Experimental Study on Structural Behavior of UHPFRC Members under Different Strain Rates

By

Nurul Mirrikh Bt Suhaimi
14509

Dissertation submitted in partial fulfillment of
the requirements for the
Bachelor of Engineering (Hons)
(Civil)

SEPTEMBER 2014

Universiti Teknologi PETRONAS,
Bandar Seri Iskandar,
31750 Tronoh,
Perak Darul Ridzuan

CERTIFICATION OF APPROVAL

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Approved by,

……………………………

(Dr. Teo Wee)

CERTIFICATION OF ORIGINALITY

This is to verify that I am responsible for the work submitted in this project, that the original work is my own except as specified in the references and acknowledgements, and that the original work contained herein have not been undertaken or done by unspecified sources or persons.

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(NURUL MIRRIKH BT SUHAIMI)

ABSTRACT

Reinforced concrete structure has suffered severe degradation due to the combined effects of aggressive environments and significantly increased live load. Furthermore, many concrete civil infrastructures such as buildings and bridges are not designed to resist highly impulsive load such as blast or impact. As a result, the structures will progressively fail when subjected to high strain rates loading such as impact or explosion. In the uncertain times facing us these days such infrastructures need to be less susceptible by using a material with high energy absorption capacity, such as Ultra-high performance reinforced concrete (UHPFRC). UHPFRC is a new class of cement composite and is characterized by highly dense microstructure and very high compressive strength more than 150 MPa. Therefore, the aim of this research is to investigate the strain rate effects on flexural strength of UHPFRC and to develop a better understanding about the behavior of UHPFRC under different loading rates. The response of UHPFRC was determined from 3-point bending test which applied three different loading rates which are 0.1 kN/s, 0.5 kN/s, and 1.0 kN/s. Besides, the behavior of UHPFRC under different loading rates is also discussed as the volume ratio of steel fibers, V_f is varies from 0% to 3%. From this study, higher fiber volume ratio results in higher flexural strength and the flexural strength of UHPFRC is increasing as higher loading rates are applied. Based on the excellence performance of UHPFRC, its behavior appears to make it an ideal material for resisting high strain rates effects.

ACKNOWLEDGEMENT

I have taken efforts in this project. However, it would not have been possible without the kind support and help of many individuals and organizations. I would like to extend my sincere thanks to all of them. I am highly indebted to Dr. Teo Wee for his guidance and constant supervision as well as for providing necessary information regarding the project and also for his support in completing the project. I would like to express my gratitude towards my family for their kind co-operation and encouragement which help me in completion of this project. I would like to express my special gratitude and thanks to Mr. Yin Hor for giving me such attention, time and guidance in completing the project. Last but not least, my thanks and appreciations also go to my colleague in developing the project and people who have willingly helped me out with their abilities.

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CHAPTER 1

INTRODUCTION

1.1 Background

The structural behaviour under high strain rate loading e.g., earthquake, impact and blast from increasing man-made accidents and natural disaster, is a major concern in civil engineering. In the last few decades, there are lot of research has been carried out to investigate concrete behaviour under high strain rate loading using both experimental and numerical studies. Due to the brittle nature of concrete materials, several methods have been developed to further increase the concrete resistance to high strain rate loading, such as adding fibre reinforcement to the concrete or use of high strength concrete materials.

Ultra high performance fibre reinforced concrete (UHPFRC) is one of promising construction materials for enhancing the resistance of civil infrastructure at high rate loading. This is because of high tensile resistance and energy absorption capacity based on their unique strain hardening and multiple micros cracking behaviour. UHPFRC is a concrete material with both high strength concrete and fibre reinforcement. UHPFRC belong to the group of High Performance Fibre Reinforced Cement Composites (HPFRCC), where HPFRCC defined as the kind of Fibre Reinforced Concretes (FRC) that exhibit strain-hardening under uniaxial tension force. In addition, UHPFRC is characterized by a dense matrix and consequently a very low permeability when compared to HPFRCC and normal strength concretes. (Yen Lei Voo, Behzad Nematollahi, Abu Bakar Bin Mohamed Said, Balamurugan A Gopal, & Yee, 2012)

UHPFRC is also characterized by a combination of high strength, ductility and toughness, thus becomes a promising material for innovative structures exposed to intensive dynamic loading such as earthquakes, industrial accidents or projectile impact. (Tran & Kim, 2013)

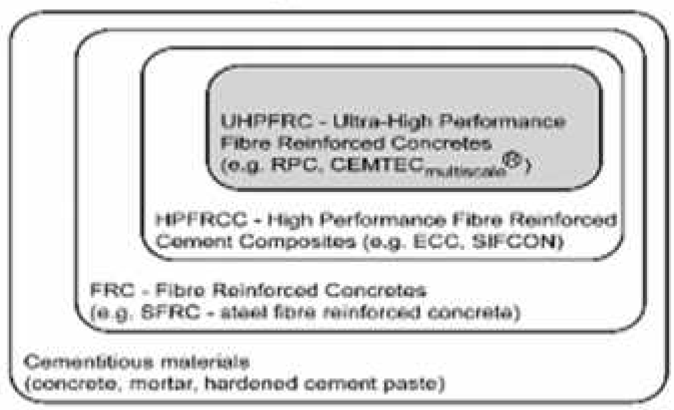


FIGURE 1. Classification of fibre reinforced concrete. (Yen Lei Voo et al., 2012)

UHPFRC offers increased strength, ductility, durability, and has much higher resistance to high strain rate loading compared to normal concrete materials. Some full scale tests have been carried out in this field and results demonstrated that under the same blast loading, UHPFRC performed much better than normal strength concrete (Mao, Barnett, Begg, Schleyer, & Wight, 2014). These improved properties are expected to improved performance and resilience in the whole structural level, even when the structures is subjected to highly impulsive loads such as blast or impact.

1.2 Problem Statement

The brittle nature of concrete material and an abrupt failure of one or more of the load bearing structural members, such as columns have caused progressive collapse of a building. Therefore, the endurance of such members under severe short duration loads is essential for the survivability of the building. (Astarlioglu, Krauthammer, Morency, & Tran, 2013). The use of innovative materials such as ultra-high performance fibre concrete (UHPFC) for the construction of new structural members is one of the cost-effective structural engineering solutions to mitigate the effects of high strain rate loads on building. (Wu, Oehlers, Rebentrost, Leach, & Whittaker, 2009)

However, the expectation for their high resistance under high rate loading is based on their static behaviour. It is well known that the dynamic behaviour of materials can be very much different from those exhibited in quasi-static conditions. Thus, the dynamic properties of UHPFRC in wide range of strain rates should be clearly understood for their application to various structures subjected to different dynamic loading conditions. In addition, it is still questionable whether the unique strain hardening behaviour of UHPFRC can be maintained even at higher strain rates.

Based on previous research, the direct tensile response of UHPFRC is strongly expected to be rate dependent since the steel fibre, cement based matrix, and their interfacial bond properties are likely dependent upon the rate of loading (Tran & Kim, 2013). Thus, it is important to develop a better understanding about the high strain rate effects on UHPFRC. Once the behaviour of UHPFRC at high strain rates are understood, the mechanical performance of UHPFRC at high strain rate can be manipulated by changing the microstructure of the cement based matrix and the geometry of fibres. The improvement of understating regarding this subject would increase the application of UHPFRRC into structures subjected to high rate extreme loadings including earthquakes, impact and blast to successfully resist such loading conditions.

1.3 Objective and Scope of Study

The purpose of this study is to evaluate the mechanical properties of Ultra-High Performance Reinforced Concrete (UHPFRC). The expected compressive strength at the age of 28 days is being more than 150 MPa. Next, to evaluate the mechanical properties of UHPFRC with respect to different steel fiber content which varies from 0% to 3% of fiber volume ratio.

Lastly, to investigate and improve the understanding toward a fundamental knowledge of the strain rate effects on the structural behaviour of UHPFRC members using high strength deformed steel fibres. According to previous research, there are limited research has been devoted to stimulate performance of UHPFRC subjected to high strain rates loading. This study will present the experimental investigation on the performance of UHPFRC under different rates of loading which is 0.1 kN/s, 0.5 kN/s and 1.0 kN/s.

CHAPTER 2

LITERATURE REVIEW AND THEORY

2.1 Material Behaviours at High Strain Rate

There is different strain rates associated with different types of loading. Blast loads typically produces very high strain rates in the range of 10^2 – 10^4 s^{-1}. This high straining loading rate would alter the dynamic mechanical properties of target structures and, accordingly, the expected damage mechanisms for various structural elements (Ngo, Mendis, Gupta, & Ramsay, 2007).

Although much research has been performed on the direct tensile behaviour for cement-based material, the investigated strain rates were mostly lower than seismic strain rates (Tran & Kim, 2014). FIGURE 2 shows a schematic diagram of the range of strain rates associated with different types of loading.it can be seen that ordinary static strain rate is located in the range 10^{-6} – 10-^{4} s^{-1}, while blast pressures normally yield loads associated with strain rates in the range 10^2 – 10^3 s^{-1}.

