transverse reinforcement, in enhancing the ultimate bond strength in (6.2) of fib-2010 is represented by $\left[K_{tr}\right]$, called transverse reinforcement factor. In absence of such reinforcement, the improvement of ultimate bond strength with increasing fibre content can be considered the function of fibre factor. To strengthen this argument, fibre factor $\left(\frac{l_\mathrm{f} V_\mathrm{f}}{d_\mathrm{f}}\right)$ is included and adjusted in the Eq. (6.4) and the plots of experimental results vs predicted values are drawn for all the specimens of NWFC (see Figure 6.12), also the mean and standard deviation values for the (6.1), (6.2) and (6.4) are presented in Table 6.8. It can be seen that proposed equation not only predicts well the experimental results with better mean values, but also the dispersion of the results from mean values expressed as standard deviation is also lower than the other bond expressions.

The limitation of $V_f \geq 0.5$ is also imposed in the proposed equation, as it is found that fibre volume fraction below this level has either negligible positive effect on bond strength as in case of NWFC pull-out test results, or in some cases has negative influence as observed in the behaviour of LWFC specimens (see Figure 6.9 (b)). Hence the contribution of fibres below this level shall not be considered i.e. $\left(\frac{l_\mathrm{f} V_\mathrm{f}}{d_\mathrm{f}}\right)$ shall be set to zero.

$$\tau_{\mathrm{u,split}} = \eta_2 .6.54. \left(\frac{f_{\mathrm{ck}}}{20}\right)^{0.25} . \left(\frac{20}{\phi}\right)^{0.2} . \left[\left(\frac{c_{\mathrm{min}}}{\phi}\right)^{0.33} . \left(\frac{c_{\mathrm{max}}}{c_{\mathrm{min}}}\right)^{0.1} + 8K_{\mathrm{tr}}\right] + \left(\frac{l_\mathrm{f} V_\mathrm{f}}{d_\mathrm{f}}\right)^{0.1} \tag{6.4}$$

Where, $V_\mathrm{f} \geq 0.5$

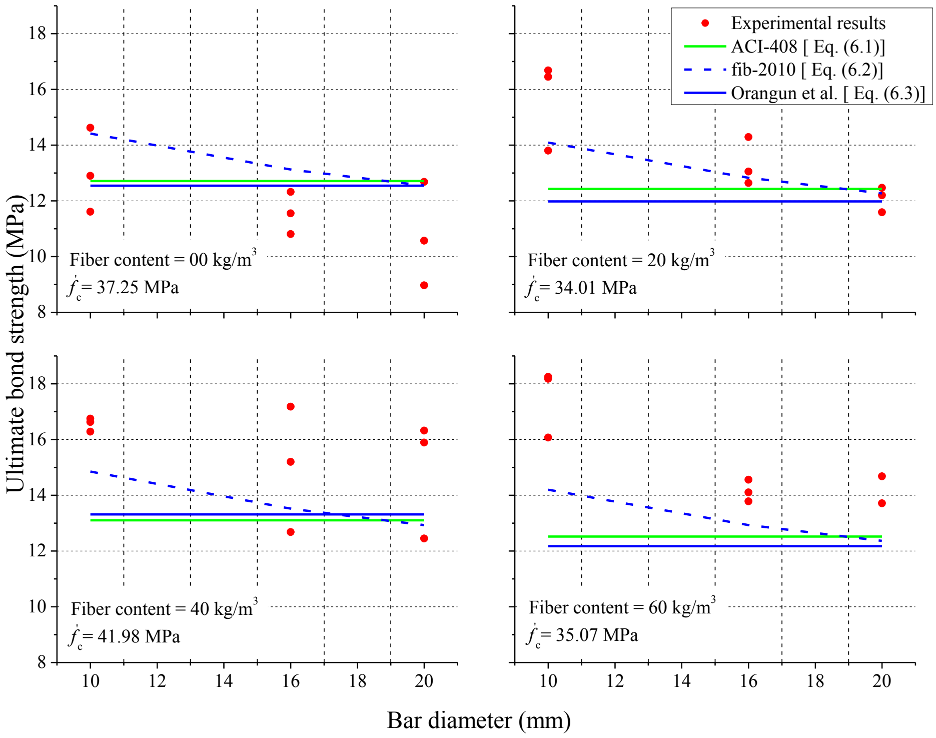


Figure 6.11 Prediction of bond strength for NWFC by different equations

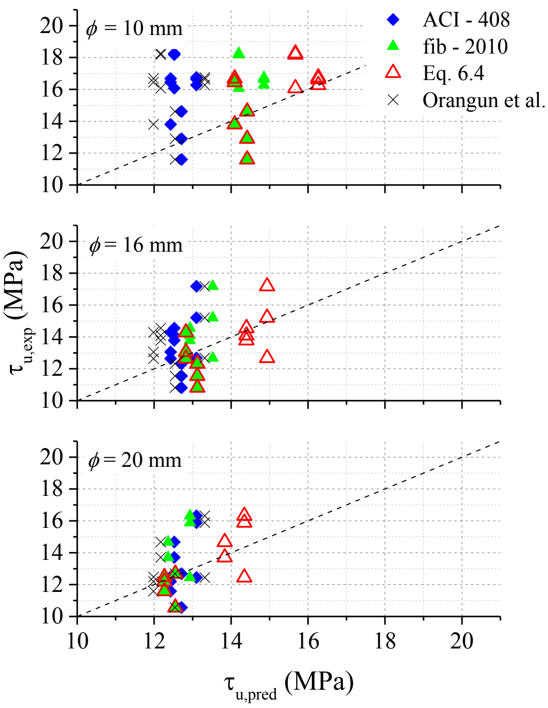


Figure 6.12 Ultimate bond strength of NWFC plotted against predicted values by different code expressions

The proposed equation (6.4) also appears to be predicting well the ultimate bond strength of LWFC bond specimens with some modification. ACI – 318 reduces the bond strength of LWC by 30% owing to its lower tensile strength (see Eq. (6.5)). This penalty of lowering the bond strength by factor 1.3 is also continued by the ACI Committee– 408 in its proposed design expression for the development length, and is represented by symbol λ as mentioned earlier in section 3.3.2. However no such parameter is incorporated in the bond expressions of fib-2010.

Therefore, ultimate bond strength of LWFC pull-out specimens is calculated using (6.1) and (6.4) after including λ = 1.3, whereas fib-2010 equation ((6.2)) remains unchanged.

$$\frac{T_{\mathrm{b}}}{f_{\mathrm{c}}'^{1/4}}=\frac{A_{\mathrm{s}}f_{\mathrm{s}}}{f_{c}'^{1/4}}=\left[1.43l_{\mathrm{b}}\ \ C_{\min}+0.5\phi\ \ +57.4A_{\mathrm{s}}\right]\left(0.1\frac{C_{\max}}{C_{\min}}+0.9\right).\frac{1}{\lambda} \tag{6.5}$$

$$\tau_{\mathrm{u,split}}=\eta_2.6.54.\left(\frac{f_{\mathrm{ck}}}{20}\right)^{0.25}.\left(\frac{20}{\phi}\right)^{0.2}.\left[\left(\frac{c_{\min}}{\phi}\right)^{0.33}.\left(\frac{c_{\max}}{c_{\min}}\right)^{0.1}+8K_{\mathrm{tr}}\right] \tag{6.2}$$

$$\tau_{\mathrm{u,split}}=\left[\eta_2.6.54.\left(\frac{f_{\mathrm{ck}}}{20}\right)^{0.25}.\left(\frac{20}{\phi}\right)^{0.2}.\left[\left(\frac{c_{\min}}{\phi}\right)^{0.33}.\left(\frac{c_{\max}}{c_{\min}}\right)^{0.1}+8K_{\mathrm{tr}}\right]+\left(\frac{l_{\mathrm{f}}V_{\mathrm{f}}}{d_{\mathrm{f}}}\right)^{0.1}\right].\frac{1}{\lambda} \tag{6.6}$$

Where; $V_{\mathrm{f}} \geq 0.5$

For LWFC specimens, the proposed equation too predicts the ultimate bond strength reasonably well as can be seen from the mean and standard deviation values (see Table 6.9) and the graphs (see Figure 6.13) of experimental results plotted against the predicted values for different bar sizes.

