University of Southern Queensland

Faculty of Engineering and Surveying

Use of Short Fibres in Structural Concrete to Enhance Mechanical Properties

A Dissertation Submitted By

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Abstract

The purpose of this research is based on the investigation of the use of short fibres in structural concrete to enhance the mechanical properties of concrete. The objective of the study was to determine and compare the differences in properties of concrete containing no fibres and concrete with fibres, as well as the comparison on the effects of different type and geometry of fibres to the concrete. This investigation was carried out using several tests, which included workability test, compressive test, indirect tensile test, flexural test and modulus of elasticity test.

A total of ten mix batches of concrete containing 0%, 0.5%, 1.0% and 1.5% fibre volume dosage rate on ‘wave cut’ steel fibres, high performance polypropylene fibres and Fibremesh were tested to determine the enhancement of mechanical properties of concrete. The workability of concrete significantly reduced as the fibre dosage rate increases. This was assessed through standard slump test, compacting factor test and VEBE consistometer test. Results of compressive strength test indicated that the use of fibre in concrete might not efficiently increase in strength. In flexural and indirect tensile test showed specimens with fibres that drastic increase in strength from specimens without fibres. A moderate increase in modulus of elasticity of the fibre reinforced concrete was indicated in modulus of elasticity test. The usage of fibres were fully utilised when it comes to post-cracking stage, as it increase on ductility and toughness of concrete. This was examined through the load/deformation curve of flexural strength test and stress/strain diagram of modulus of elasticity test.

It was found that different type and geometry of fibres influence the mechanical properties of concrete in a different manner. As to create a cost efficient fibre reinforced structure, these changes on fibres are vital to the design and construction. However, further investigations were highly recommended and should be carried out to understand more mechanical properties of fibre reinforced concrete.

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Table of Contents

List of Figures

List of Tables

Appendices

CHAPTER 1

Introduction

1.1 Introduction of Fibre Reinforced Concrete (FRC)

Concrete is acknowledged to be a relatively brittle material when subjected to normal stresses and impact loads, where tensile strength is only approximately one tenth of its compressive strength. As a result for these characteristics, concrete member could not support such loads and stresses that usually take place, majority on concrete beams and slabs. Historically, concrete member reinforced with continuous reinforcing bars to withstand tensile stresses and compensate for the lack of ductility and strength. Furthermore, steel reinforcement adopted to overcome high potentially tensile stresses and shear stresses at critical location in concrete member. The additional of steel reinforcement significantly increase the strength of concrete, but to produce concrete with homogenous tensile properties, the development of microcracks is a must to suppress. The introduction of fibres was brought in as a solution to develop concrete in view of enhancing its flexural and tensile strength, which are a new form of binder that could combine Portland cement in the bonding with cement matrices. Fibres are most generally discontinuous, randomly distributed throughout the cements matrices. The term of 'Fibre reinforced concrete' (FRC) is made up with cement, various sizes of aggregates, which incorporate with discrete, discontinuous fibres (Bentur et. al, 1990).



Polypropylene fibres Steel fibres Glass fibres

Figure 1.1: Types of fibres available on market.
(Source: Torben Lenau, 2001 for glass fibre)

1.2 Historical Development

The concept of using fibres in a brittle matrix was first recorded with the ancient Egyptians who used hair from animals and straw as reinforcement for mud bricks and walls in housing. This dates back in 1500 B.C. (Balaguru et. al, 1992). At the similar time period, about 3500 years ago, straws were used to reinforce sun-baked bricks for a 57m high hill of ‘Aqar Quf’, which is located near Baghdad. It is until the 1900’s that asbestos fibres were developed, manufactured and widely used to augment mechanical properties of cement matrix as described by Bentur and Mindess (1990). Balaguru and Shah (1992) reported that the modern developments of using only straight steel fibres began in the early 1960’s. Till now, a widely range of other type of fibres were used in cement matrices. Construction industries have led the development of type of conventional fibres such as steel, stainless steel and glass; where new types of fibres such as Kevlar and carbon; and several low modulus fibres, such as man made fibres (polypropylene, nylon) or natural fibres (jute, sisal, bamboo and wood pulp), as they are varies in their properties, cost and effectiveness. As they may produce as bundled filaments or fibrillated films, or may used as mats or woven fabrics (Bentur et. al, 1990). Primarily, the usages of fibres in modern industries are discontinuous fibres. Development of concrete with modified polymer fibres systems increases the explicit effects and mechanical properties of concrete.

In the early stage of fibre development, steel and glass fibres with geometry of straight and smooth were used, as these fibres improve in ductility, flexural strength and fracture toughness of concrete matrix. The primary factors that controlled for this composition were fibre volume fraction and length/diameter. However, the problems faced were difficulty in mixing and workability. Balaguru and Shah (1992) reported that fibres that are long and at higher volume fractions were found to ball up during the mixing process. The process called ‘balling’ occurs, causes the concrete to become stiff and a reduction in workability with increase volume dosage of fibres. This has a tendency to influence the quality of concrete and strength.

In last 40 years, discovery and acceptance of reinforcement and fibres for enhancement of concrete properties rapidly increased for use in concrete industries, research and development. Numerous types of fibres have successfully been adapted in the different applications of concrete. Technological advances bought forward the development of fibres with different geometric shapes and properties to expand the benefits in concrete structures. New manufacturing techniques and applications on fibres for concrete have been developed. These introduce various aspects of fibre-reinforced concrete and introduced to market worldwide.

All these fibres with more complicated geometric, shape and sizes have developed, mainly to modify each of their mechanical bonding with cement matrix. When fibre is added to a concrete mix, each and every individual fibre receives a coating of cement paste. Modification of fibre geometry includes hooked end fibres, deformed fibres, deformed wires, fibre mesh, wave cut fibres, large end fibres. This increases bonding without increasing in length and minimise chemical interaction between fibres and the cement matrices. This also modifies and enhances the mechanical properties and behaviour of concrete in its applications.

Fibre can be use with admixtures such as superplasticizer, air entraining, set retarding, set-accelerating admixtures and all types of cement and concrete mixtures. These produce special types of concrete with desired characteristics in fresh and hardened concrete. They increase workability, accelerated and retarded rate of hydration of cements, and resistance to freeze and thaw conditions. They provided a significant improvement to the fibre-reinforced concrete used in the fields.

1.3 Application of Fibre Reinforced Concrete in modern industries

Fibre-reinforced concrete has specialized properties and can enhance impact, abrasives, high durability, shatter, and vibration. In beginning, fibre-reinforced concrete were used for pavement and industrial slabs. But recently, applications of fibre-reinforced concrete have wide variety usage in structures such as heavy-duty pavement, airplane runways, industrial slabs, water tanks, canals, dam structure, parking structure decks, water and wastewater treatment plant, pipes, channel, precast panels, structures resist to earthquake and explosives and the techniques of shotcrete application.

A list of application for Fibre reinforced concrete:

- Floors, driveway and walks to reduce shrinkage and cracking problems are desirable.
- Increase of toughness in fibre-reinforced concrete is ideal for buildings and pavements subject to shatter, impact, abrasion, and shear.
- Its use in crack control and shrinkage for water retaining and reservoir structures to reduce the permeability and freeze-thawing conditions.
- Its replacement for temperature steel in sanitary sewer tunnels prevents corrosion and improves ductility.
- Runways are made more resistant to fuel spills with less permeable and shatter resistant fibre-reinforced concrete.
- Pumped concrete project gets easy and safe with fibre, making concrete more cohesive and prevent segregation.

Figure 1.2 below shows some application of Fibre reinforced concrete used in the modern industries.

Car park, driveways

Airport Runway

Heavy duty slabs

Sewer tunnels

Figure 1.2: Application of Fibre Reinforced Concrete (Source: Fibremesh, 1989)

1.4 Advantages and limitations of Fibre reinforced concrete (FRC)

Fibres, which are randomly distributed throughout the concrete, can overcome cracks and control shrinkage more effectively. These materials have outstanding combinations of strength and energy absorption capacity. In general, the fibre reinforcement is not a substitution for conventional steel reinforcement. The fibres and steel reinforcement has their own role in concrete technology. Therefore, many applications in which both fibres and continuous reinforcing steel bars can be use together.

However, fibres are not efficient in withstanding the tensile stresses compare to conventional steel reinforcement. But, fibres are more closely spaced than steel reinforcement, which are better in controlling crack and shrinkage. Consequently, conventional steel reinforcement used to increase the load bearing capacity of concrete member; fibres are more effective in crack control.

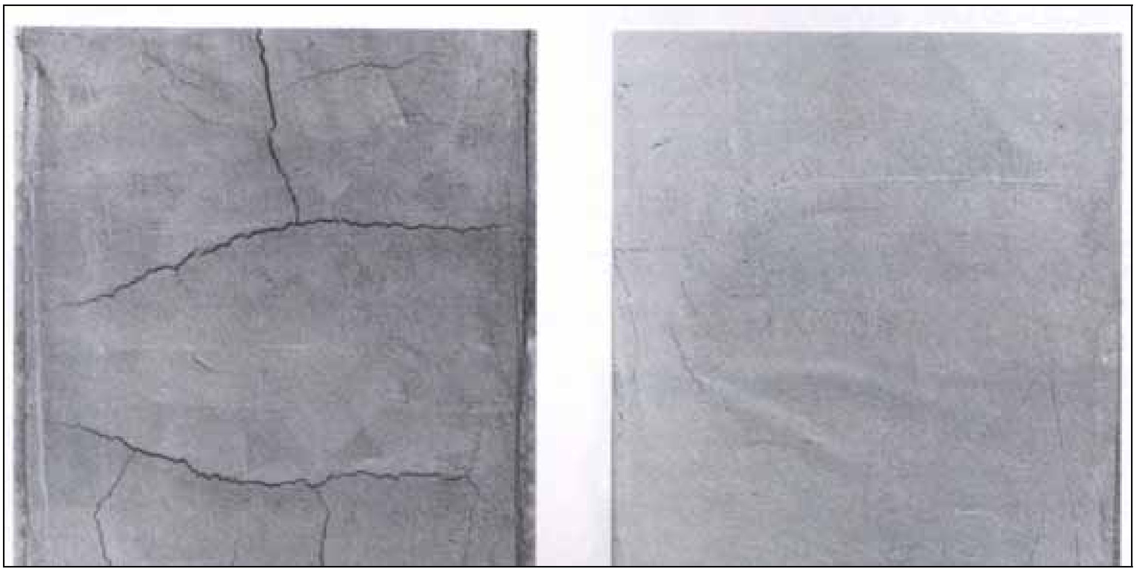
Due to these differences, there are particular applications that fibres reinforced are advance than conventional steel reinforcement. These include:

- Fibres comprise as ‘primary reinforcement’, in which the conventional steel reinforcement cannot be utilized. The fibre concentrations are comparatively high in thin sheet materials, normally exceeding 5% by volume, acts to increase in toughness and strength of mortar or concrete.
- Fibres can be components to withstand locally high loads or deformations, which applies to structures like precast piles, precast walls, blast resistant structures or sewer tunnel and linings.
- Applications that control cracks persuaded by temperature and humidity, such as pavements and slabs, where fibres offered as ‘secondary reinforcement’.

The uses of steel bars and wire mesh require unnecessary labor and material costs for structure concrete. With replacement of randomly distributed short fibres as an alternative reinforcement, will significant reduce both labour and material costs, greatly increase construction and project time.

Fibres substantially reduce formation of plastic shrinkage and settlement; enable the concrete to develop its potential long-term application to structural concrete, providing solution to exceed and meet their performance and economical prospect.

Additionally, fibres provide an effective secondary reinforcement for shrinkage and crack width control. Macro-cracks and potential problems are prevented and blocked when micro-cracks intersect fibres as concrete hardens and shrink. Effects of crack control reinforcement by additional of fibres in concrete shown in figure 1.3 below.



Without fibre-reinforced With fibre-reinforced

Figure 1.3: Comparison of cracks with and without Fibre reinforced.
(Source: Fibremesh, 1989)

Benefits of using fibres-reinforced concrete are:

- Increase impact and shatter resistance, fatigue endurance and shear strength of concrete.
- Requires no special equipments to install reinforcement.
- Increase crack resistance, long-term ductility, energy absorption capacity and toughness of concrete.
- Reduce labor and material costs in concrete applications.
- Provides multi-directional concrete reinforcement.
- Compatible with admixtures, all types of cement and concrete mixtures.
- Reduce plastic shrinkage and crack width formation.

Restrictions and limitations of using fibre-reinforced concrete are:

- Control crack as result of external stresses.
- Reduction in curling and creep.
- Justification for a reduction in the size of support columns.
- Higher structural strength development.
- Replacement of any moment for structural steel reinforcement.
- Decreasing the thickness of slab on grade.

Although short fibres cannot replace conventional steel reinforcement, they create supplementary reinforcement use to achieve increase in strength, higher ductility, greater shrinkage, crack control, fatigue, impact and abrasion resistance. However, development and advances in technologies has led to the discovery of more effects for fibres behaviour and mechanical properties of concrete.

1.5 Types of fibres

Fibres vary in types, geometry, properties and availability in construction industry. Most common types of fibres are steel fibres, glass fibres, and polypropylene fibres. These usages may alter in concrete for different applications. The fibres mostly depend and adopt on properties, effectiveness, cost and availability. Special types of fibres such as carbon, and Kevlar, natural fibres, mineral fibres, and asbestos fibres may use in harsh environment. These differences and usage of fibres depends on the requirement of behaviour and properties for a concrete, allowing the increase the explicit effects and mechanical properties. Fibre geometry varies from hooked end fibres, deformed fibres, deformed wires, fibre mesh, wave-cut fibres, large end fibres till different types and geometries.

1.6 Steel Fibres

Steel fibres are widely used in civil engineering applications and concrete reinforcement, due to its relative availability, reasonable cost and better experience in its application with conventional steel reinforcement. Bentur and Mindness (1990) stated that the early research and studies on fibre reinforced concrete in 1950's to 1960's mainly were the behaviour of steel fibre reinforced concrete. Steel fibres greatly increase toughness of concrete, which primarily is used for crack and shrinkage controls, to serves as secondary reinforcement for pavements, slabs, pipes, channel and tunnels. Its potential improvement to increase toughness, minimise cracking due to temperature changes and resistance due to extreme loading and environment such as impact, abrasion, blasting and fatigue. Furthermore, steel fibre-reinforced concrete greatly reduces the potential for fractures and spalling. Figure 1.4 to figure 1.6 below shown were different geometry of steel fibres.

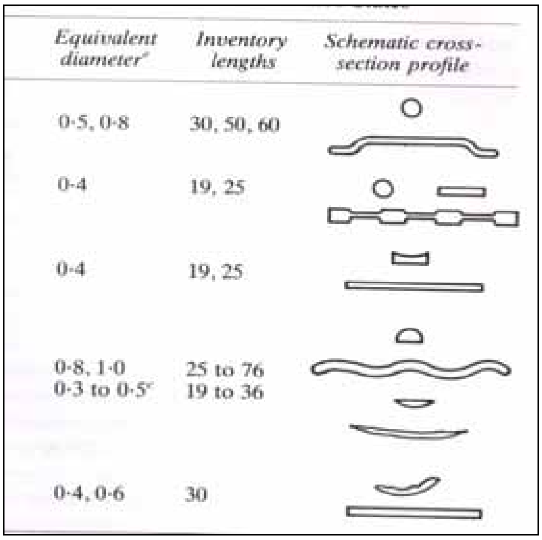
Figure 1.4: Wave-cut steel fibres.

Figure 1.5: Enlarged end steel fibres.

Figure 1.6: Deformed sheet steel fibres.

1.6.1 Shape and geometry

Cross sectional dimensions of a typical steel fibre are range 0.5mm to 1mm thick, 0.25mm to 0.90mm wide, with diameter range of 0.25mm to 0.75mm, where are created in various form of geometry. Steel fibres were produced in steel sheet form, through the process of cutting steel sheets. Depends to geometry desired, steel fibres are crimped and construct to deformed, end flat and enlarged end shapes. Using the similar process, chopped drawn wire shape of steel fibres has been produced. Steel fibres with hooked and wave shapes have been produced and well-known in use for construction industry currently. These different geometries and shapes of steel fibres are widely used in industry to fulfil the desirable behaviour and properties requirement of concrete. Figure 1.7 below shows some deformed fibres that available in the market.



Equivalent diameter[a]	Inventory lengths	Schematic cross-section profile
0·5, 0·8	30, 50, 60	
0·4	19, 25	
0·4	19, 25	
0·8, 1·0 0·3 to 0·5[c]	25 to 76 19 to 36	
0·4, 0·6	30	

Figure 1.7: Different types and geometry of steel fibres. (Source: Bentur & Mindess, 1990)

1.6.2 Durability

Steel fibre corrosion may be a major concern of durability of fibre reinforced concrete. Guidelines set in AS3600 stated that corrosion in conventional steel reinforcement could be avoided if suitable cover was provided. However, these guidelines were only applicable at particular position for conventional steel reinforcement. On fibre reinforced concrete, the steel fibres were randomly distributed throughout the matrix, as some corrosion can happen at the surface of the concrete, where it is very difficult for each fibres cover with the cement. However, with cement cover more than 1mm, the fibres are safe from corrosion. Thus, corrosion of steel fibre was considered a minor problem, as it does not affect much on the mechanical properties of the fibre reinforced concrete.

1.7 Glass Fibres

Soviet research in late 1950's explores low alkali of glass fibres in cement system, which having a low value of pH. It is until 1960's, glass fibres were classified as possible reinforcement to high pH value of cement systems (James, 1990). Glass fibres is a strong, lightweight material, which stands with tremendous fracture toughness, posses high tensile (280 to 3500MPa) and modulus of elasticity (3.1 to 3.5 GPa) in high alkaline cement systems.

1.7.1 Development of glass fibre

Glass fibres have been developed mainly in the production of thin sheet components, using glass as reinforcing bars, impregnated and saturated plastics. Glass fibres are produced in the process in which molten glass extracted by the form of filaments, at the bottom of a heated platinum tank. As filaments are extracted at the same time, they coagulated while cooling outside the tank. The filaments are collected as strands in a drum. Finally, the filaments are chopped into short strands to take random positions and applications on a cement matrix.

1.7.2 Durability

Special technologies and development are required for glass fibres as a feature of alkali-resistant glass. The problem is that a low alkali resistant of E type of glass fibres (made of borosilicate), which commonly used to reinforcing plastic, will weaken promptly in high alkali-environment of cement matrix. The alkaline nature of concrete takes place and cause damaging chemical reaction between cement and glass fibre. To inhibit this problem, a special type of alkali resistant glass, the AR type of glass fibres (made of soda-lime silicate) was developed, which reduces corrosion of glass fibre in the cement matrix (Bentur et. al, 1990). The development and research of glass fibres in a cement matrix ensures long-term durability and effectiveness of load bearing capacity when embedded in the highly alkaline environment.

Figure 1.8: Drums of glass fibre strands. (Source: J&J Trading Corp, 2002)

1.8 Synthetic Fibres

In recent years, synthetic fibres have become more attractive for reinforcement of cement and concrete material. According to James (1990), Shell Chemical Co. started the investigation on the use of polypropylene fibres in concrete around 1965. The developments of synthetic fibres were successfully utilized bonding and reinforcement in cement matrix (James, 1990). Synthetic fibres have very high tensile strength, but fibres can be differentiating into two categories, either by high or low modulus of elasticity. Most of fibres fall in the categories of low modulus of elasticity, such as polypropylene, polyester, polyethylene, and nylon. The main advantages of these fibres are alkali resistance, high melting point (up to 165^{o}C) and low cost of the raw material. Disadvantages are poor fire resistance, poor bond with cement matrix and sensitive with sunlight and oxygen. Low modulus elasticity of synthetic fibres shows the usefulness in increasing in toughness and shrinkage cracking. However, they seem less application in increase in flexural strength and ductility of concrete (Bentur et. al, 1990).

1.8.1 Properties and geometry of synthetic fibres

Synthetic fibres have high molecular isotropic and regular atomic arrangement of structure, allowing them to stretch into high degree of orientation. This type of fibres is a linear hydrocarbon polymer with a methyl group attached to alternate carbon atoms on the chain backbone (James, 1990). However, the methyl groups can affect the chemical behaviour, which can produces oxidation of the fibres. As a result, this will cause in the change of the crystallization and bulk properties of the polymer itself. Therefore, an ‘isotactic index’ (measures the percentage of polymer insoluble in the methyl compound) provides a measurement on the isotactacity, where it presents a general effects on the modulus of elasticity and the yield stress of the fibres. Synthetic fibres can be made in three different geometries: monofilaments, films and tapes. The first two forms of geometry of synthetic fibres commonly used in concrete and mortar for their mechanical anchorage effects. The filaments and fabrications provide a better bond between cement matrix, increasing the performance of cement and concrete matrix.

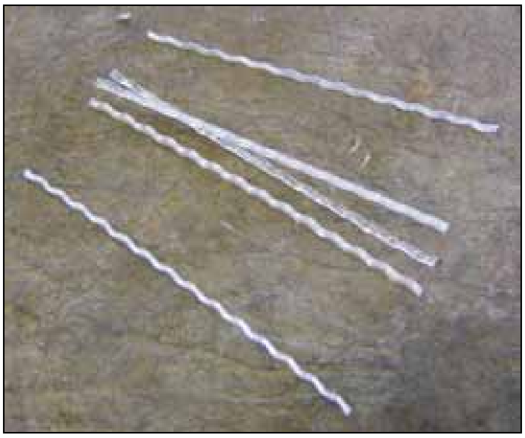


Figure 1.9: Twisted wave geometry of synthetic fibres. (Monofilament type)

Figure 1.10: Mesh geometry of synthetic fibres. (Film type)

1.8.2 Durability

The synthetic fibres have been described as chemically inert and non-toxic in the cement environment (James, 1990), as these fibres were free from environmental stress cracking problems. However, it is suspect that the oxidation of synthetic fibres may occur at elevated temperature. To encounter this problem, it was suggest that with sufficient concrete cover and synthetic fibres at 25°c will not cause any oxidation, where it can exceed the lifetime of 30 years with this temperature. Synthetic fibres may degrade when exposed to the ultraviolet radiation. Additionally, exposure of natural sunlight will also cause loss in the strength of the synthetic fibres. The synthetic fibre reinforced composites quickly degrade under fire, as standard 'flame spread tests' done in United Kingdom identified the composite belongs to the lowest class fire protection.

1.9 Other types of fibres

The above three type of fibres were the most common used fibres in the industry and construction today. There are more types of fibres were used, but their applications were limited. Such limitation of these fibres may cause by difficulty of availability in the current market, high costs of material or less effectiveness in the fibre reinforced composite. However, these fibres may have some advantages than steel, glass and synthetic fibres did not included. Some of other types of fibres include: Asbestos fibres, Natural fibres, and Carbon fibres.

1.9.1 Asbestos Fibres

Asbestos fibres are made of natural crystalline fibrous minerals. Asbestos/cement was the first fibre-reinforced composite in modern times, and still use more than any other fibre-reinforced materials. These fibres is largely in success for fibre-reinforce materials result form the compatibility between the fibres and cement matrix. Asbestos fibres relatively have high modulus of elasticity and strength, which permits effective dispersion of large fibre volume and enhance the bond between cement matrixes. These fibres are utilized with fibre-reinforced materials suitable in low cost housing and infrastructure.

1.9.2 Natural Fibres

These are the oldest form of fibre-reinforced composites, using fibres such as straw and horsehair in the structure. Recently with modern technology, natural fibres possible extract fibres economically from various vegetable and animal, such as jute, bamboo and wool. These fibres requires low amount of energy to extract. Relatively, they limited use due to high water absorption and low tensile strength compared to steel and synthetic fibres. Primary problems with these fibres are their tendency to fragment in an alkaline environment. Special treatments accept by using admixtures to improve their durability and making concrete less alkaline, allowing these fibres to increase its strength. This natural fibre-reinforced composite commonly uses for thin sheet and cement products, as well as the application for cement cladding.

1.9.3 Carbon Fibres

Carbon fibres are limiting its use in cementitious material because of its high cost in mid 1980's. Recently, low cost carbon fibres have been manufactured using petroleum and coal pitch. These two processes of making carbon fibres involve heat treatments and various grade of carbon in its chemicals. These fibres find its application to substitute cement-based pipe and wood in structural. Carbon fibres are specialized applications in high tensile and flexural strength. Typically, they have an elastic modulus as high as steel, yet they are very light. It's common uses are applications in sheeting and wrap as externally reinforced degrading concrete structures. Properties of carbon fibres greatly increase in its extremely high strengths, chemical stability and stiffness of fibre reinforced composites.

There are several precursors for production of carbon fibres. Carbon produce through controlled oxidation, orientation of graphitic crystallites, carbonisation and stretching from carbon precursors. These precursors include polyacrylonitrile (PAN), cellulosic fibres, pitch precursors, non-heterocyclic aromatic polymers, aromatic heterocyclic polymers, linear polymers and coal (James, 1990).

1.10 Field Performance of Fibre Reinforced Concrete

Concrete is the most common material used in construction field, as fibres material has brought in the play of these common sections. Fibre reinforced concrete used in slab and pavement applications general performed well than plain concrete that has the same thickness, concrete flexural strength and foundation subgrade condition. The performance of fibre reinforced concrete in the construction industry is wide, which includes industrial development, light commercial structures, residential, precast, shotcrete and transportation field. Some of fibre-reinforced concrete projects around the world are discussed in this sub-section.

1.10.1 Industrial Development

According to Synthetic Industries Concrete Company (2000), Bethlehem Steel's cold rolling mill project with estimate cost of \$389 million located in Sparrow's Point, Maryland, is a facility that produces sheet steel for the automotive and appliance industries. The 79,000m^2, 200mm thick floor slab was designed to handle 25 tonnes rolls of steel and equipment used to process the rolls. Steel fibres were added to the concrete mix at 32kg/m^3, where advantages of eliminating the need for rebar, the benefits of crack control and increased ductility.

A recent project from Novocon (2000), Fibremesh was used for reinforcement in this 51,700m^2 continuous slab in Fayetteville, Arkansas, where the construction the floor of Hannah's Candles Factory required more than 4900m^3 of fibre reinforced concrete. Fibremesh were added to the concrete mix at a rate of 0.9kg/m^3.

Figure 1.11: Continuous slab in Fayetteville, Arkansas.
(Source: Novocon, 2000)

According to Novocon (2000), Household International Office Building, a three-story structure in Brandon, Florida, was constructed using 1070m^3 of steel fibre reinforced concrete. The steel fibres were introduced at the concrete plant by using a specially designed loading rack. Workers did not have to contend with wire mesh on the metal decking as the concrete placement.

Figure 1.12: Three-storey structure in Brandon, Florida using steel Fibre reinforced slabs. (Source: Novocon, 2000)

According to Balaguru and Shah (1992), project located in Kashima, Japan, a 150mm thick fibre-reinforced slab was constructed to carry the heavy forklifts with a net gross weight of 47 tonnes. 25mm long fibres were used at a fibre dosage rate of 131kg/m^3. As a comparative evaluation, a 200mm thick conventional plain concrete slab was constructed. After one year, it was found that cracks had developed in the plain concrete slab and only one hairline crack was formed in FRC slab. This shows that FRC provides much longer pavement life and expansion joints with larger spacings are used in concrete.

1.10.2 Light Commercial Structures

A recent project done by Synthetic Industries Concrete Company (2000), a warehouse slab constructed in Springdale, Arkansas, used a combination of both steel and polypropylene fibres. The project was initially specified with wire mesh and 600mm centres along all contraction joints. Fibremesh and steel fibres were used as a replacement to the traditional method of conventional reinforcement.

In Mobile, Alabama, Fibremesh were used at a fibre volume dosage rate of 0.9kg/m^3 to enhance the quality of this 18,600m^2 parking area (Novocon, 2000). All of paving and slabs were constructed with fibre reinforced concrete for the metal frame buildings on the site. Additionally, FRC provides reinforcement throughout the entire concrete section.

Figure 1.13: Heavy duty slab for parking area located in Mobile, Alabama.
(Source: Novocon, 2000)

1.10.3 Precast

According to Synthetic Industries Concrete Company (2000), residential wall system contains of 50mm thick outer concrete skin with a wood grain texture, which made from 5,000 psi concrete, was structured in Rome, Georgia. The concrete is reinforced with a hybrid and combination mix of Fibremesh and high performance polymer fibres at an addition rate of 1.5kg/m^3.

One additional project has done by Synthetic Industries Concrete Company (2000) in Naniamo, British Columbia, portable highway median barriers was developed and constructed by fibres reinforcement. Concrete is subjected to early age stresses and can jeopardize the long-term durability of the barriers. Fibre reinforcement is the ideal solution to this problem. Synthetic fibre reinforcement holds back the development of cracks by offering an internal support system for concrete. Fibremesh were added to the precast concrete at a rate of 0.9kg/m^3 and 0.5kg/m^3.

Figure 1.14: Portable highway median barriers.
(Source: Synthetic Industries Concrete Company, 2000)

1.10.4 Residential

Fibremesh were used as secondary reinforcement in all the concrete slabs for this upscale apartment complex located in Destin, Florida. Synthetic Industries Concrete Company (2000) carried out the construction of this apartment. Fibremesh were used at an addition rate of 0.9kg/m^3 into the concrete. The concrete was placed in severe summer conditions and the fibre reinforced concrete was used to assist in controlling the drying and shrinkage cracking.

Figure 1.15: Apartment slabs reinforced with Fibremesh as secondary reinforcement located in Destin, Florida. (Source: Synthetic Industries Concrete Company, 2000)

1.10.5 Shotcrete

Drainage channel located in Bankstown, Australia, has suffered from erosion. Due to the lacking the required crane access in the installation of precast panels, 'shotcrete reinforced with Fibremesh' (Novocon, 2000) was used for this drainage channel repair.

Figure 1.16: Drainage channel repair with shotcrete fibre reinforced located in Bankstown, Australia. (Source: Novocon, 2000)

Baker's Tunnel, located in north central Tennessee, was recently rehabilitated using steel fibre reinforced shotcrete technique. This mainline railroad tunnel was estimated 80 years old. Steel fibre reinforced shotcrete was spread over to the crown and walls to prevent spalling. Fibre reinforcement used in the lining working to resist flexural and shear loading. The pre-blended shotcrete material that mixed into the concrete mix was around $50kg/m^3$ of steel fibres. (Novocon, 2000)

A recent project done by Synthetic Industries Concrete Company (2000) in Sydney, Australia, the white water rafting course was constructed for the Year 2000 Olympic games, used Fibremesh for shotcrete reinforcement. The internal walls must withstand to turbulent water and repeated battering from canoes. This structure, which is continuous, is capable of generating water-flow rates up to four meters per second along the 300-lineal meter course. Short Fibres added into the concrete mix offering an alternative system to the time and labour of placing conventional steel reinforcement.

Figure 1.17: Water rafting course located in Sydney, Australia.
(Source: Synthetic Industries Concrete Company, 2000)

1.10.6 Transportation

This leading project stated by Novocon (2000), located in Springfield, Massachusetts, and is one of ten across the state constructed to compare the use and benefits of Portland cement to bituminous concrete. This concrete road is 100mm thick consist with 1.8kg/m^3 of Fibremesh. This 100mm layer of concrete is expected to last approximately 40 years, while typical bituminous concrete lasts 15 to 20 years.

Figure 1.18: Bituminous concrete road with Fibremesh located in Springfield, Massachusetts. (Source: Novocon, 2000)

A major project recently done by Synthetic Industries Concrete Company (2000), located in Syracuse, New York, which provides road servicing to Warners Service Area Rest Stop, were rehabilitated using fibre reinforced material. A 100mm slab was poured consisting of 19 meter of concrete with 1.8kg/m^3 of Fibremesh were used for reinforcement. Another 150mm slab containing 37 meter of concrete was placed. 1.8kg/m^3 of Fibremesh dosage rate were also used in this slab. Fibre reinforced Portland cement concrete is durable and rut resistant. It creates a brighter and cooler environment with light and heat reflexiveness, and it has a longer life and lower lifecycle cost than bituminous concrete.

At Chicago's O'Hare International Airport, this airport runway was completed using steel fibres for concrete reinforcement. Synthetic Industries Concrete Company (2000) complied that the construction with the addition of 50kg/m^3 of steel fibre. This 300mm thick pavement provides fatigue resistance equivalent to a much thicker pavement, offering a better quality of shatter resistant fibre-reinforced concrete.

Figure 1.19: The construction of airport runway in Chicago O'Hare International Airport. (Source: Synthetic Industries Concrete Company, 2000)

1.11 Project Aim

This project aims to provide the improvement in mechanical properties of fibre-reinforced concrete using different types of short fibres, which will give a better understanding on the properties of fibre-reinforced concrete and its potential application in structural concrete.

1.12 Project Scope

The scope of the project was as follows:

- Review and research current usage applied to the use of short fibres in concrete.

- Construct concrete cylinders and beams by using 3 different types of short fibres with various fibre volumes, as well as casting of plain concrete cylinder and beam.

