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FLEXURAL TOUGHNESS AND DUCTILITY CHARACTERISTICS OF POLYVINYL-ALCOHOL FIBRE REINFORCED CONCRETE (PVA-FRC)

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Abstract: This paper presents the results of an experimental study investigating the effect of un-coated polyvinyl alcohol (PVA) fibres on the properties of hardened concrete. PVA fibre of varying lengths, 6 and 12 mm and aspect ratio (l/d) of 430 and 860, respectively, was utilised in different volume fractions of 0.125%, 0.25%, and 0.5%. In addition, 30% fly ash was also used as partial replacement of Portland cement in all fibre reinforced concrete (FRC) mixes. Uniaxial compression, splitting tensile, modulus of rupture (MOR) and modulus of elasticity (MOE) tests were performed following the Australian Standards to evaluate the mechanical properties of PVA-FRCs. Fracture test is also conducted in accordance with European Standard in order to evaluate the residual flexural tensile strength and limit of proportionality of PVA-FRCs. Furthermore, the structural properties of reinforced concrete (RC) beams incorporating PVA fibres are investigated for their load-deflection behaviour using 4-point loading. Flexural toughness of the test specimens and peak load deflection were measured and discussed indicating to what extent the un-coated PVA fibre can enhance the brittle-like behaviour of concrete. Results show that adding PVA fibres to the mix generally improves the mechanical properties of concrete. Regarding the strength, the optimum fibre content goes to 0.25% for both fibre lengths and in the case of toughness and ultimate deflection 0.5% shows the highest values. An increase of 30% in ductility is noted for the RC beam incorporating 0.5% by volume fraction of 12 mm PVA fibre.

1 INTRODUCTION

Concrete is a brittle material that has low tensile strength and low strain capacity. In fact, many deteriorations and failures in the concrete structures are due to the brittle nature of this material [1]. These disadvantages may be avoided by adding short discontinuous fibres to plain concrete which has been a major motivation for many research works in recent years[2]. Advancement of fibre reinforced concretes (FRCs) started in the 1970s. By that time, only glass fibre and steel fibre had been investigated [3].

Fibres are added into a brittle-matrix composite to help improve three major aspects; toughness, ductility and strength (tensile) [4]. Fibres tend to increase toughness of the composite material by bridging the cracks and provide energy absorption mechanism related to de-bonding and fibre pull-out. Furthermore, they can increase the ductility of the composite by allowing multiple cracking. They may also help increase the strength by transferring load and stresses across the cracks.

It has been investigated [5] that in a brittle composite which is subjected to uniaxial tensile loading, the first macroscopic crack forms when applied load reaches the first cracking load capacity (cracking strength) of the material. Thereafter, the load across the crack will be shared by the bridging fibres and transferred back to the matrix via the fibres interface. If sufficient load is transferred, the next crack may form and the process repeats until the matrix is broken with a series of sub-parallel cracks. This process results in a significant increase in tensile deformation (ductility) due to opening of each individual crack from the large numbers of such parallel cracks. During the multiple cracking process, the composite load may even increase and exceed the cracking strength of the composite [6]. When a macroscopic crack forms, fibres start to bridge the crack. While the crack is opening, the bridging stress increases and fibre/matrix interface de-bonds. Subsequently, the debonded part of the fibre stretches until it is ruptured or the fibre is pulled out. This is when the final failure happens.

Material resistance to breaking apart represents the energy absorbed which is called the flexural toughness, being the area under load-deflection curve for concrete prism in a 4-point bending test. This energy can show to what extent the fibre reinforced concrete can withstand loading conditions until broken. Accordingly, the additional energy which must be consumed to lead either fibre pull-out or fibre rupture enhances the toughness and post-peak behaviour (strain softening) of the matrix [1].

The optimum performance of the composite highly depends on the compatibility of the fibre and matrix in terms of strength, elasticity and surface adhesion leading to an adequate load transfer mechanism [7].

From among mentioned properties, elastic modulus of the fibre is investigated to be more important in the reinforcing effect of fibre in the matrix [8]. Fibres can only increase the strength of their composites when they have greater modulus of elasticity than that of the matrix [9]. Nevertheless, according to both theoretical and applied research, fibres with lower modulus can still improve concrete properties, in relation to strain capacity, toughness, impact resistance and crack control [9]. The improvement of the above mentioned properties is in many applications of much greater importance than a slight increase in strength.[9]

Synthetic fibres have become more attractive in recent years as reinforcements for cementitious materials. This is due to the fact that they can provide inexpensive reinforcement for concrete and if the fibres are further optimized, greater improvements can be gained without increasing the reinforcement costs [10, 11]. The properties of synthetic fibres vary widely, in particular with respect to the modulus of elasticity. However, most of the synthetic fibres have lower modulus of elasticity compared to the cementitious materials which ranges from about 15 to 30 GPa [9]. Consequently, developing fibres with a very high modulus of elasticity for cement reinforcement have been studied. The surface structure of synthetic fibres is of great importance in addition to their mechanical and elastic properties as it affects the performance of the composite, i.e. their highly hydrophobic and smooth surfaces usually tend to reduce the composite performance [12].