6.4 Bond Stress-Slip Law for LWFC

As highlighted earlier that one of the many hindrances in wide acceptability of lightweight concrete is the absence of normative rules, and this is especially true when such concrete is reinforced with fibres. Fib Model Code – 2010 for example has a well-defined bond stress-slip law/relationship for the normal weight concrete for different modes of bond failure i.e. pull-out and splitting for confined and unconfined concretes as shown in Figure 6.8. Such relationships are very useful in defining the expected material behaviour and can be used in structural analysis programs to observe overall performance of a structure before on ground construction.

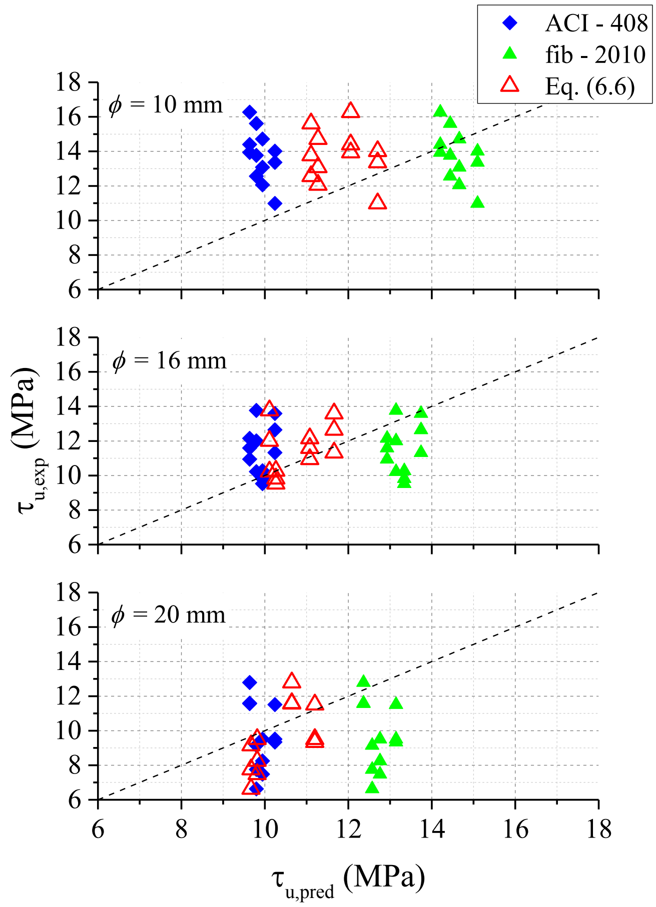


Figure 6.13 Ultimate bond strength of LWFC plotted against predicted values by different code expressions

The pull-out behaviour of NWC is defined by four different stages in fib-2010, the ascending branch, constant maximum stress, descending stress branch and then final stage reflected by resistance offered by the surface friction. Whereas, for splitting mode of failure in a concrete, confined by stirrups, the bond relationship has three stages and for unconfined concrete there are two stages. All these stages are defined by the following equations.

$$\tau = \tau_{max}\left(s / s_1 \right)^{\alpha} \qquad 0 \leq s \leq s_1 \qquad (6.7)$$

$$\tau = \tau_{max} \qquad s_1 \leq s \leq s_2 \qquad (6.8)$$

$$\tau = \tau_{\max} - \quad \tau_{\max} - \tau_f \quad \frac{s - s_2}{s_3 - s_2} \qquad s_2 \le s \le s_3 \tag{6.9}$$

$$\tau = \tau_f \qquad s > s_3 \tag{6.10}$$

Parameters used in the above equations are defined in Table 6.1. There were two distinct differences observed in the bond stress-slip profile of LWFC when it was plotted using the above four equations ((6.7 - (6.10) and Table 6.1, assuming good bond conditions. The first change observed was that the ultimate bond stress of the majority of specimens was less by a factor ranging between 1.2 to 1.3, which is the reflection of lower tensile strength of LWC; however, unlike ACI-318, such reduction is not integrated in bond stress-slip law of fib-2010. Secondly in the post peak stress region, the descending branch has mild slope for LWFC specimens compared to both the confined and unconfined bond stress behaviours defined by fib as shown in Figure 6.14. It is emphasized that for the general cases, for estimating ultimate bond strength for splitting mode of bond failure, fib suggests use of formulae $7\left(\frac{f_c'}{20}\right)^{0.25}$ and $8\left(\frac{f_c'}{20}\right)^{0.25}$ for unconfined and confined conditions respectively. For specific cases, such as current one where reinforcement and concrete details are available forehand to the designer, such calculations should be carried out using (6.2).

All these observations have been considered in the modified bond stress-slip law proposed here for LWFC, where some changes are recommended. For example, the slip values s_1 and s_2 have been fixed at 0.65 mm as all the specimens were noted to fail in splitting at slip values lower than 0.65 mm and $s_3 = 1.2s_1 + 7V_f$. Determination of s_3 using above formula is indicative of observation that slope of descending branch is affected by fibre volume fraction i.e. higher fibre volume leads to more mild slope and better ductile behaviour.

Furthermore the slope of ascending stress-slip branch has been attuned slightly by setting α equal to 0.3 instead of 0.4.

All these proposed modifications for bond stress-slip law for LWFC are presented in Table 6.1 and the resulting bond stress-slip laws are plotted for all the bars at maximum fibre content shown in Figure 6.15.

Table 6.1 Parameters for defining bond stress-slip relationship

Parameter	fib Model Code-2010			Proposed modifications
	Pull-out	Splitting-unconfined	Splitting-confined	Splitting
τ_{max}	$2.5\sqrt{f_c'}$	(6.2)	(6.2)	(6.6)
s_1	1 mm	$s\ \tau_{max}$	$s\ \tau_{max}$	0.65
s_2	2 mm	s_1	s_1	0.65
s_3	C_{clear}	$1.2s_1$	$0.5C_{clear}$	$1.2s_1 + 7V_f$
α	0.4	0.4	0.4	0.3
τ_f	$0.4\tau_{max}$	0	$0.4\tau_{max}$	$0.4\tau_{max}$

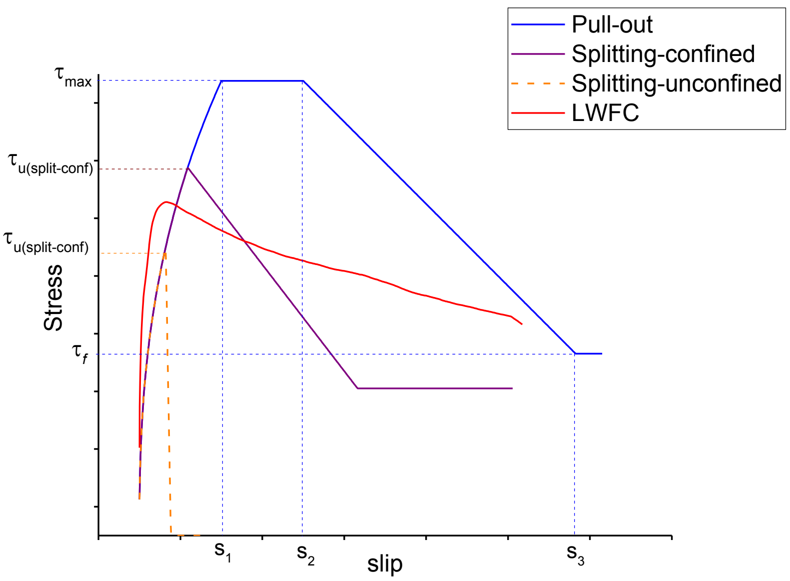


Figure 6.14 Comparison of observed bond behaviour of LWFC against the treatment of fib-2010 for the same