- Determine and compare the mechanical properties of fibre-reinforced concrete by conducting different tests, where the test parameters are based on workability, compression, tensile, flexural and modulus of elasticity.

- Analysis of the results and recommendation to further research area.

1.13 Dissertation Overview

This dissertation is structured to present the project activities in compressive manner.

Chapter 2 – Mechanical Behaviour of Fibre Reinforced Concrete

Chapter 2 deals with the mechanism of fibre-matrix interaction, where various models are used and compute the bonding between the fibres and cement matrix. As the bonding of fibre and the matrix plays a major role in the composite behaviour. Furthermore, this chapter also presents a review of literature relevant to the investigation and tests done for fibre reinforced concrete in general with a prominence of civil engineering application.

Chapter 3 – Mix Design

Chapter 3 provides the preliminary preparation, planning and testing of aggregates used for constructing fibre reinforced concrete specimens. Details such as selection of aggregates, fibre volume dosage rate and design of concrete mix will be discussed in this chapter.

Chapter 4 – Experimental Methodology

Chapter 4 discussed the brief outline of the experimental methodology, which done on the workability based on the fresh properties and the hardened properties of fibre reinforced concrete. The casting, vibrating and curing and stripping of fibre-reinforced concrete from mould were also discussed in this chapter.

Chapter 5 – Result and Discussion

This chapter features the results and analysis of all workability and hardened properties test, where the experimental results and investigations were outlined and discussed.

Chapter 6 – Conclusion and Recommendation

Lastly, Chapter 6 is structured to give the summary and conclusion of this project. Furthermore to provide possible recommendation of fibre reinforced concrete for further research and improvement.

CHAPTER 2

Mechanical Behaviour of Fibre Reinforced Concrete - (Review)

2.1 Introduction

The interactions between the fibre and cement matrix, as well as the structure of fibre reinforced cementitious material are the essential properties that affect the performance of a cement based fibre composite material. However, to understand these properties, the need for estimating the fibre contribution and the prediction of the composite's behaviour is necessitated. Such considerations included are:

- The matrix composition.
- The uncracked and crack condition of the matrix.
- Type, geometry and surface characteristic of the fibres.
- The length efficiency and orientation of fibres through the cement matrix.
- Critical volume dosage rate of fibres.
- Prediction of the behaviour and properties of fibre reinforced concrete.

This chapter discuss the mechanism of fibre-matrix interaction, where various models are used and compute the bonding between the fibres and cement matrix. As the bonding of fibre and the matrix plays a major role in the composite behaviour. Furthermore, this chapter also presents a review of literature relevant to the investigation and tests done for fibre reinforced concrete in general with a prominence of civil engineering application.

2.2 Properties of fibre reinforced concrete materials

The mechanical behaviour of fibre reinforced concrete materials are dependent on the structure of the composite, which is both the properties of the concrete and the properties of the fibre type used in the cement mix. Hence, composites analysing and prediction on their performance in various loading conditions, such internal structure on the composite must be characterised. The properties that considered were divided into three groups:

1. The structure of the cement matrix.
2. Shape, geometry and distribution of the fibres in the cement matrix.
3. The structure and the interface between the fibre and cement matrix.

2.3 The structure and main constituents of the cement matrix

The bulk cementitious can be categorised into two types of cement products, which depending on the aggregates contains in it. They are paste/mortar (mixture of cement, sand and water) and concrete (mixture of cement, sand, coarse aggregate and water). The cement in the matrix commonly consists of Portland cement. However, in some case, the cementing material can be manufacture by non-Portland cement materials. The fibre reinforced cement paste or mortars normally used for cladding, which usually applied in thin sheet components, such as asbestos cement. Fibres used in these applications acts as a primary reinforcement and the range of fibre volume dosage rate are from 5% to 20% by volume. For fibre reinforced concrete, the fibres act as secondary reinforcement, which the fibre volume dosage rate is much lower (less than 2% by volume), mainly used of crack control purpose.

The most commonly used cement in any concrete purpose is called normal Portland cement. Other available cement types including high early strength cement (HE), low heat cement (LH), sulphate resisting cement (SR) and shrinkage limited cement (SL). All these type of cement can be used in to develop fibre reinforced concrete. However, the hardened cement paste contains various sizes of air voids. This shows that the microstructure exhibits the volume changes in the hardened cement paste were from the effects of shrinkage and creep at the migration of water in the cement matrix. Furthermore, the hydration of the cement paste creates a highly alkaline environment with pH value range from 12 to 12.5. Such degree of alkalinity property in the cement matrix must be considered when selecting the type of fibre used. Other properties shows from the cement matrix are the products from the hydration. These properties include poor crystalline structure cause by calcium silicate and calcium hydroxide that create with the pozzolan in the cement matrix. The chemical compositions of the cement used in construction were shown in table 2.1.

Table 2.1: Chemical compositions of type of cement used in construction.
(Source: Balaguru and Shah, 1992, p.102)

	Chemical components (%)							**Remarks**
Cement type	C_3S	C_2S	C_3A	C_4AF	$CaSO_4$	CaO	MgO	
Normal Portland cement	49	25	12	8	2.9	0.8	2.4	General Purpose
High early strength (HE)	56	15	12	8	3.9	1.4	2.6	Faster strength gain
Low heat (LH)	30	46	5	13	2.9	0.3	2.7	Low heat generation during hydration
Sulphate resisting (SR)	43	36	4	12	2.7	0.4	1.6	Used on aggressive environment structures

Aggregates which apply on plain concrete are suitable for fibre reinforced concrete, as these aggregates were categories in two types: fine aggregates (sand) and coarse aggregates. Fine aggregates are required in both fibre reinforced mortar and fibre reinforced concrete. As the types of fine aggregate been manufactured and used were based on maximum grain size and size distribution. The fine aggregates available in the used of fibre reinforced mortar and concrete can be natural, crushed or manufactured sand. Coarse aggregates used can be normal-weight, lightweight or heavyweight type. The usage of heavyweight aggregate is limited in the construction conditions. As known that these aggregates contains in fibre reinforced concrete were successfully used in the field applications such as pavements and concrete slab, where the strength of these structure were improve by the used of fibres. Aggregates used in concrete have a major effect on the properties of concrete. Such properties of the aggregates that may affect the concrete are density, cleanliness, grading, particle shape, porosity and alkali reactivity of the aggregates. The details of the coarse aggregates were shown in Table 2.2.

Table 2.2: Details of coarse aggregate usually used in construction.

Coarse aggregate type	Material/source made from	Density (kg/m^3)
Normal weight	Natural gravel or crushed stone	2240
Light weight	Expanded clay or blast furnace slag	1440-1760

The quality of water is important as it can affect the setting time of fresh concrete and the strength of hardened fibre reinforced concrete. Furthermore, it causes the risk of corrosion of the fibres, especially to steel fibres. However, water is required for the hydration of cement and moulding and placing of concrete in the required shape and location. Balaguru and Shah (1992) stated that the sufficient water for the hydration requires a minimum water/cement ratio of 0.28. Water that fit for drinking is suitable for concrete used. If there is a high concentration of sodium, potassium salts or high-suspended solids contain in the water, the water cannot be used for concrete mixing. Care on the water must be taken to avoid contamination of water, such as split admixtures.

Chemical admixtures may also used to enhance the certain properties such as workability of the fresh concrete and the strength of concrete. The most common chemical admixture used is water-reducing admixture. This chemical admixture was developed to improve the workability of the concrete at a low water/cement ratio. The workability of a cement matrix normally reduced with addition of fibre. With the use of water-reducing admixtures, it is possible to maintain the workability of fibre reinforced concrete without adding extra water. In addition, extra water reduces the strength, increase shrinkage and have the tendency to develop cracks. As a result, these problems affect the durability of fibre reinforced concrete once it hardens. So it is desirable to minimum the amount of water used in the concrete mix.

There are two types of water-reducing admixtures; they are water reducing type and high range water reducing type. The water reducing type reduces the water demand about 10% to 20% less than the required while maintaining the strength and reducing the extra effort on the placement and compaction. The high range water reducing type was also known as superplasticiser, which these are stronger than the normal water reducing type, produce a very workable or fluid concrete. The usage of superplasticiser can be in these two conditions or combination of them:

1. Greatly increase the workability of the concrete.
2. Increase the strength of the concrete by reductions in the water content.

Other chemical admixtures are also used in fibre reinforced concrete. Such types of admixtures are air entraining, set accelerating and set retarding. The air entraining admixtures were introduced to improve the resistance of concrete to freeze thawing cycling for exposed structures. This admixture was required for exposed fibre reinforced concrete such as pavements or tunnel linings, as the fibre reinforced concrete as risk as the plain concrete. Air entrainment admixture overcomes the deficiencies in sand gradings, reduce the bleeding of concrete and improve the workability of the fresh concrete. However, care must be considered with air entrainment admixtures when the levels of air are high (about 8% to 10%), where it will results in loss of strength. Set accelerating admixtures used to increase the rate of strength development at early stage and reduce the initial setting time of concrete in cold weather. Furthermore, it reduces the likelihood on corrosion of fibres, especially to steel fibres. These admixtures usually used for shotcreting application as to speed up the setting process. Set retarding admixtures slow down the setting of concrete, as usually used in hot weather to maintain the workability of the concrete during the placing. Furthermore, this admixtures cause reduction in the heat of hydration of a concrete. So, set retarding admixtures generally used with the combination of water reducing admixtures.

Other than chemical admixtures, the most widely used mineral admixtures are fly ash and silica fumes. Fly ash benefits the fresh concrete by increasing workability and decreasing bleeding, as well as hardened concrete by increasing sulphate chloride resistance, reduce shrinkage and creep, enhance the permeability characteristics, increase the strength of fibre reinforced concrete and reduce the silicate reaction between the alkali/aggregate. Silica fumes improve the strength and durability of the concrete. Furthermore, it improves the bond between the fibres and the cement matrix. This admixture produces a denser matrix, resulting in better mechanical properties of fibre reinforced concrete.

2.4 Shape, geometry and distribution of the fibres in cement matrix

The largest influences on the fibre reinforced concrete however were the shape, geometry and mechanical properties of fibres and the dispersion of fibres in the cementitious matrix. The knowledge on the fibre properties is important for design purpose. James (1990) stated that the high ratio of fibre modulus of elasticity would have direct influences to the matrix modulus of elasticity where this facilitates the stress transfer from the matrix to the fibre. Fibre which is has a higher tensile strength is essential to reinforcing action. Furthermore, fibres that have large values of failure strain will tend to have high extend or prolongation in the composites. The most common types of fibres were steel fibres and polymers fibres, due to low cost and their availability. However, other types of fibres may be used in the concrete composites depending to the needs. The properties and their respective types of fibres were shown in table 2.3. Properties of cement matrix were also included in the table.

Table 2.3: Fibre types and their properties. (Source: James, 1990, p. 3)

Fibre type	Specific Gravity	Modulus of Elasticity (GPa)	Tensile Strength (GPa)	Failure Strain (%)
Steel	7.8	200.0	1.0-3.0	3.0-4.0
Glass	2.6	80.0	2.0-4.0	2.0-3.5
Asbestos	3.4	196.0	3.5	2.0-3.0
Nylon	1.1	4.0	0.9	13.0-15.0
Carbon	1.9	380.0	1.8	0.5
Polypropylene	0.9	5.0	0.5	20.0
Polyester	1.4	8.2	0.7-0.9	11.0-13.0
Polyethylene	0.9	0.1-0.4	0.7	10.0
Sisal	1.5	26.5	0.8	3.0
Kevlar	1.5	133.0	2.9	2.6
Wood fibre	1.5	71.0	0.9	-
Cotton	1.5	4.8	0.4-0.7	3.0-10.0
Acrylic	1.1	2.0	0.2-0.4	25.0-45.0
Rayon	1.5	6.8	0.4-0.6	10.0-25.0
Cement matrix	3.15	10.0-45.0	0.003	-

James (1990) stated that having a lower Poisson's ratio prevented such problems on fibre-matrix interface associated with the fibre debonding. Furthermore, Riley (1968) stated that most fibres have surface flaws, due to handling, processing and manufacturing, as these surface defects can affect the strength properties of the composite. Such presence of flaws was varies by fibre length and diameter, which acts to strength reduction of fibre reinforced concrete. Additionally, the 'tensile strength of the fibres decreases when the fibre length increases' (James, 1990).

Each type of fibre can be categorised into two groups:

- *Discrete monofilaments*, which each fibres were separated one from another (e.g. steel)
- *Bundles of filaments*, which all the fibres assemblies together, as each with a diameter of 10μm or less. Majority of man made fibres, such as inorganic fibres (e.g. glass), organic fibres (e.g. carbon, Kevlar) and natural fibres (e.g. asbestos).

The monofilaments fibres due to their uniform improvement were commonly used in structural concrete to enhance the fibre-matrix interaction through mechanical anchoring such as ductility, toughness and strength of concrete. The monofilaments fibres are rarely in an ideal cylindrical shape, but usually deformed into different configuration they desired. This configuration of fibres was shown in figure 2.1. Bundled fibres usually do not break up into separate filaments, as they maintain their bundled nature in the cement matrix. This was shown in figure 2.2.

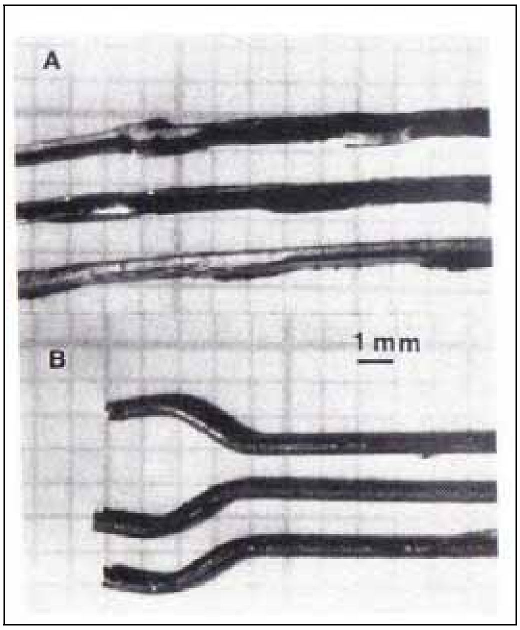
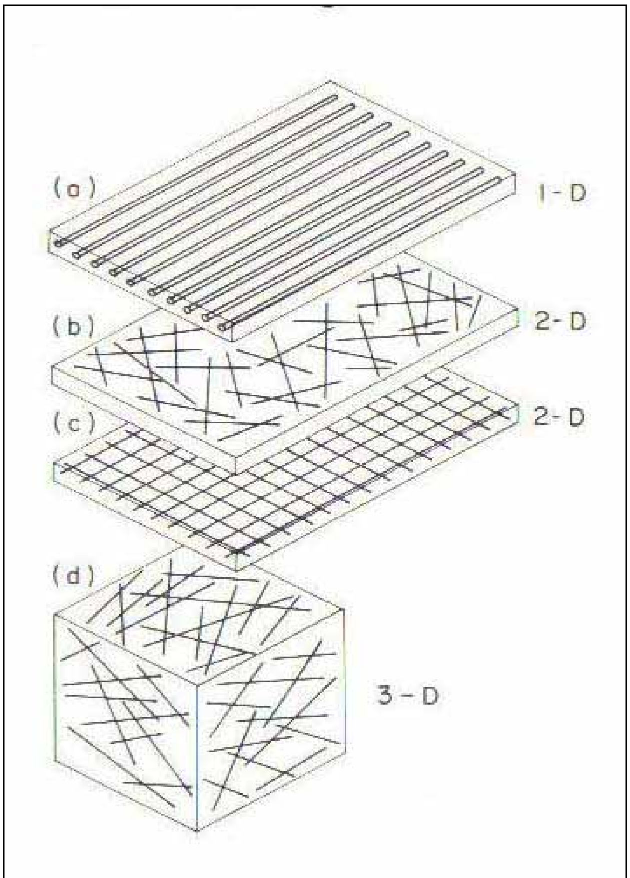


Figure 2.1: Monofilaments fibres.
(Source: Bentur and Mindness, 1990)

Figure 2.2: Bundled fibres.

The reinforcing arrays of fibres were in two different ways: *Continuous reinforcement* and *Discrete short fibres*. The continuous reinforcement was usually in the form of long fibres, which incorporated to the matrix in the methods of filament winding or layers of fibre mats. However, discrete short fibres with a length approximately 50mm or less, incorporated to the matrix in the methods of spraying and mixing. The reinforcing arrays are classified the distribution of fibres in the matrix as 1-, 2- or 3-dimensional where have large effect on the mechanical properties of fibre reinforced concrete. The classification of fibre arrangement was shown in figure 2.3.



Descriptions:

(a) 1-dimensional arrangement.

(b), (c) 2-dimensional arrangement.

(d) 3-dimensional arrangement.

(a), (c) continuous fibres.

(b), (d) short discrete fibres.

Figure 2.3: Classification of fibre arrangement in 1, 2 and 3 dimensional.
(Source: Bentur and Mindness, 1990)

2.5 Interaction between fibres and matrix

Many detailed analytical predictions and models have been developed in the interaction of fibre-matrix stress transfer and crack bridging, as well as analysing the shear stresses that develop across the fibre-matrix interface. Most of the models were done by simulate analytical solution on fibre-matrix interaction, which based on a simple pullout geometry shown in figure 2.4. These analytical models involves the shear stress and frictional stress which developed between the fibre and cement matrix, offering predictions on the efficiency of short, randomly oriented fibres in the concrete matrix. The effectiveness of fibres in the mechanical properties of the fibre reinforced concrete is influenced in two ways:

- Processes where load is transferred from the cement matrix to the fibres, and
- The bridging effect of the fibres in the concrete when the concrete cracks.

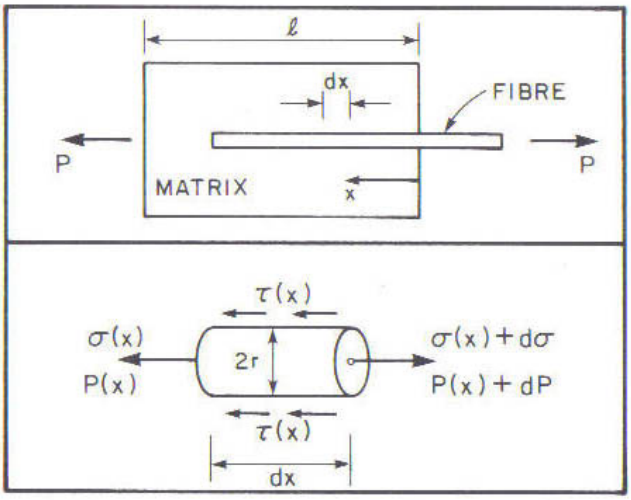


Figure 2.4: Pullout geometry to simulate the interaction between fibre and cement matrix. (Source: Bentur and Mindness, 1990)

The stress transfer effects must be considered in both pre-cracking case and post-cracking case for the brittle fibre reinforced concrete, as the processes of stress transfer are different in these two cases. Such understanding of mechanisms for the stress transfer permits the prediction of stress/strain curve on the fibre reinforced composite, the mode of fracture and a basis for developing performance on the composite with the modification of the interaction of fibre-cement matrix.

In uncracked state of the fibre-cement matrix, the major mechanism that the load transfers from the matrix to the fibre is the elastic stress. This means that the strain (longitudinal displacement) of the fibre and the matrix at the interface are almost the same. Such stress developed at the interface, which need to distribute the external load between the fibres and the matrix is shear stress. This is required in order for these two strains to remain same, where the elastic moduli of these two components are different. The elastic shear transfer was used in the prediction of limit proportionality, modulus of elasticity, elastic stress/strain behaviour and determination of the first crack stress of the fibre-matrix composite. However, the elastic shear stress distribution and deformation along the interaction of fibre and matrix was not consistent (shown in figure 2.5).

A simple fibre-matrix system containing one single fibre was shown in figure 2.5. Under unloaded stage, stresses in the fibre and the matrix were assumed to zero. The stress and deformation of the fibre and matrix was remaining same. When a load was applied, either by tension or compression, some of the load was transfer to the fibre along its surface. This mean that the stiffness of the fibre and the matrix was different as shear stress develops on the surface of the fibre. The deformation and interaction of fibre-matrix when tension and compression exerted, was also shown in figure 2.5.

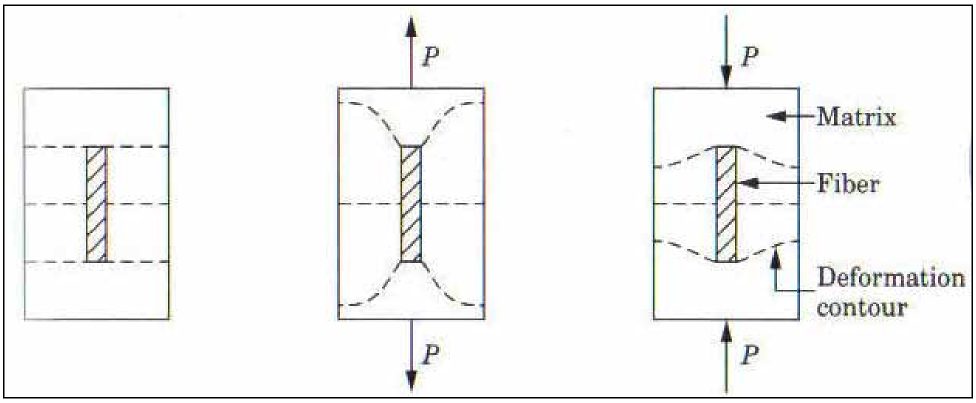


Figure 2.5: Interaction of fibre-uncracked matrix.
Left to right: unloaded, tension and compression.
(Source: Balaguru and Shah, 1992)

As the load increase, debonding around the surrounding of the interface takes place and such frictional slip occurs as process controlling stress transfer at that area. Once this situation happens, some deformation between the fibre and the matrix will develop and the frictional stress was assumed uniformly distributed at the interface of fibre and matrix. The controlling process of stress transfer was important where such properties like ultimate strength and strain can be determined, while this process is fundamental in the post-cracking case as the fibres bridge across the cracks.

In the cracked state, adhesional shear bond strength and frictional shear strength are the two major mechanisms for the stress transfer between the interaction of the fibre and matrix. The shear stress at the interface from elastic state was transfer to frictional stress and adhesional shear bond stress as the loading exceeds the fibre and matrix shear strength. When this stress exceeded, the debonding of fibre and matrix occurred, while frictional shear stress was developed on the interface at the debonded surrounding. However, the post-cracking behaviour is very difficult to predict, as the fibre orientation and the fibre length efficiency start to participate in the behaviour of the concrete. The efficiency for the fibre reinforcement will discuss in section 2.7. A further description of the stress transfer between the fibre and matrix composite was shown in figure 2.6.

Once the matrix containing with fibres cracks at a certain stage when it is loaded with tension force, the load was carried on to the fibres across the cracks and spread from one side of the matrix to the other. This interaction of fibre-matrix on cracked condition based on tension was shown in figure 2.6.

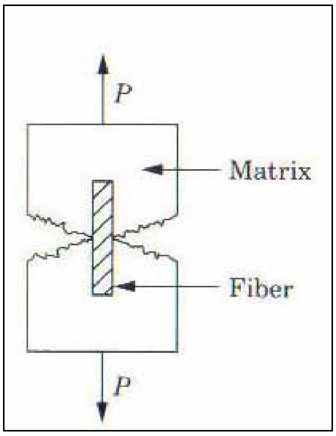


Figure 2.6: Interaction of fibre-cracked matrix.
(Source: Balaguru and Shah, 1992)

2.6 Critical fibre volume dosage

The load bearing capacity of the fibre reinforced concrete depends on the volume dosage rate applied into the concrete matrix. In this fibre cement composite, the failure strain of fibre is normally greater than the failure strain of the concrete. As to prevent the failure of fibre, the load bearing capacity of the fibre must be greater than the load applied on the concrete when the first crack appears. This was assume that the concrete does not contribute any further strength beyond the point of first crack, as the load was fully transfer to the fibre that contains in the concrete. Furthermore, the fibres are able to carry more load, result that the ultimate strength of the fibre cement composite is higher than the matrix strength itself. In this case, an equation for minimum fibre volume dosage rate, Vcr, was developed to set to equal the load bearing capacity of the fibre/cement composite and the fibre load bearing capacity.

The minimum or critical fibre volume dosage rate, Vcr, that needs to add into concrete for its loading bearing capacity or to sustain the load after the concrete occurs was given as (James, 1990, p.6):

$$Vcr > \frac{\sigma_{mu}}{\sigma_{mu} + (\sigma_{fu} - \sigma'_{fu})} \qquad (2.1)$$

where Vcr = critical/minimum fibre volume dosage rate

σ_{mu} = ultimate tensile strength of the concrete

σ_{fu} = ultimate tensile strength of the fibre

σ'_{fu} = stress on the fibres when concrete fails at its first crack

AS3600 stated that the strain of the concrete (ultimate concrete strain) at the point of first crack is 0.003. If the strain on the concrete and the fibre was assume to be same. Thus, the stress in the fibre at the point of first crack can be taken as the product of the ultimate strain of the concrete and the modulus of elasticity of fibre. The above equation 2.1 can be rearranged as:

$$Vcr > \frac{\sigma_{mu}}{\sigma_{mu} + (\sigma_{fu} - E_{st}\varepsilon_{cu})} \qquad (2.2)$$

where E_{st} = modulus of elasticity of the fibre

ε_{cu} = ultimate strain of concrete = 0.003

The equation 2.3 below taken from AS3600 was used to predict the ultimate tensile strength of the concrete, as the tensile strength of the concrete was required to obtain the minimum fibre volume dosage rate.

$$\sigma_{mu} = f'_{ct} = 0.4 \times (f'_c)^{0.5} \quad (2.3)$$

where f'_c = characteristic compressive strength of concrete

f'_{ct} = characteristic tensile strength of concrete

James (1990) stated that the minimum fibre volume dosage rate for steel, glass and polypropylene fibres in the concrete matrix is calculated approximately 0.31%, 0.40% and 0.75%. For chopped and randomly oriented fibre composites, the minimum fibre volume dosage rate is higher than the value stated, as the efficiency factor such as fibre length and orientation effects influenced the volume dosage rate. The load of the concrete at the point of first crack was enough to distribute on to the fibres when the minimum fibre volume dosage rate has been reached. It is important that equation 2.2 gives an indication of the volume of fibres required to add into the concrete, where it will increase the ductility and strength of concrete.

2.7 Efficiency of fibre reinforcement

The fibre reinforced concrete consists of distribution of short fibres in the cement matrix. Such contribution of short, inclined fibres on the mechanical properties of fibre reinforced concrete is usually less than long fibres placed parallel to the load. This means that the efficiency of the short and inclined fibres is less. However, the efficiency of the fibres in the cement matrix to enhance the mechanical properties of concrete can be judged into two ways:

- The property enhancement in the strength of the concrete, and
- The property enhancement in the toughness of the concrete.

These effects on the properties of concrete were depending on the fibre length, the orientation of fibres distributed in the concrete and the shear bond strength of the fibre/cement composite. All of these three factors were not independent as the effects on the fibre length and orientation are largely effect to the bond between the fibre and cement matrix.

In most of the engineering applications, the fibre efficiency was expressed in terms of efficiency factor, which values were from 0 to 1 (Bentur et. al, 1990). The efficiency factor was used to express the load applied on the ratio between the reinforcing effect on the short inclined fibres and the continuous fibres aligned parallel. Determination of efficiency can be obtained by empirical or analytical calculations on the factors for η_l, length efficiency and η_Θ, orientation efficiency.

2.7.1 Length efficiency

The effects of length of the fibre can be analysed by the mechanisms of stress transfer on the performance of the concrete, which explained in section 2.5. The critical length parameter, l_c, can be defined as the minimum fibre length which need to build up of a stress or load in the fibre from the frictional and shear stress transfer is equal to its failure strength (load). The definition of the critical length of fibre was shown in figure 2.4 below (Bentur et. al, 1990). In figure 2.7, curve 1 represents frictional stress transfer mechanism and curve 2 represents an elastic stress transfer mechanism. For curve 1, the fibre length is less than the critical length, where there is not sufficient embedded length to produce a stress equal to the fibre strength. If the length of the fibre exceeds l_c, the stress on most of the fibres will reach to its yield strength, as this was shown on the curve 2. The critical length of the fibre can be calculated as (James, 1990, p5):

$$l_c = \frac{df\ \sigma f}{2\tau} \qquad (2.4)$$

where df = fibre diameter

σf = ultimate strength of the fibre

τ = interfacial bond strength

Although the interfacial bond strength depends on the strength of the concrete and the bonding type of the fibre, but Balaguru and Mindness (1992) stated that it can be taken approximately 1MPa.

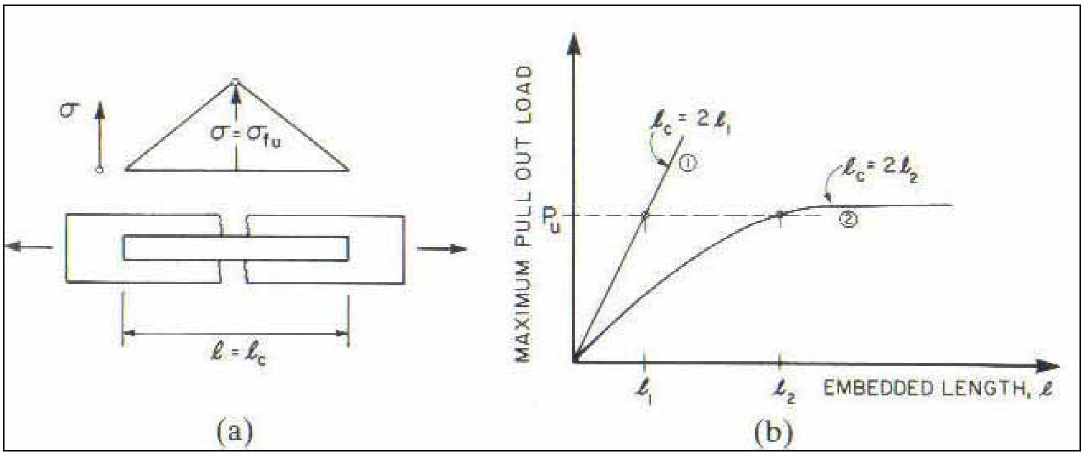


Figure 2.7: Definition of critical length: (a) Frictional stress distribution on fibres.
(b) Intersection of fibre breaking load Pu, with pullout load versus embedded length.
(Source: Bentur and Mindness, 1990)

The stress in the fibre is not constant along the entire length for discontinuous fibres. However, the stress developed linearly at the end of the fibre with a distance half of the fibre length, which shown on figure 2.7. But in most of the fibre reinforced concrete, the fibres are not placed and aligned parallel to the direction of applied stress. This shows that the fibres are not fully effective in the strengthening for fibre reinforced concrete. Furthermore, fibre placed perpendicular to the applied stress tends to have less or even no effect in the increasing strength of fibre reinforced concrete.

The equation 2.4 are the accurate method to described the required length for the fibre to transfer load, but there are another indications of efficiency factor on the load applied to the fibre, which they are the length to diameter ratio and fibre aspect ratio of the fibre. The length to diameter ratio is a simplify way to estimate the effectiveness of fibre to transfer the load. As the diameter of the fibre is larger, more loads can be transferred on to the fibres. Similar to length to diameter ratio, the fibre aspect ratio shows that more surface of the fibre commerce with the concrete matrix, the greater the load can be transferred to the fibre.

The length efficiency factors are used for the prediction of the properties of fibre reinforced concrete in pre-cracking and post-cracking state. The length efficiency factors, which take accounts of the critical fibre length on both pre-crack and post crack state, were shown by the following equation (Bentur et. al, 1990):

Pre-cracking state:

$$\eta_l = 1 - \frac{l_c}{2l}\frac{\varepsilon_{mu}}{\varepsilon_{fu}} \quad (2.5)$$

Post-cracking state:

$$\eta_l = 1 - \frac{l_c}{l} \qquad \text{for } l >> 2\,l_c \quad (2.6)$$

$$\eta_l = \frac{1}{4}\frac{l_c}{l} \qquad \text{for } l << 2\,l_c \quad (2.7)$$

where η_l = length efficiency factor,
l_c = critical fibre length (obtained from equation 2.4),
ε_{mu} = strain of the fibre (the point of first crack),
l = embedded length of fibre in the cement matrix,
ε_{fu} = ultimate strain of the fibre.

2.7.2 Fibre orientation

If all fibres were placed parallel to the direction of stress applied, the orientation efficiency is unity (James, 1990). However, the use of fibres in the concrete matrix is randomly distributed, where the orientation of the fibre was unpredictable within the concrete with either in one, two or three-dimensional arrays. In such distributions, some of the fibres were placed and axis in an angle (θ) to the load orientation or applied stress, which shown on figure 2.8. It shows that fibre at an angle can carried more load to those fibre placed parallel to the load direction by using vector analysis in the components of x, y and z. Furthermore, fibres at an inclined angle to the load direction carry more bending stress during the bridging a crack and decreases the fibre efficiency in carrying load applied to the concrete.