During the past 20 years, polyvinyl alcohol (PVA) fibre has been introduced in the production of FRCs [13-15]. PVA fibres act differently in a cement based matrix due to their surface formation and high strength [16]. The resulting composite, which exhibits a pseudo ductile behaviour, is called engineered cementitious composites (ECC). Li et al. [17] improved the performance of ECC by using PVA fibres with proper adjusted surface properties in a suitable matrix. This high performance ECC presented a multiple cracking and strain hardening behaviour. Hence, these composites are recognized as extraordinary high performance materials against durability related problems because of their ability in crack resisting and crack width minimising performance [18, 19]. Advantages of PVA fibres are; high aspect ratio; high ultimate tensile strength; good chemical compatibility with Portland cement; good affinity with water; faster drainage rate; and no health risk if used. PVA fibres have a good effect on the flexural strength of the matrix because of their adequate interfacial bond with the cement matrix. This good interfacial bond is accredited to the non-circular cross-section of the fibres, and hydrogen bonds between the fibres and the cement matrix. [9]

Beaudoin [8] investigated the flexural strength, toughness and splitting tensile strength, versus fibre content for different type of FRCs with various percentages ranging from 0 to 1% by volume fraction. Concrete proportions were water : cement : sand : gravel = 0.53 : 1.00 : 1.84 : 1.91. It has been reported that splitting tensile strength of PVA-FRCs, including 25 mm length PVA fibres, increases with increase in fibre content. Flexural strength was also having an ascending trend with fibre additions although a drop in strength was observed from 0.25% to 0.5% and optimum fibre content was given as 0.25%.

Furthermore, a dramatic slump loss with fibre additions was observed and a zero slump was recorded for 1% fibre content. It is also presented that slump loss was significantly higher in terms of PVA fibres compared to steel, carbon and glass fibres.

In the present work, the performance of using uncoated PVA fibre of two geometric lengths (6 and 12 mm) in concrete has been assessed to investigate to what extent fibres can improve the mechanical properties and flexural capacity of plain concrete.

2 EXPERIMENTAL PROGRAM

2.1 Materials

Shrinkage limited Portland cement (PC) and fly ash (FA) were used as the binder for all concrete mixes. Shrinkage limited Portland cement was used in this study to minimise concrete drying shrinkage. The fineness of FA by 45 μm sieve was determined to be 94% passing (tested in accordance with AS 3581. 1-1998).

A maximum nominal size of 20 mm aggregate was used in all mixes. All aggregates used in mix design were sourced from Dunmore, Australia, which includes 50/50 blended fine/coarse manufactured sand and 10 mm and 20 mm crushed latite gravel.

Table 1: Properties of PVA fibres

Specific gravity [g/cm^3]	Diameter [mm]	Thickness [dtex]	Cut length [mm]	Tensile strength [MPa]	Young's modulus [GPa]	Elongation [%]
1.29	0.014	1.8-2.3	6 and 12	1500	41.7	7

Table 2: Properties of steel reinforcement

Diameter [mm]	Type	Designation grade*	Uniform strain*	Ductility Class*	f_{sy} [MPa]	f_{su} [MPa]	Function
8	Plain	R250N	0.05	N	250**	-	Stirrup
10	Deformed	D250N	0.05	N	370	465	Longitudinal-top
12	Deformed	D500N	0.05	N	590	670	Longitudinal-bottom

* In accordance with AS 3600 (2009) - table 3.2.1

** Characteristic yield strength

Table 3: Mix proportions of reference concrete

kg/m^3						Lit/m^3	Water/C*
Cement	Fly ash	Sand	10 mm aggregate	20 mm aggregate	Water	HWR	
301	129	635	390	700	151	1.215	0.35

* Cementitious materials

The grading of all aggregates complies with the Australian Standard; AS 2758.1 specifications and limits. All aggregate was prepared to saturated surface dry condition prior to batching. Drinking grade tap water was used for all mixes after conditioning to room temperature (23 ± 2 °C). Furthermore, in order to improve the workability, a polycarboxylic-ether based high range water reducing admixture (HWR) was used. Non-coated polyvinyl alcohol fibre of 2 different geometries, 6 and 12 mm, with specifications mentioned in Table 1, were used in all FRC mixes. In addition, steel reinforcements with properties shown in Table 2 were utilised in order to reinforce the concrete beam elements.

2.2 Mixing and samples preparations

Mixes were prepared to obtain characteristic compressive strength at 28 days (f'_c) of 60 MPa to conform to AS 3600 requirements as structural concrete (ranging from 20 MPa to 100 MPa) even after adding fibres which may cause strength reduction, along with a slump of 80±20 mm. In order to obtain the desired slump, HWR dosage was varied. Details of the mix proportions for control concrete (non-fibre reinforced concrete, NFRC) are presented in Table 3.