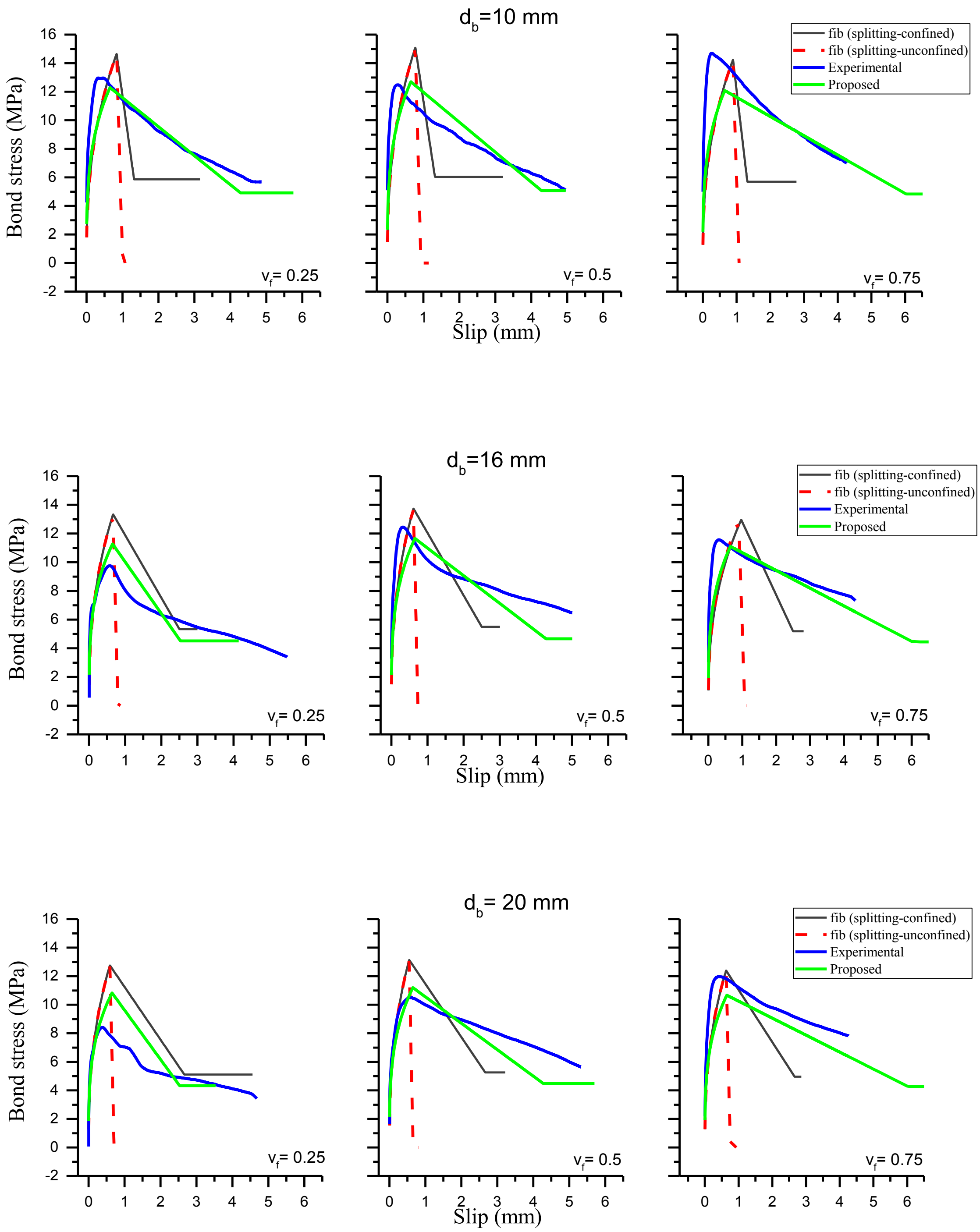


Figure 6.15 Comparison of the fib bond stress-slip law with the observed and proposed bond stress relationship for LWFC

Note: d_b = bar diameter

Table 6.2 Ultimate bond strength of LWFC Pull-out specimens

S. No.	Specimen code	Specimen size (mm) (L x W x H)	ϕ (mm)	$f_c^{'}$ (MPa)	$P_{\max}$ (kN)	τ_u (MPa)
1	LWFC-00-10-01	100 x 100 x 100	10	37.52	21.63	13.77
2	LWFC-00-10-02	100 x 100 x 100	10		24.52	15.61
3	LWFC-00-10-03	100 x 100 x 100	10		19.72	12.55
4	LWFC-00-16-01	160 x 160 x 160	16		55.38	13.77
5	LWFC-00-16-02	160 x 160 x 160	16		45.68	11.99
6	LWFC-00-16-03	160 x 160 x 160	16		41.11	10.22
7	LWFC-00-20-01	200 x 200 x 200	20		48.72	7.75
8	LWFC-00-20-02	200 x 200 x 200	20		57.47	9.15
9	LWFC-00-20-03	200 x 200 x 200	20		41.64	6.63
10	LWFC-20-10-01	100 x 100 x 100	10	39.77	23.12	14.72
11	LWFC-20-10-02	100 x 100 x 100	10		18.95	12.06
12	LWFC-20-10-03	100 x 100 x 100	10		20.55	13.08
13	LWFC-20-16-01	160 x 160 x 160	16		39.47	9.81
14	LWFC-20-16-02	160 x 160 x 160	16		38.33	9.53
15	LWFC-20-16-03	160 x 160 x 160	16		41.32	10.27
16	LWFC-20-20-01	200 x 200 x 200	20		51.84	8.25
17	LWFC-20-20-02	200 x 200 x 200	20		47.01	7.48
18	LWFC-20-20-03	200 x 200 x 200	20		59.74	9.50
19	LWFC-40-10-01	100 x 100 x 100	10	44.75	17.25	10.98
20	LWFC-40-10-02	100 x 100 x 100	10		20.99	13.37
21	LWFC-40-10-03	100 x 100 x 100	10		22.01	14.01
22	LWFC-40-16-01	160 x 160 x 160	16		54.64	13.59
23	LWFC-40-16-02	160 x 160 x 160	16		45.53	11.32
24	LWFC-40-16-03	160 x 160 x 160	16		50.85	12.65
25	LWFC-40-20-01	200 x 200 x 200	20		72.32	11.51
26	LWFC-40-20-02	200 x 200 x 200	20		59.77	9.51
27	LWFC-40-20-03	200 x 200 x 200	20		58.71	9.34
28	LWFC-60-10-01	100 x 100 x 100	10	35.41	22.62	14.4
29	LWFC-60-10-02	100 x 100 x 100	10		21.89	13.94
30	LWFC-60-10-03	100 x 100 x 100	10		25.56	16.27
31	LWFC-60-16-01	160 x 160 x 160	16		46.63	11.59
32	LWFC-60-16-02	160 x 160 x 160	16		44.02	10.95
33	LWFC-60-16-03	160 x 160 x 160	16		48.86	12.15
34	LWFC-60-20-01	200 x 200 x 200	20		72.68	11.57
35	LWFC-60-20-02	200 x 200 x 200	20		72.85	11.59
36	LWFC-60-20-03	200 x 200 x 200	20		80.35	12.79

Table 6.3 Ultimate bond strength of NWFC Pull-out specimens

S. No.	Specimen code	Specimen size (mm) (L x W x H)	ϕ (mm)	$f_c^{'}$ (MPa)	$P_{\max}$ (kN)	τ_u (MPa)
1	NWFC-00-10-01	100 x 100 x 100	10	37.25	22.96	14.62
2	NWFC-00-10-02	100 x 100 x 100	10		20.26	12.90
3	NWFC-00-10-03	100 x 100 x 100	10		18.23	11.61
4	NWFC-00-16-01	160 x 160 x 160	16		43.48	10.81
5	NWFC-00-16-02	160 x 160 x 160	16		49.56	12.32
6	NWFC-00-16-03	160 x 160 x 160	16		46.45	11.55
7	NWFC-00-20-01	200 x 200 x 200	20		66.39	10.57
8	NWFC-00-20-02	200 x 200 x 200	20		56.37	8.97
9	NWFC-00-20-03	200 x 200 x 200	20		79.70	12.68
10	NWFC-20-10-01	100 x 100 x 100	10	34.01	21.68	13.80
11	NWFC-20-10-02	100 x 100 x 100	10		26.20	16.68
12	NWFC-20-10-03	100 x 100 x 100	10		25.84	16.45
13	NWFC-20-16-01	160 x 160 x 160	16		52.49	13.05
14	NWFC-20-16-02	160 x 160 x 160	16		57.45	14.29
15	NWFC-20-16-03	160 x 160 x 160	16		50.82	12.64
16	NWFC-20-20-01	200 x 200 x 200	20		72.80	11.59
17	NWFC-20-20-02	200 x 200 x 200	20		76.65	12.20
18	NWFC-20-20-03	200 x 200 x 200	20		78.38	12.47
19	NWFC-40-10-01	100 x 100 x 100	10	41.98	25.57	16.28
20	NWFC-40-10-02	100 x 100 x 100	10		26.31	16.75
21	NWFC-40-10-03	100 x 100 x 100	10		26.13	16.63
22	NWFC-40-16-01	160 x 160 x 160	16		50.98	12.68
23	NWFC-40-16-02	160 x 160 x 160	16		61.14	15.20
24	NWFC-40-16-03	160 x 160 x 160	16		69.08	17.18
25	NWFC-40-20-01	200 x 200 x 200	20		99.83	15.89
26	NWFC-40-20-02	200 x 200 x 200	20		102.55	16.32
27	NWFC-40-20-03	200 x 200 x 200	20		78.22	12.45
28	NWFC-60-10-01	100 x 100 x 100	10	35.07	28.56	18.18
29	NWFC-60-10-02	100 x 100 x 100	10		25.25	16.07
30	NWFC-60-10-03	100 x 100 x 100	10		28.67	18.25
31	NWFC-60-16-01	160 x 160 x 160	16		56.69	14.10
32	NWFC-60-16-02	160 x 160 x 160	16		55.43	13.78
33	NWFC-60-16-03	160 x 160 x 160	16		58.53	14.56
34	NWFC-60-20-01	200 x 200 x 200	20		92.21	14.68
35	NWFC-60-20-02	200 x 200 x 200	20		86.14	13.71
36	NWFC-60-20-03	200 x 200 x 200	20		86.15	13.71