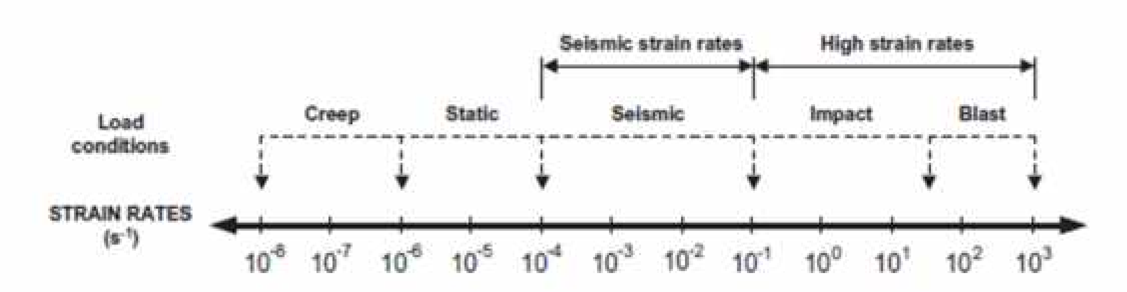


FIGURE 2. Strain rate regimes. (Tran & Kim, 2014)

2.1.1 Dynamic Properties of Concrete under High-Strain Rates

For the concrete under dynamic loading condition, the mechanical properties can be quite different from that under static loading. FIGURE 3 shows that while the dynamic stiffness does not vary a great deal from the static stiffness, the stresses that are sustained for a certain period of time under dynamic conditions may gain values that are remarkably higher that the static compressive strength than the static compressive strength.(Ngo et al., 2007)

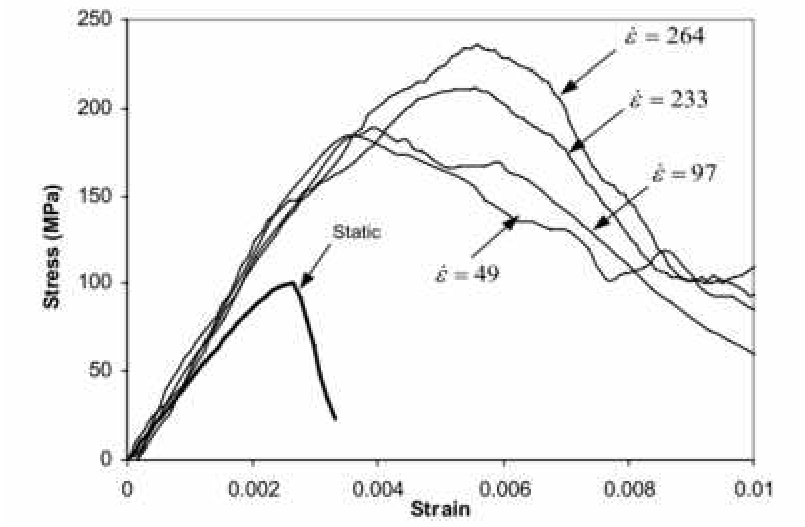


FIGURE 3. Stress-strain curves of concrete at different strain-rates (Ngo et al., 2007)

For the increase in peak compressive stress (f'$_c$), a dynamic increase factor (DIF) is introduced in the CEB-FIP (1990) model, shown in FIGURE 4 for strain-rate enhancement of concrete as follows:

$$\boldsymbol{D} \quad = \left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}}\right)^{\mathbf{1.0}} \qquad \boldsymbol{f} \quad \dot{\varepsilon} \leqslant \mathbf{3} \quad ^{\mathbf{-1}} \quad (1)$$

$$\boldsymbol{DIF} = \left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}}\right)^{\mathbf{1/3}} \qquad \boldsymbol{f} \quad \dot{\varepsilon} > \mathbf{3} \quad ^{\mathbf{-1}} \quad (2)$$

Where:

log γ = 6.156 α-2
$\dot{\varepsilon}$ = strain rate
$\dot{\varepsilon}_s$ = 30 x $10^{-6}s^{-1}$ (quasi-static strain rate)
α = $1/(5+f'_c/f_{co})$
f_{co} = 10 MPa = 1450 psi

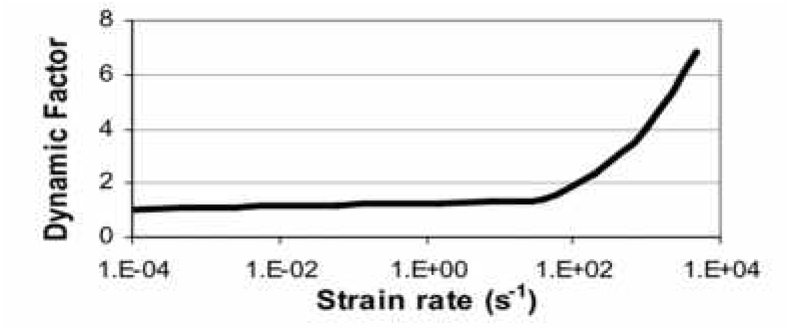


FIGURE 4. Dynamic Increase Factor for peak stress of concrete
(Ngo et al., 2007)

2.1.1 Failure modes under High- Strain Rates

Failures of a reinforced concrete member, such as columns, are classified into three major types: shear, bond splitting and flexural modes. Although it is common practice to rely on the observed crack patterns and the yielding of the steel reinforcement to determine the resulting failure mode, problems arose for specimens exhibiting features of more than one type of failure mode. Hence, there is difficulty in the evaluation of the ultimate strength of the columns, especially whether it is for shear or for bond splitting.

An alternative approach in the classification of failure modes for reinforced concrete column is presented. Aside from failure mode classification by the use of crack patterns, bar strains and ultimate strengths, an analysis of the strain distribution in the main reinforcement is considered.

Under impact or blast loading two different types of failure can be distinguished:

- Local failure;
- Global failure.

The first one is due to an explosion which occurs close to structural element; the characteristics of failure depend on:

- The dynamic local properties of the element;
- The ductility local properties of the element;
- The high-strain-rate behaviour of materials.

The second one occurs after a local failure and it is related to the attitude of the structure to withstand the loss of elements without activate progressive collapse. It depends on the global ductility properties of the structure and on the quality and the frequency of connections between elements of the structure. Obviously if local failure is more severe, then global failure becomes more probable.

Local failure

Local failure can be reached by 2 ways:

- Local failure of the material;
- Local failure of the structural element.

The first mode of failure occurs when the shock wave in air produced by the explosion impacts on the surface of the element and determines a field of compression and tensile wave propagating inside the material. This tension can cause the cracking of the concrete and the consequent projection of debris.

The phenomenon can be well modeled using Hopkinson theory in one-dimensional case. The use of more complicated models would be unnecessary, because of the high uncertainty about the pressure function acting on the structure. To this aim the results of the dynamic characterization of materials will be used to define Dynamic Incremental Factor (DIF)–strain rate curves, for the failure tension of concrete and for the yielding tension of reinforcement. These curves will be used to update the stress-strain relationship at different strain-rate levels.

The second failure mode can occur when the action of blast determines the failure of one or more sections within the element. Based on these issues it is planned to study this failure mode performing a dynamic non-linear analysis of the element. To consider strain-rate effect, non-linear moment-curvature relationships will be defined for each homogeneous portion of the element at different strain-rate. These different relationships will be the results of the material constitutive law updating due to strain-rate effect.

The occurrence of a local failure will be considered for 3 different structural elements: the deck, the piers and the arch. For each of them material failure analysis and element failure analysis will be performed.

Global failure

Global failure occurs after a severe damage of one or more structural elements; the loss of these elements in fact can determine a progressive collapse of the structure. The possibility of this failure mechanism is linked to the capacity of the structure to redistribute loads on other structural elements. It depends on:

- Redundancy of elements;
- Ductility of connections.

The first characteristic means that the static scheme of the structure is far from an isostatic configuration. The loss of some elements is then compensated by the presence of other elements which anyway can carry acting loads.

The second characteristic is also important because after the loss of some elements, the new equilibrium configurations are reached with high local deformations, which must be tolerated by the structure.

High ductility of connections is then necessary to allow these static configurations. For these reasons it is planned to perform a global failure analysis of the bridge, considering a nonlinear FEM model in which one or more elements will be removed. Then a non-linear dynamic analysis will be carried out to evaluate the new equilibrium configuration and to verify that this is compatible with deformation and resistance capacity of the structure. The choice of the elements to remove depends on the criticality of the elements role.

Surely, in the case of the bridge, the piers and the arch are fundamental to guarantee the equilibrium of the whole structure. Anyway the arch, with its massive section, has a very low probability of failure under a blast event, even if it were particularly severe. For this reason the progressive collapse analysis will be focused on the loss of the piers. Obviously this consideration will influence also the local failure analysis which will be more detailed for the piers too.

2.2 Ultra-high Performance Fibre Reinforced Concrete (UHPFRC)

According to Mao, Barnett et al. 2014, ultra high performance fibre reinforced concrete (UHPFRC) is a concrete material with both high strength concrete and fibre reinforcement. UHPFRC is a cementitious composite material, generally consisting of cement, quartz sand, silica fume and fibres. It has eminent properties: relatively high compressive strength (≥180 MPa) and tensile strength (≥10 MPa), strain-hardening behaviour under tensile stress (given a certain volume of fibres) and very low permeability because of an optimised dense matrix. (Makita & Brühwiler, 2014)
In Malaysia, UHPFRC was firstly introduced by Dura Technology Sdn. Bhd. in year 2007 with compressive strength and flexural strength of over 160MPa and 30MPa, respectively; however, it has only started its industrial-commercial penetration into the market as a new sustainable construction material since last 3 years. UHPFRC is suitable for use in;

(i) the fabrication of precast elements for civil and structural engineering,
(ii) archi-structural features,
(iii) durable components exposed to marine or aggressive environments,
(iv) blast or impact protective structures,
(v) strengthening material for repair/rehabilitation work for deteriorated reinforced concrete structures,
(vi) portal frame building construction, and others

Due to high compressive strength and tensile strength, UHPFRC has much higher resistance to high strain rate loading than normal concrete. Some full scale tests have been carried out in this field and results demonstrated that under the same blast loading, UHPFRC performed much better than normal strength concrete. (Mao, Barnett et al. 2014)

2.3 Typical mix of UHPFRC

Ordinary Portland cement, silica fume, fine aggregates, water, steel fibers and high-range water reducing agent are the main ingredient to produce UHPFRC. Table 1 demonstrates a standard UHPFRC mix design with 2% by volume of micro steel fibers. The high-range water reducing agent used is Polycarboxylate ether (PCE)-based superplasticizer and no recycled wash water shall be used in the mixture (Voo and Foster, 2010).