Mix ingredients were all measured and added to the mix by weight. All FRCs also followed the same proportioning and only fibres were added to the mixture by 0.125%, 0.25% and 0.5% of volume fraction of the mix.

For NFRC mix, mixing was performed in accordance with AS 1012.2. However, for FRC mixes, due to the presence of the fibres, the standard mixing regime suggested in Australian Standard for conventional concrete was modified. Accordingly, after a comprehensive pre-study, a mixing sequence was achieved based on Australian Standards with modifications as explained below.

Fine aggregates were firstly dry mixed with PVA fibres in a vertical pan mixer. Coarse aggregates were then added and the mixture was further mixed for 3 minutes. Thereafter, cement, fly ash and water were introduced and mixed for a further 3 minutes. In order to adjust the slump, HWR was added within the first minute of adding cementitious materials. Following the 3 minute mixing, a rest period for 2 minutes was applied followed by an additional 3 minutes of mixing to achieve a completely homogeneous concrete. Slump was taken to check the workability and, thereafter, freshly mixed concrete was placed into moulds and compacted using an external vibrating table.

Curing of test specimens was carried out in accordance with AS 1012.8. Specimens were placed in a water tank after demoulding to be cured in lime-saturated water at a temperature of 20±2 °C until the testing date. In case of reinforced concrete (RC) beams, 7 days of wet curing was applied and after that specimens were air cured for a minimum of 56 days.

2.3 Testing methods

Uniaxial compression tests and splitting tensile tests were performed on cylindrical specimens of 100 × 200 mm at the age of 28 days in accordance with AS 1012.9 and AS 1012.10 specifications and method, respectively. Cylindrical specimens were tested under load rate control condition in an 1800 kN universal testing machine with a load rate equivalent to 20 ± 2 MPa per minute for compressive test and 1.5 ± 0.15 MPa per minute for indirect tensile test.

Flexural tensile strength or modulus of rupture (MOR) is obtained from four-point bending tests on 100 × 100 × 400 mm prisms at a loading rate of 1 ± 0.1 MPa/min until fracture following AS 1012.11. Four-point loading was applied and mid-span deflection of the flexural specimens was measured by means of a linear variable differential transformer (LVDT) at the centre of each specimen. The flexural stress was calculated as

$$f_{ct.f} = \frac{PL(1000)}{BD^2} \tag{1}$$

where $f_{ct.f}$ is the modulus of rupture or flexural stress (MPa), P is the maximum applied force (kN), L is the span length (mm), and B and D are the width and depth of the specimen (mm), respectively.

Static chord modulus of elasticity (MOE) test was also carried out on 150 × 300 mm cylinders following AS 1012.17. All mechanical properties tests were conducted at 28 days of ageing and each of the following presented results is the average of 3 test specimens

Flexural tensile strength test (fracture test) was conducted over notched prismatic specimens of 150 × 150 × 550 mm following the European Standard; BS EN 14651 specifications. After 25 days of curing, a middle notch was sawn using a rotating diamond blade wet saw. This age has been selected for notching the samples due to the criteria mentioned by the standard to let the specimens be cured for a minimum of 3 days after sawing. The specimens were rotated 90° around their longitudinal axis and then sawn through the width at mid-span as shown in Figure 1. The width of the notch is less than 5 mm and the depth is approximately 25 mm. The distance h_{sp}, as shown graphically in Figure 1 , is 125 ± 1 mm. Test specimens were then returned to curing tanks for further curing until 28 days and removed from water 3 hours before testing. In order to carry out the test, a 500 kN universal testing machine, capable of measuring loads to an accuracy of 0.1 kN, with a controlled load rate was utilised to produce a constant rate of crack mouth opening displacement (CMOD). A clip gauge (extensometer) with an accuracy of 0.01 mm was also used to measure CMOD. For CMOD values less than 0.1 mm, the load rate was adjusted so that CMOD could increase at a constant rate of 0.05 mm / min. When CMOD = 0.1 mm, the machine was adjusted to increase CMOD at a constant rate of 0.3 mm / min. The values of load and corresponding CMOD were recorded at a rate of 5 Hz during the test.

4-point static flexural test was also conducted on RC beams with dimensions of 150×200×1900 mm after a minimum of 56 days of curing. The RC beams used for this test were designed in accordance with AS 3600 (2009) with specifications as mentioned in Figure 2.