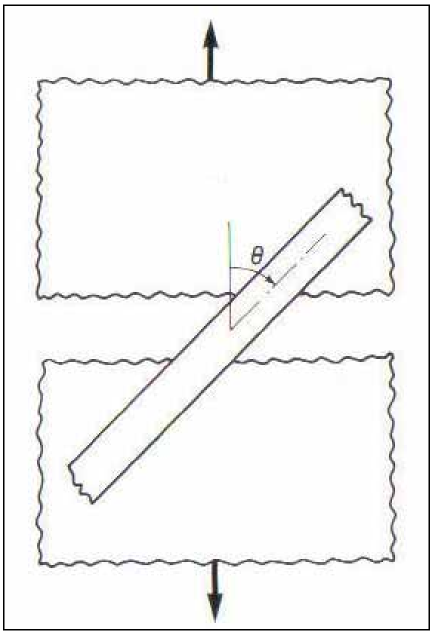


Figure 2.8: The intersection of an oriented fibre across a crack with an angle (θ).
(Source: Bentur and Mindness, 1990)

The orientation efficiency of the fibre in the concrete matrix was classified into two approaches. The first approach assumed that the fibre-reinforced composite is constrained, which the deformation of fibre is subject only in one direction of applied stress. The second approach (unconstrained) assumed that the deformation occurs in other directions of applied stress. Such example of constrained and unconstrained can be show in hardened property test like flexure strength test, where the concrete beams was subject to deformation in one direction only, and direct compressive strength test, concrete cylinders was subject to deformation in different planes. However, the vibration and compaction of the concrete rearranges the fibres, so most of the fibre reinforced concrete was assume in randomly two-dimensional orientation. The orientation efficiency factor for unconstrained and constrained with different fibre orientation was shown in table 2.4.

Table 2.4: Orientation efficiency factor for unconstrained and constrained fibre reinforced concrete. (Source: Bentur and Mindness, 1990, p81)

Fibre orientation	**η_θ, Orientation efficiency factor**	
	Unconstrained	**Constrained**
Aligned, 1-D	1	1
Random, 2-D	1/3	3/8
Random, 3-D	1/6	1/5

2.8 Prediction of the behaviour and properties of FRC

From section 2.4, the major role that fibres occur were the post-cracking state in the fibre reinforced concrete composites, where these fibres were bridge across the cracks on fibre reinforced concrete. However, the first crack on the composite will not lead to shattering failure. Eventually, this will results in the redistribution of the load between the fibres and the concrete, as discussed previously from section 2.5. As additional load was applied on the composite, more cracks were developed until the composite was separated into few numbers of segments (a to g). The separation of the composite into segments by cracks was known as 'multiple cracking'. The figure 2.9 shows the stress and the strain of failure of the fibre occurs as more of the cracks developed. The range of initial constant stress (the first crack stress, $E_c\varepsilon_{mu}$) is eventually known as the modulus of elasticity of the FRC composites (E_c). When the multiple cracks stopped and faded out, the additional load will cause the pullout of the fibres (z), as shown in figure 2.9. At this region, the slope was known as 'E_fV_f' where the aligned and continuous fibre will stretch and fail when the fibres reach their maximum load bearing capacity ($\sigma_{fu}V_f$). The final schematic description of the stress/strain curve for the FRC composite was shown in figure 2.10.

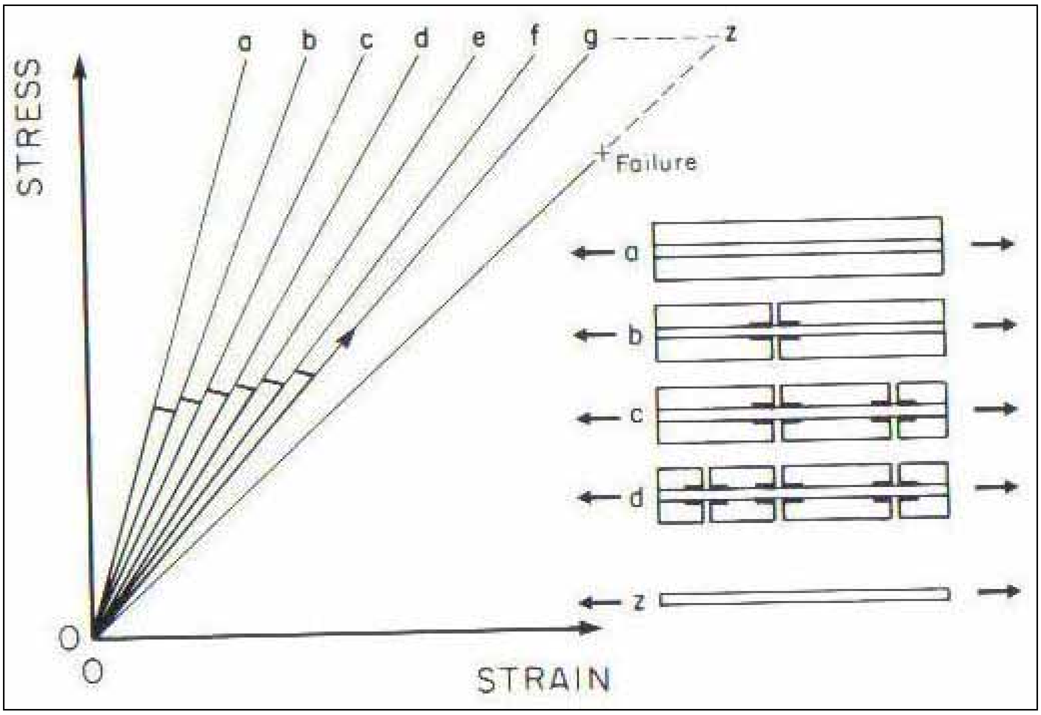


Figure 2.9: The multiple cracking process and the stages of stress/strain curve related to the multiple cracking processes. (Source: Bentur and Mindness, 1990)

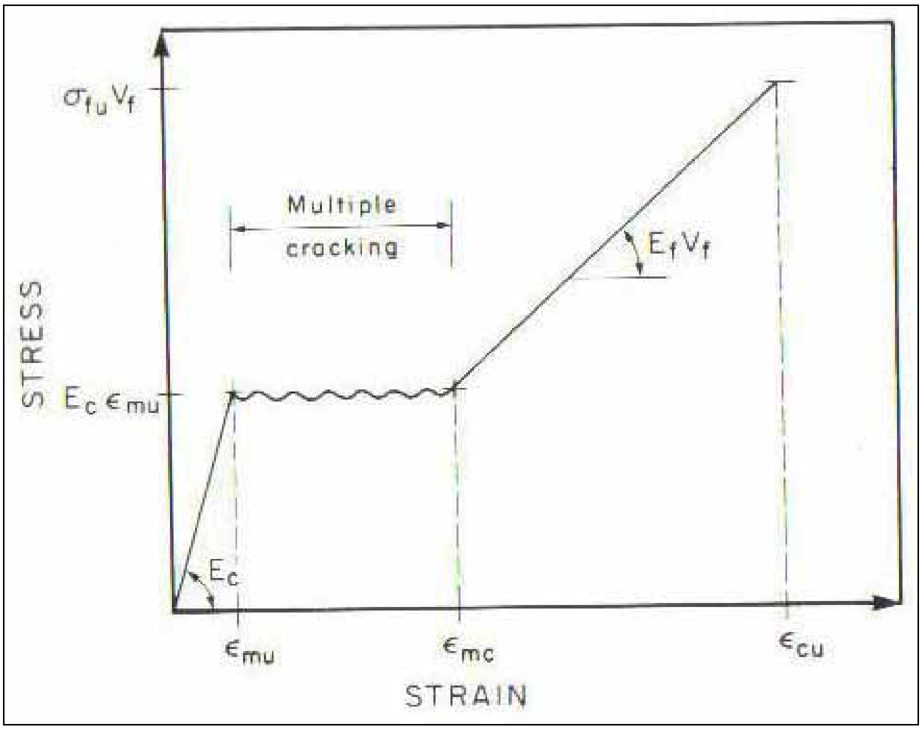


Figure 2.10: Final schematic description of stress/strain curve.
(Source: Bentur and Mindness, 1990)

Consequently from figure 2.10, the mechanical behaviour of fibre reinforced concrete can be illustrated by three stages of the tensile stress versus strain curve:

- Elastic stage.

In this stage, the load was carried by both of fibres and matrix. The stress is transfer to the fibres when the deformation in the matrix occurs, while the stress will transfer back to the matrix when the deformation stopped. This stage eventually happens up till the point of first crack, which the concrete strain arrived at a value of 0.003.

- ‘Multiple cracking’ stage.

The concrete strain has exceeded the ultimate strain of its composite, which above the stain value of 0.003, as the cracking and energy absorption takes place in this stage. When the stress continues increase between the fibres and matrix, formation of fine cracks were developed.

- Post-multiple cracking stage.

In this stage, the matrix no longer carries the load, where stress was transferred to the bridging fibres, as the pullout and stretch was occurred to fibres.

Many models and analytical predictions were used to predict the modulus of elasticity, the first crack stress and strain from the shape of the tensile stress/strain curve. In such models and predictions, attention was given to the energy involved in the failure fracture of the fibre reinforced concrete composite, as the attention comprised of composite materials approach, facture mechanics and multiple cracking.

The 'rule of mixtures' based on the composite material approach, was shown by models of the composite in figure 2.11. Bentur and Mindness (1990) stated that the 'rule of mixtures' for the properties of the composite are equal to the weight average of the properties of each individual components. The components such as modulus of elasticity and strength are valid when these two components were in the elastic stage. Hence, the 'rule of mixtures' can only applied at the pre-cracking stage of the fibre reinforced concrete composite. However, the prediction of the components by 'rule of mixtures' takes the effect of the fibre length efficiency and fibre orientation efficiency. The prediction of modulus of elasticity, E_c and first crack tensile stress of the composite, σ_{mu} were developed by Bentur and Mindness (1990), while the first crack flexural stress, σ_f was developed by Namy (2001). Through these predictions, these rules were able to apply into the concrete design and have a better utilisation and advantages of fibre reinforced concrete.

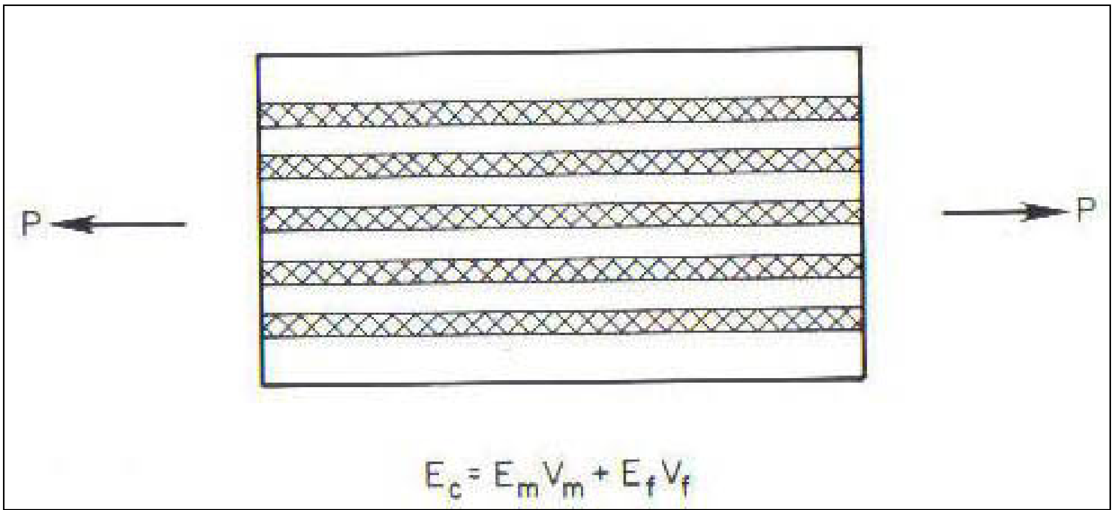


Figure 2.11: The parallel model of 'rule of mixture'.
(Source: Bentur and Mindness, 1990)

Modulus of Elasticity:

E_c = 'E_m V_m' (matrix) + 'η_l η_Θ E_f V_f' (fibre) (2.8)

where E_c = modulus of elasticity of the fibre reinforced composite

E_m = modulus of elasticity of the matrix

V_m = volume fraction of the matrix

η_l = fibre length efficiency factor

η_Θ = fibre orientation efficiency factor

E_f = modulus of elasticity of fibre

V_f = volume fraction of the fibre

First crack tensile stress:

σ_{mu} = 'σ'_{mu} V_m' (matrix) + 'η_l η_Θ σ'_f V_f' (fibre) (2.9)

where σ_{mu} = first crack tensile strength of the fibre reinforced composite

σ'_{mu} = tensile strength of the matrix at point of first crack

σ'_f = tensile strength of the fibre at point of first crack

First crack flexural stress (modulus of rupture):

σ_f = '0.843 $f_r V_m$' (matrix) + '425 V_f (l/d_f)' (fibre) (2.10)

where σ_f = first crack flexural strength of the fibre reinforced composite

f_r = stress in the matrix (modulus of rupture of the plane concrete)

l/d_f = fibre aspect ratio (ratio of length to diameter)

2.9 Previous Investigation

It has been known that fibre-reinforced concrete had been used in the early years of structural building. Between the years from 1960 to 2000, many researches have been carried out and quite a number of investigations have been performed for fibre-reinforced concrete. The use of randomly distributed fibre reinforcement can be considered to be a lucrative method of providing higher structural strengths to concrete structures, pavements and slabs. However, stresses caused by shrinkage to concrete itself historically been a problem to control because of their unpredictable and irregular occurrence.

2.9.1 Fibre effects and parameters on behaviour of fibre reinforced concrete

Fibre reinforced concrete was successfully used in variety of engineering applications, because of its satisfactory and outstanding performance in the industry and construction field. However, most of the engineers and researches do not fully understand how and why the fibres perform so successfully. So, to recognize the usage of fibres in concrete, in these last four decades, most of the researches were done on mechanical behaviour of fibre reinforced concrete and the fibres itself.

The fibre reinforced concrete in many applications is subject primarily to bending rather than axial loading, as this indicates the performance in flexure is remark as important. Johnston (1982) conducted tests by determining the factors influencing the flexural strength measurement on fibre reinforced concrete. He proposed that such parameters that affect the performance of the flexural were the loading mode in flexure, specimen's size, shape and span, fibre length, dimension of fibres and fibre volume fraction.

After 10 years, Johnston and Skarendahl (1992) conducted similar tests by examined 117 beams (150 x 100 x 750mm) under a three point loading with 5 types of steel fibres in amounts from 30 to 100 kg/m^3. They concluded that the first-crack strength was primarily depends on the matrix characteristic, while secondary depends on fibre parameters such as type, size and amount. At the post cracking state, the toughness of concrete depends on the fibre type, amount and fibre aspect ratio. However, Tat et. al (1998) reported that the higher fibre concentration and longer fibres lead to better performance while bond stress between the matrix and fibres is a major influence to the flexural strength of fibre reinforced concrete.

Few years later, Banthia and Dubey (2000) used Residual strength test method (RSTM) to measure the flexural toughness of fibre-reinforced concrete in terms of its post peak residual strength, which was investigated. This method has the ability to identify the influence of different fibre characteristics such as type, length, configuration, volume fraction, geometry, and the modulus of elasticity. The results were based on two set of testing. Test of set 1 was clearly stated that fibrillated polypropylene fibres provide a better toughness than monofilament polypropylene fibres. Test of set 2 noted that hooked-end steel fibres verified a better toughening strength than crimped steel fibres in fibre-reinforced concrete.

Some investigations were based on the effect of fibre content and damaging load on fibre reinforced concrete stiffness. Patton and Whittaker (1983) investigated on the steel fibre content for dependence of modulus of elasticity and correlation changes on damage due to load. They found out that there is approximately 3.3 percent increase over the modulus of elasticity of plain concrete for every 1.0% increase in fibre content by volume. Furthermore, the investigation shows that degeneration of stiffness starts at approximately 30 percent of the ultimate load before the first visible crack appears.

Rossi et. al (1987) analysed that the effects of steel fibres on the cracking at both local level (behaviour of steel fibres) and global level (behaviour of the fibre/cement composite) were dependant to each other. The results of this analysis showed that 1.0% volume content of steel fibres could replace approximately 0.15% of flexural steel reinforcement.

Although fibre's material was same, there is some difference in behaviour of fibre reinforced concrete if the geometry of the fibres were different. Barros and Figueiras (1999) used two types of steel fibres in the fibre reinforced concrete for their research. These two fibres had similar tensile strength, however, their aspect ratio was different.

Two tests parameters were conducted: uniaxial compression tests and three-point loading flexural tests. They noted that increase in fibre percentage has significant improved the load carrying capacity and decreased the crack opening and crack spacing. Furthermore, the higher fibre aspect ratio of steel fibres exhibited an ultimate load twice the ultimate load of the other steel fibres.

There are relation between the flexural strength with the compressive strength and tensile strength of the concrete. Dwarakanath and Nagaraj (1991) predicted flexural strength of steel fibre concrete by these parameters such as direct tensile strength, split cylinder strength and cube strength. This experimental test results and evaluates determination of direct tensile strength for the composite, was reflects by the combined effects of fibre volume and ratio of length and diameter parameters in steel fibre reinforced concrete.

Investigation shows that the toughness on the fibre reinforced concrete increase rapidly than plain concrete. Trottier et. al (1994) investigated the toughness of fibre reinforced concrete by using different geometry of steel fibres, which include hooked end, crimped circular, crimped crescent and twin cone end steel fibres. One fibre volume fraction (40kg/m^3) was used throughout the research. The test included compressive strength test and flexural strength test, with measurement of deformation of specimen as the load applied. They found out that fibres brought significant improvement in the toughness and energy absorption capacity of concrete. Based on four fibre geometries, fibres with deformations only at end appear more effective than those with deformations over the entire length.

In the same year, Chen et. al (1994) conducted the similar test concept (toughness) to Trottier et. al (1994), by determining the first crack and flexural toughness of steel fibre reinforced with different dimensions. The research used hooked end steel fibre with 30mm long and 0.5mm in diameter. Four groups of dimension were used. First group in width:depth:span ratio of 1:1:3 as the spans beams in 150, 225, 300 and 450mm. The second group used same cross-sectional area (75 x 75mm) with four different spans length of 150, 225, 300 and 450mm. The third group maintained the same width (75mm) and spans (300mm), while the depths are 75, 100 and 150mm. The last group keeps the same depth (100mm) and span (300mm) with five different widths (25, 50, 75, 100 and 150mm). They investigated that all toughness parameter were affected by the width of the beam, even the depth and span were unchanged. Furthermore, the specimen size not only influenced toughness, but also stress and deflection at first crack and ultimate flexural strength.

2.9.2 Different types of fibres in fibre reinforced concrete

In section 2.9.1 discussed some usage of the different geometries of fibres in concrete, but the researches in this section were based on the behaviour and mechanical properties of other types of fibres (material) that used in the concrete. In year 1992, Nanni et. al (1992) conducted an investigation on the use of newly developed aramid fibres for the reinforcement of Portland cement based concrete. The aramid fibres were produced in chopping a bundle made of epoxy-impregnated braided into aramid filaments. In this investigation, the behaviour of reinforced concrete of aramid fibres was compared to steel fibres and polypropylene fibres. Beams of 100 x 100 x 350mm were tested under four point flexural loading. It was found that aramid fibres acts similar to steel fibres and is superior to polypropylene fibres. They concluded aramid fibres were lack in corrosion problems while having a higher performance than polypropylene fibres. However, the use of aramid fibres was not very economical.

Wang et. al (2000) applied recycled fibres as reinforcement in concrete. The recycled fibres included tire cords/wires, carpet fibres, feather fibres, steel shavings, wood fibres from paper waste and high-density polyethylene. The research conducted was based on shrinkage, durability and toughness characteristics test. The results of each test showed that recycled fibre effective improving the toughness, shrinkage and durability characteristics of concrete. Wang et. al (2000) recommended and encouraged the use of low cost waste fibre for reinforcement could lead to improved infrastructure with better durability and reliability, as these applications are reduced the solid waste from industrials and consumers.

Perry (2003) used large and small synthetic fibres to reinforced external pavements. He reported that the abrasion of pavement surface had exposed the steel fibres used, creating health and safety hazards. Two tests were done. First test method conducted in a smaller area of external concrete pavement and compares the evaluation of steel fibres (hooked end, 60mm long) at a dosage of 30kg/m^3 and synthetic fibres (50mm long) at a dosage of 6.9kg/m^3. Flexural strength and flexural toughness test were conducted as second test method under three-point loading. The results of flexural test demonstrated the steel fibre reinforced concrete has an equivalent flexural strength ratio of 53%, while synthetic fibre reinforced concrete was recorded as 78%. On the external concrete pavement, steel fibre has an equivalent flexural strength ratio of 20% and synthetic fibre was 41%. Perry (2003) concluded that synthetic fibre could provide concrete with the same level and even more of post-crack performance to steel fibres.

2.9.3 Usage of fibres with conventional steel reinforcement

The usage of fibres also can be applied with the conventional steel reinforcement. Swamy and Sa'ad (1981) had done an investigation on deformation and ultimate strength of flexural in the reinforced concrete beams under four point loading with the usage of steel fibres, where consists of 15 beams (dimensions of 130 x 203 x 2500mm) with same steel reinforcement (2Y-10 top bar and 2Y- 12 bottom bar) and variables of fibres volume fraction (0%, 0.5% and 1.0%). As they concluded that fibres were effective in resisting deformation at all stage of loading from first crack to the failure and increasing the flexural stiffness at the failure stage of the beams. Furthermore, this investigation shows that role of steel fibres prevents any advancing cracks and increase the ductility and post-cracking stiffness of the beam right till to failure.

Similar crack behaviour investigation, which based on combination of 5 full scale reinforced concrete beams (350 x 200 x 3600mm) with steel fibres (volume fraction of 0.38% and 0.56%) were done by Vandewalle (2000). In this investigation, the experimental results and theoretical prediction on the crack widths was compared. Vandewalle (2000) also concluded that the addition of steel fibres decreases the cracking spacing and crack width. However, he reported that prediction of crack widths stated Eurocode 2 on the combination of fibres with conventional steel reinforcement overestimates measured values. Thus, he established a simple empirical expression on the final cracking spacing of steel fibre reinforced member.

Sener et. al (2002) calibrated the size effect of the 18 concrete beams under four-point loading. The all beams thickness are uniform at 40mm and length of 800mm, but the height of the beams were varies at 40mm, 80mm and 160mm. Steel fibres was used with the same length/height ratio of 5 and volume fraction of 0.6%, and 1.2%, while the cement/aggregate/sand ratio of 1:2:4. It results that as height of the beam increased, the ultimate flexural strength increased. Also, the bending failure in fibre reinforced concrete exhibits a greater size effect and higher brittleness than concrete containing no fibres.

Most of the investigation of steel fibre reinforced concrete was based on flexural strength and crack width. In Singapore, Tan et. al (1993) conducted some investigation on the shear behaviour of steel fibre reinforced concrete. Six simply supported I-beams were tested under two-point loading with hooked steel fibres of 30mm long and 0.5mm diameter, as the fibre volume fraction increased every 0.25% from 0% to 1.0%. This investigation confirms that the shear strength increased as much as 70 percent by adding small quantities of steel fibres (1.0%) into ordinary reinforced concrete. Furthermore, the steel strains on steel fibre reinforced concrete are less than reinforced concrete at diagonal cracking of the web.

2.9.4 Other applications and test methods on fibre reinforced concrete

Most of the investigations on fibre reinforced concrete were based on the basic mechanical properties and behaviour. However, the investigations and researches of fibre reinforced concrete can be extent into more further to other types of structure and applications.

Sanjuan et. al (1998) investigated the effect of polypropylene fibre reinforced mortars on steel reinforcement corrosion induced by carbonation. In this investigation, crack control by fibres in plastic state mortars and crack evolution with time has been studied. Furthermore, the influence of crack width on steel bar corrosion induced by carbonation has been monitored. The objective of the investigation is to assess the effectiveness of polypropylene fibre as secondary reinforcement to delay the initiation of reinforcement corrosion induced by carbonation. The fresh polypropylene fibre reinforced mortar was cast into a cylindrical ring and a solid cube of 70mm (containing 5 steel reinforcement bars) was located inside the mortar. The tests were conducted on day 800 after the casting of mortar. They found that polypropylene fibres were able to control crack width in inadequately cured mortars and the addition of fibres reduced the corrosion rate on the steel reinforcement. However, there is no relationship between the corrosion rate and crack width.

Gupta et. al (2000) conducted impact test on fibre reinforced wet mix. It is known that shotcrete is often subjected to impact and dynamic load. Ten different commercially available shotcrete fibres were investigated in wet-mix shotcrete. The ten fibres included: four deformed steel fibres, two straight polypropylene fibres, one crimped polypropylene fibre, two straight carbon microfibres and one deformed polyvinyl alcohol (PVA) fibre. The mixes were shot onto wooden forms (600 x 500 x 100mm) with fibre volume fraction of 10 to 60 kg/m^3, and eight beams (100 x 100 x 350mm) were sawn after demoulded and cured for 28 days. Four beams were tested under impact loading with 60kg hammer dropped from a height of 0.45m, producing potential energy of 266J and velocity of 2.97m/s. The remaining four beams were tested under static loading with a circular 100mm diameter-loading cylinder and all four edges were supported on a rigid support frame. The results showed that fibre reinforcement in wet-mix shotcrete improves the fracture energy absorption and toughness under impact loading. However, the improvement does not happen under static conditions. Furthermore, Gupta et. al (2000) concluded that wet-mix shotcrete is highly sensitive to the rate at which load is applied.

After one year, Luo et. al (2001) studied and conducted test on the mechanical properties and resistance against impact on steel fibre reinforced high-performance concrete. Five different geometry of fibres included steel-sheet-cut fibres and steel-ingot-milled fibres with four fibre volume fractions (4%, 6%, 8% and 10%) were applied into the mix. Beams (100 x 100 x 400mm) and cubes (100 x 100 x 100mm) were casted. The projectiles used in the test were armor penetration projectiles with diameter of 37mm and weight of 0.9kg. The projectile was launched at a high velocity between 365m/s and 378m/s. The investigation shows that increase in fibre percentage improves the mechanical properties, where peak compressive strength and flexural strength reached 140MPa and 80MPa, respectively increased 61% and 774% compared to specimens containing no fibres. In impact test, the specimens containing no fibres were smashed up and steel fibre reinforced high-performance concrete were kept intact with some radial cracks developed in front faces and minor cracks in side faces.

Fatigue is an important consideration with regard to the durability of thin concrete repairs. Repeated loading and restrained shrinkage can cause damages and debonding of repair layer. Mailhot et. al (2001) studies the flexural fatigue behaviour of steel fibre reinforced concrete by conducting series of flexural fatigue test (under three point-loading) with fibre volume dosage of $40kg/m^3$. Three different types of steel fibres (hooked, nail-anchored and crimped) and two-water/cement ratio (0.35 and 0.45) were applied into the mix design. Six slabs (125 x 425 x 500mm) were made with each batch. The slabs were carried out at three different repeated stress levels: 85, 75 and 70% of the first crack strength. Number of cycles was performed up till 7 days with maximum limit cycles of $3x10^5$. When the maximum number of cycles reached, the specimen was recorded either uncracked or cracked. The survival life under repeated loadings was defined as the difference the number of cycles at failure and number if cycles at onset of the first crack. The investigation found that the specimen with fibres exceeded 80% of the overall life cycle while survival life of specimen containing no fibre were extremely short, as the parameters affecting were water/cement ratio and type of fibres used.

In last two decades, steel fibres have replaced the conventional reinforcement in industrial ground floors. Research and practice have shown that steel fibre reinforcement is more efficient and economic for industrial floors. Experimental comparative done by Chen (2004), investigated the strength of 15 steel fibre reinforced and plain concrete ground slabs. The slabs were 2 x 2 x 0.12m, reinforced with hooked end steel fibres and mill cut steel fibres. All slabs were centrally loaded hydraulic and electric pump through 100 x 100mm steel plate. He concluded that the load bearing capacity of could be effectively increased when the slabs are reinforced with steel fibres. In addition, he also indicates that the energy absorption capacity of steel fibre reinforced concrete specimens can be used in assessing the effect on the load carrying capacity of steel fibre reinforced concrete ground slabs.

2.9.5 Guides and practice of fibre reinforced concrete

As discuss above, the fibre reinforced concrete were so successfully used in the construction and industry. However, there is no standard and a few generally accepted the practice on fibre reinforced concrete. Thus, this obstructs the understanding of the fibres and probably tends to discourage potential users from specifying on fibres. To overcome this problem, guide and good practice must be provide and apply to fibre reinforced concrete.

A report done and prepared by ACI Committee (1984), giving guidance of specifying, mixing, placing and finishing of fibre reinforced concrete. The guide emphasised the difference between conventional concrete and fibre reinforced concrete and method to deal with them. The report warned that calcium chloride should not add with fibre reinforced concrete, but recommended the usage of water reducing and air-entraining admixtures with fibres. Furthermore, ACI Committee suggested that fibres must stored in care to prevent deterioration. The fibres have a tendency to protrude sharp corners, as this can be hazardous to personnel. The guide suggested the sharp corners should be chamfered.

The guide by ACI Committee (1984) suggested that two methods of adding the fibres into the fresh concrete mix, as these methods provide good dispersion of fibres and prevent clumping (balling). The first method point out that fibre can be added last into the fresh concrete mix, while second method indicates that fibres was mix with the aggregates before the addition of water into the mixer. All fibres must be clumping free (as rain of individual fibres) during the addition of fibres into the mixer. Furthermore, the guide stated that the causes of balling may occurs if the fibre volume fraction is more than 2% or even 1% with high aspect ratio and the other reason was clumping of fibres before and during adding the fibres. On the placing consideration, the fibre tends to be stiff and not workable. The guide recommended that vibration must be done to improve the placability. Again, the guide specified that water/cement ratio must be range from 0.40 to 0.50.

Furthermore, the guide by ACI Committee (1984) specified the transporting and placing of fibre reinforced concrete with conventional equipment must be properly designed, maintained and clean. If pumping were used on transporting fibre reinforced concrete, some important point were suggested by the guide, where the pump must be capable to handle the volume and pressure required, the diameter of pump hose must at least 150mm wide and avoid flexible hose if possible. However, the guide did not suggest any special attention on the finishing, but it indicated that overwork on the surface could result in bringing excessive fines and bleeding. The guide also indicates that curing of fibre reinforced concrete were same as conventional concrete.

On the practice of purpose, Dunstan et. al (1986) recommends that key to good practice dealing with fibre reinforced concrete and fibres are emphasis on the manufacture, design and construction, as all materials used for engineering or building purpose, quality and design are interdependent. Fail in performing adequately in practice will results customer dissatisfaction, inadequately quality control and potential of defects appear on structures.

CHAPTER 3

Mix Design

3.1 Introduction

To effectively research the improvement in the mechanical properties of the fibre reinforced concrete, preliminary planning, procedures and methods must be wisely chosen. The criteria to assess mechanical properties are based on the activities to plan and preparation, which carried out by before the testing of the fresh and hardened properties of Fibre reinforced concrete. These activities are:

- Aggregate Testing (Particle density and water absorption).
- Sieve analysis (aggregate grading).
- Fibre volume dosage rate.
- Mix design.
- Preparation of test specimens.
- Concrete mixing.

Experimentation is an activity required by the majority of the engineering researches, where it comprises all preparation and plan of action to be taken and being situated into operation afterwards. This chapter 3 describes preliminary design and planning such as experimentation of the coarse and fine aggregates, selection of fibres with fibre volume dosage rate, target strength of concrete specimens, mix design and number of mix batches and concrete specimens required to meet the scope of this project.

3.2 Particle Densities and Water Absorption of Coarse and Fine Aggregates

Particle density is one of the parameter used to classify the properties of an aggregate. It is not influence by the particle shape or a measure of the quality, but particle density reflects the character of the constituents of the aggregate, which is related to strength. Particle density of an aggregate is affected by several factors, including the amount of moisture present, the amount of compactive effort used in filling the container and the geological properties of an aggregate. In another words, it involves the determination of the geometric space occupied within the space of a solid material including any interior voids, cracks or pores (Neville, 1997).

For any concrete mix design, aggregate density is mainly used in proportioning the weight, volume and space of aggregate particles in the concrete. According to Australian Standard HB64 (2002), substituting a different density of aggregate into the concrete mix will influence the yield, unit mass of the concrete and the quantity of aggregate required for a given volume of concrete. Characteristic of aggregate density effects plastic and hardened properties of the concrete into some extent. Coarse aggregates usually have a higher bulk density, because 'the pores space is less and fewer voids to be filled than finer aggregates' (Neville, 1997). As AS1465 stated, the particle density of aggregates varies a minimum of 2300kg/m^3 to maximum of 3000kg/m^3. The determination of particle density is carried out accordance with AS1141.5 and AS1141.6.1. Particle density test is carried out as particular reason to acquire the volume and weight of the coarse and fine aggregates required for the concrete mix in this project.