Table 1: Typical mix design of UHPFRC

Ingredient	Mass (kg/m^3)
UHPC Premix	2100 – 2200
Superplasticizer	30 – 40
Steel Fiber	157
Free Water	144
3% Moisture	30
Targeted W/B Ratio	0.15
Total Air Void	< 4%

2.4 Properties of Ultra High Performance Fibre Reinforced Concrete (UHPFRC)

2.4.1 Compressive strength

Typical Compressive strength for normal strength concrete is around 20 - 40 MPa. Contrasted with the conventional concrete, UHPFRC has higher compressive strength. As per the past studies in year 2012 by Hassan et al, the compressive quality of UHPFRC can achieved more than 150 Mpa. The most well-known test on hardened concrete is the compressive strength test (Hassan, Jones, & Mahmud, 2012).

Numerous organized standards have given definite rule to the estimation of the compressive and modulus of elasticity for normal concrete utilizing cylinder and cube specimens. This study has taken after the test design depicted in the BS EN 12390-3-2009 for the determination of compressive stress-strain values for UHPFRC. The compressive strength is ascertained using the equation below:

$$C \qquad S \qquad (F_C) = \frac{L \quad (P)}{C \quad s\epsilon \quad a \quad (A)} \qquad (3)$$

2.4.2 Flexural strength

UHPFRC is a relatively new construction material with higher strength, deformation capacity and toughness than conventional normal strength, normal weight concrete. The sample stress-strain curve for UHPFRC is shown in Figure 4. The behaviour of UHPFRC under flexure loading can be characterized by three phases that is (i) the linear elastic behaviour up to the first cracking strength of the material, (ii) a displacement-hardening phase up to the maximum load, and (iii) a deflection-softening phase after the maximum load is reached.

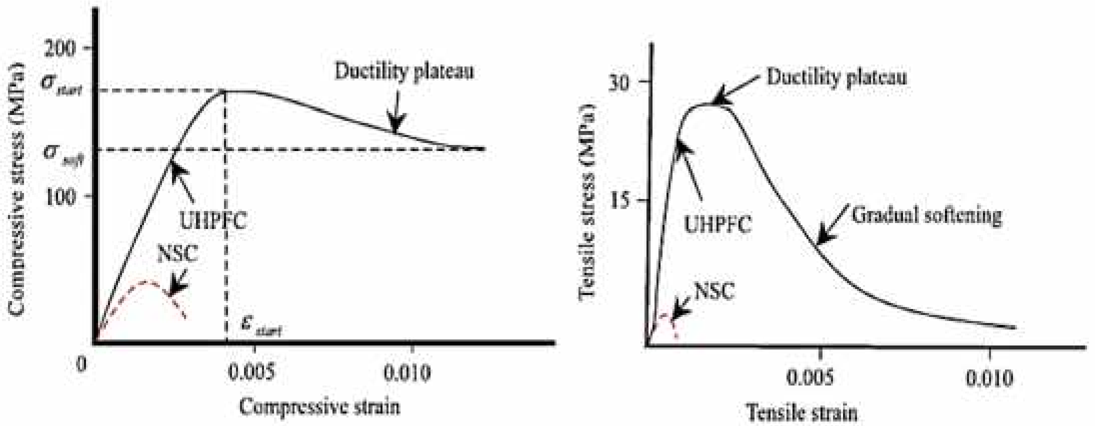


FIGURE 5. Mechanical properties of conventional concrete and UHPFRC (Wu et al., 2009)

BS EN 12390-5-2009 testing hardened concrete part 5 is used to test Flexural strength of specimens. The flexural strength can be calculated using the following equation (3 point bend testing):

$$f_c = \frac{3 \times F \times I}{2 \times d_1 \times d_2^2} \qquad (4)$$

Where f_C is the flexural strength (MPa), F is the maximum load (N), I is the distance between the supporting rollers (mm), and d_1 and d_2 are the lateral dimensions of the cross section (mm).

Smooth rectangular specimens without notches are commonly used for bend testing under three-point or four-point bend arrangements as shown in figures 6 a) and b) respectively. Figure 7 outlines three-point bending which is equipped of 180^{o} bend angle for welded materials.

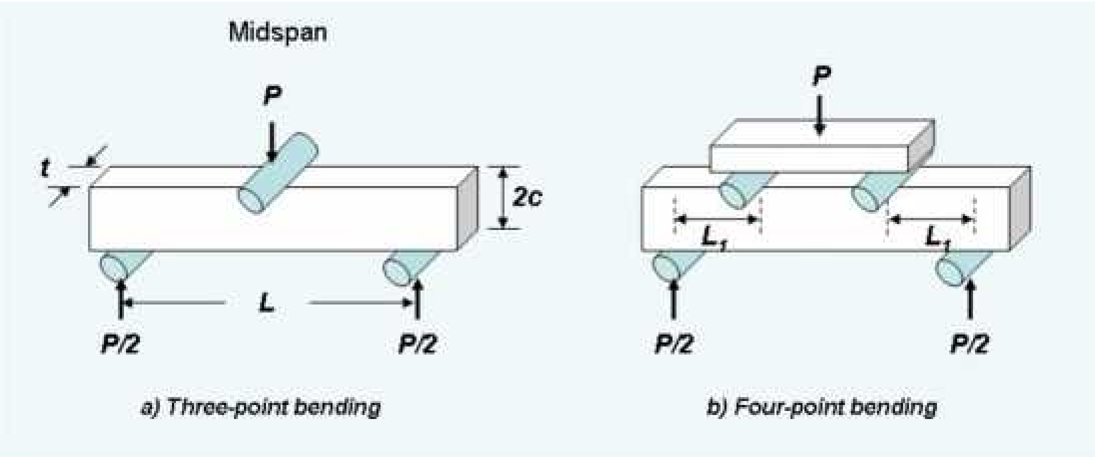


FIGURE 6. Bend testing of a rectangular bar a) three-point bending b) four-point bending

Considering a three point bend test of an elastic material, when the load P is applied at the mid-span of specimen in an x-y plane, stress distribution across the specimen width (w = 2c) is demonstrated in figure 8 a). The stress is essentially zero at the neutral axis N-N. Stresses in the y axis in the positive direction represent tensile stresses whereas stresses in the negative direction represent compressive stresses. Within the elastic range, brittle materials show a linear relationship of load and deflection where yielding occurs on a thin layer of the specimen surface at the mid-span. This in turn leads to crack initiation which finally proceeds to specimen failure. Ductile materials however provide load-deflection curves which deviate from a linear relationship before failure takes place as opposed to those of brittle materials previously mentioned.

Moreover, it is likewise hard to focus the start of yielding for this situation. The stress distribution of a ductile material after yielding is given in figure 8 b). Along these lines, it can be seen that bend testing is not suitable for ductile materials because of complexity in deciding the yield point of the materials under bending and the acquired stress-strain curve in the elastic region may not be linear. The results got may not be approved. Therefore, the bend test is subsequently more fitting for testing of brittle materials whose stress-strain curve demonstrate its linear elastic behavior just before the materials fail.

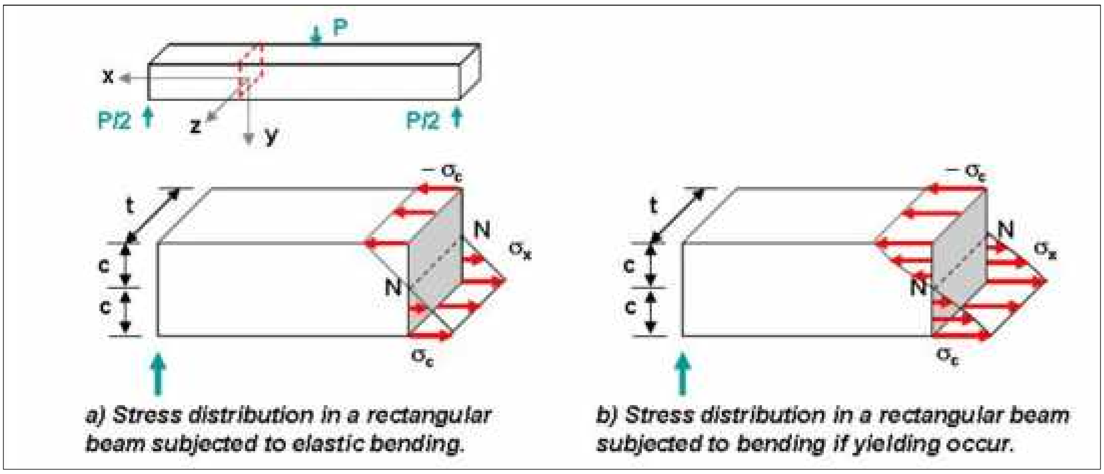


Figure 7. Stress distributions in a rectangular bar when a) elastically bended and b) after yielding

2.4.3 Durability

UHPFRC not only results in higher mechanical strength but also leads to greater durability properties. This makes UHPFRC both a high strength and a high performance material. The very low porosity of UHPFC, particularly capillary porosity, leads to great improvements in the durability properties of UHPFC. (Tayeh, Bakar, Johari, & Voo, 2013a). The great durability of UHPFRC may lead to reduce maintenance costs for the material and a possible reduction in the cover concrete required to resist weathering effects compared to normal concrete.