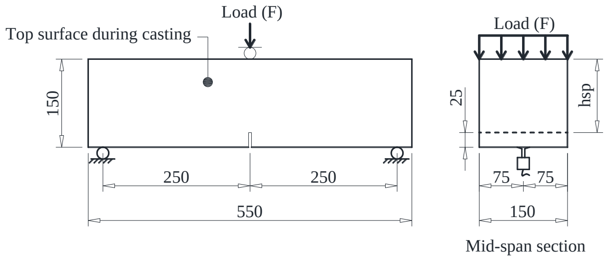

Figure 1: Typical arrangement of 3-point bending test on notched sample measuring CMOD (EN 14651)

LVDTs were also mounted along the beam axis, to measure the deflection of the beam in middle section and one-third spans. Several strain gauges are mounted at various locations of the beam to monitor the behaviour of the test specimen during the test. Strain gauges attached to steel bars (imbedded in concrete) can capture the stress-strain behaviour of rebars during the test, where, concrete strain gauges mounted on the beam face, can measure the strain changes on top (compression zone), bottom (tension zone) and middle (neutral axis) of the mid span section. The steel strain gauges were attached on bottom reinforcements at the centre and one-third span.

The test set-up is arranged in a way to make sure that the simply supported beam condition is satisfied. Thus, supports are designed in a way to allow beam to rotate freely and minimise the friction which may impose bending moment at the end of the beam (in the supports location). The support is also designed in a way that the vertical displacement at both supports is removed but horizontal displacement at one end can take place. The specimens are loaded at one-third span by the means of two hydraulic jacks.

3 RESULTS AND DISCUSSION

3.1 Mechanical properties

Table 4 presents the mechanical properties of FRCs and control concrete. Compressive strength, indirect tensile strength, MOR and MOE values were improved to different extents with the following mentioned details in response to the fibre volume fractions.

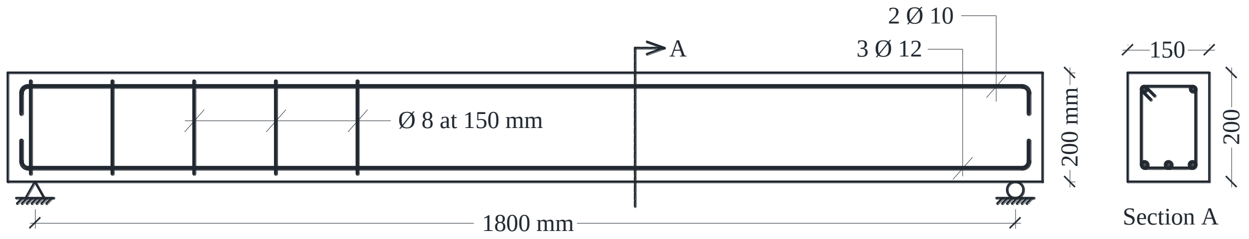

Figure 2: Schematic of RC beam

Table 4: Mechanical properties of different mixes at 28 days of ageing

Mix Reference	Fibre length [mm]	V_f [%]	Compressive strength[1]- $f_{c,28}$ [MPa ± SD]	Splitting tensile Strength[2]- $f_{ct.sp,28}$ [MPa ± SD]	Modulus of Rupture[2]- $f_{ct.f,28}$ [MPa ± SD]	Modulus of Elasticity[3]- $E_{c,28}$ [GPa]
NFRC	-	-	60.0 ± 3.2	3.7 ± 0.5	5.6 ± 0.2	39
FRC.1	6	0.125	65.0 ± 4.6	4.6 ± 0.2	6.8 ± 0.3	39
FRC.2	6	0.250	67.0 ± 3.2	4.9 ± 0.2	6.8 ± 0.2	40
FRC.3	6	0.500	61.5 ± 2.5	4.2 ± 0.3	6.3 ± 0.1	38
FRC.4	12	0.125	63.0 ± 1.8	4.3 ± 0.1	6.6 ± 0.4	38
FRC.5	12	0.250	64.5 ± 3.2	4.7 ± 0.2	6.7 ± 0.2	39
FRC.6	12	0.500	58.5 ± 2.8	4.1 ± 0.4	6.2 ± 0.5	36

[1] Compressive strength calculated to the nearest 0.5 MPa in accordance with AS 1012.9.
[2] Splitting tensile strength and modulus of rupture calculated to the nearest 0.1 MPa in accordance with AS 1012.10 and AS 1012.11, respectively.
[3] Static chord modulus of elasticity determined to the nearest 1 GPa in accordance with AS 1012.17.

By looking at the results, it can be noted that most of PVA-FRCs have higher compressive strength at 28 days ($f_{c,28}$) compared to the control except for FRC.6 which includes 0.5% volume fraction of 12 mm fibres. It can also be observed from Figure 3 that with a same fibre content shorter fibres act better than longer ones in terms of compressive strength and the optimum fibre volume fraction goes to 0.25% for both fibre lengths with approximately 11.5% improvement in compressive strength.

As indicated in Table 4, splitting tensile strength at 28 days ($f_{ct.sp,28}$) of plain concrete is significantly enhanced by introducing PVA fibres to the mix. The $f_{ct.sp,28}$ values of all FRCs are higher than that of control concrete, ranging from 11% for FRC.6 to 32.5% for FRC.2 including 0.25% volume fraction of 6 mm fibres.

Similar to splitting tensile strength, results of flexural strength at 28 days ($f_{ct.f,28}$) of FRCs versus control show considerable improvement ranging from 11% up to 21.5%.