Table 6.4 Average results of ultimate bond strength of different bar sizes of LWFC and NWFC

		Ultimate bond strength, τ_u (MPa)		
		ϕ (mm)		
Concrete type	f_v (kg/m^3)	10	16	20
LWFC	0	13.98	11.99	7.84
	20	13.29	9.87	8.41
	40	12.79	12.52	10.12
	60	14.87	11.56	11.98
NWFC	0	13.04	11.56	10.74
	20	15.64	13.33	12.09
	40	16.55	15.02	14.89
	60	17.5	14.15	14.03

Table 6.5 Residual bond strength of LWFC at different slip values

		Residual bond strength, τ_{res} (MPa)		
f_v (kg/m^3)	τ_u (MPa)	δ = 1 mm	δ = 2 mm	δ = 4 mm
		ϕ = 10 mm		
0	13.98	7.74	1.45	-
20	13.29	11.43	9.23	6.44
40	12.79	10.50	8.78	6.29
60	14.87	13.07	10.52	7.34
		ϕ = 16 mm		
0	11.99	-	-	-
20	9.87	8.04	6.32	4.82
40	12.52	10.15	8.84	7.25
60	11.56	10.48	9.41	7.74
		ϕ = 20 mm		
0	7.84	7.50	6.08	2.38
20	8.41	7.04	5.20	4.11
40	10.12	9.98	8.95	7.08
60	11.98	11.20	9.79	8.02

Note: values are the average of three test results

Table 6.6 Residual bond strength of NWFC specimens at different slip values

f_v (kg/m^3)	τ_u (MPa)	Residual bond strength, τ_{res} (MPa)		
		δ = 1 mm	δ = 2 mm	δ = 4 mm
	ϕ = 10 mm			
0	13.04	11.96	10.26	6.69
20	15.64	13.73	9.55	5.53
40	16.55	15.67	12.75	8.67
60	17.5	16.83	14.21	9.95
	ϕ = 16 mm			
0	11.56	10.84	8.39	2.86
20	13.33	12.23	9.12	6.43
40	15.02	14.54	11.73	8.11
60	14.15	14.01	12.84	9.90
	ϕ = 20 mm			
0	10.74	10.56	9.05	3.84
20	12.09	11.28	8.46	6.20
40	14.89	14.64	12.12	9.60
60	14.03	13.83	12.45	10.28

Note: values are the average of three test results

Table 6.7 Estimation of bond strength of NWFC by different bond expressions

S. No.	Specimen size (mm x mm)	ϕ (mm)	f_c' (MPa)	$P_{\max}$ (kN)	τ_u (MPa)	fib – 2010 (MPa)	ACI – 408 (MPa)	Orangun et al. (MPa)
1	100 x 100	10		22.96	14.62	14.42	12.71	12.54
2	100 x 100	10		20.26	12.90	14.42	12.71	12.54
3	100 x 100	10		18.23	11.61	14.42	12.71	12.54
4	160 x 160	16		43.48	10.81	13.12	12.71	12.54
5	160 x 160	16	37.25	49.56	12.32	13.12	12.71	12.54
6	160 x 160	16		46.45	11.55	13.12	12.71	12.54
7	200 x 200	20		66.39	10.57	12.55	12.71	12.54
8	200 x 200	20		56.37	8.97	12.55	12.71	12.54
9	200 x 200	20		79.70	12.68	12.55	12.71	12.54
10	100 x 100	10		21.68	13.80	14.09	12.43	11.98
11	100 x 100	10		26.20	16.68	14.09	12.43	11.98
12	100 x 100	10		25.84	16.45	14.09	12.43	11.98
13	160 x 160	16		52.49	13.05	12.83	12.43	11.98
14	160 x 160	16	34.01	57.45	14.29	12.83	12.43	11.98
15	160 x 160	16		50.82	12.64	12.83	12.43	11.98
16	200 x 200	20		72.80	11.59	12.27	12.43	11.98
17	200 x 200	20		76.65	12.20	12.27	12.43	11.98
18	200 x 200	20		78.38	12.47	12.27	12.43	11.98
19	100 x 100	10		25.57	16.28	14.85	13.10	13.31
20	100 x 100	10		26.31	16.75	14.85	13.10	13.31
21	100 x 100	10		26.13	16.63	14.85	13.10	13.31
22	160 x 160	16		50.98	12.68	13.52	13.10	13.31
23	160 x 160	16	41.98	61.14	15.20	13.52	13.10	13.31
24	160 x 160	16		69.08	17.18	13.52	13.10	13.31
25	200 x 200	20		99.83	15.89	12.93	13.10	13.31
26	200 x 200	20		102.55	16.32	12.93	13.10	13.31
27	200 x 200	20		78.22	12.45	12.93	13.10	13.31
28	100 x 100	10		28.56	18.18	14.20	12.52	12.17
29	100 x 100	10		25.25	16.07	14.20	12.52	12.17
30	100 x 100	10		28.67	18.25	14.20	12.52	12.17
31	160 x 160	16		56.69	14.10	12.93	12.52	12.17
32	160 x 160	16	35.07	55.43	13.78	12.93	12.52	12.17
33	160 x 160	16		58.53	14.56	12.93	12.52	12.17
34	200 x 200	20		92.21	14.68	12.36	12.52	12.17
35	200 x 200	20		86.14	13.71	12.36	12.52	12.17
36	200 x 200	20		86.15	13.71	12.36	12.52	12.17

Table 6.8 Mean and Standard Deviation values calculated against proposed and other bond expressions for NWFC specimens

Equation	ϕ					
	10 mm		16 mm		20 mm	
	Mean	S. Dev	Mean	S. Dev	Mean	S. Dev
ACI – 408 (Eq. 6.1)	1.274	0.151	1.076	0.119	1.001	0.145
Fib – 2010 (Eq. 6.2)	1.124	0.133	1.042	0.115	1.014	0.146
Proposed (Eq: 6.4)	1.024	0.107	0.982	0.089	0.993	0.109

Table 6.9 Mean and Standard Deviation values calculated against proposed and other bond expressions for LWFC specimens

Equation	ϕ					
	10 mm		16 mm		20 mm	
	Mean	S. Dev	Mean	S. Dev	Mean	S. Dev
ACI – 408 (Eq. 6.5)	1.387	0.154	1.17	0.128	0.931	0.186
Fib – 2010 (Eq. 6.2)	0.941	0.105	0.872	0.095	0.726	0.145
Proposed (Eq: 6.6)	1.158	0.134	1.029	0.114	0.898	0.142

7 Conclusions and Recommendations

7.1 General Remarks

This research work was aimed at studying the bond behaviour of lightweight fibre-reinforced concrete. Lightweight concrete has numerous advantages over conventional concrete, but it lags behind its true market potential due to higher initial cost and poor ductility. For these reasons, engineers have shown restraint in its acceptance as a construction material, especially in seismic areas. Efforts are continuously being made to reduce the brittleness of lightweight concrete without much affecting its lower density quality. Use of discrete fibres in lightweight concrete is one such step in this dimension. There are normative rules and bond stress-slip relationship defined in codes for conventional concrete and substantial data for lower diameter bar sizes; however, these are not available for concretes reinforced with discrete fibres. An effort was made in current research work to address some of these issues by performing pull-out tests on lightweight and normal weight concretes with and without hooked-end steel fibres. Following section contains the conclusions which could be drawn from these tests, followed by the recommendation for future work.

7.2 Conclusions

7.2.1 Bond

It is possible to produce structural LWFC of similar strength class as that of NWFC with small variations in material quantities except for coarse aggregates. For example, cement quantity required for producing 1 cubic meter of LWFC was only 10 kg higher than that needed for NWFC. Maximum difference of 4.92 MPa in the compressive strength of both the LWFC and NWFC was recorded at fibre dosage of 00 kg/m^3.