Water absorption is known as the amount of moisture absorbed into the aggregate. According to Civil Material Introductory Book (2003), water absorption reflects the porosity, which related as the strength, durability and shrinkability of a particular aggregate. All aggregates contain with pores are available to filled with water moisture (HB64, 2002), where surface moisture presented on the aggregate, are capable to give them a damp or wet appearance. The capacities of absorption are based on two conditions: saturated surface dry and oven dried conditions. The water absorption is an important parameter, where it can affect the amount of water to achieve a given water/cement ratio used in a concrete mix. Furthermore, the water in the concrete mix has a direct influence to the setting time and compressive strength of concrete (Australian Standard HB64, 2002). The surface moisture contents of sands are significant to these effects.

The absorption of an aggregate is a useful guide to the permeability of the concrete. This helps to obtain the correct amount of water required in the concrete mix. Thus, in preparing a particular mix design for a particular aggregate, the moisture content of aggregate in saturated surface-dry condition must be first determined. If the moisture content of the concrete is not to required target, additional water will need to add to avoid the loss of workability, as the aggregate absorbs moisture. If the free moisture on the surface of aggregate exceeds the required target, less water should be added (HB64, 2002). This shows that the water adjustment in concrete may require. The absorption of aggregate is expressed in terms of percentage. As stated in AS1465, the maximum water absorption of a particular aggregate may reach to 5 percent. The water absorption of the aggregate may determine accordance to AS1141.5 and AS1141.6.1.

The particle density and water absorption parameter of the aggregate will determine and based on natural aggregate with grain size of 20mm, 10mm, 7mm and natural. Since natural aggregate (coarse aggregate) and natural sand (fine aggregate) are used, the experimental methodology and apparatus used will be significant different. The sampling and testing of fine aggregate are accordance to AS1141.5 and coarse aggregate may determine accordance to AS1141.6.1. All testing on particle density and water absorption was carried out at the engineering laboratory of University of Southern Queensland.

3.2.1 Apparatus for particle density and water absorption of coarse aggregate

The following apparatus and equipment complying with the relevant provisions of AS1141.2 were used.

Wire Basket – of appropriate mesh and size with wire hanger for connecting it from the balance.

Water Bath – of appropriate size and shape to contain the basket and supply a cover at least 50mm water above the top of the immersed basket.

Balance – of sufficient capacity with a limit of performance not more or less than 0.5g, and have a type that will allow a basket containing the sample to be attach to it and weight in water.

Container – suitable size may require use for containing the sample.

Oven – thermostatically controlled to carry out at a temperature of 105°c to 110°c.

Dishes – of suitable sizes.

3.2.2 Test Procedure (coarse aggregate)

The test procedure was in accordance to AS1141.6.1. The procedure of the testing was as follow.

1. The aggregate was immersed in the water at room temperature, for a period of at least 24 hour, with a cover of a least 20mm of water above the aggregate. The entrapped air bubbles were removed by stirring the aggregate infrequently. Ensure the aggregates must remain completely immersed during the soaking period.

2. The aggregate was transferred into a basket after the immersion. The basket was immediately immersed into the water containing in a bath below the balance. The entrapped air bubbles were removed by rattling the basket. The basket was attached to the basket hanger at the balance. The basket and the aggregates were weight and recorded (C).
3. The basket and aggregates was removed from the water and water from the basket and aggregates was drained. All of the aggregate was transfer to a container. The material was not allowed to dry out if the particle density and water absorption was required to be determined.
4. The empty basket was returned to the water and air bubbles were removed by rattling the basket. The weight of the basket in the water was weighed to the nearest 1g (D). The empty basket in the water may need to set to zero prior to the procedure in step 2.
5. The aggregates were surface dried by rolling the aggregates in the absorbent cloth. Large aggregates were dried as individually and smaller aggregates were rolled on a dry cloth. All the aggregates were spread one stone deep over a dry cloth to allow it to surface dry. The drying was continued until the surface of the aggregates' appearance change to a lighter colour, but the surfaces of particle shall still remain damp. The mass of the surface dry material was weighted and recorded (B).
6. The aggregates were dried in the oven at 105°c to 110°c for 24 hours. After 24 hours, the aggregates were weighted and recorded (A).

The calculations of bulk density (Dry), bulk density (SSD) and water absorption of coarse aggregates had shown as below.

$$\text{Bulk Density (Dry)} = \frac{A \times 1000}{B - (C - D)} \qquad (3.1)$$

$$\text{Bulk Density (SSD)} = \frac{B \times 1000}{B - (C - D)} \qquad (3.2)$$

$$\text{Water Absorption} = \frac{(B - A) \times 100\%}{A} \qquad (3.3)$$

Figure 3.1: Natural aggregate size from 20mm, 10mm and 7mm.

Figure 3.2: Immersion of aggregates and wire basket
into the water for weight record.

Figure 3.3: Placement of aggregates in the container before oven drying.

Figure 3.4: The thermostatically controlled oven.

3.2.3 Apparatus for particle density and water absorption of fine aggregate

The following apparatus and equipment complying with the relevant provisions of AS1141.2 were used.

Balance – of sufficient capacity with a limit of performance not more or less than 5g.

Metal Mould – mould made of 0.8mm sheet metal in the shape of a truncated cone 73mm high with a large diameter of 90mm decreasing to a small diameter of 38mm.

Tamping rod – a mass of 350g with a flat circular tamping face of 25mm diameter.

Oven – thermostatically controlled to carry out at a temperature of 105°c to 110°c.

Container – suitable size may require use for containing the sample.

Glass container and Flask – a glass container of 400mm diameter and 600mm high to contain water and a flask of 500mL volumetric flask with a lid or stopper

Heater or dryer – able of provide a gentle flow of warm air.

Dishes – of suitable sizes.

3.2.4 Test Procedure (fine aggregate)

The test procedure was in accordance to AS1141.5. The procedure of the testing was as follow.

1. The sand was immersed in the water at room temperature for a period of not less than 24 hours. The air entrapped in the sand was gentle disturbed with a rod until no air bubbles rise to the surface.

2. The water was drained off and the sand was spread on a flat impervious surface. The sand was surface dried by exposing to a gently moving current of warm air and stirring it regularly to achieve uniform drying. Additionally, the sand can be dried under the sun for few hours.
3. The sand was filled into the conical mould by loosely placing part of the sand in it. Surface of the sand was tamped with tamping tool 25 times, allowing the tamping tool to fall 10mm above the surface of the sand. Conical mould was lifted vertically. If free moisture was presented, the cone of sand will retain its shape. If the sand was too dry, additional water was required to add.
4. The drying with constant stirring was continued. Step 3 was repeated until the cone of sand slump on the removal of the mould. This state the sand has reached to a saturated dry condition. The mass of the saturated dry sand was weighted and recorded (B).
5. The sand was placed into a volumetric flask and water was filled. The mass of the sand, flask and water was weighted and recorded (C).
6. The sand was removed from the flask without losing any particle into a container.
7. The flask was filled with water. The mass of flask and water was weighted and recorded (D).
8. The sand on the dish was dried in the oven at 105°c to 110°c for 24 hours. After 24 hours, the sand was weighted and recorded (A).

The calculations of bulk density (Dry), bulk density (SSD) and water absorption of fine aggregates shown as below.

$$\text{Bulk Density (Dry)} = \frac{A \times 1000}{D - (C - B)} \quad (3.4)$$

$$\text{Bulk Density (SSD)} = \frac{B \times 1000}{D - (C - B)} \quad (3.5)$$

$$\text{Water Absorption} = \frac{(B - A) \times 100\%}{A} \quad (3.6)$$

Figure 3.5: Bottle of distilled water and flask containing the natural sand and water.

Figure 3.6: Drying of wet natural sand was under the sun.

Figure 3.7: Placement of natural sand in the container before oven drying.

Figure 3.8: Natural sand in the container after oven drying.

3.2.5 Results of particle density and water absorption

Table 3.1: Weight of coarse aggregate during and after the testing.

Size of aggregate	20mm	10mm	7mm
A, Oven dry mass of aggregate (g)	1445.0	1482.7	1724.5
B, Mass of SSD aggregate in air (g)	1463.2	1502.6	1751.8
C, Mass of SSD aggregate and wire basket in water (g)	1097.6	1125.7	1292.0
D, Mass of wire basket in water (g)	135.0	135.0	133.3

Calculations for coarse aggregate:

$$\text{Bulk Density (Dry)} = \frac{A \times 1000}{B - (C - D)}$$

$$\text{Bulk Density (SSD)} = \frac{B \times 1000}{B - (C - D)}$$

$$\text{Water Absorption} = \frac{(B - A) \times 100\%}{A}$$

20mm natural aggregate:

$$\text{Bulk Density (Dry)} = \frac{1445.0 \times 1000}{1463.2 - (1097.6 - 135)} = 2886.54 \text{ kg/m}^3$$

$$\text{Bulk Density (SSD)} = \frac{1463.2 \times 1000}{1463.2 - (1097.6 - 135)} = 2922.89 \text{ kg/m}^3$$

$$\text{Water Absorption} = \frac{(1463.2 - 1445.0) \times 100\%}{1445.0} = 1.26\%$$

10mm natural aggregate:

$$\text{Bulk Density (Dry)} = \frac{1482.7 \times 1000}{1502.6 - (1125.7 - 135)} = 2896.46 \text{ kg/m}^3$$

$$\text{Bulk Density (SSD)} = \frac{1502.6 \times 1000}{1502.6 - (1125.7 - 135)} = 2935.34 \text{ kg/m}^3$$

$$\text{Water Absorption} = \frac{(1502.6 - 1482.7) \times 100\%}{1482.7} = 1.34\%$$

7mm natural aggregate:

Bulk Density (Dry) $= \dfrac{1724.5 \times 1000}{1751.8 - (1292.0 - 133.3)}$ = 2907.60 kg/m^3

Bulk Density (SSD) $= \dfrac{1751.8 \times 1000}{1751.8 - (1292.0 - 133.3)}$ = 2953.63 kg/m^3

Water Absorption $= \dfrac{(1751.8 - 1724.5) \times 100\%}{1724.5}$ = 1.58%

Table 3.2: Weight of fine aggregate during and after the testing.

Description	Mass (g)
A, Dry mass of fine aggregate	71.0
B, Mass of SSD fine aggregate	77.3
C, Mass of flask, fine aggregate and water	384.4
D, Mass of flask and water	340.4

Calculations for fine aggregate:

Bulk Density (Dry) $= \dfrac{A \times 1000}{D - (C - B)}$

Bulk Density (SSD) $= \dfrac{B \times 1000}{D - (C - B)}$

Water Absorption $= \dfrac{(B - A) \times 100\%}{A}$

Fine aggregate (sand):

Bulk Density (Dry) $= \dfrac{71.0 \times 1000}{340.4 - (384.4 - 77.3)}$ = 2132.13 kg/m^3

Bulk Density (SSD) $= \dfrac{77.3 \times 1000}{340.4 - (384.4 - 77.3)}$ = 2321.32 kg/m^3

Water Absorption $= \dfrac{(77.3 - 71.0) \times 100\%}{71.0}$ = 4.01%

Table 3.3: Result of particle density and water absorption of all aggregates.

Aggregate types and sizes	Particle Density (Dry) (kg/m^3)	Particle Density (SSD) (kg/m^3)	Water Absorption (%)
20mm natural aggregate	2886.54	2922.89	1.26
10mm natural aggregate	2896.46	2935.34	1.34
7mm natural aggregate	2907.60	2953.63	1.58
Fine aggregate (sand)	2132.13	2321.32	4.01

The results obtained from the test shown that the particle density of coarse aggregate has a higher value than fine aggregate (sand). The average dry particle density of coarse aggregate is about 2900 kg/m^3 and fine aggregate (sand) has an average value of 2200 kg/m^3. As a contrast, there are a difference of 25% in particle density between coarse aggregate and fine aggregate (sand). This shows that the fine aggregate (sand) are lighter.

The water absorption capacities of fine aggregate (sand) are higher than coarse aggregate. The water absorption of fine aggregate is about 4% while the average water absorption of coarse aggregate is 1.4%. This shows that fine aggregate has 2.6 times more water absorption capacity than coarse aggregate.

The results of particle density of each aggregate are valuable data in mix design. These data are used in the proportioning the require weight, volume and space of aggregates particles in the concrete. For water absorption results, the water/cement ratio may need to be adjusted in order to meet the required concrete strength. As a result, more water is needed to add during the concrete mixing to overcome the loss of water and obtaining the required workability.

3.3 Sieve Analysis

Sieve analysis is used to acquire the proportion and amount of different size of aggregates required for any concrete mixing. It is a method for the determination of particle size distribution in coarse and fine aggregates, by shaking the aggregates through a series of sieves, placing with the smallest sieve size at the bottom. These sieves have round or square openings and usually constructed of wire mesh. It is a laborious procedure to sieve by hand. In any laboratory, if a considerable amount of sieving need to be done, it is advisable to avoid wastage of labour. To ensure better accuracy, using of sieving machine with several stack of sieve can be used. Mechanical sieve machines were shown in figure 3.9 and 3.10.

Before any sieve analysis was performed, all the aggregates must be air-dried. Neville (1997) noted that main reasons are to avoid lumps of fine particles being classified as large particles and prevent clogging of finer sieves. A ‘sample reduction’ is a method, where aggregates were taken out of the stockpile by riffling. The aggregates were discharge on the riffle and separate out into two boxes at the bottom of the chute of the riffler. One of the boxes will be discharge and the next box was riffled again until the mass of the aggregates meets the specification. The actual sieve operation can be performed after the correct mass of aggregate reached. The determination of sieve analysis was carried out accordance with AS1141.11. The sieve operation was separate into two operations: coarse and fine aggregate sieve operations. In coarse aggregate sieve operation, sieve sizes were range from 19mm to 1.18mm. Fine aggregate are too fine to pass through these larger size sieve. The sieve sizes for fine aggregate were range from 2.36mm to 75µm.

The results of sieve analysis can be represent graphically in a manner way. Grading charts are very significantly used. It is possible to see whether the grading of the sample match up to the specification, or is too coarse or too fine, or ‘deficient in a particular size’ (Neville, 1997). According to Australian Standard HB64, the aggregate grading influences the water demand and workability of the concrete. Furthermore, it can affect the strength and other properties of the hardened concrete.

3.3.1 Apparatus

The following apparatus and equipment complying with the relevant provisions of AS1141.11 were used.

Balance – of sufficient capacity with a limit of performance not more or less than 5g for coarse aggregate and not more or less than 0.5g for fine aggregates.

Sieves and riffler – sieves and riffler complied with AS1152 and AS1141.2.

Brush – a soft and fine brush to clean the sieve.

Mechanical sieve machine – in a good and usable condition.

3.3.2 Test procedure

The test procedure was in accordance to AS1141.11. The procedure of the testing was as follow.

1. The required sieves were placed in the order of decreasing size of opening from top to bottom. The aggregates were placed in the top sieve.
2. A mechanical sieve process begins with at least 5 minutes for a rate of 100 stokes per minute.
3. All the aggregates are not allowed to forced through by hand pressure, but for sieve size more than 19mm and greater, hand placing of aggregate was allowed.
4. Sieving was completed when no more than 1 percent of mass stayed on any individual sieve after 1 minute of continuous hand sieving.
5. The mass of aggregates on each sieve was determined and no aggregates were allowed to retain on each sieve.

Figure 3.9: Mechanical sieve machine for fine aggregates.

Figure 3.10: Mechanical sieve machine for coarse aggregate.

3.3.3 Results of sieve analysis

Table 3.4: Percentage passing for all aggregate.

	Percentage Passing (%)			
Sieve Size	20mm aggregate	10mm aggregate	7mm aggregate	Fine aggregate (sand)
19.00mm	100.00	100.00	-	-
13.20mm	44.48	100.00	-	-
9.50mm	8.09	90.51	100.00	-
4.75mm	0.53	1.21	18.06	-
2.36mm	0.37	0.34	2.09	98.86
1.18mm	0.35	0.31	0.75	96.93
600µm	-	-	-	90.45
300µm	-	-	-	58.54
150µm	-	-	-	19.28
75µm	-	-	-	4.72
Pan	0.13	0.21	0.57	2.80

From the results shown at the table, it was found that fine aggregate (sand) has more dusts left in the in sieve than coarse aggregate. This gave an indication that the fine aggregate (sand) has more water absorption capacity. Majority of the coarse aggregates passed through the 19mm sieve size, but not more than 1% of coarse aggregate may pass through 1.18µm sieve. Average of 60% of coarse aggregate passed through the sieve size of 9.5mm and 4.75mm. For fine aggregate (sand), majority of the sand particles (40%) passed through the sieve size of 300µm. The results of sieve analysis are an important parameter to determine the proportion amount of aggregate used in concrete mix design. The full details and results of sieve analysis were shown in Appendix B.

3.4 Strength

According to Construction Engineering Study Guide (2001), concrete must be able withstand the environment in which it is constructed over the expected useful life of the structure. Therefore it must be designed to have the required durability. These durability (compressive strength) requirements are taken out from AS3600 (Appendix B), which are known as characteristic strength of the concrete. Once the characteristic strength required for task of concrete is known, target strength of the concrete can be calculated. Target strength of concrete is a value for at least 95 percent of the concrete strength must be average when tested. This means that the target strength must be somewhat higher than the characteristic strength. A standard deviation was included in calculation as measure of quality control for the average strength. Characteristic of a concrete was aimed around 32 MPa where it is a reasonable strength to be used for structural concrete. The target strength of concrete was calculated from the following equation 3.7.

$$T = C + 1.65S \qquad (3.7)$$

Where T = Target strength of the concrete

C = Characteristic strength of the concrete

S = Standard deviation (a measure of quality control)

Characteristic strength of the concrete (C) = 32MPa

Standard deviation (S) for 32MPa = 5.34 (Appendix B)

$$\begin{aligned} T &= C + 1.65S \\ &= 32 + 1.65 \times 5.34 \\ &= 40.81\text{MPa} \approx 40\text{MPa} \end{aligned}$$

Target strength of the concrete is one of the parameter used to determine the required water/cement ratio and aggregate/cement ratio. Generally, lower ratio of the water to cement tends to produce a higher strength of the concrete. As for aggregate/cement ratio, the maximum aggregate size influences the strength of concrete. Of course, this neglected the effects of weak aggregates.

3.5 Fibre types, fibre volume dosage rate and mix batches

The fibres used in the concrete mix were fibre that was available from the market, construction and industry. Fibre types selected consist of Masterfibre™ wave cut steel fibre, Novamesh™ high performance polypropylene (HPP) fibre and INFORCE™ Fibremesh (Appendix B). The figure 3.11 below shows the types of fibre used in concrete mix.

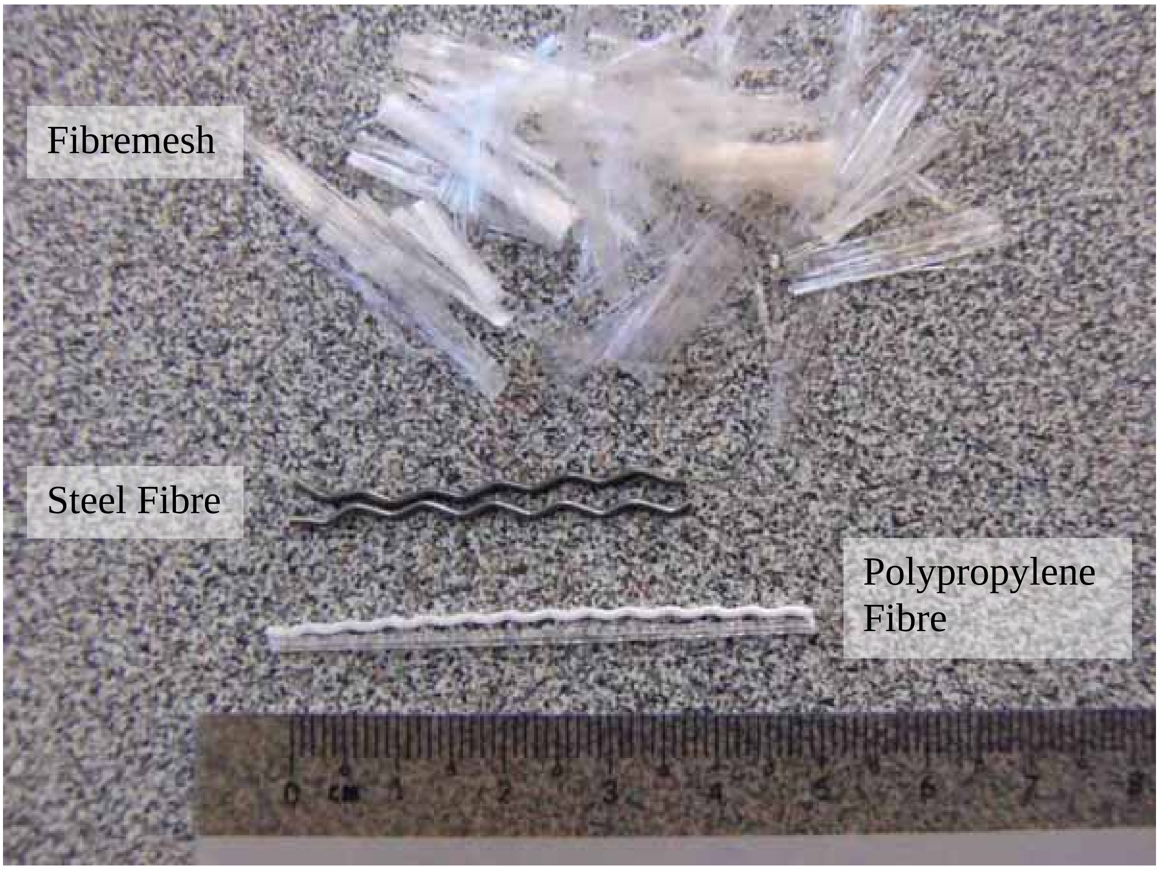


Figure 3.11: Types of fibre used in the concrete mix.

The minimum dosage rate (critical volume fraction) of fibres was obtained from the equation 2.1, which shown below.

$$V_{cr} > \frac{\sigma_{mu}}{\sigma_{mu} + (\sigma_{fu} - \sigma'_{fu})}$$

The tensile strength of concrete was obtained from the equation 2.3, which shown below.

$f'_{ct} = 0.4 \times (f'_c)^{0.5}$

$= 0.4 \times (40)^{0.5}$

$= 2.53$ MPa

So, the tensile strength of concrete (σ_{mu}) is 2.53MPa.

The yield strength of the steel fibre (σ_{fu}) is 500MPa.

If the strain of the concrete and steel fibre was assumed to be same at the point of first crack, then the stress in the steel fibre can be calculated as follows:

Strain of concrete at first crack (ε_{cu}) = 0.003

Modulus of elasticity for steel fibre (E_{st}) = 200MPa

Stress of the steel fibre (σ'_{fu}) = $\varepsilon_{cu} \times E_{st}$

$= 0.003 \times 200$

$= 0.6$MPa

Therefore, the critical volume fraction of steel fibres is shown below.

$$V_{cr} > \frac{2.53}{2.53 + (500 - 0.6)}$$

$V_{cr} > 0.005 \approx 0.5\%$

The minimum amount of steel fibres should add in the concrete mix was therefore 0.5% by volume. The value obtained corresponds to the minimum volume percentage used in the past research. Although the yield strength of polypropylene fibre and Fibremesh are different from steel fibres, still 0.5% volume dosage rate was taken for all types of fibre. Some comparison on fresh and hardened properties of concrete can be made (among the same volume dosage rate with different types of fibres). To alter and further the effects of fibre reinforced concrete, fibre dosage rates of 0.5%, 1.0% and 1.5% by volume were selected. As a result, a total of 10 mix batches were required for this project, which involves 3 different types of fibres with 3 different fibre volume dosage rates and a control mix batch. From current information, when fibre volume dosage is around 2%, the concrete is not workable and too stiff.

3.6 Mix Design

Mix design is known as the selection of mix ingredients and their proportions required in a concrete mix. As last subchapter shows that a total of 10 mixes were required. In this case, some calculations and knowledge of the proper proportioning of concrete mixes will be desirable. There are several methods of mix design used throughout the world. Eventually, all of these methods follow the same procedure and produce similar results. The most common mix design used in Australia is termed 'British Method' (Ryan et. al, 1992). The mix design involves that amount cement, fine aggregate, and coarse aggregate be available and the relation between water/cement ratio and target strength must be known. When these data were available and ready, calculation of mix design can be carried out.

3.6.1 Water/cement ratio

Most of the concrete specification requires that mixing water shall be potable, which 'is fit for drinking or it is clean and free from impurities harmful to concrete' (Australian Standard HB64, 2002). The strength of a concrete depends upon to the water/cement ratio. To obtain a higher strength of concrete, it is known that the water/cement ratio must be low. The water/cement ratio can be determined from the strength and water/cement curve specified in Australian Standard HB64 (Appendix B). The target strength of the concrete must be known. From the graph, a horizontal line was drawn from the strength axis and intersected the two curves of age. The two corresponding values of water/cement ratio were read off at the intersection. The water/cement ratio to meet strength requirement was done by averaging the two values of water/cement ratio.

Target strength at day 28 = 40 MPa (obtained from section 3.3)

Water/cement ratio $= \dfrac{0.51 + 0.61}{2}$ (obtained from Appendix B)

$= 0.56$

Usually the aggregates obtained from site or stockpiles are saturated wet and water is excess if added to the concrete mix. Therefore, water/cement ratio must make it clear if free water is considered. To correct the ratio of water and cement, the water absorption of each aggregate must be known, where this can be done by the water absorption test according with AS1141.5 and AS1141.6.1.

3.6.2 Proportioning of aggregates

Various grading of aggregates with different materials may affect the quality of concrete made. Grading of aggregates has a major influence on workability and consistence of a mix and even more on the strength of a concrete. The proportioning of aggregate may involve two or more type of aggregate blend with different size of aggregates. The target grading chosen depends on the largest nominal aggregate size used in concrete mix. The ratio of the aggregates which is necessary to meet the requirements of target grading is known as aggregate proportioning, where it can achieve by these two methods: graphical method and algebraic method.

When two or more aggregates were necessary to blended, this can be performed by graphical method using Rothfuch's method, which is most common method in Australia. The method involves the percentage passing of each size for each aggregate size done by sieve analysis. The target grading used in British method was taken from grading curve 3 for a nominal aggregate size of 20mm (Appendix B). Once the percentage passing for each aggregate was known, the ratio of each aggregate required can be obtained (Appendix B). The proportion of each aggregate can be done, by plotting a graph showing all the percentage passing for each aggregate. The results of proportioning of aggregate were shown in table 3.5.

Table 3.5: Proportion of aggregate calculated.

Aggregate size	Percentage
20mm natural aggregate	36%
10mm natural aggregate	7%
7mm natural aggregate	21%
Fine aggregate (sand)	36%

3.6.3 Aggregate/Cement Ratio

The other main influence on the 'workability of a concrete is aggregate/cement ratio, apart from water/cement ratio' (Australian Standard HB64, 2002). Some requirements using the British Method must be known to acquire aggregate/cement ratio. These requirements of obtaining aggregate/cement ratio was dedicated by water/cement ratio which meet the strength requirements of a concrete, the target grading chosen, the required slump selected, the shape of aggregates and the nominal size of aggregate used. If the aggregate in the grading has two or more blend of aggregate size, the largest nominal size aggregate was used in the determining of aggregate/cement ratio. The ratio applies to all remaining of different size of aggregate. When all these requirements have been determined, an appropriate aggregate/cement ratio can be selected from the tables.

The selection of aggregate/cement ratio is based on British method. The largest nominal size of aggregate used in this project is 20mm natural aggregate. Therefore, the aggregate/cement ratio table must base on 20mm aggregate. The shape of the aggregate was irregular shape gravel and a high slump value was required. Referring to section 3.5.1 and 3.5.2, the water/cement ratio was 0.56 and the target-grading curve 3 was selected. Once the required data was determined, the aggregate/cement ratio can be read off from the table (Appendix B).

Water/cement ratio	= 0.56	(Obtained from section 3.5.1)
Grading curve selected	= 3	(Obtained from section 3.5.2)
Required slump selected	= High	
Shape of the gravel	= Irregular shape gravel	
Aggregate/cement ratio	= 4.90	(Obtained from Appendix B)

3.6.4 Calculation of mix quantities

With the available data, such as particle density, water absorption, proportion of aggregates, water/cement ratio, aggregate/cement ratio and target strength, the calculation of mix quantities can be obtained. Calculation of mix quantities is based on one bag of cement of weight 40kg. At the beginning of the calculations, the mix proportion was calculated with the mass of 40kg per bag of cement. The volume of each mix ingredients (show as Litre) was determined by dividing the weight of each mix ingredients with density. The table 3.6 below shows the initial data for calculation of mix quantities. Table 3.7 below shows the density of all mix ingredients. The densities of fibres were obtained from table 2.3. The proportion of aggregates was shown in table 3.6.

Table 3.6: Initial data for calculation of mix quantities.

Target Strength (MPa)	40
Water/cement ratio	0.56
Aggregate/cement ratio	4.9
Weight of bag of cement (kg)	40

Table 3.7: Density of all mix ingredients.

Cement	3.15
Water	1.00
20mm natural aggregate	2.89
10mm natural aggregate	2.90
7mm natural aggregate	2.91
Fine aggregate (sand)	2.13
Steel fibre	7.80
Polypropylene fibre	0.90
Fibremesh	0.91

Table 3.8: Weight of mix ingredients (based on 40kg per bag of cement).

Mix ingredients	**Ratio**		**Weight (kg)**
Cement	40 x 1	=	40.00
Water	40 x 0.56	=	22.40
20mm natural aggregate	40 x 4.9 x 0.36	=	70.56
10mm natural aggregate	40 x 4.9 x 0.07	=	17.64
7mm natural aggregate	40 x 4.9 x 0.21	=	33.32
Fine aggregate (sand)	40 x 4.9 x 0.36	=	74.48
		Total =	258.40

Table 3.9: Volume of mix ingredients (based on 40kg per bag of cement).

Mix ingredients	**Ratio**		**Volume (L)**
Cement	40.00 / 3.15	=	12.70
Water	22.40 / 1.00	=	22.40
20mm natural aggregate	70.56 / 2.89	=	24.42
10mm natural aggregate	17.64 / 2.90	=	6.08
7mm natural aggregate	33.32 / 2.91	=	11.45
Fine aggregate (sand)	74.48 / 2.13	=	34.98
		Total =	112.03

A target volume of concrete must be known. The mixer available in University of Southern Queensland is a capacity of 60-litre pan mixer. Using this target volume, a ratio was done to scale the weight of the mixing ingredients, as this provides the required weight of the mix proportion of all ingredients.

The scale of the required mix batch = 60 / 112.03
= 0.54

Table 3.10: Weight of mix ingredients (based on the actual mix proportion).

Mix ingredients	**Ratio**		**Weight (kg)**
Cement	40.00 x 0.54	=	21.44
Water	22.40 x 0.54	=	12.01
20mm natural aggregate	70.56 x 0.54	=	37.82
10mm natural aggregate	17.64 x 0.54	=	9.46
7mm natural aggregate	33.32 x 0.54	=	17.86
Fine aggregate (sand)	74.48 x 0.54	=	39.92
		Total =	138.51

All aggregates contain with pores are available to filled with water moisture. Therefore, an adjustment of correct amount of water was required in the concrete mix. To correct this, less water is required in the mix calculations. Table 3.11 shows the amount of excessive water of each aggregate. Table 3.12 shows the final adjustment of mix calculations.

Table 3.11: Calculation of excess water of each aggregate.

	Weight (kg)	Water absorption (%)	Additional free water (kg)
20mm natural aggregate	37.82	1.26	0.47
10mm natural aggregate	9.46	1.34	0.11
7mm natural aggregate	17.86	1.58	0.28
Fine aggregate (sand)	39.92	4.01	1.54
		Total =	2.39

Table 3.12: Final adjustments of each mix ingredients.

Mix ingredients		Weight (kg)
Cement	21.44 =	21.44
Water	12.01 - 2.39 =	9.62
20mm natural aggregate	37.82 + 0.47 =	38.29
10mm natural aggregate	9.46 + 0.11 =	9.56
7mm natural aggregate	17.86 + 0.28 =	18.14
Fine aggregate (sand)	39.92 + 1.54 =	41.46
	Total =	138.51

Although the quantities of mix ingredients were obtained, the quantities of short fibres cannot be neglected. The volume of the fibre is done by ratio the total volume of the concrete mix to the percentage of the fibres. From section 3.5.4, the target volume of concrete mix batches was 60 litres. As to know the total volume of each type of fibres, the volume of each fibre was summed. Using the total volume of each type of fibres, the weight of each type of fibres can be obtained by specific gravity. Specific gravity of each type of fibres can be obtained from table 3.7. The table 3.13 shows the total weight required for each type of fibres. The total weight of each type of fibres was separated into the required weight by the categories of 0.5%, 1.0% and 1.5% volume dosage rate.