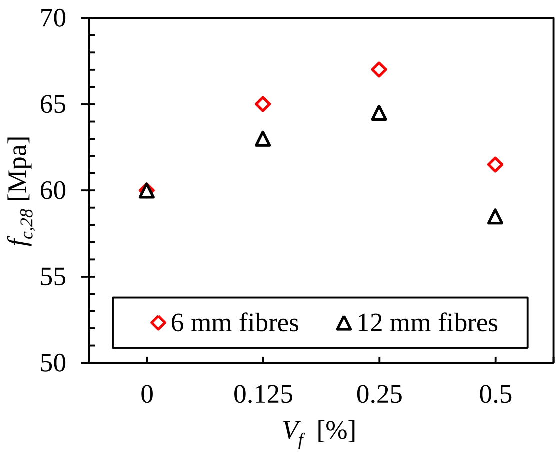

Figure 3: 28 days compressive strength of FRCs

Recurrently, in terms of flexural strength, optimum fibre content goes to 0.25% volume fraction. Up to this optimum value, more fibres provide more bridging effects and enhance the multiple cracking process which leads to strength improvement, however, higher fibre contents had reverse effects. This can be caused by the weak fibre distribution and improper orientation due to large number of fibres in the mix. Fibres which are not parallel to the cracks, can contribute to the stress bridging process by preventing a proper fibre orientation.

Furthermore, as it is inferred from the results, shorter fibres had a better effect on improving the mechanical properties of the concrete, particularly the flexural strength. Higher aspect ratio of the fibres can be responsible for this effect. It is assumed that these fibres may bend and not stay straight in the matrix, therefore, their total length cannot contribute in load bearing process and stress control mechanism.

As stated in the literature [20, 21] the flexural toughness and ductility can be determined from the flexural load-deflection curves. The flexural toughness is calculated as the area under the entire load-deflection curve and ductility is determined from deflection in post-peak region.

As shown in Figure 4, the sum of all finite area increments over the entire region under ABC curve, known as the work done leading to fracture, represents total flexural toughness.

The flexural toughness can be mathematically calculated from Equation 2.

$$T = \sum \left(\frac{P_i + P_j}{2} \right) \times \left(X_j - X_i \right) \qquad (2)$$

where P_i and P_j introduce two different applied loads for a finite increment and $(X_j - X_i)$ is the change of deflection within the finite increment.

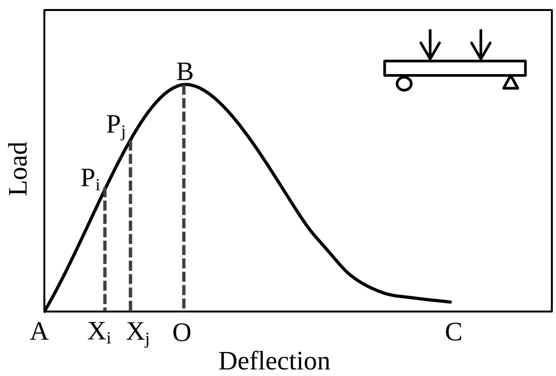

Figure 4: Schematic of a typical load-deflection curve in flexure [20]

As shown in Figure 4, the total ABC area can be divided into two regions. The deflection prior to the peak (AO) in ABO region is more related to rigidity, strength and micro-cracking process of the composite, where the post-peak deflection (OC) is a measure of ductility.

The region ABO which is limited to initial point of the curve (A) and the peak load (B) is the area in which elastic and some inelastic deformations occur due to strain hardening.

Figure 5 illustrates the Load-mid span deflection of test specimens in flexure. It can be observed that, the load-deflection curves for composites vary significantly depending on the fibre content.

As shown in Figure 5, fracture and total failure in the plain concrete occur suddenly just after exceeding the maximum load with a straight drop in the load-deflection curve. The same behaviour in fracture is also observed for all FRCs although a very small improvement can be noticed in their post peak region.

FRCs show significantly larger deflections at the ultimate state as well as higher loads compared to the control concrete. This will result in a bigger area under the load-deflection curves (ABC) which can be an indication of increasing toughness.

Consequently, it can be stated that the flexural toughness of concrete enhances by introducing PVA fibres to the mix and it will be further increased if more fibres are added.

Figure 6 shows normalized peak load deflection of FRCs. As illustrated, the peak-load deflection is increasing systematically as the amount of fibre increases. At 0.125% volume fraction, the peak load deflection for both fibre lengths was twice as much as the control. This deflection was improved by 300% and 340% where 6 mm and 12 mm fibres were added by 0.25% volume fraction, respectively. However, for fibre content more than 0.25% no significant improvement in peak load deflection were observed and FRCs normalized values are 3.3 and 3.5 for 6 and 12 mm fibres, respectively.

As a well-known fact, the slope of the load-deflection curve in flexure represents the flexural stiffness. As can be observed from Figure 5, the FRCs flexural stiffness is lower than the control concrete although they have higher load bearing capacities.