2.5 UHPFRC in Rehabilitations and strengthening

Reinforced concrete structure has suffered severe degradation since their construction due to the combined effects of aggressive environments and significantly increased live load. Furthermore, many concrete civil infrastructures (e.g. buildings, bridges, tunnels, etc.) are not designed to resist highly impulsive load such as blast or impact. As a result, the structures will progressively fail during unexpected event of explosion. In the uncertain times facing us these days such infrastructures need to be less susceptible by using a material with high energy absorption capacity, such as UHPFRC.

Repair and strengthening in order to improve the durability of these structures has become critical. The main concern of civil engineers is to save, retrofit, and these deteriorating structures. The implementation and development of new, cost-effective repair methods are required to extend the service life. Therefore, ultra-high performance reinforced concrete (UHPFRC) properties in terms of durability and strength are fully exploited in rehabilitation and strengthening.

An original concept is presented for the durable rehabilitation and strengthening of concrete structures. The main idea is to use Ultra-High Performance Fibre Reinforced Concrete (UHPFRC) complemented with steel reinforcing bars to "harden" and strengthen those zones of the structure that are exposed to severe environmental influences and high mechanical loading. This concept combines efficiently protection and resistance properties of UHPFRC and significantly improves the structural performance of the rehabilitated concrete structure in terms of durability. (Bruhwiler, 2012)

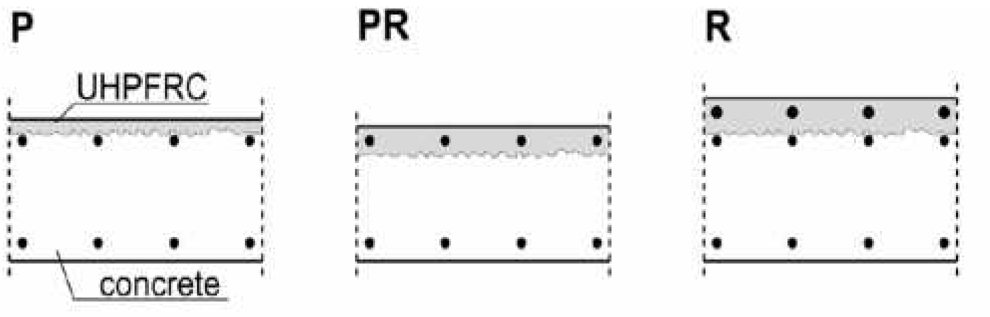


Figure 8. Basic configuration for composite structural elements combining UHPFRC and conventional structural concrete

The combination of the protective and load carrying properties with the mechanical performance of reinforcement bars provides a simple and efficient way of increasing the stiffness and load carrying capacity with compact cross sections (Figure 5). UHPFRC is applied on existing reinforced concrete elements as thin watertight overlays (in replacement of currently used waterproofing membranes), as reinforcement layers combined with reinforcement bars, or as prefabricated elements. The relatively high cost of these materials imposes to use them only where they take the maximum benefit of their outstanding mechanical properties.

CHAPTER 3

METHODOLOGY / PROJECT WORK

This study was conducted experimentally with a series of tests to investigate the structural behavior of Ultra-high Performance Fiber Reinforced Concrete (UHPFRC) members under different loading rates by conducting three-point bending tests. Compressive strength of UHPFRC also determined from compression tests. The design of the experiment is described below.

3.1 Design of experiment

3.1.1 Specimen specifications

The compressive strength of UHPFRC was determined in compression test of 21 UHPFRC cube specimens. These specimens had a length of 100 mm, a width of 100 mm and a depth of 100 mm. For the static and impact behavior of UHPFRC was determined in bending on 21 UHPFRC prism specimens. Each prism had a length of 500 mm, a width of 100 mm and a depth of 100 mm. Additional of 7 cube, 7 cylinder and 7 prism specimens of normal strength concrete for control specimens.

3.1.2 UHPFRC composition and specimen preparation

The UHPFRC mix design was based on the composition developed by Mohan, 2014. Mix proportions are provided in Table 2. The mix design of UHPFRC differs significantly from that of normal and high-strength concrete. UHPFRC mix compositions are characterized by high cement, super plasticizer, and silica fume content. The water-binder ratio is determined as w/b = 0.16 and a high percentage of silica fumes is implemented.

Furthermore, in order to achieve sufficient strain-hardening behavior, various percentages of steel fibers in excess of 1% were incorporated. In this study fiber content was selected as the main test variable and was classified into three groups corresponding to volume ratio, which was increased in increment of 1% to 3%. The chemical properties of cement were presented in Table 3.

Table 2: Mix design of UHPFRC

Cement Content (kg/m3)	**Relative weight ratios to cement**							**Steel fiber* (V_f, %)**
	Cement	Silica fume	Quartz powder	Quartz sand	River sand	Super plasticizer	Water	
1000	1.00	0.25	0.25	0.40	0.80	0.04	0.20	0
1000								1
1000								2
1000								3

* Steel fiber volume expressed as volumetric ratio to the whole volume.

Table 2. Chemical properties of cement

Material	**Chemical composition (%)**										
OPC	Na_2O	MgO	Al_2O_3	SiO_2	P_2O_5	K_2O	CaO	TiO_2	Fe_2O_3	SO_3	MnO
	0.20	2.42	4.45	21.45	0.11	0.83	63.81	0.22	3.07	2.46	0.20

3.2 Specimen preparation and experimental setup

The specimens for UHPFRC and normal strength concrete were subsequently tested at 14 and 28 days after casting. To eliminate variations in casting and testing procedures, careful consideration was given at all stages. In particular, the same equipment and procedures for casting and testing were used at all times. The mixing procedures are described below:

1. Silica fume, river sand and quart sand are dry mixed for about 4 minutes.
2. Cement and quartz powder are added and mixed for 4 minutes.
3. All water is added followed by all super plasticizer while mixer runs.
4. Mix until the homogenous mix is achieved
5. Steel fibers are added.
6. Mix until all fibers are properly coated and dispersed.

3.2.1 Curing Method

Curing is the process of controlling the rate and extent of moisture loss from concrete during cement hydration. In order to obtain good quality concrete, an appropriate mix must be followed by curing in a suitable environment during the early stages of hardening. Curing must be undertaken for a reasonable period of time if concrete is to achieve its potential strength and durability.

For this study, curing method applied is the conventional curing which involves dipping the specimens in water at 25^{o}C at the end of 24 hours of casting after allowing for air drying. The specimens are dipped in the water for 14 days and 28 days before the compressive strength and flexural strength is determined. Figure 9 shows the curing method.

Figure 9. Curing method

Simple testing procedures to determine the compressive strength and the flexural strength as described below.

3.2.2 Compression Test

This study has followed the test configuration described in the BS EN 12390-3-2009 for the determination of compressive stress-strain values for UHPFRC. Seven test specimens with dimension 100 x 100 x 100 are manufactured for each mix design described in Table 2. The compressive strength test will be conducted using Digital Compressive Testing Machine. During the test, the specimens will be loaded with constant load without any sudden shock. The compressive strength value of the concrete will be recorded and analyzed.

Figure 10. Setup of the compression test

3.2.3 Three-point Bending Test

The test methods utilized to focus the tension softening property of UHPFRC, uni-axial tensile test, and compact tension (CT), comparing to those generally used for normal concrete. Among them, the ending test is the most broadly practiced method, because of its simplicity. While the uni-axial tensile test offers the point of interest of specifically deciding the tension softening curve, difficulties are experienced in securing precision.

The CT test displays a discernible advantage in being essentially free from the effects of self-weight of the specimen due to the extensive failure area developed in specimens with small volumes. However, with this method the tension softening curve is resolved by implication, as with the bending test, and is seldom connected because of the need of special equipment.(Kang, Lee, Park, & Kim, 2010)

This study performed a 3-point bending test for the determination of the UHPFRC behavior under different loading rates which are 0.1 kN/s, 0.5 kN/s, and 1.0 kN/s. This method was performed using the guidance from the BS EN 12390-5-2009. For each concrete mixture shown in Table 2, seven test specimens were casted for each mix design. The dimensions of the specimens are 100 x 100 x 500 mm. The location where the load will be applied under 3-point bending is marked. Bend testing is carried out using a dynamic testing machine until failure takes place. Figure 11 and 12 show the 3-point bending test arrangement.