Total volume percentage required by each type of fibres

= 0.5% +1.0% +1.5%

= 3.0%

Total volume of each type of fibres required

= 60L x 3.0%

= 1.8L

Table 3.13: Total weight required for each type of fibres.

Fibre types	Ratio	Weight (kg)
Steel fibre (kg)	7.80 x 1.8L =	14.040
Polypropylene fibre (kg)	0.90 x 1.8L =	1.620
Fibremesh (kg)	0.91 x 1.8L =	1.638

Table 3.14: Final mix design proportions.

Mix ingredients	Weight (kg)
Cement	21.44
Water	9.62
20mm natural aggregate	38.29
10mm natural aggregate	9.56
7mm natural aggregate	18.14
Fine aggregate (sand)	41.46

Table 3.15: Quantities of fibre required for all mix batches.

	Fibre volume dosage rate			
Fibre types	0.50%	1.00%	1.50%	Total
Steel fibre (kg)	2.340	4.680	7.020	14.040
Polypropylene fibre (kg)	0.270	0.540	0.810	1.620
Fibremesh (kg)	0.273	0.546	0.819	1.638

The table 3.14 shown was the mix design chosen for each batch. Table 3.15 shown was the quantities of fibre required for all mix batches (not including the control mix batch). These tables are the mix ingredients needed for the total 10 concrete mix batches, which required by the scope of this project.

The water was reduced approximately 2kg for the water absorption from the actual mix design, where else the weights of the aggregates were increase to meet the target strength requirement. This shows that has a weight deduction of 19% of the water and increased of 2.3% of the weight of all aggregates to the mix design. The addition of sand required the most because of the higher water absorption capacity in this fine aggregate.

Although the volumes of the fibres were same, the weights of each type of fibres were significantly different. The main parameter influences the required quantities of fibres was the specify gravity. From the table, the weight of steel fibres is approximately 8.6 times heavier than polypropylene fibre and Fibremesh. As polypropylene fibre and Fibremesh are the same type of material (different geometry and shape), the specify gravity of these two fibres are similar. As no doubt, the weight of polypropylene fibre and Fibremesh are almost equivalent.

3.7 Mixing of concrete batches

The process of mixing influences the quality of the concrete in hardened sate. The mix material is required to be in uniform distributed and consistency in the concrete mix in order ‘to reduce the ‘weak spots’ within concrete specimens’ (Ryan et. al, 1997). Furthermore, the strength of the bond between particles and full coating of cement binder to the aggregate and fibre will be increased encouraged by proper mixing.

The mixing of concrete batches was carried out, with a small drum mixer or small electrical pan mixer. The mixer used for this project is a 60-litre pan mixer, as shown in Figure 3.12. To encourage a uniform distribution of fibres throughout of the concrete, fibres were added to the concrete mix by slowly and evenly after the water, aggregates and cement have been fully mixed. This prevents the congregation of the fibres on the paddle, which leads to balling of fibres. All mixing was performed at the engineering laboratory of University of Southern Queensland. The same mix sequence was undertaken for all the mix batches throughout of the project to ensure uniformity.

Once the concrete mixing was finished, the fresh concrete was ready for workability test and casting of concrete specimens into the concrete moulds to meet the scope of this project. The workability test, casting of concrete specimens and hardened properties test will be discussed in chapter 4.

3.7.1 Apparatus

Mixer – a useable horizontal pan mixer, which have a capacity to conduct sixty litres of concrete mix.

Buckets – buckets capable to store mix materials before the mixing.

Wheel barrier – a capable to contain the fresh concrete for workability test and placement of fresh concrete into moulds.

3.7.2 Mixing process

1. All material was weight according to mix design and prepared.
2. The surface of the mixer was moistened with a damp cloth before the mixing begins.
3. The aggregates were added into the mixer and mix thoroughly till the aggregates are evenly.
4. Cement was added into the mixer and mix until the mix was uniformed.
5. Water was added into the mixer slowly after the cement was placed.
6. The concrete was mixed around 3 minutes.
7. Fibres were added into the mixer uniformly by hand and the concrete was mixed continually mix around 3 minutes.
8. The mixer was switched off after step 7 achieved.
9. The concrete in the mixer was poured into the wheel barrier and the fully mix concrete was ready for workability test.

Figure 3.12: 60 litres pan mixer.

Figure 3.13: The preparation of the mixing material in the buckets.

Figure 3.14: Additional of fibres after the main materials were mixed.

CHAPTER 4

Experimental Methodology

4.1 Workability Test

The term workability is hard to define precisely, and Newman (1965) has proposed that it can define in three separate properties:

1. *Compatibility*. This means the ‘ease and ability of the concrete can be compacted and the air voids are removed’ (Murdock et. al, 1968).
2. *Mobility*. It terms as the ‘ease and ability of the concrete can pour into the moulds, around the steel and be remoulded’ (Murdock et. al, 1968).
3. *Stability*. It is the ability of the ‘concrete to stay a stable coherent homogenous mass while handling and vibration’ (Murdock et. al, 1968) without the constituents segregating.

There is another term for workability. Cement Association of Canada (2004) stated that workability could be defined as the ease of placing, consolidating, and finishing fresh mixed concrete and the degree to which it resists segregation. Concrete must be workable but the constituents should not be separate during transport and handling.

The fresh concrete has difference in the consistency and variations in the uniformity. So, there are few factors that influence the workability of concrete. The influences were:

(1) Method and duration of transportation of concrete.
(2) Quantity and characteristics of all constituents.
(3) Consistency of concrete, such as slump.
(4) Grading, shape, and surface texture of fine and coarse aggregates.
(5) Percentage of entrained air available in the fresh concrete mix.
(6) Water content in the aggregates.
(7) Air temperatures around the concrete.

Troxell (1968) described that consistency is a practical consideration in securing a workable concrete. It is taken to denote the fluidity and wetness as indicated by the slump or corresponding tests.

Properties such as 'consistency, segregation, mobility, bleeding, and finishability are related with workability' (Cement Association of Canada, 2004). Slump test is a used to measure consistency of a concrete, which have a close indication to workability. A low slump concrete has a stiff consistency. Concrete will be difficult to place when the consistency is too dry and harsh. It may results that large aggregate particles may separate from the concrete mix. However, more workable mix does not necessarily means a more fluid mix. Segregation and honeycombing can occurred if the mix is too fluid.

Indications show that concrete mix with addition of fibres may have a stiff effect. Therefore, slump test is not recommended as the only test used to measure the workability of fibre-reinforced concrete. Furthermore, Murdock (1968) reported that the compacting factor test does not provide an accurate measure to compact, dry and harsh mixes when compacting factors ratio is less than 0.80. To prevail over the dry and harsh mix, VEBE consistometer test is more reliable. VEBE consistometer has its own limitation. Some human error can be occurred during the measurement of time when the fresh concrete was vibrated and collapse in the apparatus of VEBE.

This section 4.1 describes the outline of the experimental methodology and workability test of fresh properties of fibre-reinforced concrete. The discussion of casting, vibrating and curing of fibre-reinforced concrete and stripping of concrete mould was followed, once the workability test was done. After the casting of concrete, the background and methodology of hardened properties test will discuss in section 4.3. The measurement of consistency of a concrete mix in this project was according with AS1012.3, and these methods include:

- Slump test.
- Compacting factor test.
- VEBE Consistometer test.

4.1.1 Slump test

Slump test is the most common test to evaluate the workability of a fresh concrete in worldwide. Although the slump test 'does not measure the workability of concrete' (Neville, 1997), it is useful to 'obtain the difference in the consistency of fresh concrete' (Murdock et. al, 1968) and detecting 'variations in the uniformity of concrete mix' (Neville, 1997). Variation in slump indicates that some changes occurred in the batching system or mixing system. The water content in concrete is the most obvious cause, as other factors such as aggregate grading and particle shape may varies the slump. The apparatus of slump test is simple, portable and suitable for laboratory and on-site testing. After the concrete was fully mixed, the fresh concrete was undertaken for use in the slump test. The test procedure was carried out accordance with AS1012.3.1. The apparatus of slump test was shown in figure 4.1.

The slump test has its limitation. Australia Standard HB64 (2002) noted that this test does not well for concretes with either very high or very low workabilities. A very workable concrete will lose their shape by flowing and collapse, and very low workability concrete will not collapse at all. Murdock (1968) sated that the temperature affects the slump, where the temperature of the mixed concrete increased; the slump decreases.

4.1.1.1 Apparatus

The following apparatus and equipment complying with the relevant provisions of AS1012.3.1 were used.

Mould – a hollow frustum of a cone made from galvanized steel sheet with thickness of between 1.5mm to 2.0mm. The bottom and the top of the mould are open and at a right angles to the axis of the cone. The mould includes with suitable footpieces and outer handles for holding in place during filling and internal surface must be smooth. Dimensions of the mould were as below.
Bottom diameter = 200 ± 5 mm.
Top diameter = 100 ± 5 mm.
Vertical height = 300 ± 5 mm.

Rod – a metal rod of 16 ± 1 mm in diameter, approximately 600 mm long and having at least one end tapered for a distance of approximately 25 mm (a spherical shape) having diameter of 10mm.

Scoop – a appropriate size which large enough to accommodate the maximum size of aggregate in the concrete mix.

Base plate – a smooth, rigid and non-absorbent material of base metal plate with minimum 3.0mm thickness.

Ruler – appropriate steel ruler is required for measurement of slump height.

4.1.1.2 Test Procedure

The test procedure was in accordance to AS1012.3.1. The procedure of the testing was as follow.

1. The internal surface of the mould was cleaned (free from set concrete) and moistened with a damp cloth immediately before beginning of each test.
2. The mould placed on a smooth and horizontal surface. The mould was hold firmly by standing on the footpieces against the base plate while the mould is being filled.
3. The mould was filled in three layers approximately one-third of the height of the mould. Each layer was rodded 25 strokes with the metal rod. The strokes were distributed in a uniform manner over the cross-section of the mould.
4. After the top layer has been rodded, the excessive concrete on the top of the mould strike off or rolled off with the rodded. A firm downward pressure was maintained at all times until the mould is removed.
5. The mould was immediately removed from the concrete by raising its lowly and carefully in a vertical direction, allowing the concrete to collapse.
6. The mould was placed upside down next to the collapse concrete. The steel rod was positioned on to the mould.

7. The slump immediately was measured by determining the difference between the height of the mould (300 mm) and the average height of the top surface of the concrete.

Figure 4.1: Apparatus for slump test.

Figure 4.2: A typical slump test.

4.1.2 Compacting Factor test

Compacting factor test is not widely used in Australia, but it presents a better measurement of workability of concrete than slump test and this test suited better for controlling the production of low slump concrete mixes. The degree of compaction, called 'compacting factor', is measured by the density ratio (Neville, 1997), which can described as the ratio of the density actually obtained in the test to the density of the same concrete when it is fully compacted.

This method describes the degree of fresh concrete will compact by itself when allowing it to fall freely by force of gravity and without any other external compactive influence. The apparatus consists of two conical hoppers, each in the shape of a frustum of a cone, and one cylinder, with three of them are above one another in an axis. Hinged doors are located at the bottom of hoppers. The apparatus of compacting factor test was shown in figure 4.3. Table 4.1 shows the dimension of the apparatus. The degree of self-compaction is compared to the maximum compaction achievable for the fresh concrete. Australian Standard AS1012.3.2 (1998) stated that the extent of fresh concrete will compact itself under these conditions will not vary between individual batches of the concrete. Furthermore, the characteristics and proportions of the ingredients in the concrete mix do not vary from batch to batch of the concrete made.

Compacting factor test is usually not recommended for on-site testing. The main reason is the apparatus was heavy and not portable. But on larger jobs where a site laboratory is established there, appears no reasons why the test should not be used as a check on workability on time to time. In laboratory, the compacting factor test provides a reliable guide for use in the field. This test recommended for type of work that deals workability parameter with the compacting equipment while containing the less quantity of mixing water in concrete mix. Hence, for on-site situation, still slump test was the best recommendation for workability tests. The test procedure was carried out accordance with AS1012.3.2.

4.1.2.1 Apparatus

The following apparatus and equipment complying with the relevant provisions of AS1012.3.2 were used.

Compacting factor apparatus – (a) consists of two conical hoppers mounted above a cylinder.
(b) The hoppers and cylinder are made of rigid materials, which is smooth inside (not readily attacked by cement paste).
(c) The rim of the cylinder is perpendicular from plane surface to axis of the hopper. The lower ends of the hoppers can be closed with tightly fitting and trapdoors can be hinged to having quick-release catches. The doors is approximately 3mm thick sheet brass plate.
(d) The frame mounted the hoppers and cylinder must made of rigid construction and can be firmly locate them in the relative positions. The cylinder is detachable from the frame.

Trowels – two trowels are required.

Scoop – a appropriate size which large enough to accommodate the maximum size of aggregate in the concrete mix.

Rod – a metal rod of 16 ± 1 mm in diameter, approximately 600 mm long and having at least one end tapered for a distance of approximately 25 mm (a spherical shape) having diameter of 10mm.

Balance – a balance that capable of weighing to an accuracy of 10g.

Level – an appropriate level is required.

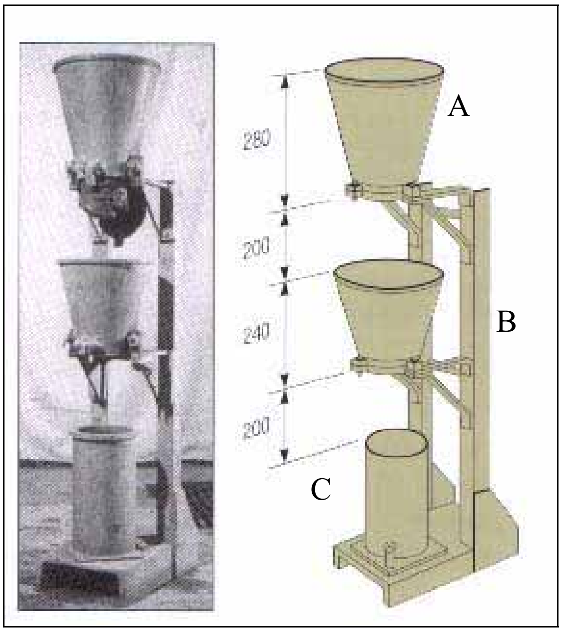


Figure 4.3: Apparatus for compacting factor tests. (Source: Australian Standard HB64, 2002)

Table 4.1: Dimension for the compacting factor apparatus. (Source: AS1012.3.2, 1998)

Detail	Dimensions* mm
Upper hopper A:	
Top internal diameter	260
Bottom internal diameter	130
Internal height	280
Lower hopper B:	
Top internal diameter	240
Bottom internal diameter	130
Internal height	240
Distance between bottom of upper hopper and top of lower hopper	200
Distance between bottom of lower hopper and top of cylinder	200
Cylinder C:	
Internal diameter	150
Internal height	285
Radius between wall and base	20

*Tolerance on all dimension ±1 mm.

4.1.2.2 Test Procedure

The test procedure was in accordance to AS1012.3.2. The procedure of the testing was as follow.

1. The internal surface of the mould was cleaned (free from set concrete) and moistened with a damp cloth immediately before beginning of each test.
2. The apparatus was placed on a level rigid surface that is free from vibration or shock. The cylinder was placed below at the bottom hopper.
3. The fresh concrete was poured in the upper hopper using the scoop gently, until the hopper is filled.
4. The trapdoor was opened to allow the concrete fall into the lower hopper, after the hopper is filled immediately.
5. After the concrete has come to rest, the trapdoor of the lower hopper was opened again and allowing the concrete to fall into the cylinder.
6. The excessive concrete that over the level of the cylinder was cut off by holding a trowel in each hand. The trowels were moved simultaneously from each side across the top of the cylinder with the plane of the blades horizontal. The outside layer of the cylinder was wiped clean.
7. The mass of the concrete in the cylinder was weighted and recorded to the nearest 10 g (M_1). This mass was recorded as the 'mass of the partially compacted concrete'.
8. The cylinder was empty. Fresh concrete was filled into the cylinder again in layers approximately 50 mm deep. Each layers was rodded with the metal rod, until full compaction is achieved. The top surface of the fully compacted concrete was struck off carefully and was finished by leveling. The outside layer of the cylinder was wiped clean.
9. The mass of concrete in the cylinder was weighted and recorded again to the nearest 10 g. This mass was recorded as the 'mass of fully compacted concrete' (M_2).
10. The overall procedure must be completed with minimum delay.

The compacting factor was determined from the following equation:

$$\text{Compacting factor} = \frac{\text{Mass of partially compacted concrete } (M_1)}{\text{Mass of fully compacted concrete } (M_2)} \qquad (4.1)$$

Figure 4.4: The compacting factor test. The concrete in the cylinder mould is about to measure for its weight.

4.1.3 VEBE Consistometer test

VEBE test is the most suitable for the measurement of the workability of fresh concrete at a very low workability. Another word, the VEBE test is basically a mechanical version of slump test for concrete with low workability. It determines the consistency of the concrete by measuring the time taken for the concrete to collapse in the mould under the action of vibration. This test measures the workload which is required to compact the freshly mix concrete. The name 'VEBE' is derived from the initials of V. Bährner of Sweden who developed the test (Neville, 1997).

The method was carried out in similar version to the slump test. A truncated shape cone was prepared on a vibrating table and was filled with fresh concrete. The cone was then removed and a transparent disc was lowered on the subsided fresh concrete. Once the disc was in place, a standard vibration was done to allow the concrete and the transparent disc to collapse together in the mould. The compaction rotates at 50 Hz and a maximum acceleration of 3g to 4g. The time was recorded (in seconds) when the whole surface of the transparent disc was covered with the cement grout, which is the moment when full compaction of the fresh concrete was attained. Although it is often and difficult to obtained an accurate reading of time, the VEBE shows the true evaluation of the workability of the fibre reinforced concrete. More over, this test demonstrates the stiffening effect cause by the fibres. The apparatus of VEBE consistometer test was shown in figure 4.5.

The test is widely used in the laboratory investigations. It is more sensitive to the changes in material properties than slump test. Such sensitivity is the early hydration rate of cements and very dry mix. Hence, the application of this test is limited for controlling consistence in the field. An additional advantage of this test is that 'the treatment of the concrete was comparatively close related to the method of placing in practice' (Neville, 1997). The importance factor for this type of apparatus is cleanliness when conducting the test. Due to distortion, the test must be done in a careful manner to avoid errors. The test procedure was carried out accordance with AS1012.3.2.

4.1.3.1 Apparatus

The following apparatus and equipment complying with the relevant provisions of AS1012.3.3 were used.

Consistometer – the apparatus is consists of these equipments:

(a) *Container* The metal cylindrical container where the internal diameter and height of which is 240 ±5 mm and 200 ±5 mm, fitted with handles, and capable of protection from corrosion. Footpieces of the container is able to be securely clamped to the top of the vibrating table.

(b) *Mould* A frustum of a cone constructed with metal of thickness not less than 1.5 mm. The internal surface of the mould must be smooth and provided with handles for lifting from the moulded concrete specimen in a vertical direction. The mould consists of following internal dimensions:

(i) Bottom diameter = 200 ±5 mm.

(ii) Top diameter = 100 ±5 mm.

(iii) Vertical height = 300 ±5 mm.

(c) *Disc* The transparent disc is 230 ±1 mm in diameter and 10 ±1 mm in thickness.

(d) *Vibrating table* The vibrating table is 380 mm in length and 260 mm in width. The table is supported on four rubber shock absorbers. A vibrator unit was securely fixed under rubber feet at the base of the table. The vibrator operates at an amplitude of 0.5 ±0.02 mm and at a frequency of 50 ±1 Hz

Rod – a metal rod of 16 ± 1 mm in diameter, approximately 600 mm long and having at least one end tapered for a distance of approximately 25 mm (a spherical shape) having diameter of 10mm.

Stopwatch – a readable stopwatch to at least 0.5 second.

Scoop – a appropriate size which large enough to accommodate the maximum size of aggregate in the concrete mix, able to deal with by cement paste.

Figure 4.5: Apparatus of VEBE consistometer. (Source: Australian Standard HB64, 2002)

4.1.3.2 Test Procedure

The test procedure was in accordance to AS1012.3.3. The procedure of the testing was as follow.

1. The internal surface of the mould was cleaned (free from set concrete) and moistened with a damp cloth immediately before beginning of each test.
2. The apparatus was placed on a rigid surface free from external vibration. The surface of the table must be horizontal. The container was placed firmly on the table with two wingnuts. The conical mould was placed in the container and funnel was positioned over the mould.
3. The mould was filled in three layers approximately one-third of the height of the mould. Each layer was rodded 25 strokes with the metal rod. The strokes were distributed in a uniform manner over the cross-section of the mould.
4. After the top layer has been rodded, the excessive concrete on the top of the mould strike off and taken away with the scoop. The mould must be taken care where the mould does not lift from the bottom of the container during these operations.
5. After the excessive concrete was removed, the setscrew was loosened. The funnel was swing back through 90 degrees and the setscrew then retightened.

Carefully remove any surplus concrete, which has fallen from the mould during the filling and leveling.

6. The mould was immediately removed from the concrete by raising its lowly and carefully in a vertical direction, allowing the concrete to collapse.
7. The setscrew was loosened and the transparent disc was swing into position over the subsided cone of concrete. The screw was released allowing the transparent disc to touch the concrete carefully.
8. The setscrew was retightened. The stopwatch was kept ready before the vibration commerce.
9. The stopwatch was start simultaneously once the vibration started.
10. The remoulding of the concrete in the container was observed through the transparent disc. At the moment when the whole of the lower surface of the transparent disc was covered with cement grout and the concrete has been fully compacted, the stopwatch was stopped and the vibrator switched off.
11. The time was recorded in the nearest 0.5 second.

Figure 4.6: The VEBE consistometer test. The concrete in the mould is positioned before the disc is place.

4.2 Concreting of concrete specimens

Once the workability test was done, the concrete specimens were then prepared by pouring the concrete into 150mm diameter x 300mm large cylindrical moulds, 100mm diameter x 200mm small cylindrical moulds and 150mm x 150mm x 700mm beam moulds. In total there were three batches for each type of fibre corresponding to the various fibre volume dosages (0.5%, 1.0% and 1.5%) and one control batch with no added fibres. This shows grand total of 90 concrete specimens (all 10 mix batches), which include five small cylindrical specimens, two large of cylindrical specimens and two beam specimens for each concrete mix batch. The figure 4.7 shows the mould for small and large cylindrical specimens. Figure 4.8 shows the mould for concrete beam.

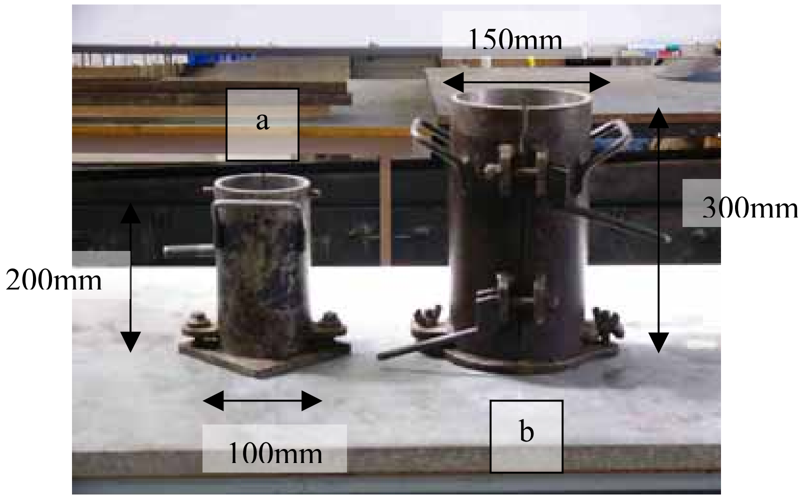


Figure 4.7: 100mm diameter x 200mm (a) and 150mm diameter x 300mm (b) cylindrical moulds.

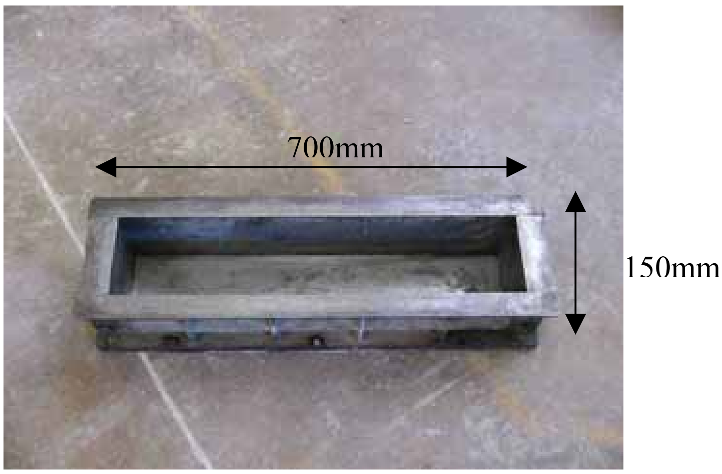


Figure 4.8: Beam mould with dimensions of 150mm x 150mm x 700mm.

As the description for the hardened properties test specimens of this project, 3 of the small concrete cylinders were used to acquire the compressive strength by performing cylinder compressive test; remaining 2 small concrete cylinders were used to obtain the modulus of elasticity test. The 2 large concrete cylinders were used for indirect-tensile test and the last 2 concrete beams were used to obtain the flexural strength of fibre-reinforced concrete. The details of the hardened property test of fibre-reinforced concrete will be discussed in section 4.3.

4.2.1 Casting of concrete specimens

Before any fresh concrete was poured into the concrete moulds, all concrete moulds must be cleaned from the existing concrete stain and diesel oil was applied inside and around the moulds. The fresh concrete was placed into the mould with the scoop and vibrated with an immersion vibrator. Once the concrete moulds were filled, the surface of the concrete was leveled with a lever.

The concrete specimen's surface for steel fibre and polypropylene fibre reinforced concrete was easy to leveled when the fibre volume dosage rate were 0.5% and 1.0%. Once 1.5% volume dosage rate was applied into the concrete for these two fibres, the surfaces of the concrete specimens were difficult to level. However, for Fibremesh, the surface of concrete was very difficult to level when the fibre volume dosage rate was 0.5%. The effects can be shown significantly during the leveling of the concrete beam specimens. This effect showed that the workability of wire cut shape and wave geometry fibres was better than the mesh type of fibres. Hence, the geometry of the fibres may be one of factor that influences the workability of the fresh concrete.

4.2.2 Compaction of fresh concrete

The moulds were filled in third until full and vibrated using an immersion vibrator to achieve an adequate compaction, allowing the entrapped air in the concrete to expel out or reach the surface. Compaction packs the aggregate particles together, increases the strength and enhances the bonding of the concrete. The immersion vibrator was driven by an electric motor situated within the tubular casing, with the frequency of 130Hz. This vibrator is relatively light in weight, with a switch located at the vibrator, and easy to handle. Figure 4.9 shows an immersion vibrator used for the compaction of fibre-reinforced concrete. Figure 4.10 was shown that the use of the immersion vibrator to achieve adequate compaction.

All of the fibres were easy to compact when 0.5% volume dosage rate was applied into the concrete. When 1.0% and 1.5% of volume dosage rates were applied into the concrete, the compaction of the concrete was difficult (except steel fibres). Hence, rubber hammer was used and tapped at the mould to allow the entrapped air to reach the surface. Figure 4.11 was shown that the concrete specimens were left to the next day, allowing the concrete specimens to set.

Figure 4.9: An immersion vibrator.

Figure 4.10: Use of immersion vibrator.

Figure 4.11: Concrete casting.

4.2.3 Stripping of mould and curing of concrete specimens

The concrete was left overnight to set so that the moulds could be removed easily. The concrete specimens were stripped and placed a side for identification of mix batches. The moulds were cleaned and oiled for the next concrete mix. Figure 4.12 and figure 4.13 shows the set concrete specimens after 24 hours. All concrete specimens were placed in the curing room with a controlled environment of 25°C degree of a further of 27 days for hardened property testing.

Curing is designed to keep the concrete moist by preventing the loss of moisture from the concrete while it is gaining its strength. It is a poor practice to not allowing the concrete specimens to cure. This results that the concrete to perform less well. Such effects of inadequate curing often lead the concrete specimens to have unexpected cracking and steel corrosion. All concrete specimens were removed from the curing room after 28 days and were conducted for the appropriate hardened property tests. Figure 4.14 shows the concrete specimens in the curing room.

Figure 4.12: The concrete specimens in the mould after 24 hours.

Figure 4.13: Concrete specimen after stripping.

Figure 4.14: Curing of concrete specimen in curing room.

4.3 Hardened properties Test

Concrete is a strong material in compression, where it can resist high static crushing loads. However, it is relatively weak in tension, as it is easily to fail and cause cracks when subjected to tension and bending. To overcome this weakness, reinforcement is required. The characteristic of concrete generally was used in relation to the quality for any construction purpose. It is important to understand the properties of concrete as they indicate the potential qualities to this purpose. Nevertheless, characteristics of concrete strength and durability should not consider as essential material properties. Such factors like specimen geometry and preparation, temperature, loading rate, moisture content, and type and method of testing will affect the mechanical behaviour. Majority of these properties of concrete were used in laboratory work, and especially in research, where it is based to the knowledge of the influence on these tests and the measured property is important. Such hardened properties tests can categorise to destruction and non-destructive tests, which permit repeated test on the same specimen, making potential study of change in properties with time. The tests conducted for this project are destructive test.

This section 4.3 mainly deals with the hardened property test of fibre reinforced concrete. Each test and methodology was outlined and discussed. The test procedure to measure the hardened property of fibre reinforced concrete in this project was according with AS1012.3, and these methods include:

- Compressive strength test.
- Indirect tensile strength test.
- Flexural strength test.
- Modulus of Elasticity.

4.3.1 Compressive strength test

Compressive strength of a concrete is a measure of its ability to resist static load, which tends to crush it. Most common test on hardened concrete is compressive strength test. It is because the test is easy to perform. Furthermore, many desirable characteristic of concrete are qualitatively related to its strength and the importance of the compressive strength of concrete in structural design. The compressive strength gives a good and clear indication that how the strength is affected with the increase of fibre volume dosage rate in the test specimens.

In Australia, concrete specimens for compressive strength test were 100mm diameter and 200mm height. Although in AS1012 stated that the specimens for compressive strength can be 150mm diameter and 300mm height, but this only applies to the maximum aggregate size more than 20mm. Concrete may tested in cube specimen with 150mm each side, but this commonly practice in some other countries like United Kingdom. In standard also required that the test specimens to be capped or ground plane at each end to promote symmetrical loading of the specimen. In this compressive strength test, the test specimens were end capped using mould rubber capping.

This test was performed to find the increase and differences of strength according the increasing percentage of fibre in the concrete. The compressive strength test was conducted in the engineering laboratory of University of Southern Queensland after the concrete specimens were cured for 28 days. The test procedure was carried out accordance with AS1012.9.

The compressive strength of concrete can be calculated using the following formula:

$$f'_c = \frac{P \times 1000}{A} \qquad (4.2)$$

Where: f'_c = Compressive strength of concrete (MPa).

P = Maximum load applied to the specimen in kN.

A = Cross sectional area of the specimen (mm^2).

4.3.1.1 Apparatus

Testing Machine – ‘Avery’ hydraulic loading machine.

Rubber Capping – a suitable size mould of rubber cap.

Vernier calliper – a Vernier calliper readable at least of 0.2mm accuracy.

Balance – a balance that capable of weighing to an accuracy of 10g.

Ruler – ruler at least 400mm long with scale of 0.5mm interval.

4.3.1.2 Test Procedure

The test procedure was in accordance to AS1012.9. The procedure of the testing was as follow.

1. All moist cured specimens that need to be test and the measuring of the concrete specimens were conducted immediately after the specimens were removed from the curing room.
2. The diameter and height of the concrete specimens were measured with the Vernier calliper and ruler. All measurement was recorded.
3. The mass of each specimen was weight and recorded.
4. The platens of the testing machine were cleaned with a clean rag to ensure it is free from films of oil and particles of grit.
5. The uncapped surfaces of the specimens was wiped and brushed to remove free loose particles.
6. The specimen was placed in the testing machine (between the two platens). The axis of the specimens was carefully aligned with the center of thrust of the spherically seated platen.
7. The rubber cap was placed on the rough surface of the specimens.
8. The upper platen was lowered down to the capped specimen so that uniform bearing is obtained.

9. Force was applied without shock and increase continuously at a rate equivalently to 20±2MPa compressive stress per minute until the specimen can sustain no force.
10. The maximum force was read from the testing machine meter and the reading was recorded.
11. The compressive strength of the specimen was obtained by through the maximum load form the meter over the cross sectional area of the specimen.