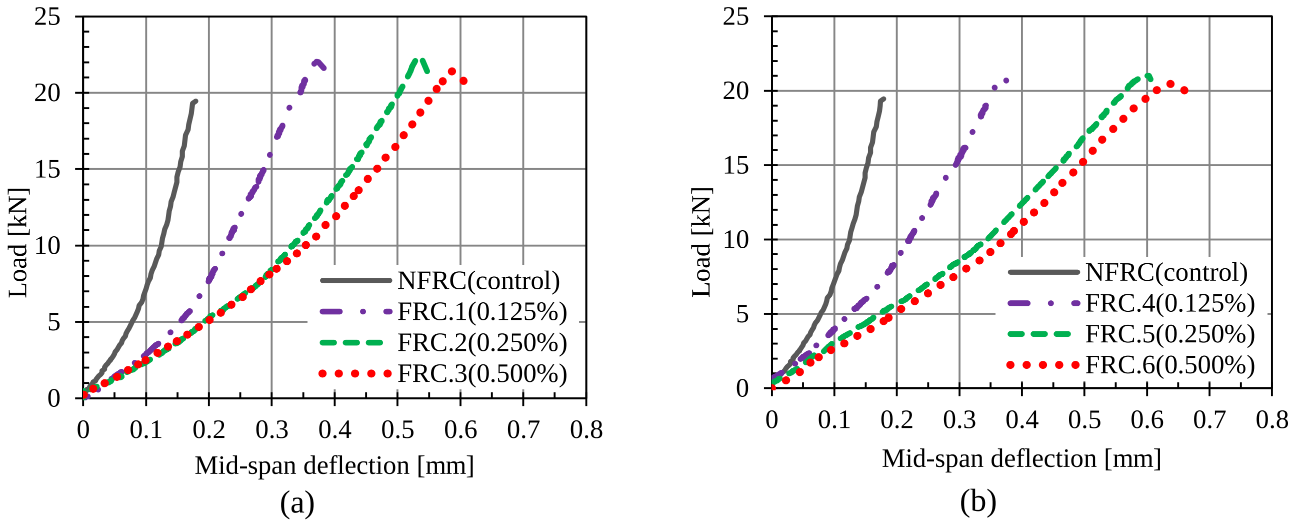

Figure 5: Load-mid span deflection curves in 4-point bending test (a) FRCs with 6 mm fibres versus control (b) FRCs with 12 mm fibres versus control

This behaviour which is a result of fibre bridging in concrete matrix although may bring up few concerns regarding the serviceability characteristics by imposing larger deflections within serviceability state, is a proper behaviour for concrete in seismic applications. This kind of behaviour provides large deflections before failure and dissipates more energy compared to the conventional concrete in critical situation such as earthquake.

An ideal concrete can be produced for seismic region structures, if the post peak behaviour of concrete can also be improved and a strain softening behaviour and high ductility be achieved. This may be attainable by using longer fibres which can be targeted as the future works.

Static Chord modulus of elasticity at 28 days ($E_{c,28}$) of FRCs and control are compared in Table 4.

From the results it can be noted that PVA fibres in low volume fractions used in this study do not significantly affect the modulus of elasticity.

However, it is noted that within the FRCs incorporating the same fibre, 0.25% volume fraction shows the highest $E_{c,28}$ and in a same fibre content longer fibres have lower $E_{c,28}$.

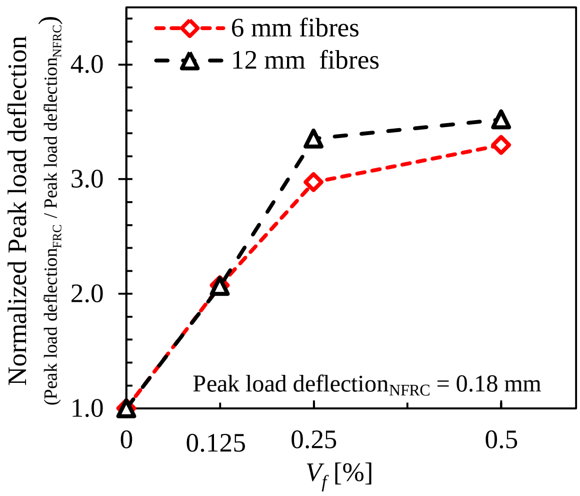

Figure 6: Normalized peak load deflection of FRCs in 4-point flexural test

3.2 Residual flexural tensile strength

From the results of the mechanical properties, it has been observed that although FRCs including 0.5% volume fraction of PVA fibres show lower mechanical properties compared to control, they provide a relatively higher deflection at peak load and flexural toughness. Therefore, this fibre volume fraction has been selected to be tested in order to evaluate its residual flexural tensile strength and limit of proportionality. Carrying out the 3-point bending test over prismatic notched samples, a load-crack mouth opening displacement (F-CMOD) relationship is established by mounting a clip-gauge on the opening of the notch. These results are illustrated in Figure 7.