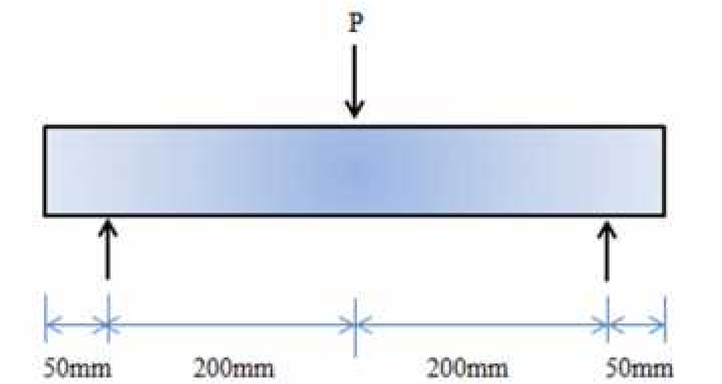


Figure 11. 3-point bending test arrangement

Figure 12. Example of bend testing under a three-point bend arrangement

3.3 Project Key Milestones

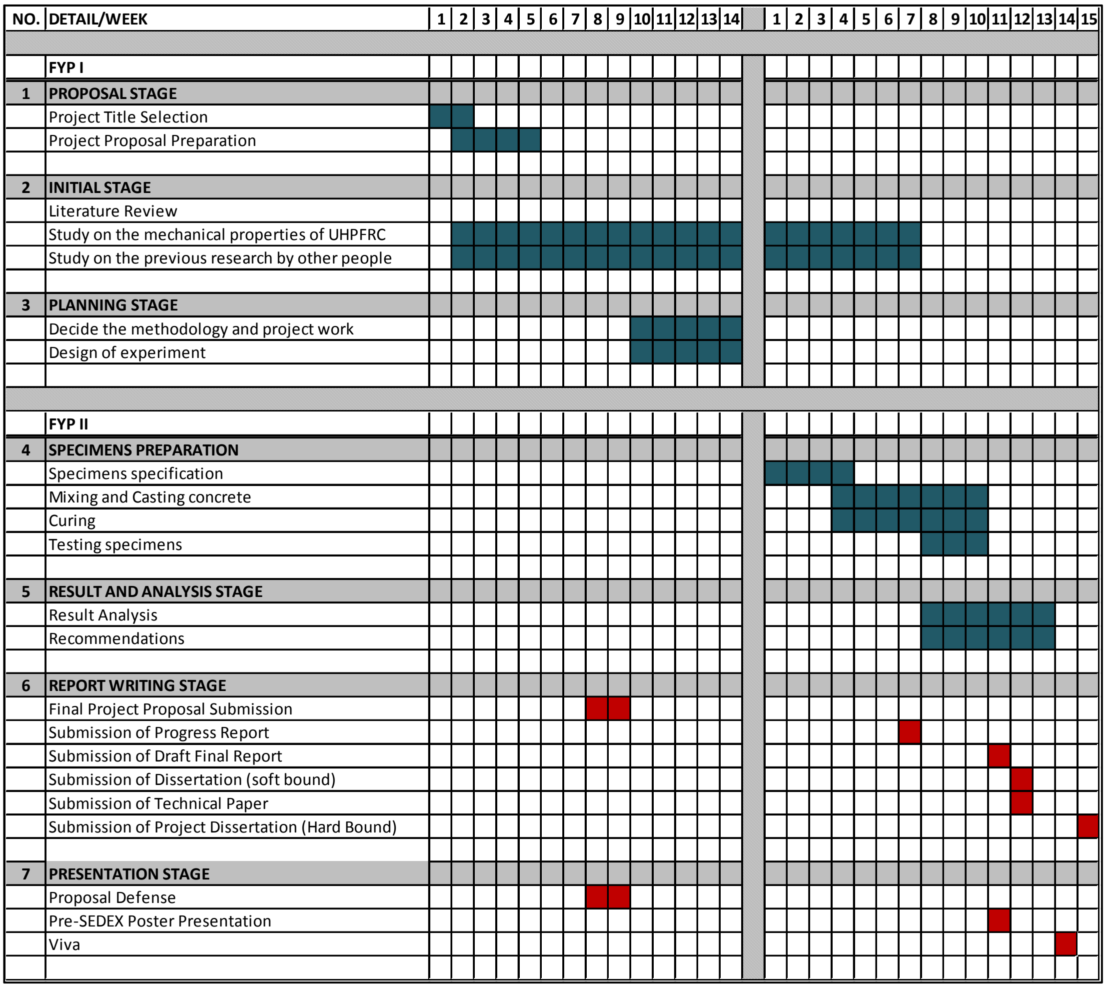
Milestone planning is used to show the major steps that are needed to reach the goal on time. When several tasks have been completed the milestone is reached. In this project, there are few major stages have been identified which are:

1. Proposal stage
2. Initial stage
3. Planning stage
4. Specimens preparation
5. Results and analysis stage
6. Report writing stage
7. Presentation stage

These steps are the planning milestone toward the completion of the Final Year Project I and II.

3.4 Project Timeline (Gantt Chart)

The project timeline is shown below.



CHAPTER 4

RESULTS AND DISCUSSION

4.1 Compressive strength

Figure 10 shows the UHPFRC configuration before and after the compression test. The specimen's age is 28 days under water curing method. Subjected to uniaxial compression loading, the failure cracks generated are approximately parallel to the direction of applied load (Figure 10) with some cracks formed at an angle to the applied load.

a) Before test b) After test

FIGURE 13. UHPFRC configuration a) before and b) after compression test

Table 4 present the compressive strength result of UHPFRC after 14 and 28 days. SFC1, SFC2 and SFC3 denote the specimen with different steel fiber volume ratio, V_f which is 1%, 2% and 3%, while the control specimens contained 0% of steel fiber.

A number of specimens from each mix design are tested in order to obtain more accurate result. The summary of the compressive tests as follows.

Table 4. Compressive strength result for 14 days and 28 days water curing

Specimens		**14 days**		**28 days**	
		Compressive strength (MPa)	**Average (MPa)**	**Compressive strength (MPa)**	**Average (MPa)**
Control ($V_f = 0$ %)	S1	125.00	**120.00**	155.1	**148.43**
	S2	115.00		152.9	
	S3	120.00		137.3	
	S4	-			
SFC1 ($V_f = 1$ %)	S1	131.00	**139.08**	151.70	**152.90**
	S2	152.50		154.10	
	S3	150.00		-	
	S4	122.80		-	
SFC2 ($V_f = 2$ %)	S1	146.70	**140.00**	156.70	**156.00**
	S2	134.50		155.30	
	S3	147.70		-	
	S4	131.10		-	
SFC3 ($V_f = 3$%)	S1	152.60	**151.28**	170.90	**173.25**
	S2	146.10		175.60	
	S3	163.10		-	
	S4	143.30		-	

As described in the Table 4, there are two series of tests conducted which are after 14 days water cured and 28 days water cured. For 14 days water cured, the compressive strength of the concrete cube specimens is increasing as the fiber volume ratio is increased. The same trend is observed for the 28 days water cured.

At 28 days of age, the concrete has reached its design strength. Thus, the compressive strength for 28 days is higher as compared to 14 days of age. For this study, the UHPFRC is designed to achieve more than 150 MPa using the available local material. From the results obtained, all the UHPFRC cube specimens have exceed the design strength. The highest compressive strength obtained is 175.60 MPa.

The variation of compressive strength of UHPFRC for 14 days and 28 days water curing and compressive strength between UHPFRC and conventional concrete is presented in Figure 14. The figure shows the comparison of compressive strength for different fiber volume ratio which varies from 0% to 3% of steel fibers. As seen in Figure 14, higher fiber volume ratio results in higher compressive strength. UHPFRC have higher compressive strength compared to conventional concrete which does not contain steel fibers.

Based on the results obtained, it is seen that UHPFRC reflects the superior behavior with respect to high strength and ductility, respectively. Therefore, UHPFRC is suitable for use in civil and structural engineering which required high strength and ductility or use as strengthening material for repair/rehabilitation work for deteriorated reinforced concrete structures. This is supported by the previous study by Yen Lei Voo et al, 2012.

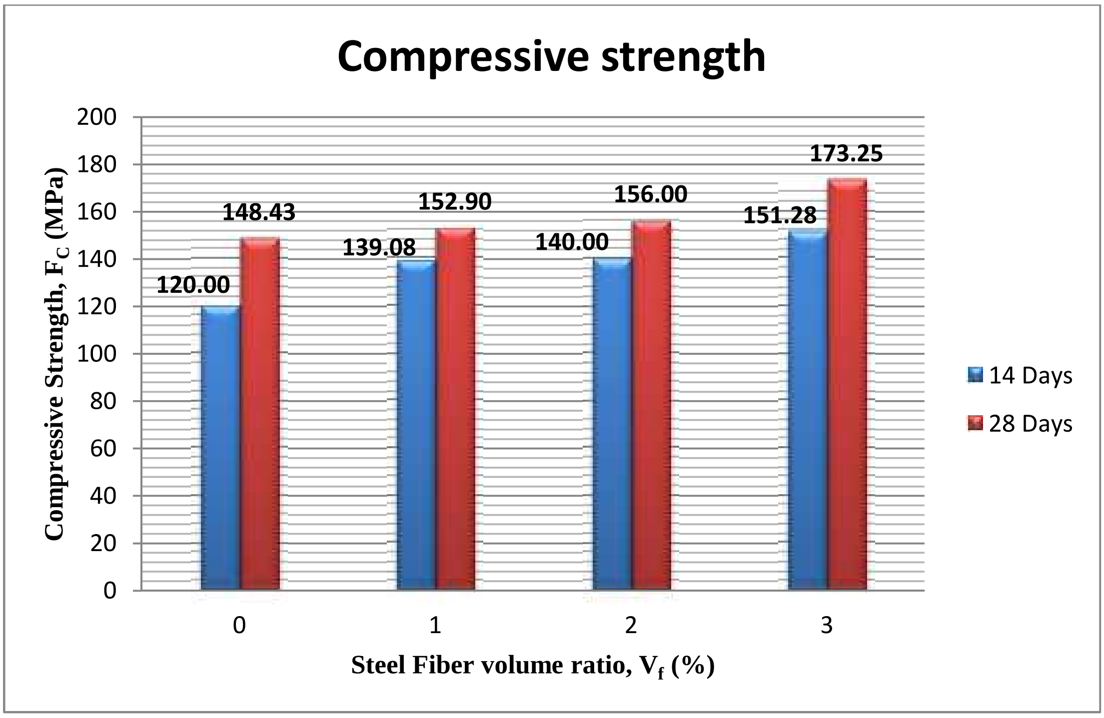


Figure 14. Variation of compressive strength with respect to steel fiber volume ratio

4.2 Flexural strength

As described before, this study performed a 3-point bending test for the determination of the UHPFRC behavior under different loading rates which are 0.1 kN/s, 0.5 kN/s, and 1.0 kN/s. This method was performed using the guidance from the BS EN 12390-5-2009. Figure 13 shows the UHPFRC failure configuration before and after the 3-point bending test. The specimen's age is 28 days under water curing.

a) Before test

a) After test

Figure 15. UHPFRC configuration a) before and b) after 3-point bending test

The results of the 3-point bending tests are presented in Table 5, which compare the behavior of each specimen at different loading rates. SFB1, SFB2 and SFB3 denote the specimen with different steel fiber volume ratio, V_f which is 1%, 2% and 3%, while the control specimens contained 0% of steel fiber. The flexural strength for each specimen is calculated by using equation below.

$$f_c = \frac{3 \times F \times I}{2 \times d_1 \times d_2^2}$$

Where f_c is the flexural strength (MPa), F is the maximum load (N), I is the distance between the supporting rollers (mm), and d_1 and d_2 are the lateral dimensions of the cross section (mm).