Although there is some improvement (29%) in ultimate bond strength of NWFC specimens, such improvements were not observed for LWFC, especially at lower fibre volumes i.e. for $V_f < 0.5\%$. Since NWFC mixes were rich in fine aggregate, therefore it is possible that better bond of fibres with NWFC matrix resulted in improvement in ultimate bond strength. Post-cracking bond strength of both the LWFC and NWFC is however significantly improved and this improvement is higher in case of LWFC.

Fibres are not as effective as the compressive strength of concrete in enhancing the maximum bond strength and strong variation in bond strength is observed with the variation in compressive strength.

Pre-cracking stage of bond stress-slip profile is little affected by fibres, nevertheless, in the later stage i.e. in the post maximum bond stress region; bond stress-slip relationship is highly affected by fibre volume. In other words, ductility and bond strength in post cracking region can be improved considerably by fibre addition due to restriction of splitting cracks by fibres.

Similar specimen size for all bars in RILEM test methodology favours smaller bars, therefore same cover to bar size ratio was adopted for current experimental work. However, even with similar c/ϕ ratio, the bond strength of larger diameter pull-out bars was lesser than the bond specimens having smaller diameter pull-out bars. For example on an average the bond strength of 10 mm bar size specimens was higher than 16 mm and 20 mm bar sized specimens by 13.6% and 17.5% respectively.

For all sizes of bars, the mode of failure recorded in current experimental work is splitting type. Pull-out specimens, without fibres, and having highest diameter bar (20 mm bar) failed in somewhat explosive way.

The contribution of fibres in enhancing post-cracking bond strength is found to be higher for higher bar sizes in LWFC. For example at slip of 1 mm percent increase for 20 mm and 16 mm bar is 59% and 30% respectively, whereas for 10 mm bar this increase was only 14%.

The existing fib Model Code – 2010 does not deal the case of fibre-reinforced concrete and hence the improvements, especially in the post-cracking phase of bond stress-slip profile of concrete are not reflected. A modified bond stress-slip law for lightweight concrete reinforced with fibres is presented therefore, with some changes in existing fib Model Code – 2010.

7.2.2 Material properties

Effects of fibre addition, particle density and shape of aggregates were clearly observed during handling and measurement of fresh concrete properties. All these parameters favoured LWFC over NWFC. Due to lower density of lightweight aggregates: 1.19 compared to 2.5 of gravel, LWFC was on an average 21% lighter than that of NWFC as determined by density tests. Because of their higher specific gravity, fibres tend to raise the density of concrete mix. For fibre volume fraction of 0.75%, density of LWFC increased by 6.7% and density of NWFC increased by 2.2%. Possible reason for higher density values of LWFC could be the easiness in vertical travel of lightweight aggregates while vibration due to their regular shape. Thus more lightweight concrete could be filled in moulds compared to conventional concrete; this is also supported by the higher air content values of NWFC (5.58%) than LWFC (2.02%).

Fibre addition lowers slump values, thus seriously affecting the workability of fresh concrete mix. Nevertheless it can be improved through several means, for example, by using superplasticizers. Higher reduction in slump flow of LWFC compared to NWFC due to fibre addition was noted. It is because the design of flow table test is such that while jolting operation, heavier normal weight aggregates collapsed easily overcoming the resistance of fibres. Even at lower slump values handling of LWFC is better than NWFC of higher slump values. The degraded workability of fresh concrete mix due to fibre addition is compensated to some extent by regular, round shaped aggregates of lower density. All these parameters can play significant role in minimizing the efforts required at site for handling mixes with medium to high fibre volume fraction.

No clear variation is seen on compressive strength of NWFC with increasing fibre content. Although, for LWFC it seems that compressive strength is increasing but then at maximum fibre content level it starts decreasing, mainly due to the disturbance caused by fibres to concrete in attaining full compaction thereby raising the air void content. It is therefore concluded that steel fibres in general have no influence on concrete’s compressive strength.

Both the LWFC and NWFC attained tensile strength that is on an average 6.7% of their respective compressive strengths. It is concluded from the trend followed by the tensile strength test results that it is highly dependent on compressive strength. It is further determined that for the selected range of fibre dosage (up to 0.75% V_f) first cracking tensile strength of concrete remains unaffected. Although it has been claimed in a research [204], that it can be improved by using larger fibre volume fraction, but current laboratory experience and other literature review cited suggest restriction of fibre dosage up to 2% for practicality and economic reasons. Nevertheless, there is some marginal improvement in tensile strength of both the concretes and it is more pronounced in LWFC than NWFC, possibly due to higher brittleness of former. LWFC at maximum fibre content level attained splitting tensile strength equivalent to that of conventional concrete without fibres. Addition of fibres does influence the failure pattern of tensile test specimens. Specimens without fibres failed with single vertical major crack compared to those with fibres which developed multiple cracks at failure and the number of cracks increased with increasing fibre volume.

Flexural performance of the concrete was judged by three parameters: first peak strength of test beams, toughness and the residual capacity.

Like splitting tensile strength test, first peak strength also called modulus of rupture remained in general unaffected by fibre addition and is strongly dependent on the compressive strength of the concrete. It is therefore confirmed from both these tests that initiation of cracking in matrix is independent of presence of fibres.

Energy absorption capacity evaluated from beams point out that increase in toughness is higher for NWFC than LWFC for the similar percent increase in fibre content. Overall up to higher fibre volume fraction energy absorption capacity of the LWFC is 55 times higher than the control specimens, whereas for NWFC this increase was 73 times. Effectiveness of fibres in enhancing the toughness property can be judged from the fact that energy absorption capacity of lightweight concrete reinforced with lowest fibre amount (20 kg/m^3) was 32 times higher than conventional concrete without fibres. It is further established that fibre dosage of 40 kg/m^3 (V_f = 0.5%) is the limit point beyond which beams when loaded under flexure start showing signs of flexural hardening.

Like toughness, residual flexural capacity also improves noticeably as a result of fibre addition. Flexural capacities determined at 1 mm and 4 mm beam deflection indicate that conventional concrete reinforced with discrete fibres has advantage over LWFC.

Even though the compressive strength of lightweight concrete was higher at some fibre dosages, yet it underperformed in terms of residual flexural capacity primarily due to the coarse aggregates.

The slight decrease in elastic modulus of LWFC, about 5.8%, and for NWFC variation of it with compressive strength support the observation and hence the conclusion that fibres have no significant impact on this property of concrete. On an average elastic modulus of lightweight concrete with or without fibres is lower than NWFC by 14 GPa.

7.3 Recommendations

Current research was carried out to better understand the effect of fibres on the bond performance of lightweight concrete, apart from it, behaviour of conventional concrete with and without fibres was also considered since the existing descriptive and design expressions are based on the results of conventional concrete. Other parameters that were included in the work were bar sizes, fibre content. However, a full study and understanding of bond property is beyond the scope of single PhD research due to the diversity of parameters affecting it. Nevertheless based on the findings of current work following recommendations may be considered for future research.

Steel fibres having aspect ratio $\left(\frac{l_\mathrm{f}}{d_\mathrm{f}}\right)$ of 64 were considered in this experimental program. It is suggested that further investigation should be carried out to see the effect of different fibre aspect ratios. It would also be interesting to find out the effect of steel fibres in combination with other fibre types like synthetic fibres on bond performance.

In present study, although efforts were taken to use similar quantities of materials, yet it was not possible completely in case of fine aggregate which had higher amount in conventional concrete than LWFC. It is believed that if similar or higher quantity of fine aggregate is used in lightweight concrete its bond strength would increase, this aspect needs to be verified in presence of fibre reinforcement.

A numerical study using proposed bond stress-slip law for LWFC may be carried out for further verification and improvement. It may also be improved by carrying out experimental works using higher fibre volume fractions and by studying their effect on bond stress-slip behaviour.

Selective numbers of parameters were studied in this study; research work on bond performance of LWFC may be carried out with other test parameters as well, such as bond length, fibre geometry, bar location etc.

Since it is established that post cracking phase of bond stress-slip law is highly affected by presence of fibres, therefore this effect be incorporated and reflected by making some modifications as suggested in previous chapter.

A test procedure may be devised for determination of fracture energy of fibrous mixes. Fibre-reinforced concrete has higher fracture energy compared to conventional concrete. The knowledge of fracture energy is useful for numerical simulations and modelling of cracking. Although a RILEM [215] test methodology, using 3-point bending tests on notched beams exists for conventional concrete, there is no standard procedure documented to obtain fracture energy for fibrous mixes.