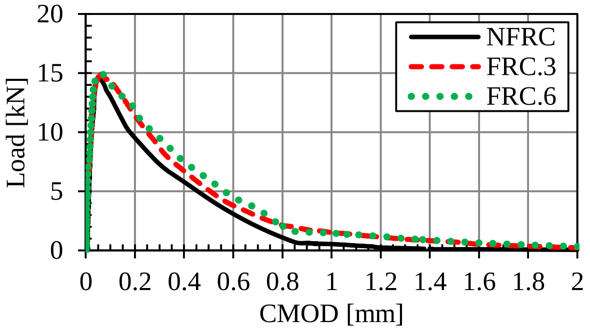

Figure 7: Load-CMOD curves of FRCs versus control

From this relationship, the load at limit of proportionality (F_L) can be evaluated. F_L is equal to the highest value of the load recorded up to a CMOD of 0.05 mm according to the European standard. Accordingly, concrete limit of proportionality (LOP) can be calculated using the expression given below;

$$f_{ct}^{f}L = \frac{3F_L l}{2bh_{sp}^2} \quad (3)$$

where $f_{ct}^{f}L$ is the limit of proportionality (LOP) in Newtons per square mm (MPa), F_L is the load corresponding to the LOP in Newtons, b (150 mm) and l (500 mm) are the width and span length of the specimen and h_{sp} (125 mm) is the distance between the tip of the notch and the top of the specimen. All these expressions were defined assuming a linear stress distribution on the cross section.

From the analysis of the F-CMOD relationship of NFRC (control), the value of 14.5 kN can be characterised for F_L and LOP value is calculated as 4.6 MPa using Equation 3. Following the same analysis for FRCs, F_L and LOP were calculated and the results are presented in Figure 5. F_L can be defined as the load corresponding to the first crack and limit of proportionality (LOP) is the stress at first crack [22].

From the load-CMOD curve, residual flexural tensile strength can also be calculated representing the load capacity at different CMODs. In this study, the procedure proposed in European standard; BS EN 14651, is used to determine the residual flexural tensile strength of FRCs and plain concretes with the expression given in Equation 4.

$$f_{R,j} = \frac{3F_j l}{2bh_{sp}^2} \quad (4)$$

where $f_{R,j}$ is the residual flexural tensile strength corresponding to CMOD = CMOD$_j$ (j = 1,2,3,4) in MPa which are CMOD$_1$ = 0.5 mm, CMOD$_2$ = 1.5 mm, CMOD$_3$ = 2.5 mm and CMOD$_4$ = 3.5 mm. F_j is the load corresponding to CMOD$_j$ and the other parameters are the same as per the description for Equation 3.

According to the above mentioned standard, the residual flexural tensile strength values are calculated by assuming a linear elastic distribution of stresses at the fracture point. The load-CMOD curves of all concrete mixes indicate that after the peak load, a load decay has occurred and the value of F_j corresponds to CMOD$_2$ = 1.5 mm and is approximately equal to zero, which gives zero residual flexural tensile strength ($f_{R,j}$) and only $f_{R,j}$ can be calculated for CMOD$_1$ = 0.5 mm. The results for $f_{R,1}$ are presented in Table 5.

Table 5: Residual strength and LOP at 28 days

Mix reference	F_L [kN]	LOP [MPa]	F_1 [kN]	$f_{R,1}$ [MPa]
NFRC	14.5	4.6	4.2	1.3
FRC.3	15.0	4.8	5.0	1.6
FRC.6	15.5	5.0	6.0	1.9

Comparing control with FRCs, it is noticeable that all concretes have almost the same F_L and LOP although FRCs showed marginally higher values. It is also noticeable that FRC.6 including 0.5% of 12 mm PVA fibres shows the highest value for LOP while its compressive and tensile strength is the least.

In the case of the residual flexural tensile strength corresponding to crack opening of 0.5 mm ($f_{R,1}$), a slight increasing trend is observed by PVA fibre addition to plain concrete. Generally, the synthetic fibres show a noticeable drop in strength after the first peak, even with high fibre content. This can be mostly due to the elastomeric nature and properties of synthetic fibres, which is known to have low strength characteristics. Once concrete begins to crack, synthetic fibres require a large deformation to occur before fibres are stretched enough and begin to carry the load [23]. However, in this study, no significant effect was observed by using PVA fibres. This may be either due to the short length and small diameter of monofilament fibres used in the matrix or the strong chemical bond present between the fibre and cementitious matrix. This very strong chemical bond created due to the hydrophilic surface of uncoated PVA fibre, leads to tendency of fibre to rupture and limits the multiple cracking effect which results in a lower tensile strain capacity and strain hardening profiles of the resulting composite [14]

3.3 Flexural strength and ductility

In order to study the effect of PVA fibre addition on structural properties of concrete, two RC beams with and without PVA fibre were fabricated and tested. For this purpose, 12 mm PVA fibre with volume fraction of 0.5% was selected having demonstrated the highest value for limit of proportionality and residual flexural strength.