The table shows the comparison of flexural strength for different fiber volume ratio which varies from 0% to 3% of steel fibers. As seen in Table 5, higher fiber volume ratio results in higher flexural strength. UHPFRC have higher flexural strength compared to conventional concrete which does not contain steel fibers.

Table 5. Summary of 3-point bending test result

Specimens	Loading rates (kN/s)	Maximum Load (kN)	Flexural Strength (MPa)	Deflection (mm)
Control	**0.1**	**27.589**	**16.553**	**0.362**
SFB1		**28.389**	**17.033**	**0.343**
SFB2		**47.281**	**28.369**	**1.197**
SFB3		**53.479**	**32.087**	**0.610**
Control	**0.5**			
SFB1		**33.986**	**20.392**	**0.326**
SFB2		**48.281**	**28.969**	**1.010**
SFB3		**51.679**	**31.007**	**0.781**
Control	**1.0**			
SFB1		**36.285**	**21.771**	**0.438**
SFB2		**44.482**	**26.689**	**0.629**
SFB3		**45.482**	**27.289**	**0.553**

From the results obtained, the behavior of UHPFRC is differing as different loading rates are applied. For SFB1, the flexural strength is increasing as higher loading rates are applied. Different trend is observed for SFB2 which increased at first and then drop as 1.0 kN/s loading rate is applied. While, the flexural strength of SFB3 is decreasing as higher loading rates are applied.

The response of UHPFRC under different loading rates is presented in the Figure 16, 17, and 18. Applied load is plotted as a function of mid-span deflection for the three-point bending tests. By analyzing the load-deflection curves for each loading rate, SFB3 has the highest applied load compared to the others. For applied loading rate 0.1 kN/s, the maximum applied load for SFB3 is 53.479 kN, while for 0.5 kN/s and 1.0 kN/s the maximum applied load for SFB3 is 51.679 kN and 45.482 kN respectively. As discussed earlier, the flexural strength of SFB3 is decreasing al higher loading rates are applied.