Figure 4.15: A typical compression test of 100mm diameter x 150mm height concrete specimen with rubber capping.

4.3.2 Indirect tensile strength test

Tensile strength of a concrete is a measure of its ability to resist forces, which stretch or bend it. Unlike steel, the concrete is sufficient in strength only in one direction. The tensile strength of concrete is approximately one-tenth of the compressive strength and it is not generally used in the design of concrete structure. Nevertheless, it is an important property in many applications. Addition of fibre is one of the primary reasons to increase the tensile strength.

The splitting indirect tensile is also known as the Brazil test, which developed originally in Brazil. The testing of specimens in pure tension is very difficult and usually determines by indirect mean, applying tension in the form of splitting. Concrete specimens for indirect tensile test were 150mm diameter and 300mm height. The specimens were placed with its axis horizontal, between the platens of a compression-testing machine. The apparatus set up was shown in figure 4.16. Load was applied until the specimen fails in its vertical diameter. The splitting test is simple to conduct and gives more consistent results than other tension tests. It is believed that the strength obtain by splitting test is nearer that the true tensile strength of the concrete than modulus of rupture.

The task of this test was performed to find the increase and differences of strength according the increasing percentage of fibre in the concrete. The indirect tensile strength test was conducted in the engineering laboratory of University of Southern Queensland after the concrete specimens were cured for 28 days. The test procedure was carried out accordance with AS1012.10.

The indirect tensile strength of concrete is calculated using the following formula:

$$f'_{ct} = \frac{2P \times 1000}{\pi \times L \times D} \qquad (4.3)$$

Where: f'_{ct} = Indirect tensile strength of concrete (MPa).

P = Maximum load applied to the specimen in kN.

L = Length of the specimen in mm.

D = Diameter of the specimen in mm.

4.3.2.1 Apparatus

The following apparatus and equipment complying with the relevant provisions of AS1012.10 were used.

Testing machine – 'Avery' hydraulic loading machine.

Testing jig – an appropriate steel jig illustrated in Figure 4.16. The thickness of the jig bearing plate is about 20mm.

Supplementary bearing bar or plate – supplementary steel bearing plate width of 50mm and thickness of 20mm.

Bearing strips – two bearing strips of 5mm thick, 25mm wide and 300mm long.

Vernier calliper – a Vernier calliper readable at least of 0.2mm accuracy.

Ruler – ruler at least 400mm long with scale of 0.5mm interval.

4.3.2.2 Test Procedure

The test procedure was in accordance to AS1012.10. The procedure of the testing was as follow.

1. The diameter and length of the specimens were measured by averaging at least three diameters and at least two length measurements. All measurements were recorded.
2. The specimen was placed on the testing jig. Handboard bearing strips were aligned between the top and bottom platen of the specimen.
3. A small initial force was applied and the side constraint was removed.
4. Force was applied without shock and increase continuously at a rate equivalently to 15±0.15MPa indirect tensile stress per minute until the specimen can sustain no force. The maximum force applied was recorded.

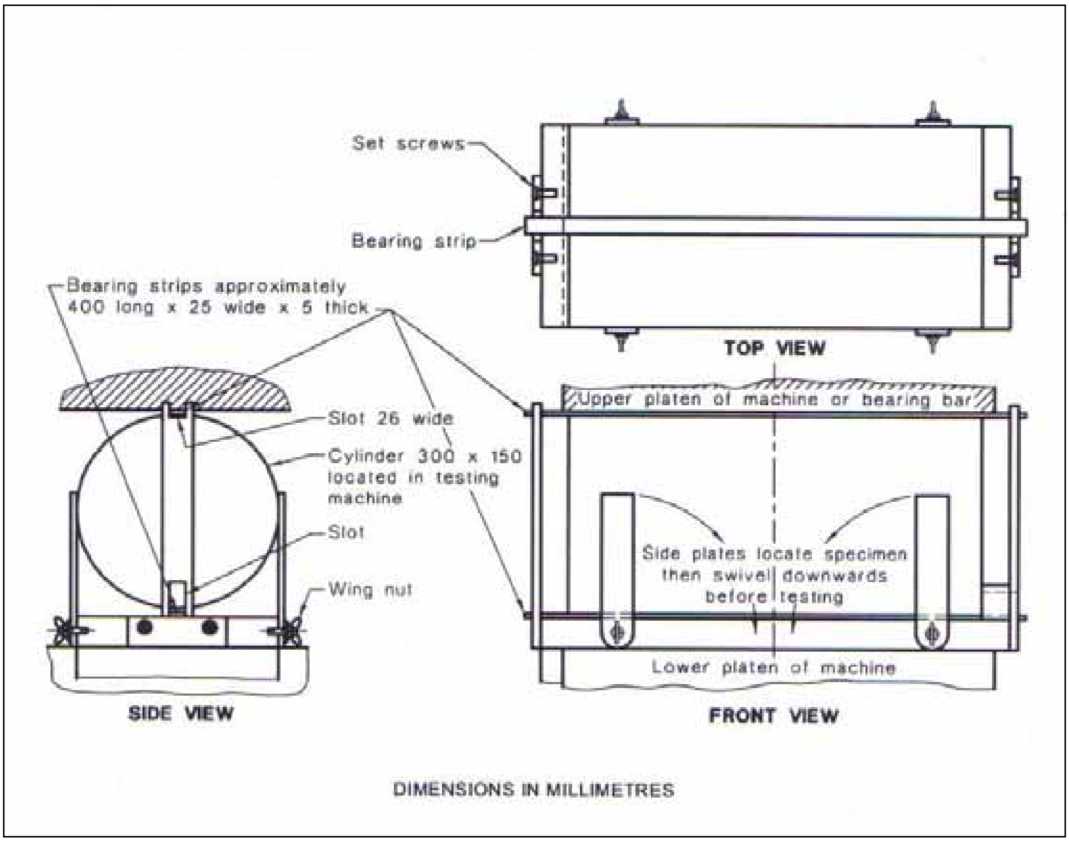


Figure 4.16: Apparatus of indirect tensile strength test with dimension.
(Source: Australian Standard - AS1012, 2000).

Figure 4.17: A typical set up of indirect tensile test.

4.3.3 Flexural strength test

Flexural strength of a concrete is a measure of its ability to resist bending. Flexural strength can be expressed in terms of 'modulus of rupture'. Concrete specimens for flexural strength were cross sectional area of 150mm width with 150mm depth and length of 700mm concrete beam. The specimen is subjected to bending, using four-point loading until it fails. The distance of the loading point (*l*) is 150mm and the supporting point (L) is 450mm. Figure 4.18 shows the setting up of the concrete beam specimen. The test was carried out under computer-controlled conditions, which measures the load and deflection of concrete beam specimens. As an accurate load/deformation curves can be plotted to obtain the toughness of the concrete with the computer software. In this test, the load/deformation behaviour is much more important than the modulus of rupture. It tells more information about the behaviour and effect of each type of the short fibres applied in the concrete, once it reaches to the post-crack condition. Because of this perspective, the test is desirable to conduct under a computer-controlled condition.

The task of this test was performed to find the increase and differences of strength according the increasing percentage of fibre in the concrete, in both pre-crack and post-crack behaviour, as fundamental to assess and evaluate the effects of the additional of short fibres on the behaviour of concrete. The flexural strength test was conducted in the material laboratory of University of Southern Queensland after the concrete specimens were cured for 28 days. The test procedure was carried out accordance with AS1012.11.

The flexural strength of concrete can be calculated using the following formula:

$$f'_{cf} = \frac{P \times L \times 1000}{B \times D^2} \quad (4.4)$$

Where:

f'_{cf} = Flexural strength of concrete (MPa).

P = Maximum load applied to the specimen in kN.

L = Length of the specimen in mm.

B = Width of the specimen in mm

D = Diameter of the specimen in mm.

4.3.3.1 Apparatus

The following apparatus and equipment complying with the relevant provisions of AS1012.11 were used.

Testing machine – Computerised ‘Avery’ hydraulic testing machine.

Measuring tape – a measuring tape about 2 meters long with scale of 1mm interval.

Vernier calliper – a Vernier calliper readable at least of 0.2mm accuracy.

Marker pen – a useable marker pen to draw the loading and supporting line on the specimen.

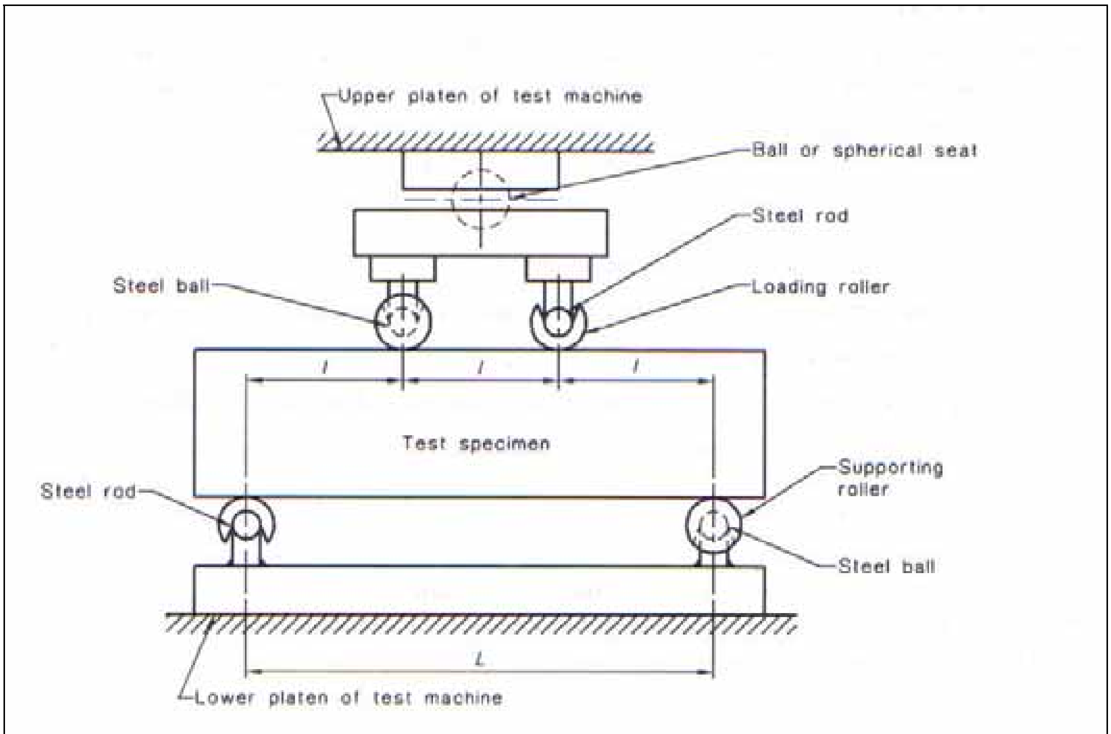


Figure 4.18: The flexural strength test. (Source: Australian Standard - AS1012, 2003)

4.3.3.2 Test Procedure

The test procedure was in accordance to AS1012.11. The procedure of the testing was as follow.

1. Any girt from the surface was removed and surplus water was wiped on the concrete specimen before commencing the test.
2. The average width and average depth of the concrete specimens was measured with the Vernier calliper. All measurement was recorded.
3. The measuring tape was placed on the concrete beam specimen and lines were drawn at the every interval of 150mm at each side, respect to the center of the specimen.
4. The specimen was turned to its side respect to its position as the mould and placed on the supporting rollers. The two outer lines drawn on the specimen were aligned to the center of the supporting roller.
5. The loading rollers were brought in contact to the top of the specimen, applying a seating load not exceeding 100N. Once the loading rollers reached, the uniformity of bearing of the rollers was checked. The positions of the loading rollers were aligned to the two inner lines drawn on the specimen.
6. Force was applied without shock and increase continuously at a rate equivalently to 1±0.1MPa extreme fibre stress per minute until the specimen can sustain no force. The maximum force applied to the concrete specimen was recorded through the software of the computerised test machine.
7. The appearance of concrete and type of fracture was notice during the load process.
8. If the fracture happens to occur outside the middle third of the span length, the modulus of rupture was not calculated. However, the distance for section of failure was measure and recorded from the nearest supporting roller.

Figure 4.19: The servo system of computerised ‘Avery’ testing machine.

Figure 4.20: A typical flexural strength tests under 4-point loading.

Figure 4.21: Computerised ‘Avery’ testing machine.

4.3.4 Modulus of elasticity

Modulus of elasticity of the concrete is one of the important mechanical properties of any material when reflecting its use in design, through determination of deflection of structural element. It is a measure of the stiffness, or the resistance of the material to deformation (Troxell, 1968). Modulus of elasticity is a fundamental factor to estimate the deformation occurs on the structure, as well as the determination of modular ratio used as design parameter for members subjected to flexure.

The modulus of elasticity test was carried out accordance to AS1012.17. According to the standard, it stated that this method follows the same procedures used in the compression test, while measuring the deflection of the concrete specimen at a particular distance during the loading by using strain-measuring equipment. However, this test can be done through using a computer-controlled machine, where the load applied to the specimen can be uniformly controlled and the displacement can be measure in short intervals during the loading process. Hence, more accurate results for the elasticity of the material can be obtained. Modulus of elasticity for each concrete specimen was determined through 60% of the maximum peak stress respect to the strain (deformation) at this 60% of the maximum peak stress.

Capping of the specimen was required in this test, as to enable the load uniformly distributed over the specimens. Deflection measuring of the specimen is essential in the determination of modulus of elasticity, it is important that the deformation does not occur to the specimens itself, but may influence by other material such as capping and testing machine plate used. Rubber capping deflects while the load is transferring on to the specimen. As a result, an inaccurate measurement of deflection occurs. Hence, a form of rigid capping that does not reflect under the load is required. Sulphur capping is a rigid form of capping which deflects almost similar to the concrete. Therefore, it is a practice to sulphur capped all the concrete specimens before commencing the modulus of elasticity test. AS1012.9 stated that sulphur capping of all specimens must at least 1 hour old before the conducting of the test.

The task of this test was performed to for the determination of static chord modulus of concrete specimens to investigate the effects of the additional of short fibres on the behaviour of concrete. All mix proportion of the concrete specimens was same, as any change of the modulus of elasticity will only cause by the replacement of different volume dosage of fires. The modulus of elasticity test was conducted in the material laboratory of University of Southern Queensland after the concrete specimens were cured for 28 days. Concrete specimens for modulus of elasticity were 100mm diameter and 200mm height.

4.3.4.1 Apparatus

Testing machine – Computerised 'Avery' hydraulic testing machine.

Vernier calliper – a Vernier calliper readable at least of 0.2mm accuracy.

Ruler – ruler at least 300mm long with scale of 0.5mm interval.

Balance – a balance that capable of weighing to an accuracy of 10g.

Mould – a mould which capable to mould the specimen with sulphur, as shown in figure 4.23.

Heater – a heater with adjustable control of every 1°C to heat the sulphur.

Scoop – an appropriate scoop to fill the sulphur liquid.

Rubber hammer – a useable rubber hammer to assist the specimen out of the mould.

4.3.4.2 Test Procedure

The test procedure adopted was based on the procedure below. The procedure of the testing was as follow.

1. The diameter and height of the concrete specimens were measured with the Vernier calliper and ruler. All measurement was recorded.
2. The mass of each specimen was weight and recorded.
3. The uncapped surfaces of the specimens was wiped and brushed to remove free loose particles.
4. Sulphur was capped on the rough surface on the specimen and left more than 1 hour before commencing the test.
5. The platens of the testing machine were cleaned with a clean rag to ensure it is free from films of oil and particles of grit.
6. The specimen was placed on the platen of the testing machine after 1 hour.
7. The loading platen was brought in contact to the top of the specimen, applying a seating load not exceeding 100N. Once the loading platen reached, the specimen was checked and aligned with the center of thrust of the spherically seated platen.
8. Force was applied without shock and increase continuously at a rate equivalently to 15±2MPa compressive stress per minute until the specimen can sustain no force. The maximum force applied to the concrete specimen and the deformation of the specimen was recorded through the software of the computerised test machine.

The stress of the specimen was calculated with the following formula:

$$\sigma = \frac{P \times 1000}{A} \qquad (4.5)$$

Where: σ = Stress of concrete (MPa).

P = Maximum load applied to the specimen in kN.

A = Cross sectional area of the specimen (mm^2).

The strain of the specimen was calculated with the following formula:

$$\varepsilon = \frac{\text{Deformation of the concrete specimen obtained from computer}}{\text{Height of the concrete specimen}} \qquad (4.6)$$

Where: ε = Strain of the specimen.

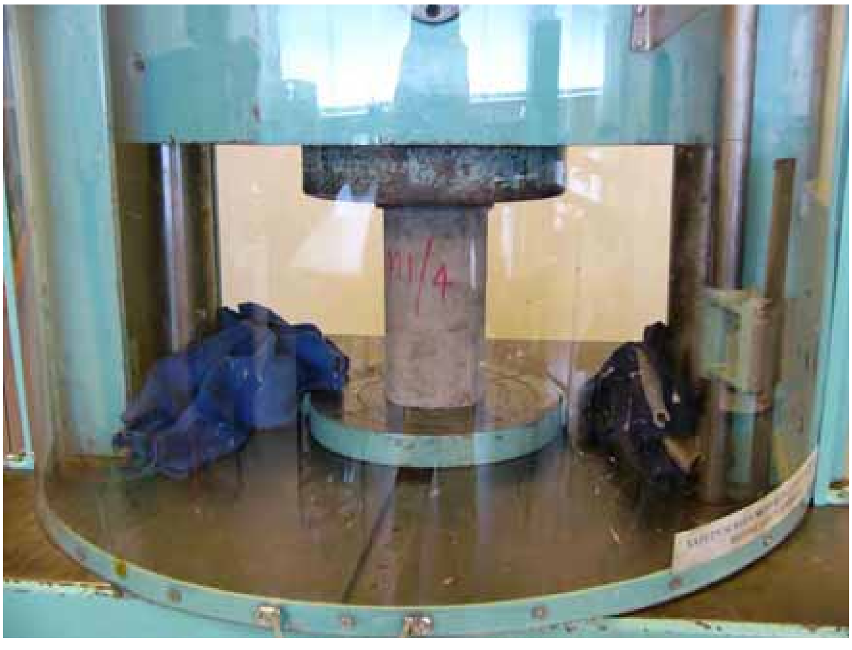


Figure 4.22: A typical test for modulus of elasticity.

Figure 4.23: Mould used to form sulphur cap for specimens.

4.4 Summary

It is worth mentioning that all the experimental methodologies adopted in this project conform to the standard test procedures. The test set up was arranged bearing in mind in the laboratory and to its resource limitations. All of the tests were operated successfully and safely through out the project. The results of each test were discussed in chapter 5.

CHAPTER 5

Results and Discussions

5.1 Introduction

This chapter focuses on the experimental results obtained from each test and analysis of the test results. The experimental tests were carried out to obtain the mechanical properties and behaviour of fibre reinforced concrete, while also compared to the conventional plain concrete. The comparisons of mechanical properties and behaviour include the workability, compressive strength, indirect tensile strength, flexural strength, modulus of elasticity, pre and post-cracking state behaviour and the ductility of fibre reinforced concrete. With the discussions and results obtained from the experimental tests, it is clearly to know the effect of steel fibres, polypropylene fibres and Fibremesh used in the structural concrete.

5.2 Slump Test

The slump test indicates a decreasing trend when the percentage of fibre increased. Table 5.1 below shows the slump height recorded during the test and the difference in percentage dropped for all mix batches compared to control batch. Figure 5.1 below shows a graphical representation of slump height for concrete containing no fibres and concrete containing different amounts and types of fibres.

Table 5.1: Slump values recorded and percentage drop for each mix batch.

Type of mix batch	Slump height (mm)	Percentage Drop (%)
Control (0%)	140	0
0.5% Polypropylene Fibre	80	43
1.0% Polypropylene Fibre	65	54
1.5% Polypropylene Fibre	30	79
0.5% Steel Fibre	135	4
1.0% Steel Fibre	105	25
1.5% Steel Fibre	50	64
0.5% Fibremesh	50	64
1.0% Fibremesh	35	75
1.5% Fibremesh	20	86

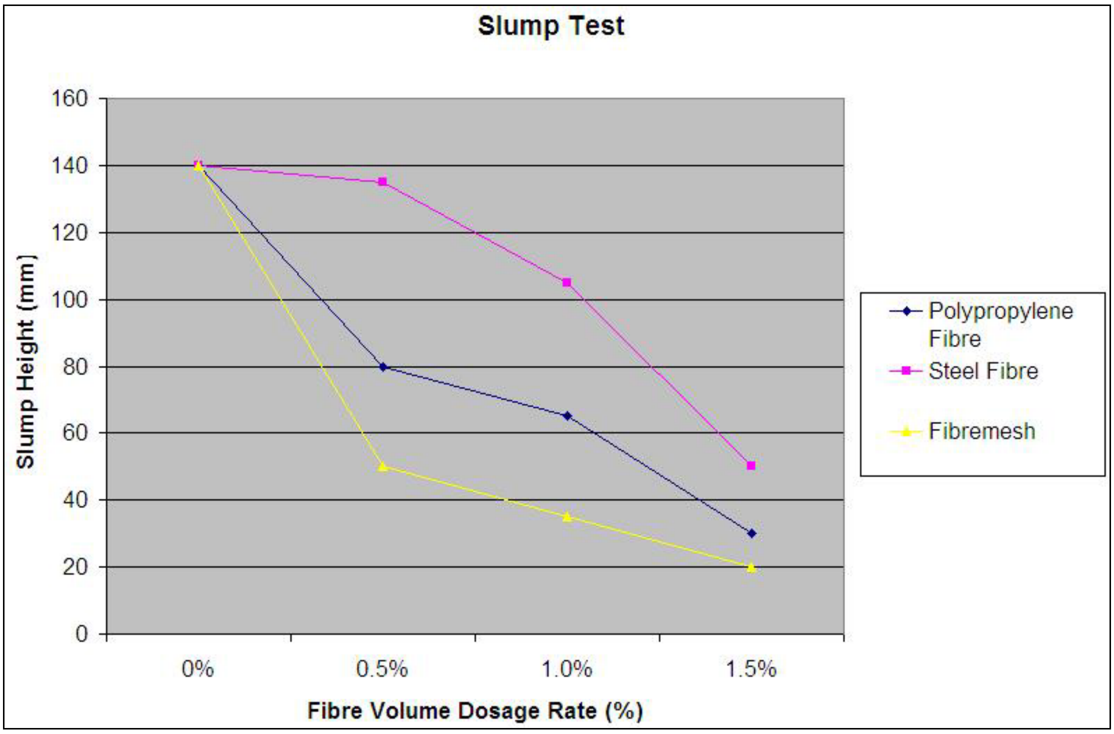


Figure 5.1: Average slump height vs. fibre volume dosage rate of each batch.

The experimental results showed that the slump of the fibre reinforced concrete has a decreasing trend when the fibre volume dosage rate increases. Furthermore, the decreasing of slump has a different trend when different type and geometry of short fibres were used. Figure 5.1 indicates that Fibremesh has less workability where else steel fibre has more workability than other type of fibres. Polypropylene fibre is considered moderate as the measurement of slump falls in between of steel fibres and Fibremesh.

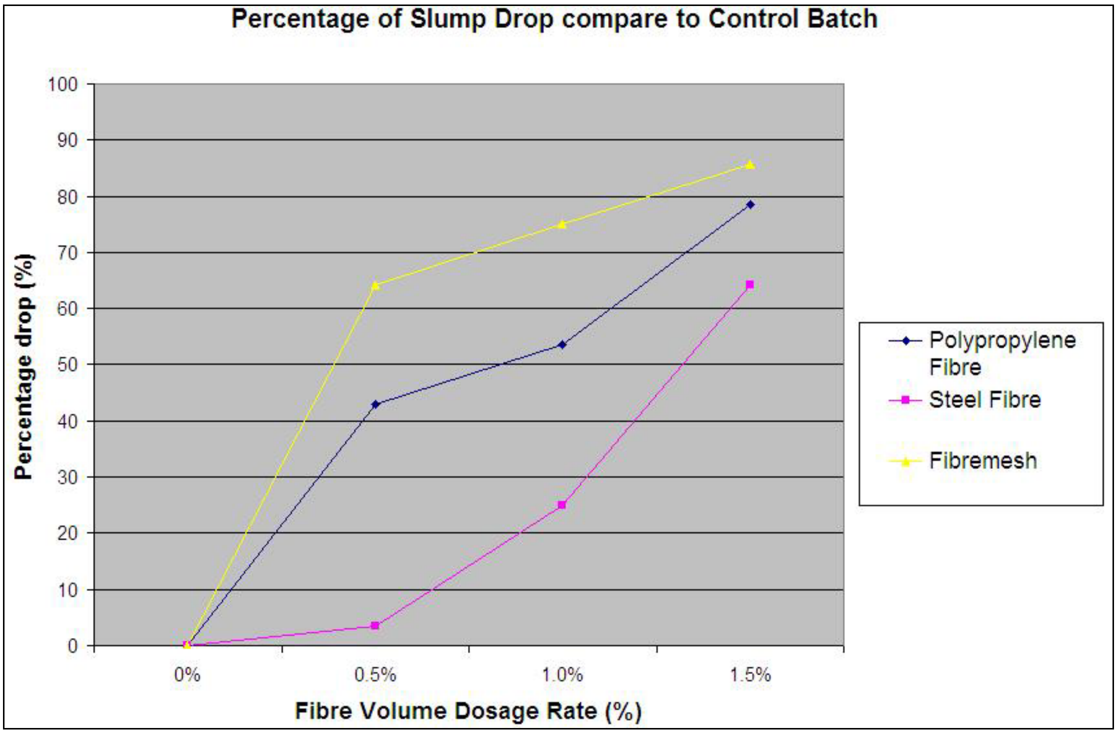


Figure 5.2: Percentage drop of slump vs. fibre volume dosage rate of each batch.

Table 5.1 indicates a slump value of 140mm for control mix with no fibre added to the concrete. Once the fibre was added into the concrete, all the fibres had an average slump drop of 88mm for 0.5%, 68mm for 1.0% and 33mm for 1.5% fibre volume dosage rate. An average of 23mm to 32mm of slump decreased when every 0.5% of fibre volume dosage rate increased. Figure 5.1 shows the comparison of slump dropped for each batch. Figure 5.2 shows the comparison of percentage dropped for each batch, comparing to the control batch.

The slump of steel fibre has a tendency to drop more (about 135mm of slump and 4% dropped compare to control batch) when fibre volume dosage rate of 0.5% were added to the concrete mix. Once the fibre volume dosage rate exceeds more than 0.5%, the slump decreases rapidly where 105mm for 1.0% dosage and 50mm for 1.5% dosage of steel fibre. An average of 32% of slump drop occurred for every increase of 0.5% fibre volume dosage rate compare to control mix. The slump of steel fibre was large and that might be caused by its geometry (wave cut geometry). It indicated that the concrete mix was still workable before 1.0% volume dosage rate of steel fibre was added.

Polypropylene fibre has a moderate slump drop compare to steel fibre and Fibremesh. Polypropylene fibre has less drop of slump once the fibre had been mixed into the concrete mix, which is similar to Fibremesh. Even though polypropylene fibre has the similar decreasing trend to Fibremesh, but the concrete was still easy to be handled. Table 5.1 shows that there was 80mm slump drop for 0.5% of polypropylene fibre volume dosage rate, which about 43% of slump drop compare to control batch. The slump for 1.0% and 1.5% of fibre volume dosage rate were 65mm and 30mm. Such percentage dropped for these two dosage rates compare to control batch is approximately 54% and 79%.

There was a big decrease of slump when Fibremesh was added in the concrete mix. The slump dropped to 50mm when fibre volume dosage rate was 0.5% as indication that the concrete mix was not workable as the concrete was difficult to compact. As the volume dosage rate of Fibremesh increased to 1.0% and 1.5%, the slump gradually decreased to 35 and 20mm. The average percentage of Fibremesh slump drop compare to control mix was approximately 75%. There was no big variation of slump drop for Fibremesh compare to polypropylene fibre and steel fibre after the fibre was added.

The slumps obtained from the concrete mix were considered satisfactory. These results show the workability of concrete with fibres is less than the concrete without fibres, as slump test may take into account that it is satisfactory test for workability of fibre reinforced concrete. Judging from the difference in slump, it is said that if fibre volume dosage rate increased, a lower workability of concrete would occur. The geometry and shape of the fibres is another factor influences the workability of concrete.

5.3 Compacting Factor Test

Similar to slump test, the compacting factor test indicates a decreasing trend when the percentage of fibre increased. Table 5.2 below shows the slump height recorded during the test. Figure 5.3 below shows a graphical representation of slump height for concrete containing no fibres and concrete containing different amounts and types of fibres.

Table 5.2: Compacting factor ratio recorded for each mix batch.

Type of mix batch	M_1 (kg)	M_2 (kg)	Compacting Factor Ratio
Control (0%)	26.19	26.31	0.995
0.5% Polypropylene Fibre	25.79	26.32	0.980
1.0% Polypropylene Fibre	23.13	24.61	0.940
1.5% Polypropylene Fibre	23.44	25.76	0.910
0.5% Steel Fibre	26.35	26.38	0.999
1.0% Steel Fibre	25.31	26.16	0.968
1.5% Steel Fibre	24.78	26.11	0.949
0.5% Fibremesh	23.26	24.54	0.948
1.0% Fibremesh	22.68	24.59	0.922
1.5% Fibremesh	21.66	24.63	0.879

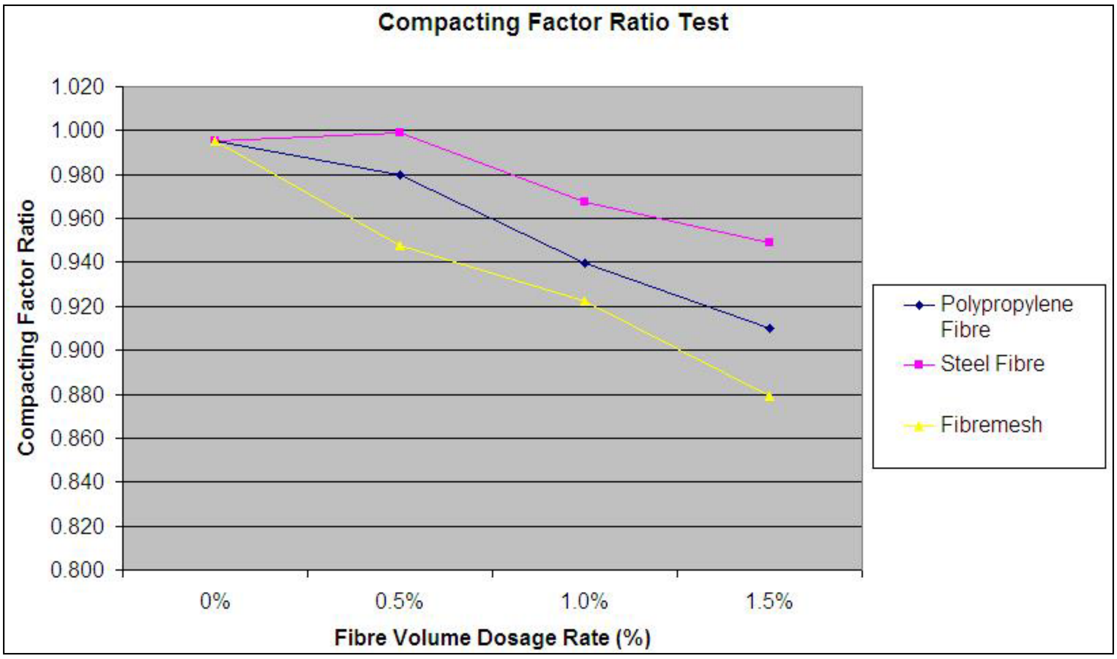


Figure 5.3: Average compacting factor ratio vs. fibre volume dosage rate of each batch.

Figure 5.3 show that all type of fibres has a similar decreasing trend when the fibre volume dosage rate increased. The results shows that the workability of the Fibremesh reinforced concrete was less than polypropylene fibre reinforced concrete, where the steel fibre reinforced concrete are more workable than the other two types of fibre reinforced concrete. The results of compacting factor test are very similar to slump test. It was show that whenever addition of fibre was applied into the concrete, the workability of the fibre reinforced concrete decreased.

When 0.5% volume dosage rate was applied into the concrete, the compaction for Fibremesh decreased the most compare to other types of fibres, where the compacting factor ratio was 0.948. The polypropylene fibre reinforced concrete was 0.98 and steel fibre reinforced concrete was 0.999. All fibres have an average of 0.976 of compacting factor ratio compare to control batch. It is very interesting that the compacting ratio of the steel fibre reinforced concrete is higher than control batch. The reason of this may cause by the refilling of fresh concrete into the mould for first weight measurement, which the mass of partially compacted concrete. The fibres was randomly distributed in the fresh concrete as the steel fibres has a higher specific gravity, which gives a higher value of weight.