From the comprehensive set of data collected within 4-point static flexural test, the load-deflection curves of RC beams are focused on in this paper.

The load-deflection curves, as shown in Figure 8, are calculated from the raw data captured by two load-cells which were mounted on loading jacks and an LVDT placed at mid-span to measure the deflection.

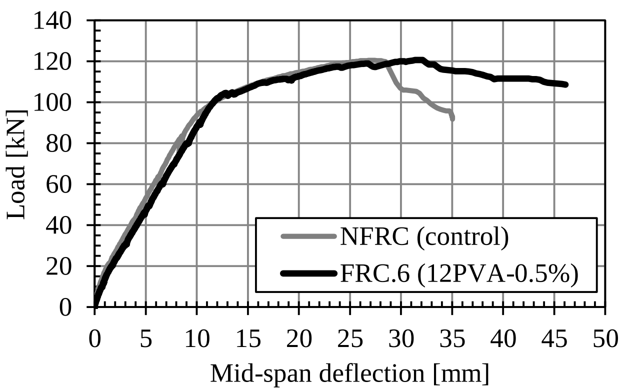

Figure 8: Load-deflection curves of PVA-FRC beam versus control

The term ductility in seismic design is used to explain the ability of a structure to withstand large amplitude cyclic deformations in the inelastic range without a considerable strength reduction [24]. Ductility factor, which is defined as the maximum deformation divided by the corresponding deformation when yielding occurs, permits the maximum deformation to be expressed in non-dimensional terms. This can be used as indices of inelastic deformation for seismic design and analysis. The displacement ductility factor (μ), which is usually determined in inelastic time history dynamic analysis, may vary between 1 for elastically responding structures to as high as 7 for ductile structures. However, it is typically ranging from 3 to 6 [24]. The displacement ductility factor can be calculated using below mentioned equation;

$$\mu = \frac{\delta_u}{\delta_y} \tag{5}$$

where δ_u is the ultimate (maximum) displacement and δ_y is the yield displacement.

In order to evaluate the yield and ultimate displacements, the definition proposed by Park [24] is utilised which may be considered as the most realistic description of the yield and ultimate displacement for reinforced concrete structures. Accordingly, δ_y is the yield displacement of the equivalent elasto-plastic system with reduced stiffness found as the secant stiffness at 75% of the ultimate load of the real system and δ_u is the post-peak displacement when the load bearing capacity has undergone a small reduction which has been taken as 85% of the ultimate load.

The summary of all above mentioned displacement analyses and ductility calculations for control and PVA-FRC beams are presented in Table 6. From the results, it can be observed that although the FRC beam incorporating 0.5% volume fraction of 12 mm fibre has lower concrete compressive strength, it provides approximately 30% higher value for ductility factor. This can be due to the fibre bridging effect which contributed as secondary or intrinsic reinforcement. In case of PVA-FRC beam, when concrete matrix was cracked, more energy was needed to enforce fibres to be fractured through the crack. This consequently resulted in higher load bearing capacity, especially beyond the peak load.

Table 6: Ductility factor of control and PVA-FRC beam

Beam lable	f_c [1] [MPa]	F_u [2] [kN]	δ_y [mm]	δ_u [mm]	μ
NFRC	80.0	120.5	12.7	32.0	2.5
FRC.6	75.0	120.6	13.9	46.1	3.3

[1] Compressive strength of concrete (cylindrical samples) at test day (> 56 days)
[2] Ultimate (maximum) load applied

4 CONCLUSIONS

The following conclusions can be drawn from the results and discussion presented in this paper;

The compressive strength of concrete is marginally increased with increasing fibre content and the optimum volume fraction goes to 0.25% for both geometric lengths, 6 and 12 mm, with approximately 10% improvement compared to the plain concrete.

The same trend for tensile and flexural strength is also observed with increasing fibre content. The average strength development of 20% in flexure and 30% in splitting tensile at 0.25% volume fraction shows to what extent fibres can improve the properties of plain concrete.

The flexural toughness of concrete is improved by introducing PVA fibres to the mix. This value is further increased by adding more fibres.

FRCs showed lower flexural stiffness together with providing a higher load bearing capacity compared to the control. This results in a higher ultimate deflection for FRCs where the peak load values remained constant.

Furthermore, from the flexural tensile test results, it has been observed that prior to peak response of all concrete series was almost similar, however, a subtle improvement has occurred in post-peak flexural response of FRCs compared to control. It is quite possible that PVA fibres with 6 mm and 12 mm length have very low effect on the post-peak flexural capacity of plain concrete.

In terms of structural properties, it has been observed that, PVA fibre addition to the mix has significantly improved the ductile behaviour of RC beam. 30% higher value of ductility factor is calculated for the beam incorporating 0.5% volume fraction of 12 mm PVA fibres.

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