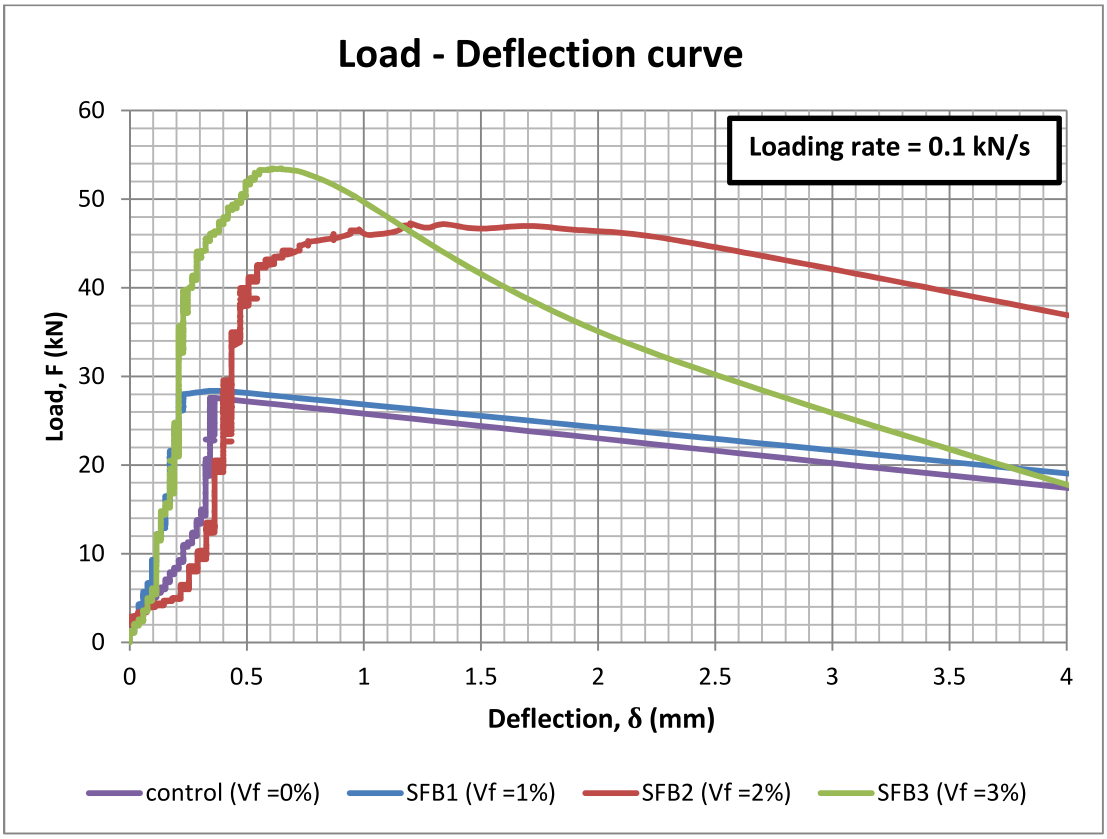


Figure 16. Load-deflection curve for 0.1 kN/s loading rate

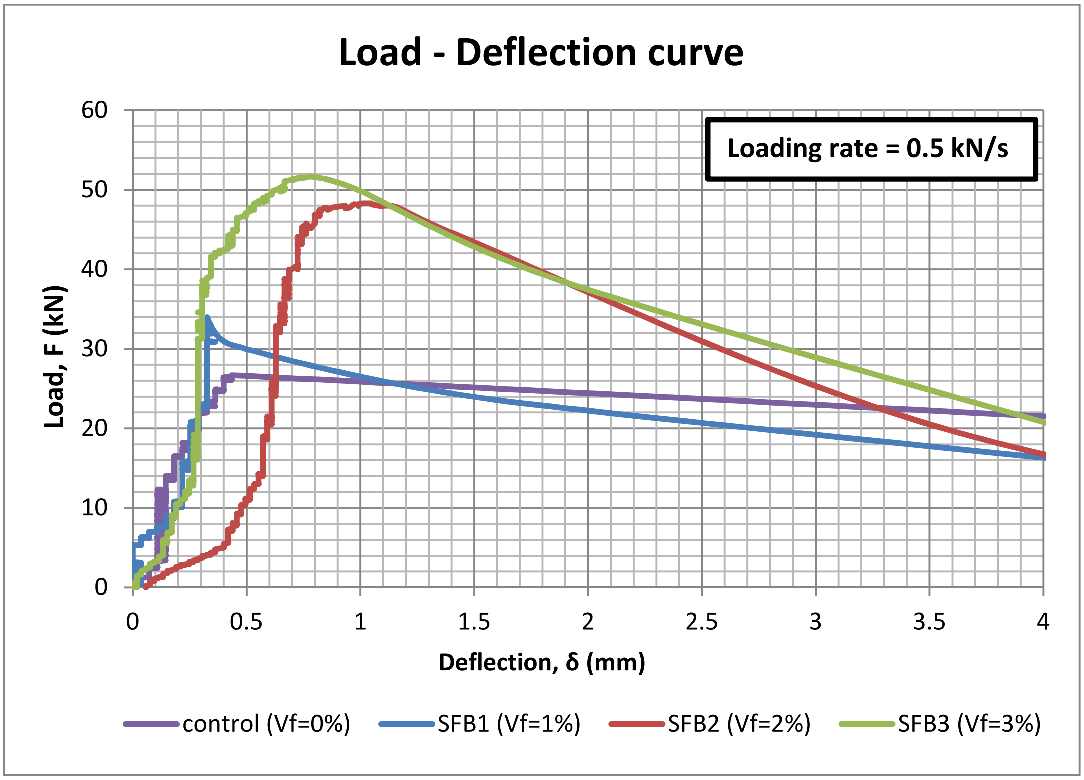


Figure 17. Load-deflection curve for 0.5 kN/s loading rate

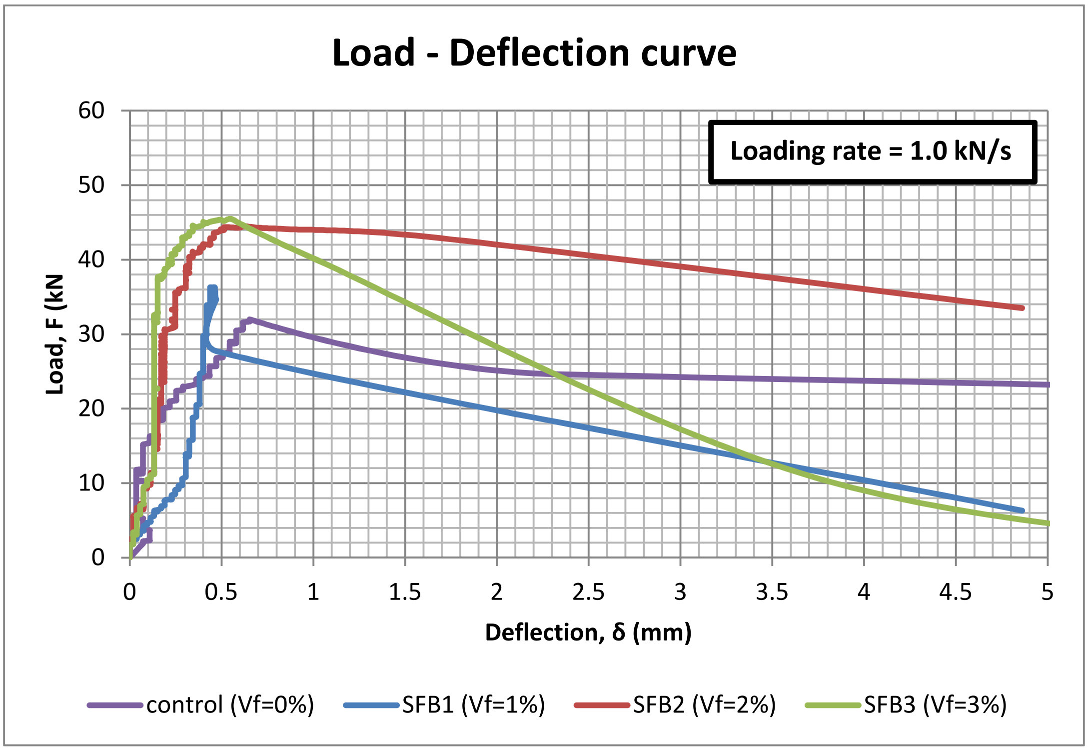


Figure 18. Load-deflection curve for 1.0 kN/s loading rate

Figure 19, 20, 21 and 22 present a comparison of Load-deflection curve for four fiber volume ratios. They showed that the maximum load increases gradually together with the gradual change of brittle behaviour in the softening section. Figure 19 and figure 20 show the behaviour of specimens with 0% and 1% fiber volume ratio. By comparing the two figures, the flexural strength does not have an obvious trend with the the fiber volume ratios. However, figure 21 and 22 which show the performance of UHPFRC with 2% and 3 % respectively show a significant increase in flexural strength and show the increase in ductility.

In addition, the figures also show that the flexural strength is increasing as higher loading rates are applied. However, in case of SFB3 (V_f=3%) shows different trend, which is decreasing as higher loading rates are applied. Although it is decreasing, the different in flexural strength is not much different. Besides, the loading rates also not much different to reflect the actual behaviour of UHPFRC. Thus, the inconsistency in results obtained is might be due to the equipment error which is not calibrated properly during the test is conducted.

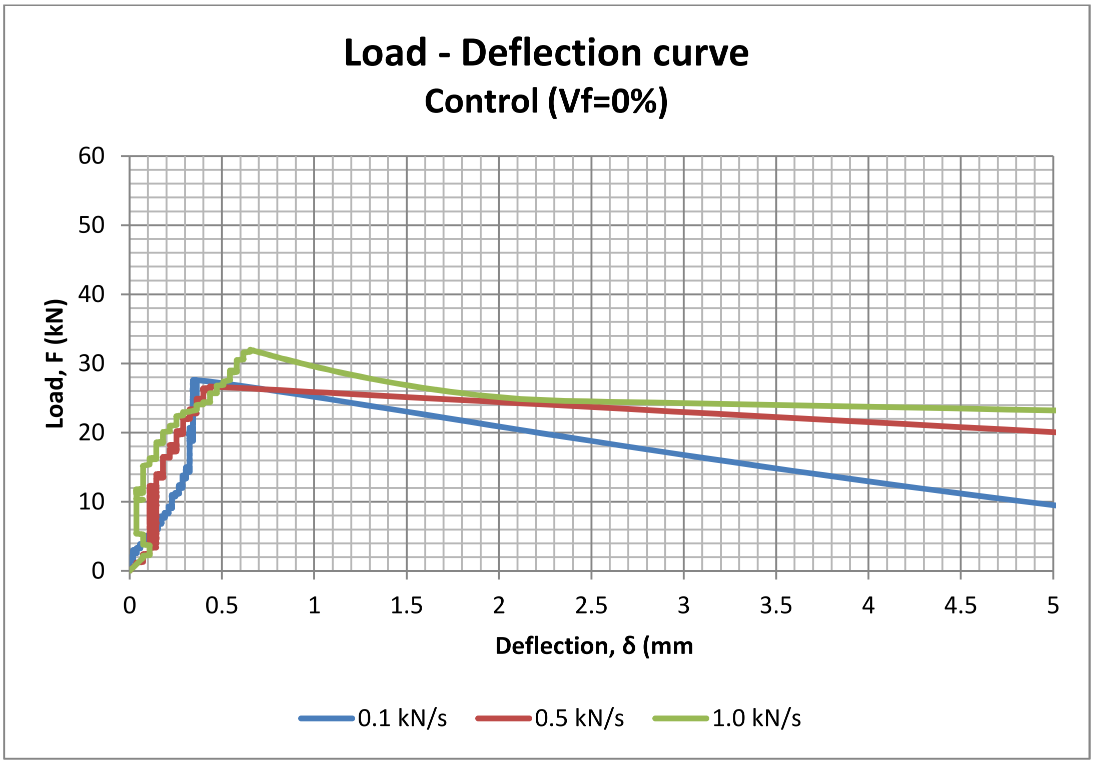


Figure 19. Load-deflection curve for control (Vf=0%)

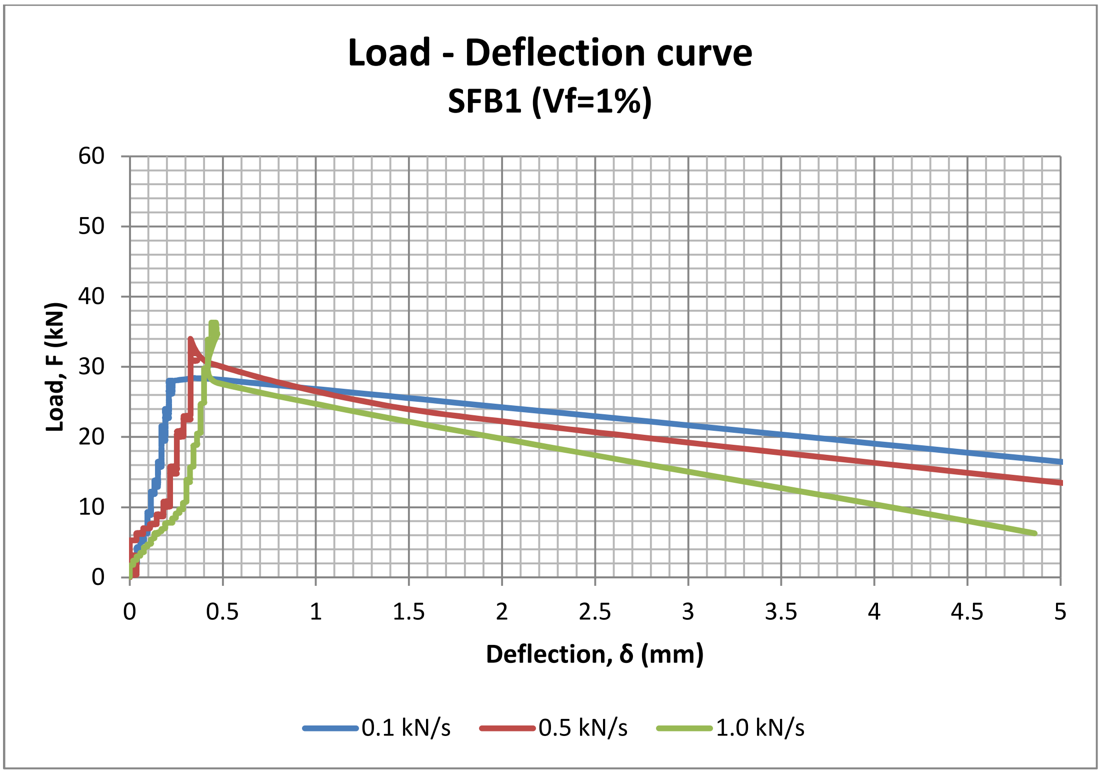


Figure 20. Load-deflection curve for SFB1 (Vf=1%)