German Committee for Structural Concrete (Deutscher Ausschuss für Stahlbeton - DAfStb) [109] has proposed methodology to obtain design residual tensile strength values from the fibre performance class of SFRC. It is recommended that this approach may further be expanded and mechanism be devised to get the residual bond strength values and ultimately the design values for development length for bond.

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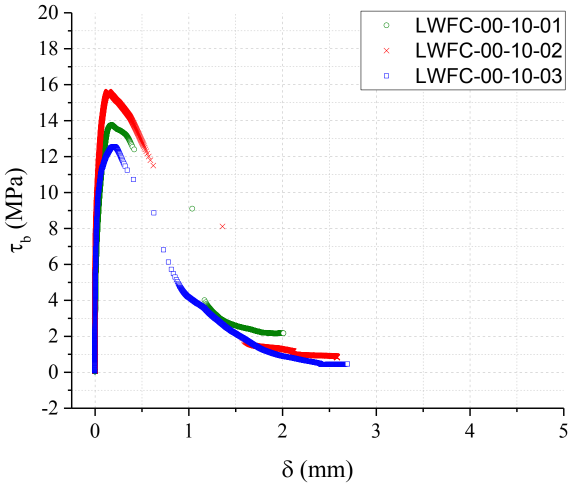
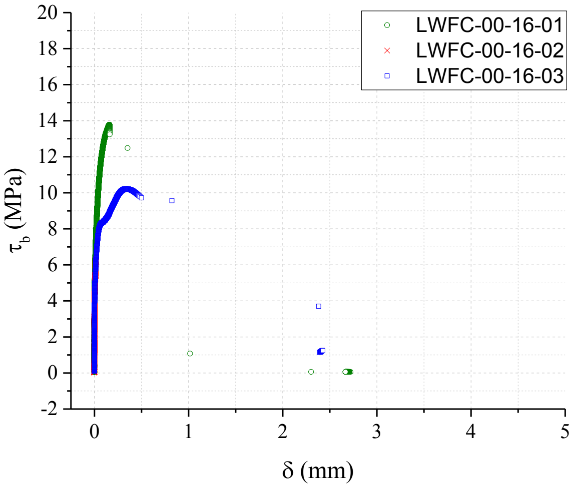
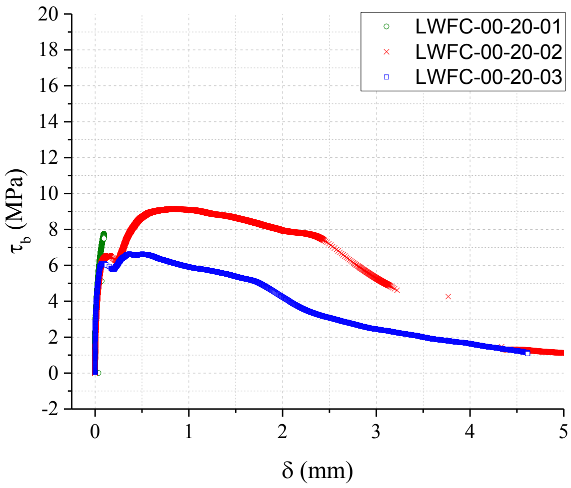
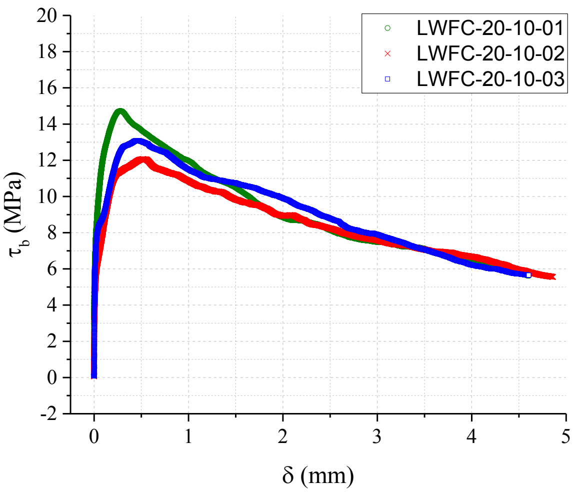
Annexes

Annex A: Bond stress – slip profiles of LWFC specimens

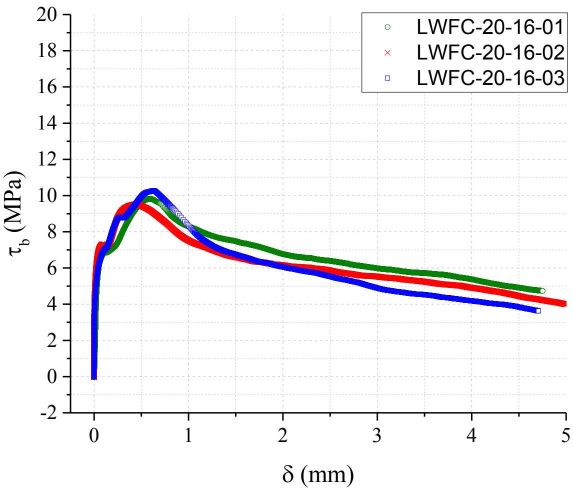




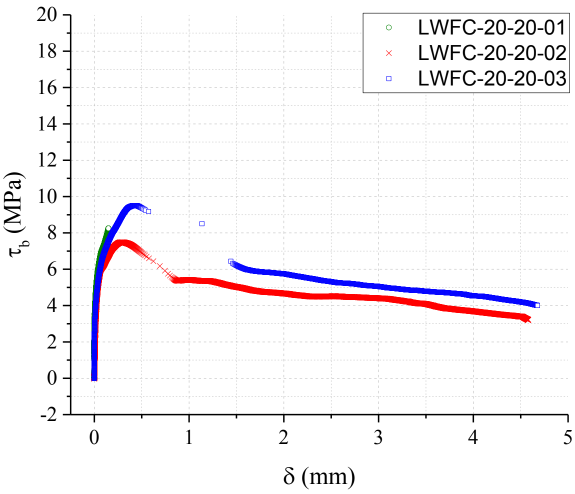




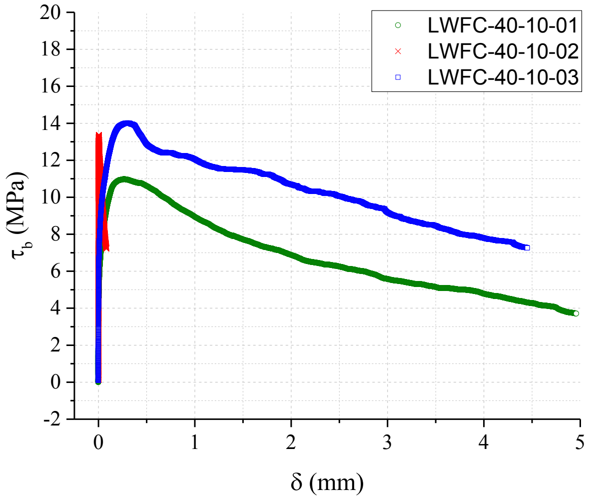
LWFC-20-10-01
LWFC-20-10-02
LWFC-20-10-03
τ_b (MPa)
δ (mm)

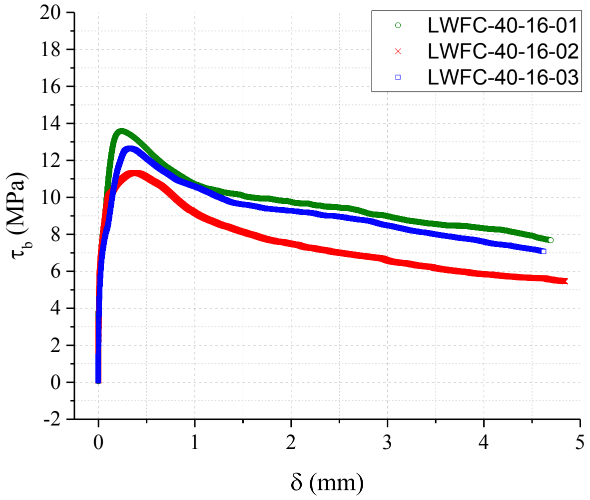


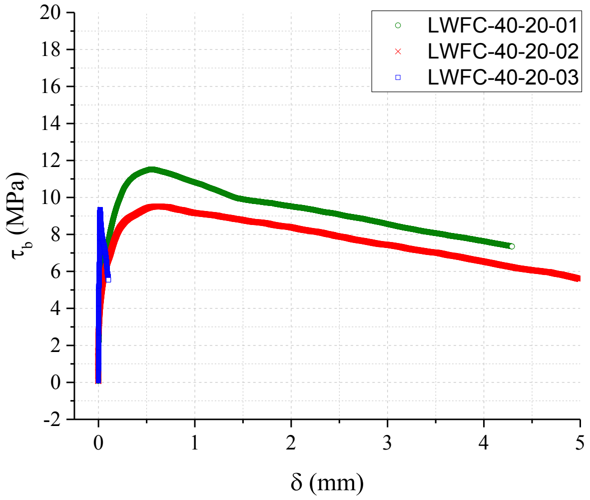
LWFC-20-16-01
LWFC-20-16-02
LWFC-20-16-03
τ_b (MPa)
δ (mm)