In 1.0% volume dosage rate, the compacting factor ratio of all type of fibres decreased gradually, with an average of 4% reduction of compaction. All fibres have an average of 0.943 of compacting factor ratio compare to control batch. The compacting factor ratio for polypropylene fibre, steel fibre and Fibremesh was 0.940, 0.968 and 0.922. There was a difference of 3%, 6% and 7% of compacting factor ratio for steel fibre, polypropylene fibre and Fibremesh, comparing to control batch.

The compacting factor ratio decreased gradually for all type of fibres, when 1.5% volume dosage rate was applied in the concrete. There was not much difference in the decreasing rate for 1.5% volume dosage rate, compare to 0.5% and 1.0% volume dosage rate. But comparing to control batch, the difference of compacting factor ratio was large. Steel fibre has a ratio of 0.949, polypropylene fibre has a ratio of 0.91 and Fibremesh has a ratio of 0.879, where the control batch has a ratio of 0.995. The difference ratio to the control batch is approximately 5%, 9% and 12% respectively to steel fibre, polypropylene fibre and Fibremesh.

The compacting factor test gives a reasonable evaluation of workability to fibre reinforced concrete. The test has some limitations. All mixes tend to stick in one or both hoppers and the concrete material has to be eased gently by poking with the steel rod. Moreover, the amount of work required for full compaction depends on the richness of the mix. Compacting factor test does not meet this requirement, where leaner mixes required more work than richer ones. Another limitation is the orientation of fibre placed in the concrete will influence the degree of compaction for the concrete. Hence, an accurate measurement of workability of fibre-reinforced concrete in required, which is VEBE consistometer test.

5.4 VEBE Consistometer Test

VEBE consistometer test gave more accurate indication of the workability of the fibre reinforced concrete than standard slump test and compacting factor test. The vibration of the VEBE consistometer apparatus overcomes the stiffening effects of the fibres. This means that the indication of the true workability of fibre reinforced concrete can be obtained accurately. Table 5.3 shows the value obtained from the VEBE consistometer test and figure 5.4 represented the graphical trend of the test results for all fibre reinforced concrete and concrete containing no fibres.

Table 5.3: VEBE time recorded for each mix batch.

Type of mix batch	Time (sec)
Control (0%)	1.0
0.5% Polypropylene Fibre	2.0
1.0% Polypropylene Fibre	3.5
1.5% Polypropylene Fibre	5.5
0.5% Steel Fibre	1.5
1.0% Steel Fibre	2.0
1.5% Steel Fibre	4.5
0.5% Fibremesh	6.0
1.0% Fibremesh	7.0
1.5% Fibremesh	10.0

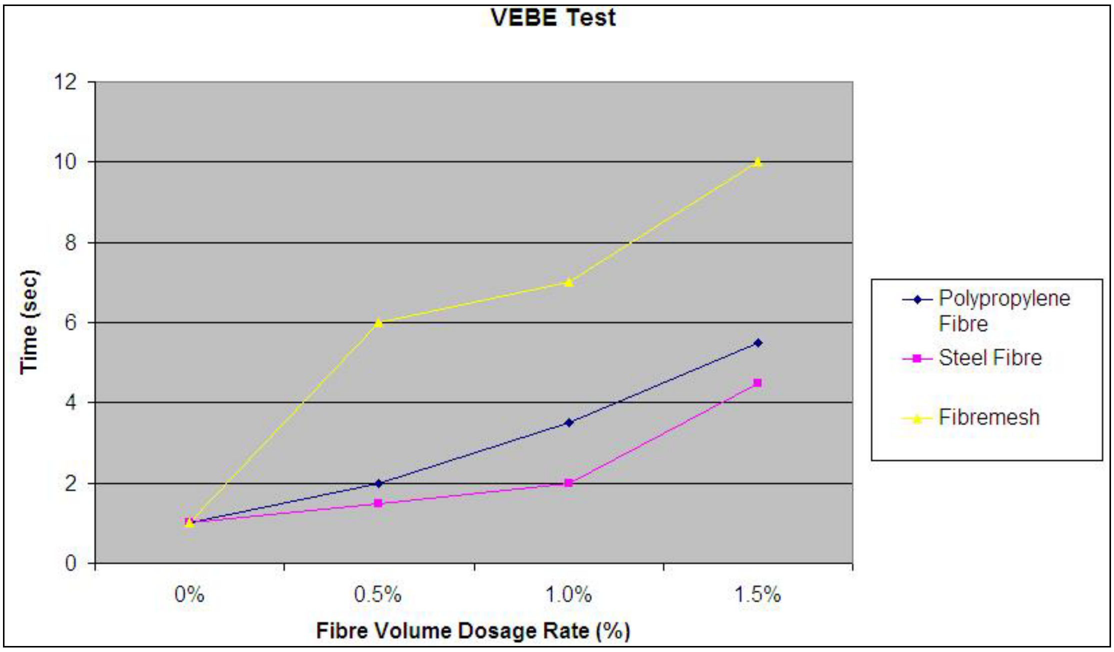


Figure 5.4: Average VEBE time vs. fibre volume dosage rate of each batch.

Figure 5.4 shows linear relationship between the volume dosage rates added for all type of fibres and the time taken for the concrete to collapse that gives the moment of compaction to level surface by vibration. This direct increasing trend was shows that the addition of fibre in the concrete will lead to the decrease of workability of a concrete. Steel fibre and polypropylene fibre tends to have similar rising trend, but the rising trend for Fibremesh to reach a full compaction by vibration increasing rapidly. The steel and polypropylene fibre reinforced concrete was considered workable, as the time was less than 6 seconds. It is obvious that the Fibremesh fibre reinforced concrete was not workable, where the time beyond more than 6 seconds.

When 0.5% volume dosage rate was applied into the concrete, Fibremesh took 6 seconds to reach to a full compaction state. Steel fibre and polypropylene took less than 2 seconds. This shows that the steel fibre and polypropylene fibre took an average time of 1.8 seconds. The time difference between steel fibre and polypropylene and control batch was very close to each other, but time taken for Fibremesh was too much far from the control batch.

In 1.0% volume dosage rate, the time taken by polypropylene fibre increase more than other fibres (2 seconds to 3.5 seconds, difference of 1.5 seconds). Steel fibres increased from 1.5 seconds to 2 seconds, difference of 0.5 seconds and Fibremesh increased from 6 seconds to 7 seconds, difference of 1 second. This shows that there was some reduction in the time taken when the fibre volume dosage rate was 1.0% for steel fibre and Fibremesh.

The time taken was increase rapidly for all type of fibres, when 1.5% volume dosage rate was applied in the concrete. Steel fibre increased from 2 seconds to 4.5 seconds, a difference of 2.5 seconds, where polypropylene increased from 3.5 seconds to 5.5 seconds, a difference of 2 seconds and Fibremesh increased from 7 seconds to 10 seconds, a difference of 3 seconds. This shows that all type of fibres have very poor workability in concrete. The analysis indicates that 1.5% volume dosage rate may be the extent limit of volume of fibre can applied into a concrete.

The VEBE consistometer test gave a more accurate indication of the workability of the fibre reinforced concrete that the slump test and compacting factor test. The VEBE test provides an indication of the true workability of the fibre reinforced concrete and overcomes the stiffening effects of the fibres to the concrete. Although the there are some inaccuracies due to human error when conducting this test, the results shown that there is a reasonable assessment of workability to the fibre reinforced concrete.

5.5 Compressive Strength Test

The results of compressive strength test shows that the compressive strength did not increase when the percentage of fibre increased more than 1.0% fibre volume dosage rate. Table 5.4 below shows the average compressive strength recorded during the test and the strength difference in percentage for all mix batches compared to control batch. Figure 5.5 below shows a graphical representation of average compressive strength for concrete containing no fibres and concrete containing different amounts and types of fibres.

Table 5.4: Average compressive strength and percentage difference compare to control batch for each batch.

Type of mix batch	Average compressive strength (MPa)	Percentage difference (%)
Control (0%)	34.1	0
0.5% Polypropylene Fibre	48.5	42
1.0% Polypropylene Fibre	42.9	26
1.5% Polypropylene Fibre	41.6	22
0.5% Steel Fibre	44.4	30
1.0% Steel Fibre	45.4	33
1.5% Steel Fibre	48.6	42
0.5% Fibremesh	45.3	33
1.0% Fibremesh	37.6	10
1.5% Fibremesh	28.6	-16

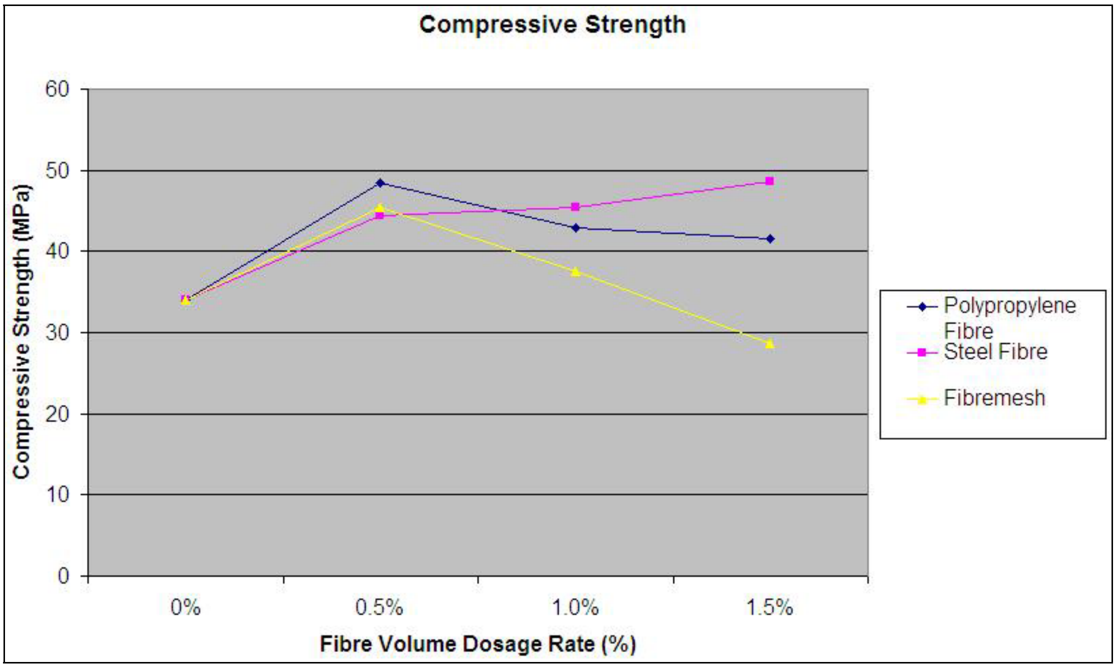


Figure 5.5: Average compressive strength vs. fibre volume dosage rate of each batch.

The results of the compressive strength test indicate the addition of different geometry and type of fibres to concrete has various effects on the ultimate capacity of the concrete in compression. Generally, polypropylene fibre and Fibremesh are the same type of material and different type of geometry. But from figure 5.5, it indicates that a decreasing trend of average compressive strength occurs to these fibre-reinforced concrete after the fibre volume dosage rate exceed more than 0.5%. However, steel fibre reinforced concrete tends to have an increasing trend when the fibre volume dosage rate increased.

From figure 5.5, it shows that the compressive strength of the all fibre-reinforced concrete was near to each other when the fibre volume dosage rate was 0.5%, as the average compressive strength for all fibre-reinforced concrete was around 46 MPa. Additional information shows that the polypropylene fibre and Fibremesh has higher increased in compressive strength than steel fibre. This indicates that 0.5% fibre volume dosage rate was possible the best and economical dosage rate applied into a structure.

When 1.0% fibre volume dosage rate was applied to the concrete, the difference of the compressive strength of each type of fibre appeared gradually. Eventually, in 1.0% fibre volume dosage rate, the steel fibre improved the ultimate compressive strength, while some reduce in strength was developed for polypropylene fibre and Fibremesh. Steel fibre has slightly increased in the compressive strength (1.4MPa), but polypropylene fibre and Fibremesh has a great decrease on compressive strength (5.7MPa for polypropylene fibre and 7.7MPa for Fibremesh).

The difference of the compressive strength among all type of fibre reinforced concrete was obvious when the fibre volume dosage rate was 1.5%. The decreasing strength for Fibremesh was critical, where it decreased 9MPa from 1.0% volume dosage rate. Polypropylene fibre has slightly decreased in compressive strength, which it was about 1.3MPa reduced in strength. However, the steel fibre at this volume dosage rate greatly increased in strength from 1.0% volume dosage rate, which approximately 3.2MPa increased.

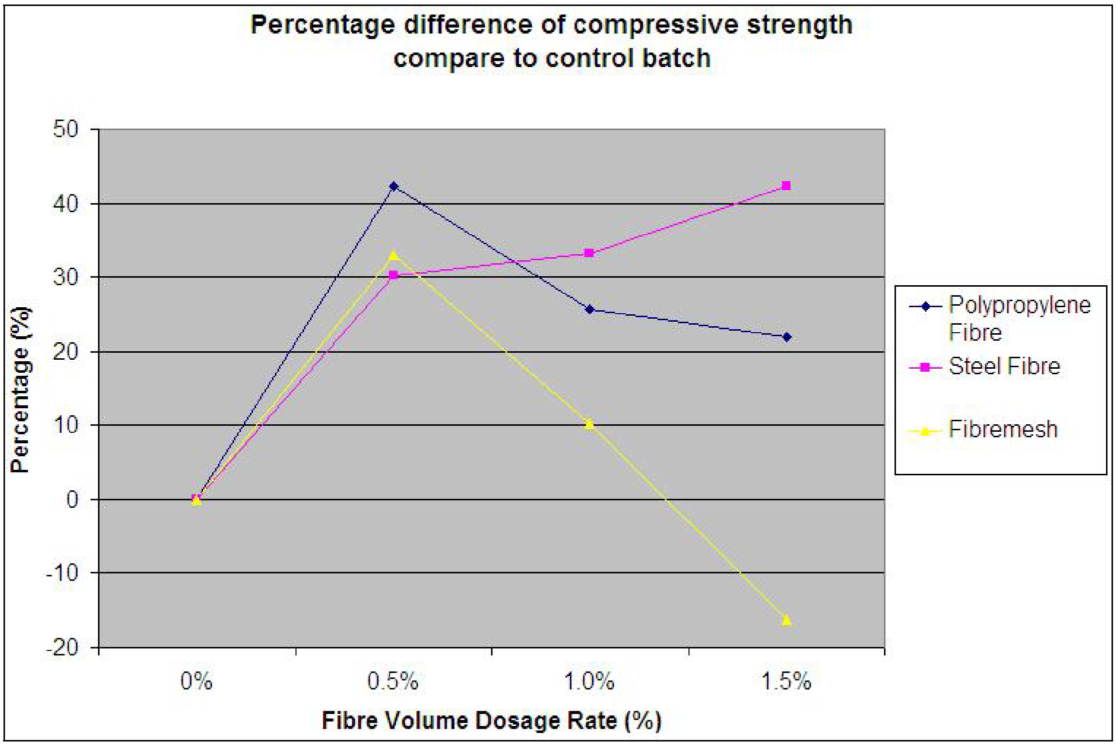


Figure 5.6: Percentage difference of compressive strength
vs. fibre volume dosage rate of each batch.

Figure 5.6 shows the percentage differences of compressive strength of different type of fibre reinforced concrete to the control batch. The comparison of percentage on figure 5.6 has a similar trend to figure 5.5 among these three types of fibre reinforced concrete, as it clearly shows the variations and effects of the compressive strength. It is vital that these comparisons were useful information for manufacture and designer to acquire the required volume dosage rate to their structure and design.

Almost all the fibre has increasing strength when 0.5% fibre volume dosage rate was applied into the concrete. Taking control batch as the independent comparison, the steel fibre increased 22%, polypropylene fibre increased 43% and Fibremesh increased 33% of their own compressive strength. As in figure 5.5 shows that the variations of all fibre reinforced concrete do not have many differences in compressive strength, but in figure 5.6 clearly show the difference among them.

The percentage difference of 1.0% and 1.5% fibre volume dosage rate to control batch greatly increased among all fibre reinforced concrete. For 1.0% fibre volume dosage rate, the steel fibre increased 33%, polypropylene fibre increased 26% and Fibremesh increased 10% of compressive strength compare to control batch. Steel fibre increased approximately 11% of strength, while polypropylene fibre and Fibremesh decreased about 17% and 23% of strength from 0.5% fibre volume dosage rate. However, the variations appears clearly on 1.5% fibre volume dosage rate, where the steel fibre increased 42%, polypropylene fibre increased 22%, but Fibremesh decreased 16% of compressive strength compare to control batch. From 1.0% to 1.5% fibre volume dosage rate, Fibremesh decreased the most, as approximately 26% difference in percentage of compressive strength. Polypropylene fibre has slightly decreased around 4%. For steel fibre, the percentage of compressive strength increased greatly, where 9% of compressive strength difference from 1.0% to 1.5% fibre volume dosage rate.

Generally, a lower workability concrete mix tends to provide a higher strength concrete. However, after evaluated the compressive test, it indicates there is no relationship between the additions of fibres for the compressive strength to the workability of each concrete mix. Hence, the ultimate compressive strength for all fibre reinforced concrete does not depend on their workability. It shows that the factors control for this parameter was the geometry and type of fibres used in the concrete and the cement adhesion or chemical reaction between the fibres. Especially for Fibremesh, such high volume of percentage of fibres will prevent the concrete from being adequately mixed.

5.6 Indirect Tensile Strength Test

The tensile strength of fibre reinforced concrete increased in indirect tensile strength test when the percentage of fibre increased. Table 5.5 below shows the average indirect tensile strength recorded during the test and the strength difference in percentage for all mix batches compared to control batch. Figure 5.7 below shows a graphical representation of average indirect tensile strength for concrete containing no fibres and concrete containing different amounts and types of fibres.

Table 5.5: Average indirect tensile strength and percentage difference compare to control batch for each batch.

Type of mix batch	Average indirect tensile strength (MPa)	Percentage difference (%)
Control (0%)	3.0	0
0.5% Polypropylene Fibre	3.9	32
1.0% Polypropylene Fibre	4.0	34
1.5% Polypropylene Fibre	4.2	42
0.5% Steel Fibre	3.8	28
1.0% Steel Fibre	4.9	66
1.5% Steel Fibre	5.5	86
0.5% Fibremesh	3.7	26
1.0% Fibremesh	3.9	33
1.5% Fibremesh	4.1	40

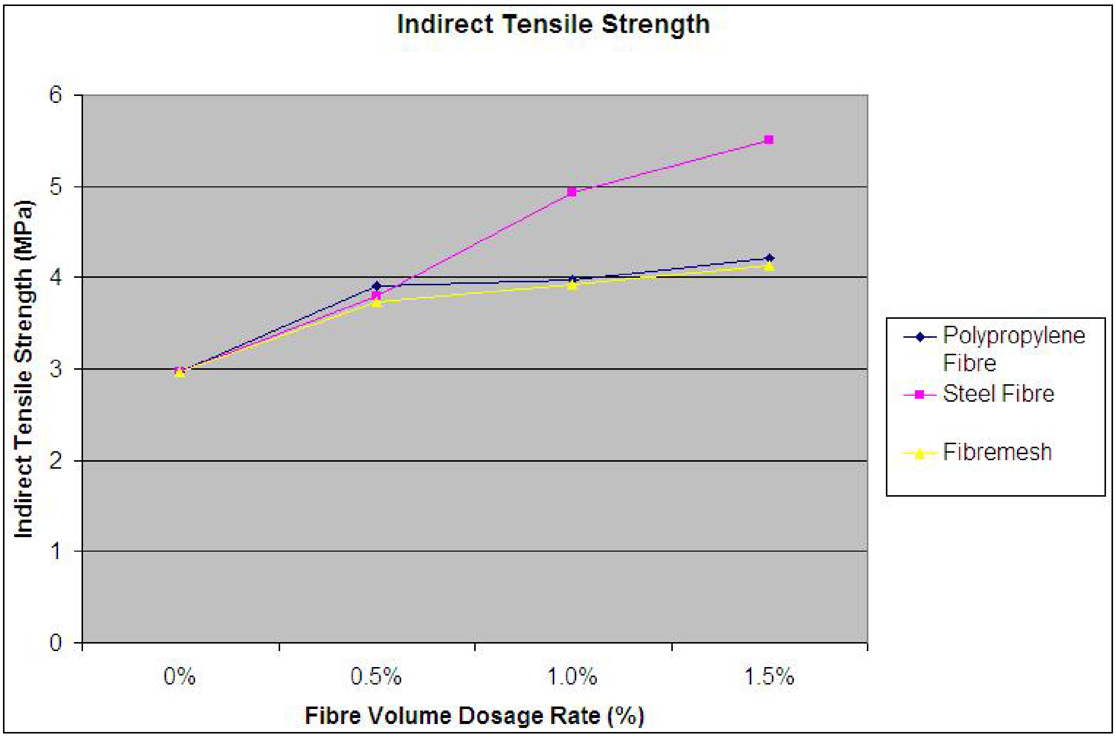


Figure 5.7: Average indirect tensile strength vs. fibre volume dosage rate of each batch.

Figure 5.7 showed that the indirect tensile test results have an increasing trend of average tensile strength for all type of fibre reinforced concrete when the fibre volume dosage rate increased. It shows that the increasing trend of tensile strength is different from the compressive strength's trend. This increase in tensile strength was due to the nature of binding of fibre available in concrete. When the reinforced concrete was force to split apart in the tensile strength test, the load was transferred into the fibres as pullout behaviour when the concrete matrix began to crack where it exceeded the pre-crack state. The control batch specimens containing no fibres failed suddenly once the concrete cracked, while the fibre reinforced concrete specimens were still intact together. Figure 5.7 shows the difference of indirect tensile strength of concrete specimens containing no fibres and with fibres. This shows that the fibre reinforced concrete has the ability to absorb energy in the post-cracking state.

When the fibre volume dosage rate was 0.5%, it shows that there was increased of tensile strength for all fibre reinforced concrete. Similar to compressive strength test, the tensile strength at 0.5% fibre volume dosage rate was close to each other, as the average tensile strength was 3.8MPa for all fibre reinforced concrete. The figure 5.7 indicates that 0.5% fibre volume dosage rate was not strong enough in tensile strength compare to 1.0% and 1.5% fibre volume dosage rate. But for economy situation, 0.5% fibre volume dosage rate may adopt to be the best dosage in structural.

However, the difference of the tensile strength appeared greatly when the fibre volume dosage rate was 1.0% and 1.5%. It shows that the tensile strength of polypropylene fibre and Fibremesh reinforced concrete specimens was almost similar and has a gradually strength rate increased when the dosage rate increased, as it was around 2MPa per 0.5% volume dosage rate, where the average tensile strength of these two were 4.0MPa and 4.2MPa for 1.0% and 1.5% fibre volume dosage rate. Eventually, steel fibre reinforced concrete specimens were greatly increased as the increasing of fibre volume dosage rate. Approximately 2.0MPa and 2.5MPa of tensile strength was increased for 1.0% and 1.5% fibre volume dosage rate compare to control batch.

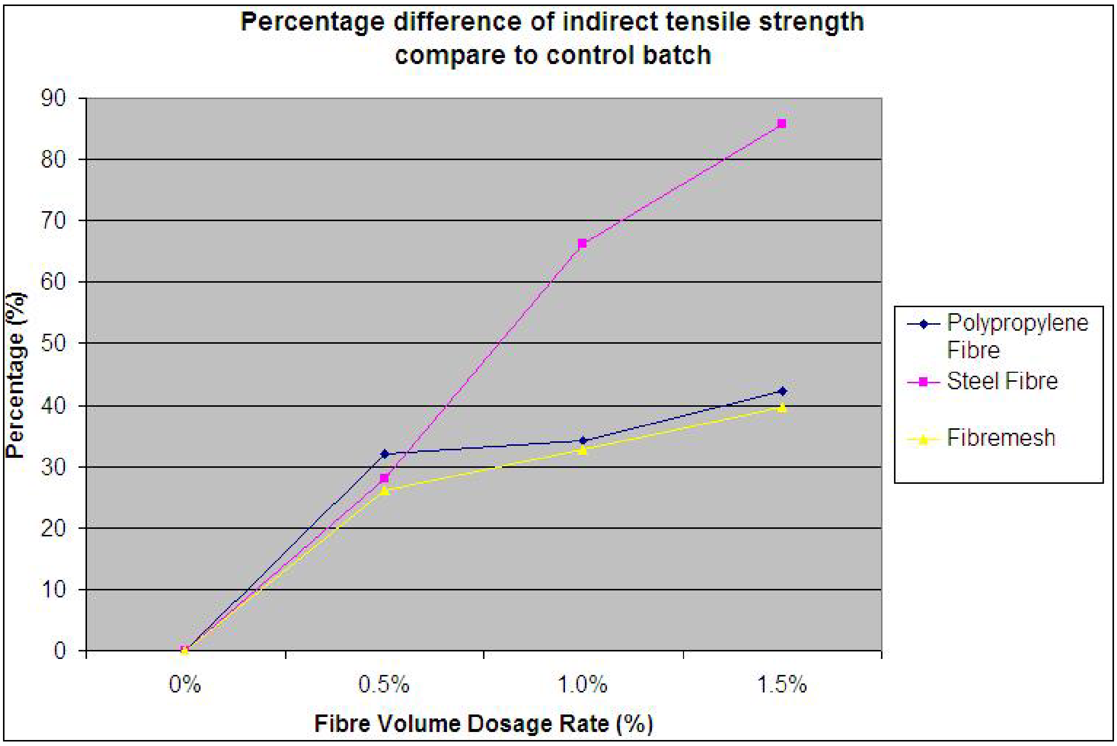


Figure 5.8: Percentage difference of indirect tensile strength vs. fibre volume dosage rate of each batch.

Figure 5.8 illustrated the comparison of percentage difference in indirect tensile strength for each fibre reinforced concrete to the control batch. The percentage comparison trend of tensile strength for all fibre reinforced concrete against control batch on figure 5.8 and figure 5.7 was similar. Here, the difference among the fibre reinforced concrete was clear, as control batch was taken as an independent reference. The figure shows that the most variation of the percentage increased was steel fibre reinforced concrete when the fibre volume dosage rate was 1.0% and 1.5%.

When the 0.5% fibre volume dosage rate was applied into the concrete, the tensile strength increased was approximately 29%. The polypropylene fibre was around 32% where it is the highest increased percentage in this dosage rate. Steel fibre increased 28% and Fibremesh increased 26%. The percentage difference among these three fibre reinforced concrete was approximately 3%.

In 1.0% fibre volume dosage rate, the trend of percentage of tensile strength increase for steel fibre reinforced concrete to control batch was great, where it was 66% increased. Polypropylene fibre and Fibremesh were gradually increased in the percentage of tensile strength where it was approximately 33% increased. Such percentage difference among these two fibre reinforced concrete to steel fibre reinforced concrete was 33% of gap.

Similar to 1.0% fibre volume dosage rate, the steel fibre increased the most percentage of tensile strength compare to control batch in 1.5% fibre volume dosage rate, where it was approximately 86% increased. This shows the steel fibre reinforced concrete has almost one times higher in the tensile strength compare to control batch concrete, as the energy absorption was great in post-crack state. Polypropylene fibre increased 42% and Fibremesh increased 40%. The tensile strength of these two fibre reinforced concrete was about 0.5 times capable to receive more forces after cracks were developed on the concrete specimens.

In Australian Standard AS3600 stated that the tensile strength of the concrete has a relationship to the compressive strength. However, this theory cannot apply to fibre reinforced concrete. Evidence shown by comparing to the compressive strength test and tensile strength test, the strength trend between these two tests do not match together. It shows that the factor may affect the tensile strength of fibre reinforced concrete was the yield strength and modulus of elasticity of these fibres itself. Figure 5.9 below shows the specimen after the test. The test shows that fibre reinforced concrete was still intact together.

(a) With fibre reinforced.

(b) Plain Concrete.

Figure 5.9: Specimens after the indirect tensile test.

5.7 Flexural Strength Test

The flexural strength trend on all fibres varies when the percentage of fibre increased. Table 5.6 below shows the average flexural strength recorded during the test and the strength difference in percentage for all mix batches compared to control batch. Figure 5.10 below shows a graphical representation of slump height for concrete containing no fibres and concrete containing different amounts and types of fibres. The behaviour of post-cracking state of fibre reinforced concrete was also discussed.

Table 5.6: Average flexural strength and percentage difference compare to control batch for each batch.

Type of mix batch	Average flexural strength (MPa)	Percentage difference (%)
Control (0%)	3.9	0
0.5% Polypropylene Fibre	5.1	32
1.0% Polypropylene Fibre	5.1	34
1.5% Polypropylene Fibre	4.4	14
0.5% Steel Fibre	4.5	17
1.0% Steel Fibre	5.2	34
1.5% Steel Fibre	6.6	72
0.5% Fibremesh	5.5	42
1.0% Fibremesh	5.1	31
1.5% Fibremesh	4.9	28

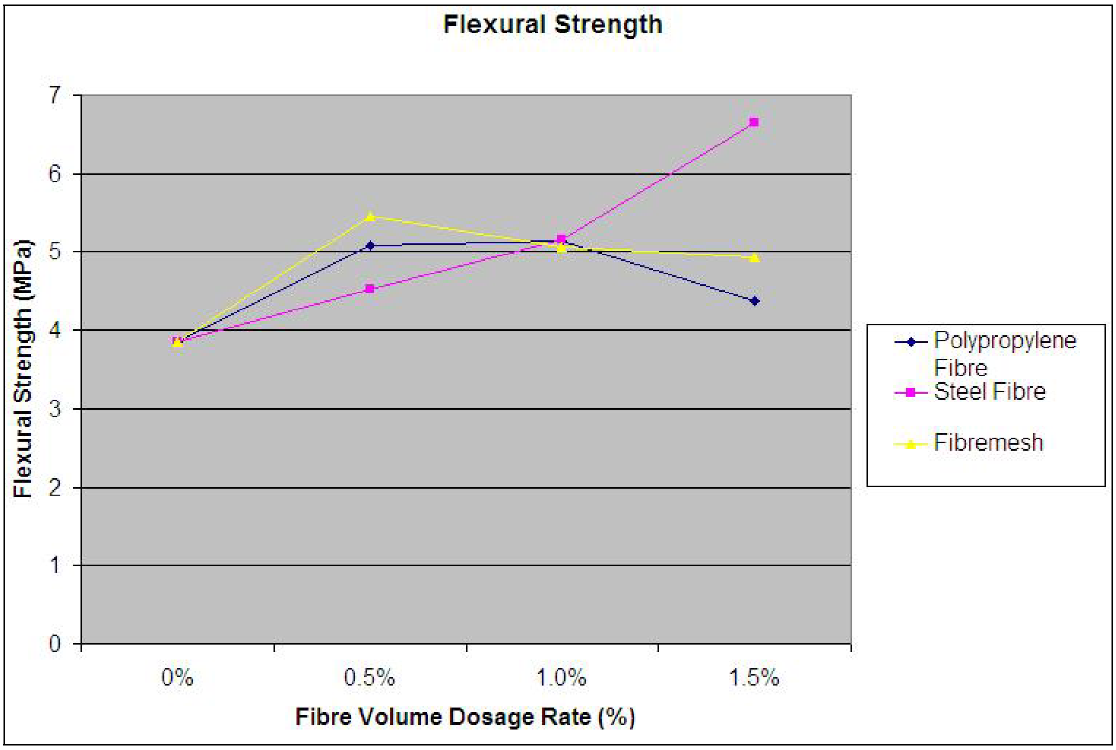


Figure 5.10: Average flexural strength vs. fibre volume dosage rate of each batch.

The figure 5.10 showed the average modulus of rupture of the concrete corresponding to the increase in the amount of fibres applied to the concrete. It indicated that increases of flexural strength were occurred as the fibre volume dosage rate was increased. Furthermore, it shows the general upward trend in the flexural strength for all fibre reinforced concrete. The concrete specimens containing no fibres were cracked and failed in brittle condition when it reached the ultimate strain in the concrete. However, fibre reinforced concrete also cracked at ultimate strain, but it is capable to carry the load well after the crack developed on the concrete. This indicates that the fibre reinforced concrete has the ability to hold on the crack of the concrete and preventing the concrete beam to fall apart. Figure 5.12 below in this section showed the crack developed for concrete containing no fibres and fibre reinforced concrete. The test showed that the fibre-reinforced beams were still intact together.

The results of 0.5% fibre volume dosage rate for flexural strength test were similar to compressive strength test and indirect tensile strength test. When fibre was added into the concrete, it shows that the concrete has the ability to carry more strength compare to control batch concrete. But the flexural strength of all fibre reinforced concrete was far from each other. This flexural test indicates that Fibremesh reinforced concrete has an initial strength gain when the volume dosage rate was 0.5%. However, steel fibre reinforced concrete was less compare to other fibre reinforced concrete. Flexural strength on steel fibre reinforced concrete was 4.52MPa, while polypropylene fibre and Fibremesh was 5.08MPa and 5.46MPa. Hence, the increase in flexural strength for steel fibre reinforced concrete specimens to control batch specimens was approximately 0.67MPa, while polypropylene fibre was 1.23MPa and Fibremesh was 1.61MPa.