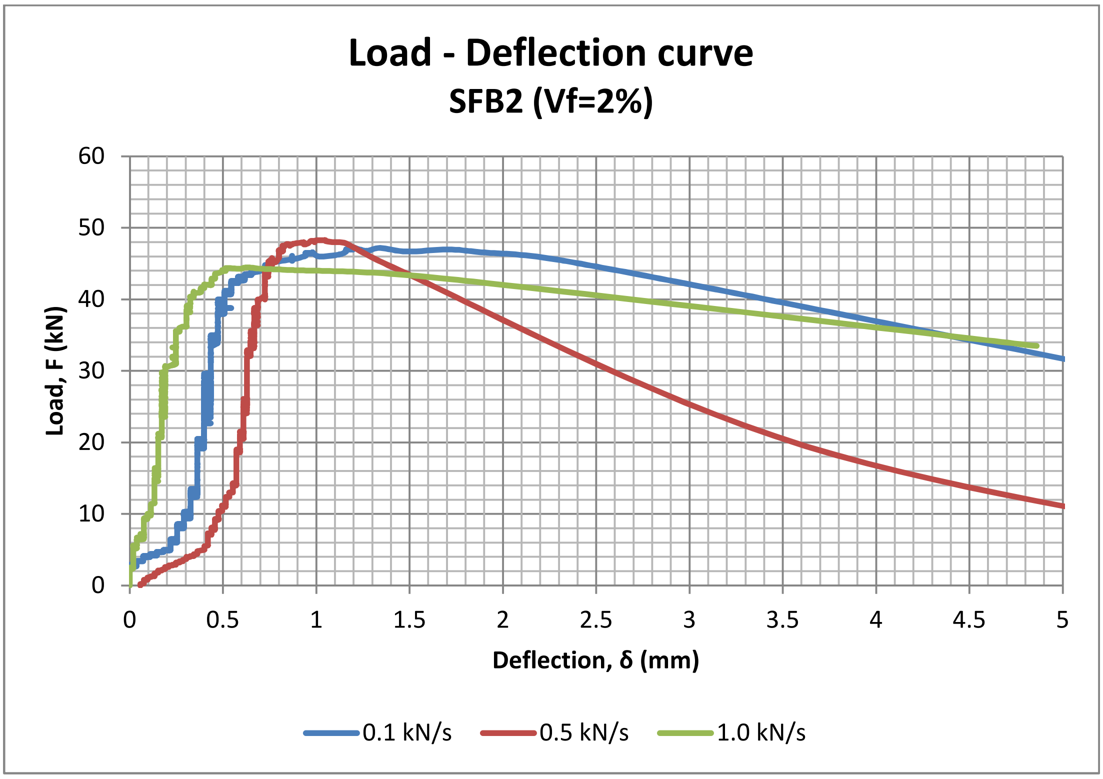


Figure 21. Load-deflection curve for SFB2 (Vf=2%)

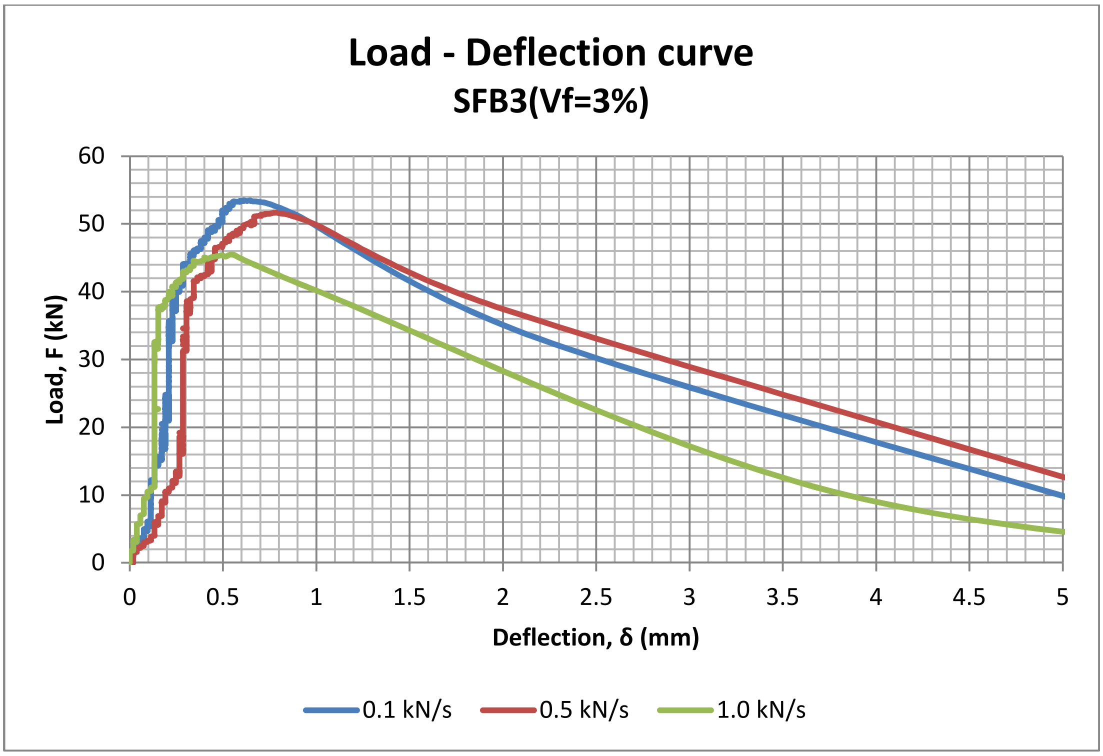


Figure 22. Load-deflection curve for SFB3 (Vf=3%)

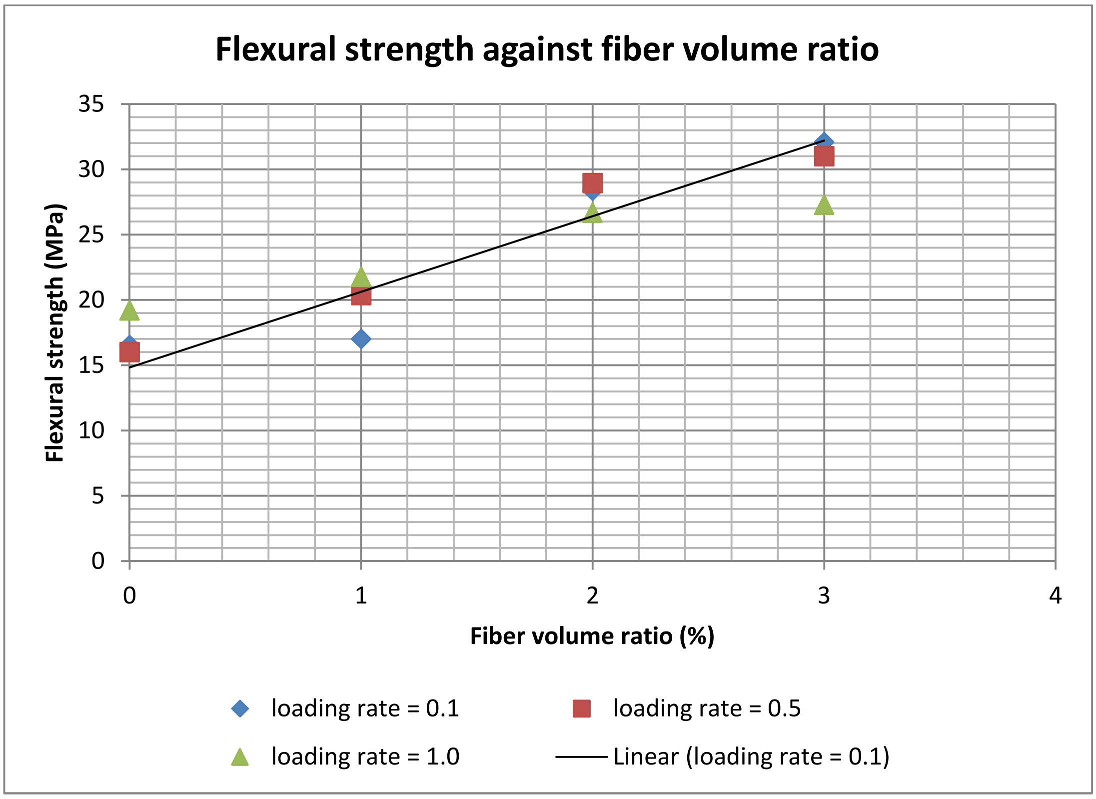


Figure 23. variation of flexural strength with respect to fiber volume ratio

Figure 23 shows the relationship between the flexural strength and the corresponding fiber volume ratio obtained from the experiment. Based on this figure, the flexural strength of UHPFRC is linearly dependent on the fiber content. This figure also shows the comparison between the flexural strength and the loading rates with respect to four group of fiber volume ratio which varied from 0% to 3%.

From the figure, the overall concrete performance showed high flexural strength at high loading rate. The phenomenon of strain rate sensitivity v=can be explained by the Griffith's theory, which state that if the load is applied very slowly, the subcritical flaws have time to grow and thus the failure occurs at a lower value of load. However, if the load is applied at very high rate, there is little or no time available for the growth of the subcritical flaws, and higher load can be reached by structural element before failure occurs. The result obtained is also can be supported by the previous study conducted by Fanlu Min et al. in 2014.

CHAPTER 5

CONCLUSION AND RECOMMENDATION

Based on the mechanical properties of UHPFRC with high strength and its outstanding performance, the behavior of UHPFRC appears to make it capable of resisting high loading rates. UHPFRC is also capable in resisting mechanical loading and severe environment (Bragov et al., 2013). The improved durability characteristics and the high compressive strength suggest that (UHPFRC) could be used as an attractive choice to conventional materials and solutions. UHPFRC structure is able to guarantee a long-term durability which helps avoid multiple rehabilitation interventions on concrete structures during their service life.

The mechanical characteristics and high density of UHPFRC make it an ideal material for resisting high strain rates effects, but only a few tests have been conducted to characterize the high strain resistance on the combine material of the bond UHPFRC and normal concrete. Thus, there may be more experimental tests to be carried for the bond strength between UHPFRC and normal concrete for example split test and pull off test to further proven that this material is one of the most compatible and advisable material to be used as a concrete structures repair material. Furthermore, further study can be done by using different type of steel fiber such as hooked and twisted steel fibers. In order to obtained more accurate behavior of UHPFRC under high strain rate, higher strain rates should be apply in the testing. In addition, long-term test for bond strength should also be carried out to collect statistics and to review the performance of UHPFRC against time effects. (Tayeh, Bakar, Johari, & Voo, 2013b)

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7.0 APPENDICES