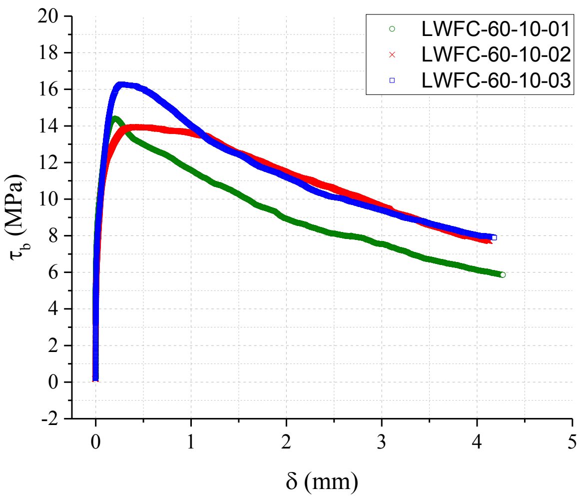


LWFC-20-20-01
LWFC-20-20-02
LWFC-20-20-03
τ_b (MPa)
δ (mm)


LWFC-40-10-01
LWFC-40-10-02
LWFC-40-10-03
τb (MPa)
δ (mm)


LWFC-40-16-01
LWFC-40-16-02
LWFC-40-16-03
τb (MPa)
δ (mm)


LWFC-40-20-01
LWFC-40-20-02
LWFC-40-20-03
τb (MPa)
δ (mm)



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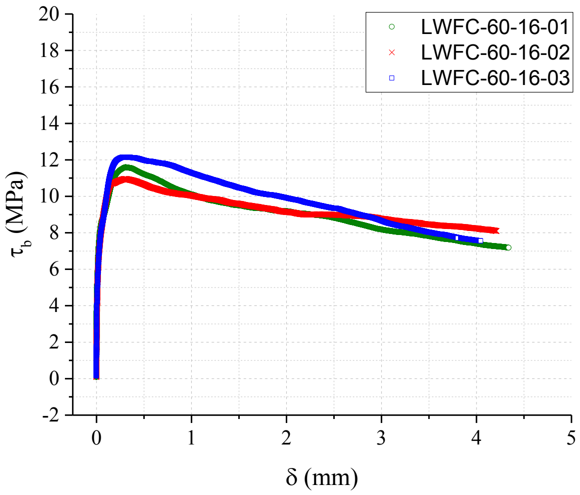
LWFC-60-10-02
LWFC-60-10-03
τ_b (MPa)
δ (mm)



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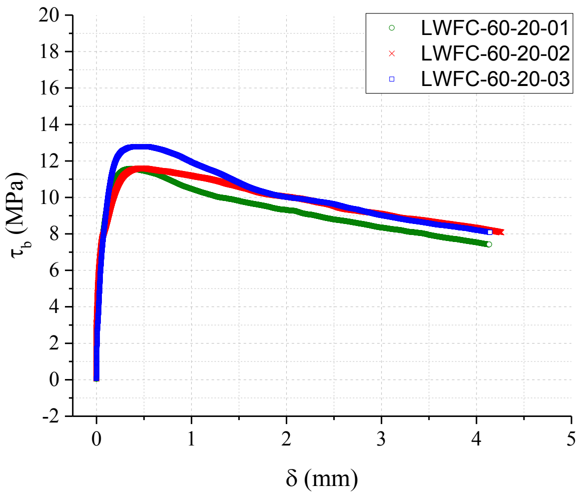
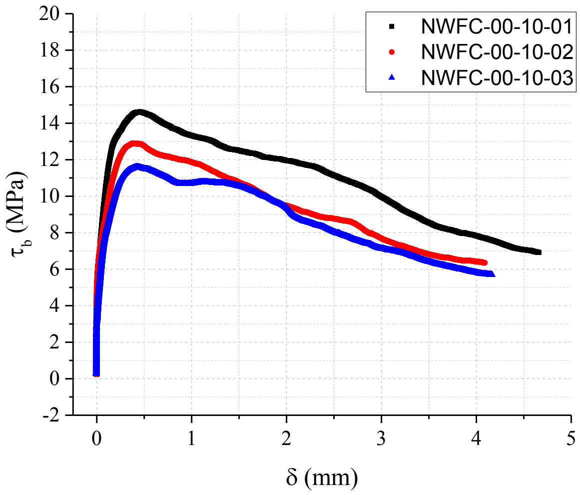
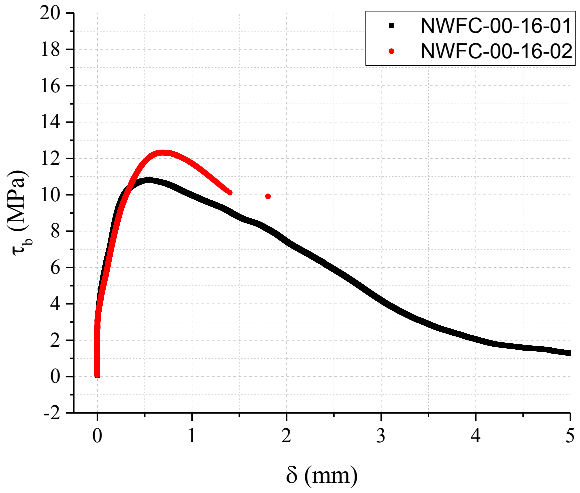
LWFC-60-16-02
LWFC-60-16-03
τ_b (MPa)
δ (mm)

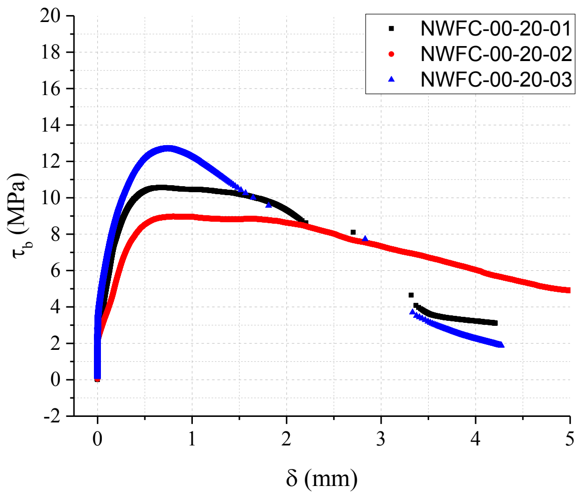


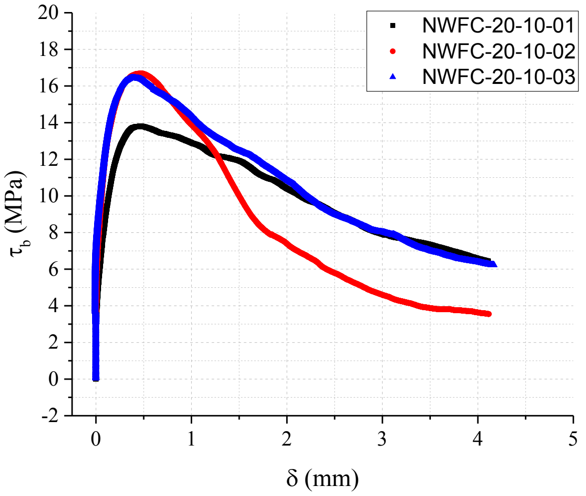
LWFC-60-20-01
LWFC-60-20-02
LWFC-60-20-03
τ_b (MPa)
δ (mm)

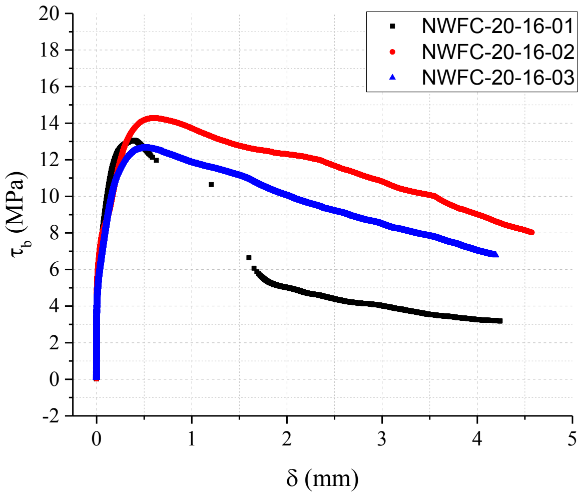
Annex B: Bond stress – slip profiles of NWFC specimens

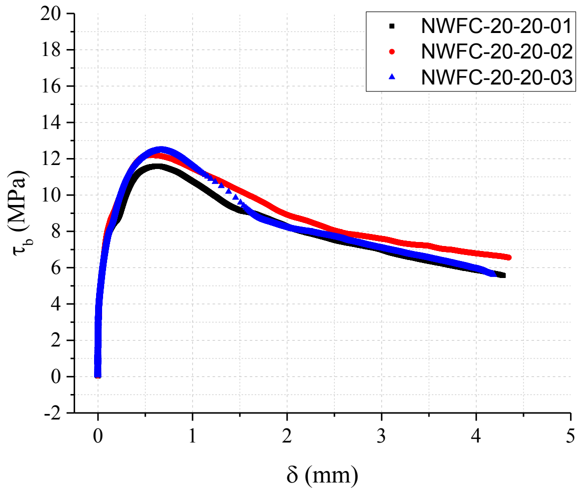


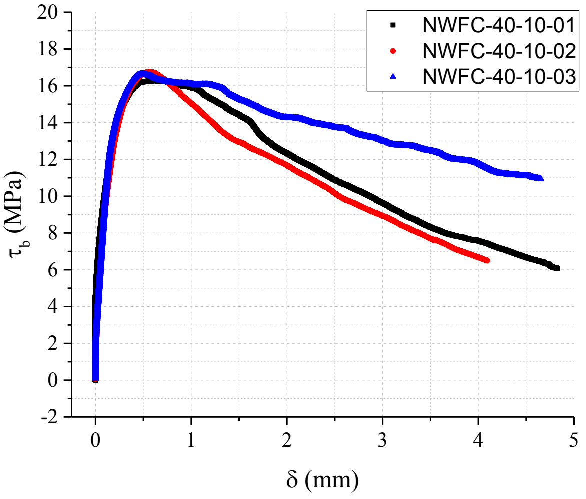


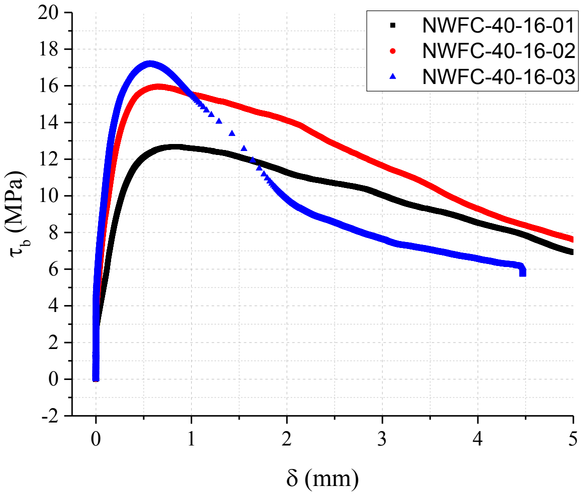


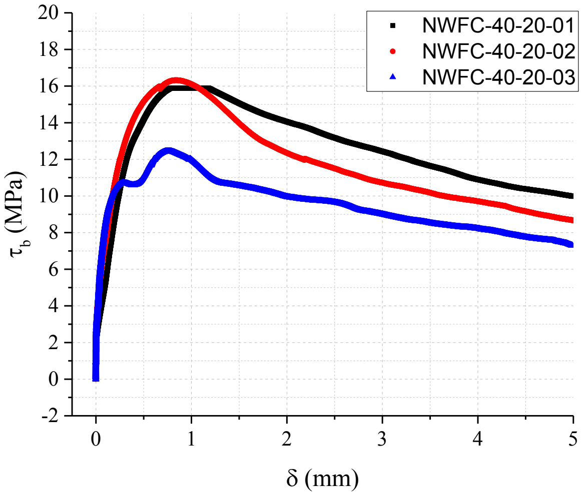

NWFC-20-10-01
NWFC-20-10-02
NWFC-20-10-03
τb (MPa)
δ (mm)

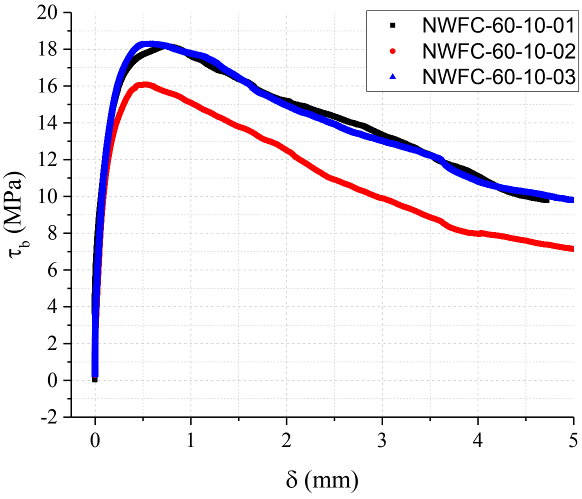

NWFC-20-16-01
NWFC-20-16-02
NWFC-20-16-03
τb (MPa)
δ (mm)

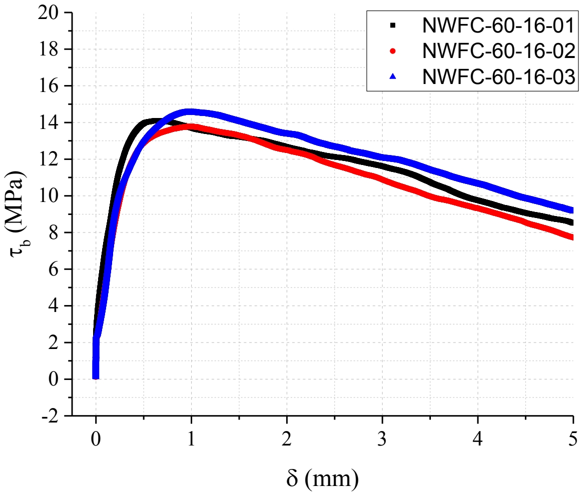

NWFC-20-20-01
NWFC-20-20-02
NWFC-20-20-03
τb (MPa)
δ (mm)


NWFC-40-10-01
NWFC-40-10-02
NWFC-40-10-03
τb (MPa)
δ (mm)


NWFC-40-16-01
NWFC-40-16-02
NWFC-40-16-03
τb (MPa)
δ (mm)


NWFC-40-20-01
NWFC-40-20-02
NWFC-40-20-03
τb (MPa)
δ (mm)


NWFC-60-10-01
NWFC-60-10-02
NWFC-60-10-03
τ_b (MPa)
δ (mm)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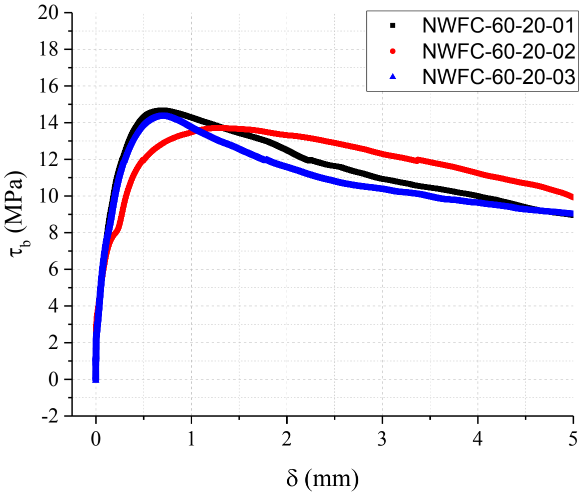
NWFC-60-16-02
NWFC-60-16-03
τ_b (MPa)
δ (mm)


NWFC-60-20-01
NWFC-60-20-02
NWFC-60-20-03
τ_b (MPa)
δ (mm)