A decreasing trend of flexural strength happens for polypropylene fibre and Fibremesh when the fibre volume dosage rate exceeded 0.5%. But, an increasing trend of flexural strength developed for steel fibre from 0.5% fibre volume dosage rate. However, most of all fibre reinforced concrete tends to have similar value of the flexural strength occurs on 1.0% fibre volume dosage rate, where an average of 5.12MPa increased from control batch specimens for all fibre reinforced concrete. The difference of flexural strength of all fibre reinforced concrete does not have large variation, where it is approximately 0.06MPa.

In 1.5% fibre volume dosage condition, steel fibre largely increased in flexural strength, where the flexural strength was about 6.64MPa and 2.79MPa increased from control batch. However, polypropylene fibre and Fibremesh have slightly reduced in flexural strength for 1.5% volume dosage rate, where polypropylene fibre was 4.38MPa and Fibremesh was 4.94MPa. Such strength increased for polypropylene fibre and Fibremesh were 0.53MPa and 1.09MPa from control batch.

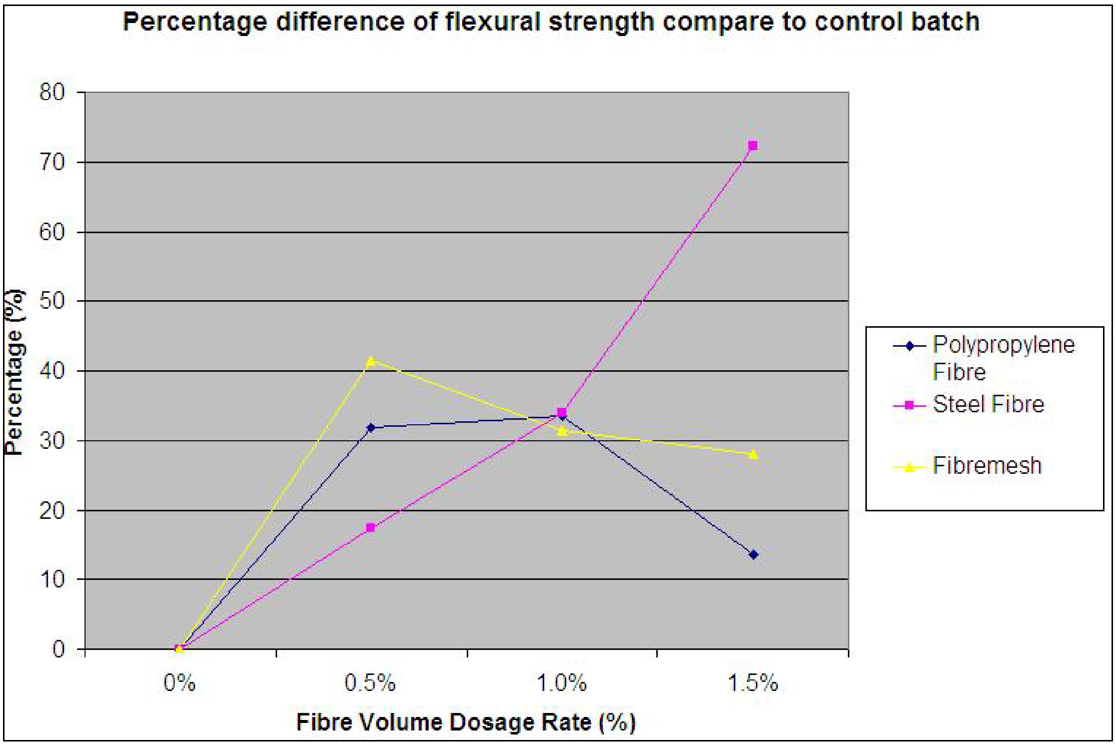


Figure 5.11: Percentage differences of flexural strength vs. fibre volume dosage rate for each batch.

Figure 5.11 illustrated the comparison of percentage difference of flexural strength for each fibre reinforced concrete to the control batch. This shows that the percentage comparison trend of flexural strength for all fibre reinforced concrete against control batch on figure 5.10 and figure 5.11 was similar. The figure above indicates that steel fibre have an increased of percentage in flexural strength where it is similar to compressive strength test and tensile strength test. However, polypropylene fibre and Fibremesh had percentage strength increased only for 0.5% fibre volume dosage rate, while it gradually decreased when the fibre volume dosage rate was 1.0% and 1.5%.

The difference of percentage of flexural strength in 0.5% fibre volume dosage rate for all fibre reinforced concrete was 17% (steel fibre), 32% (polypropylene fibre) and

42% (Fibremesh). It showed that Fibremesh concrete specimens were 0.5 times higher in flexural strength compare to control batch concrete specimens. The amount of percentage differences in flexural strength for 0.5% volume dosage rate of concrete specimens compare to control batch was average of 31%.

There was not much difference of percentage of flexural strength compare to control batch when the fibre volume dosage rate was 1.0% for all fibre reinforced concrete. All fibre reinforced concrete has an average of 33% differences of flexural strength compare to control batch. Steel fibre increased about 34%; polypropylene fibre increased around 34%, while Fibremesh increased about 31% from control batch. This showed that 1.0% fibre volume dosage rate may be the best dosage rate can apply into the structural member for all fibres. The average percentage differences for 0.5% (31%) and 1.0% (33%) fibre volume dosage rate was very near to each other.

Great variations of percentage differences of flexural strength appeared when the fibre volume dosage rate was 1.5%. The most significant percentage difference was steel fibre where it was 72% compare to control batch. This indicated that steel fibre concrete specimens have approximately 1 times higher in flexural strength to control batch specimens when volume dosage rate was 1.5%. Fibremesh gradually decreased where it was 28% of flexural strength difference. However, polypropylene fibre was the least percentage increased of flexural strength in the entire fibre volume dosage rate and all type of fibre reinforced concrete, where it was 14% of flexural strength increased compare to control batch.

Control batch concrete beam.

Fibremesh fibre reinforced concrete beam.

Polypropylene fibre reinforced concrete beam.

Steel fibre reinforced concrete beam.

Figure 5.12: Concrete beam specimen after the tests.

(a) **Fibremesh fibre concrete beam**.
Hairline cracks developed after the test.
(1.5% Fibremesh volume dosage rate)

(b) **Polypropylene fibre reinforced concrete beam**.
Polypropylene fibres tends to pull out of the concrete beam (The figure shows the bottom view of the concrete beam with 1.5% polypropylene fibre volume dosage rate).

(c) **Steel fibre reinforced concrete beam**.
The crack with a distance of approximately 10mm occur below the concrete beam.
(1.5% steel fibre volume dosage rate)

5.7.1 Post crack behaviour of same type of fibres

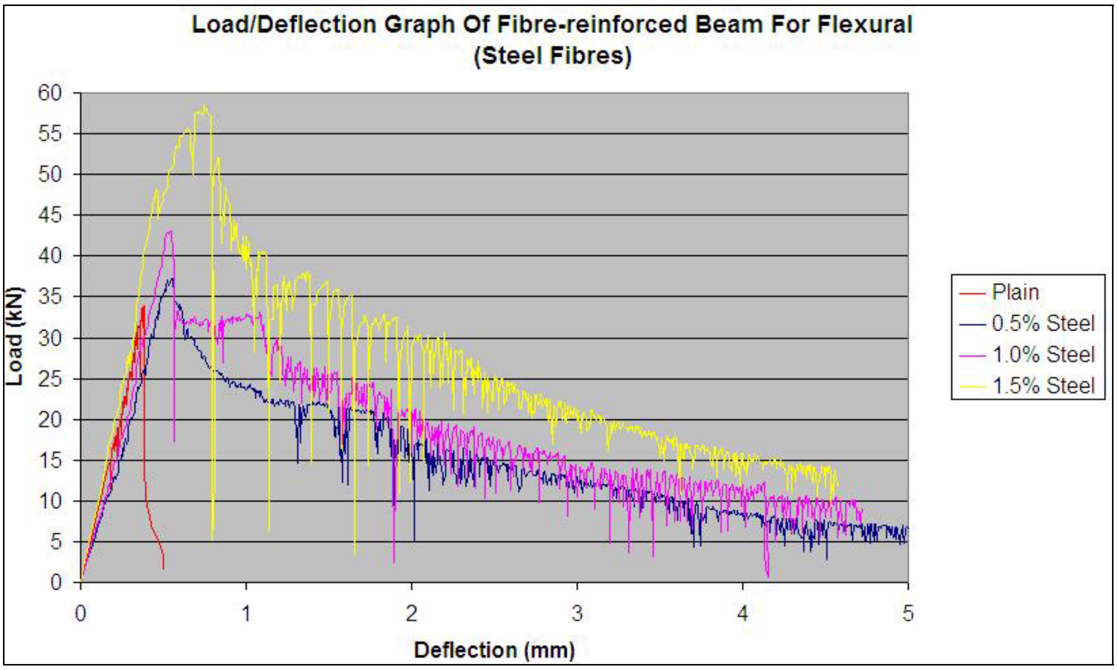


Figure 5.13: Load/deformation curves for concrete containing varying amounts of steel fibres.

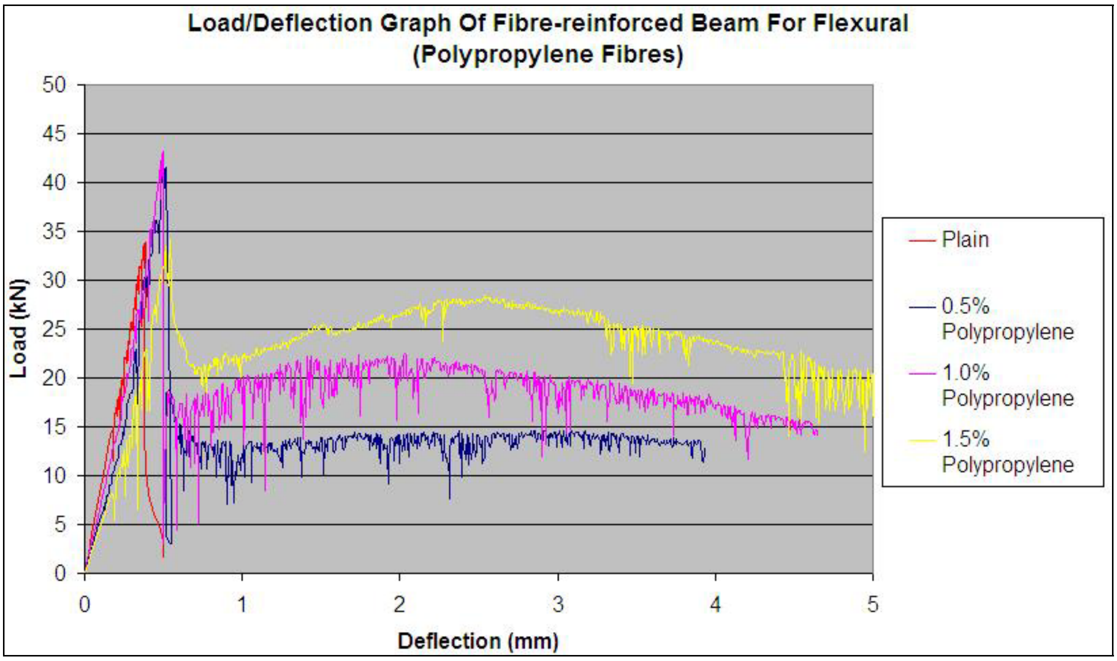


Figure 5.14: Load/deformation curves for concrete containing varying amounts of polypropylene fibres.

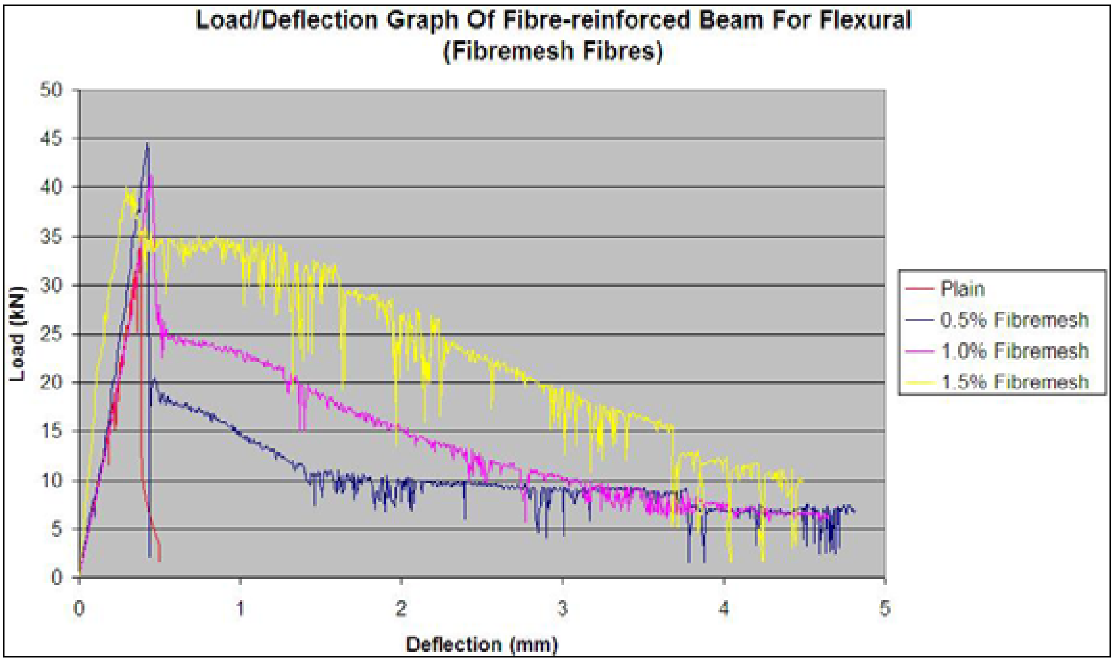


Figure 5.15: Load/deformation curves for concrete containing varying amounts of Fibremesh.

The fibre reinforced concrete not only shows their effect on the pre-cracking state, but it is vital on the post-cracking state and effects showed that an increase of ductility developed for fibre reinforced concrete after the first crack of concrete. Figure 5.13, 5.14 and 5.15 showed the load/deformation curve of different type of fibre reinforced beam where the each fibre volume dosage rate increased in flexural strength test. These figures tend to compare the effect of each type of fibre reinforced concrete when the fibre volume dosage rate was increased in the post-cracking state.

From the figures shown above, most of the strength was transferred to the fibre element in post-cracking state. Evidences showed that specimens containing no fibres could not sustain any load and fails suddenly when the first crack was developed. However, fibre reinforced concrete has the ability of energy absorption to the fibres after first crack appeared where it tends to hold the concrete beam together, without causing it to break into two parts. In figure 5.14 and 5.15, the strength initially dropped in a sudden and has approximately half of the peak load increased after when the crack appeared for polypropylene fibre and Fibremesh. Steel fibre did not have the initial strength dropped, but eventually the load was transferred to the fibre immediately, as this was shown in figure 5.13.

From all of the load/deformation curves (figure 5.13, 5.14 and 5.15), showed that the post-crack strength of all fibre reinforced concrete specimens were greatly increased as the increasing in fibre volume dosage rate. This showed that 1.5% fibre volume dosage rate applied into the concrete have the most post-crack energy absorption while in 0.5% fibre volume dosage rate, the energy absorption by the fibres were low.

As the deformation of the concrete beam increased, the load capacity of the fibres was decreased. From figure 5.13 and 5.15, the load transferred to steel fibre and Fibremesh gradually decreased where it is approximately 25% of the peak load. However, polypropylene fibre did not have this effect of load capacity decrease. From figure 5.14, the load capacity of polypropylene fibre tends to uniformly hold whenever the deformation of the concrete increased. This shows the pullout strength of polypropylene fibre was better compare to the other two types of fibres.

5.7.2 Post crack behaviour of same fibre volume dosage rate

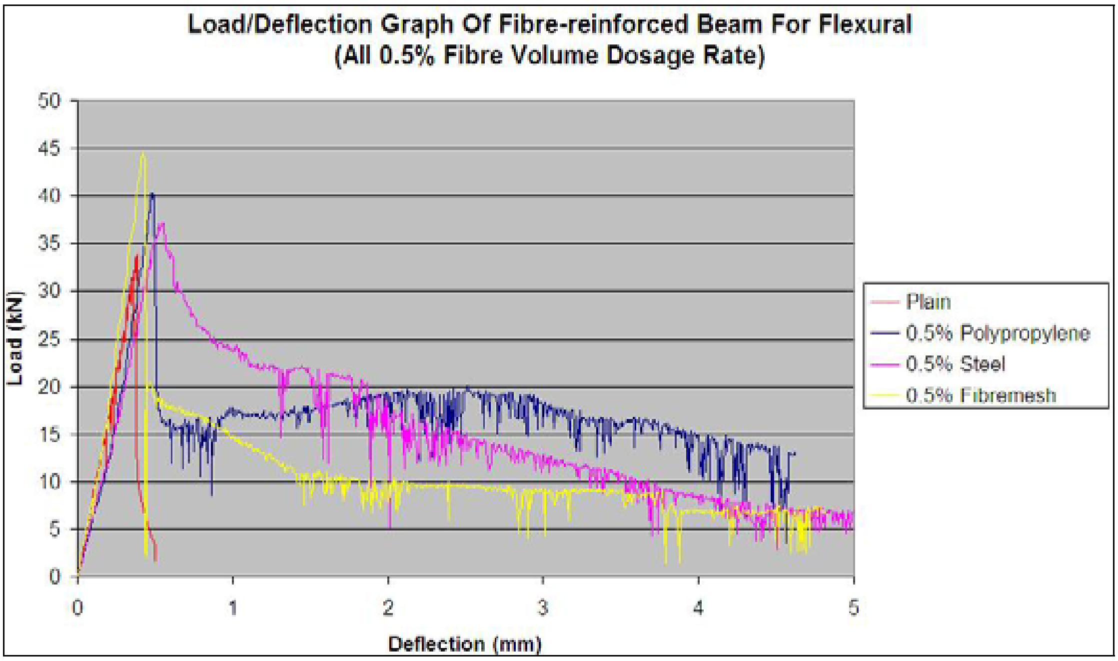


Figure 5.16: Load/deformation curves for concrete containing 0.5% fibre volume dosage rate of different type of fibres.

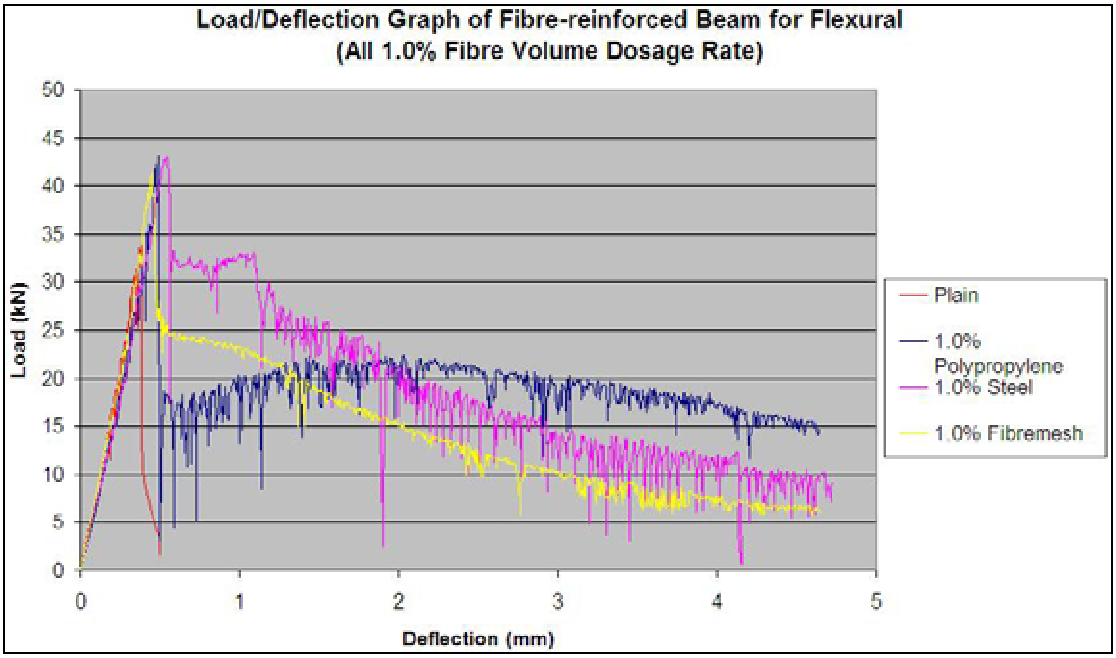


Figure 5.17: Load/deformation curves for concrete containing 1.0% fibre volume dosage rate of different type of fibres.

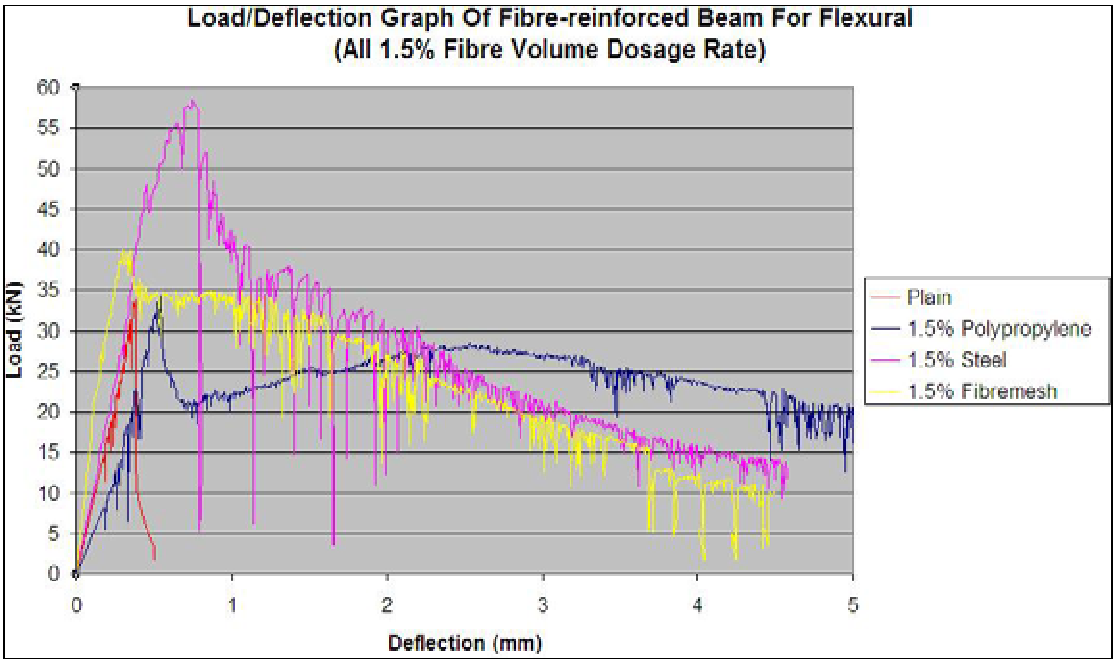


Figure 5.18: Load/deformation curves for concrete containing 1.5% fibre volume dosage rate of different type of fibres.

Similar to figure 5.13 to figure 5.15, figure 5.16 to figure 5.18 shows the load/deformation curve for all fibre reinforced concrete beam, but these figures compare the variations of the between the load capacity of fibres as the fibre volume dosage rate was remaining the same. It showed that load transferred after the first crack appeared to polypropylene fibre was initially less, but as the deformation of the beam increased, the polypropylene fibre tends to have higher and uniform load absorption compare to the other two fibres. However, the effect on steel fibre and Fibremesh was the reverse from the polypropylene fibre. The load capacity of steel fibre and Fibremesh gradually reduced when the deformation of the concrete beam increased.

From the results of flexural strength test, it shows that 1.0% fibre volume dosage rate was the suitable dosage rate applied to the concrete. The factor that influences the energy absorption of fibre reinforced concrete was the efficiency length of the fibres. As a general overview, fibres do increase the flexural strength of concrete on the pre-cracking and post-cracking stage.

5.8 Modulus of Elasticity

The slope of stress-strain diagrams in elastic range for fibre reinforced concrete gives an indication of modulus of elasticity of each concrete specimen. Table 5.7 below shows the average modulus of elasticity recorded during the test and the modulus of elasticity difference in percentage for all mix batches compared to control batch. Figure 5.19 below shows a graphical representation of average modulus of elasticity for concrete containing no fibres and concrete containing different amounts and types of fibres. The behaviour of post-cracking state of fibre reinforced concrete was also discussed.

Table 5.7: Average modulus of elasticity and percentage difference compare to control batch for each batch.

Type of mix batch	Average modulus of elasticity (MPa)	Percentage difference (%)
Control (0%)	6992	0
0.5% Polypropylene Fibre	5504	-21
1.0% Polypropylene Fibre	6231	-11
1.5% Polypropylene Fibre	6522	-7
0.5% Steel Fibre	6519	-7
1.0% Steel Fibre	7248	4
1.5% Steel Fibre	7435	6
0.5% Fibremesh	7196	3
1.0% Fibremesh	7618	9
1.5% Fibremesh	7840	12

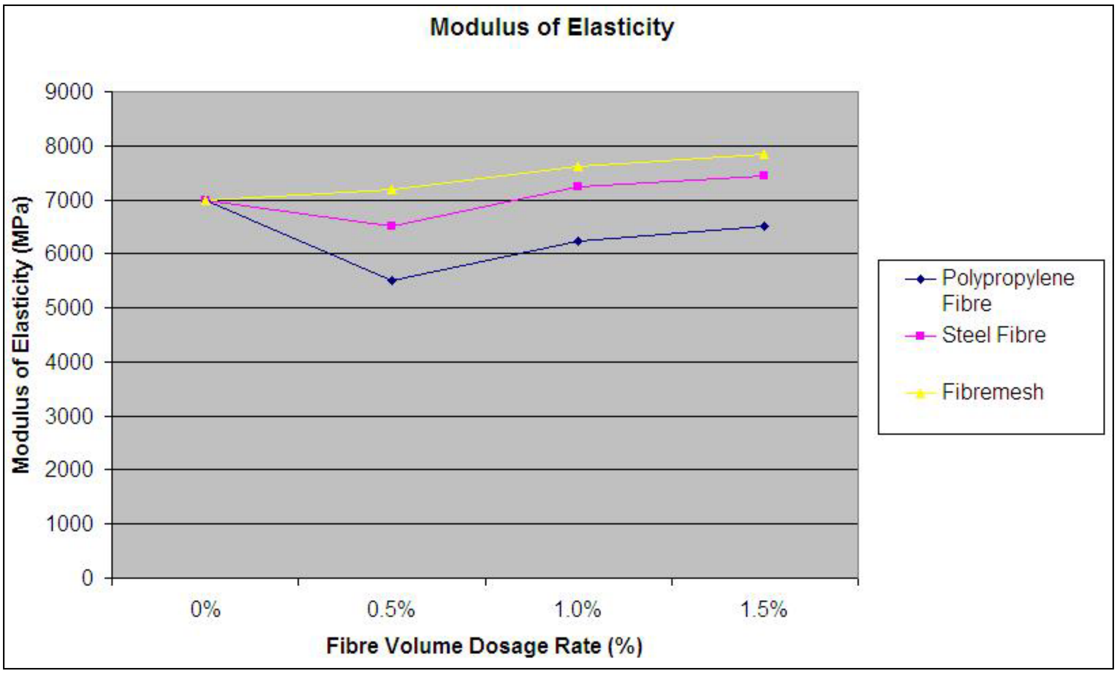


Figure 5.19: Average modulus of elasticity vs. fibre volume dosage rate of each batch.

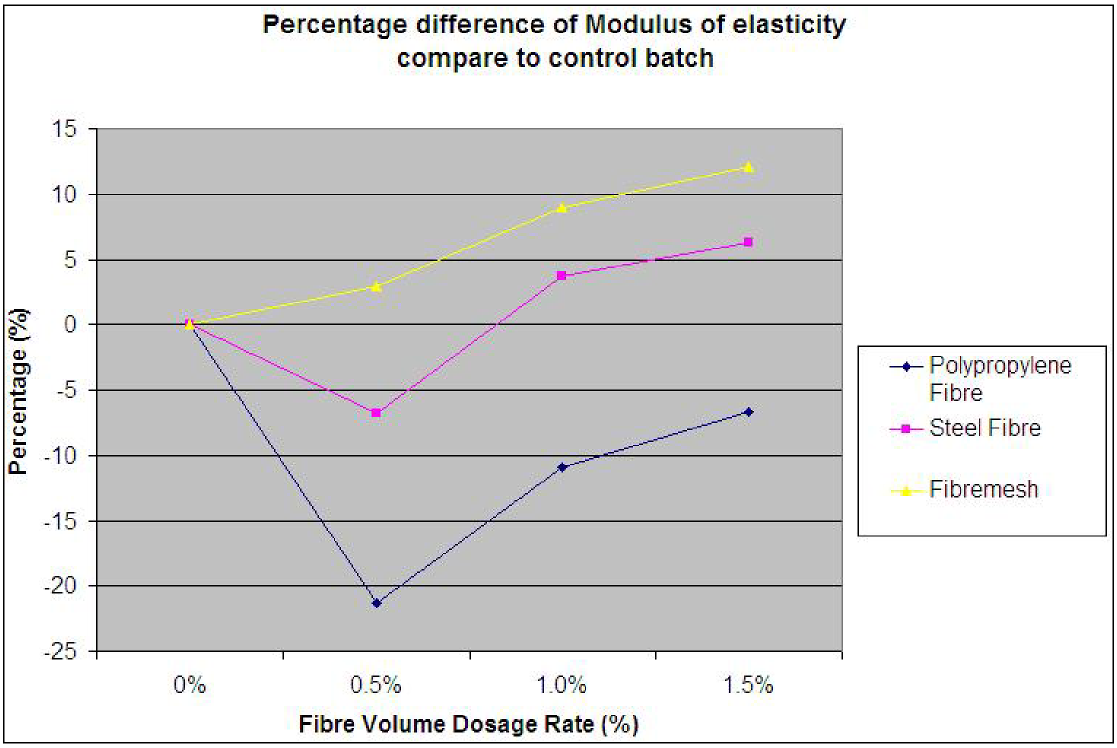


Figure 5.20: Percentage differences of modulus of elasticity vs. fibre volume dosage rate for each batch.

Figure 5.19 shows the comparison between the average modulus of elasticity for all fibre reinforced concrete. Figure 5.20 illustrated the percentage difference of modulus of elasticity of all fibre reinforced concrete compare to the control batch. Majority of the fibre reinforced concrete will have decrease of modulus of elasticity when the fibre volume dosage rate was 0.5% (except Fibremesh). However, gradually increasing trend appeared when the fibre volume dosage rate exceeded 0.5% for all fibre reinforced concrete. Previous experimental showed that the strength of the fibre reinforced concrete was increased whenever fibre was applied to the concrete. But, the experimental results here shows that the modulus of elasticity of fibre reinforced concrete does not need to be high when fibre was added. Polypropylene fibre reinforced concrete was the lowest among all the fibre reinforced concrete. The modulus of elasticity of the control batch was 6992MPa.

In 0.5% fibre volume dosage rate, steel fibre and polypropylene fibre has a reduce value of modulus of elasticity, where is was around 6519MPa and 5504MPa; while it was 6% and 21% reduce in modulus of elasticity from control batch specimen. Fibremesh have slightly increased, where it was approximately 7196MPa and 3% increase from control batch. The difference of modulus of elasticity from polypropylene fibre to Fibremesh was approximately average of 865MPa, where it was 12% of difference between these three fibre reinforced concrete.

When the fibre volume dosage rate was 1.0%, an increasing in modulus of elasticity was developed for all fibre reinforced concrete. Steel fibre was 7248MPa and polypropylene fibre was 6231MPa, about 6% increased and 11% reduced compare to control batch. Steel fibre and polypropylene fibre greatly increased where it was an average of 728MPa (11% increased compare to control batch) of modulus of elasticity. However, the increasing rate for Fibremesh was not so high from 0.5% volume dosage rate, where the value for modulus of elasticity was 7618MPa (9% increased compare to control batch and 6% increased from 0.5% fibre volume dosage rate).

As for 1.5% fibre volume dosage rate, all fibre reinforced concrete has slight increased in modulus of elasticity from 1.0% fibre volume dosage rate. An average of 233MPa and approximately of 5% of modulus of elasticity value increased for all fibre reinforced concrete from 1.0% to 1.5% fibre volume dosage rate. Steel fibre was 7435MPa; Fibremesh was 7840MPa and polypropylene fibre was 6522MPa, where it was 6% and 12% increased and 7% decreased compare to control batch.

Figure 5.21: Specimens after the modulus of elasticity test.

5.8.1 Post crack behaviour of same type of fibres

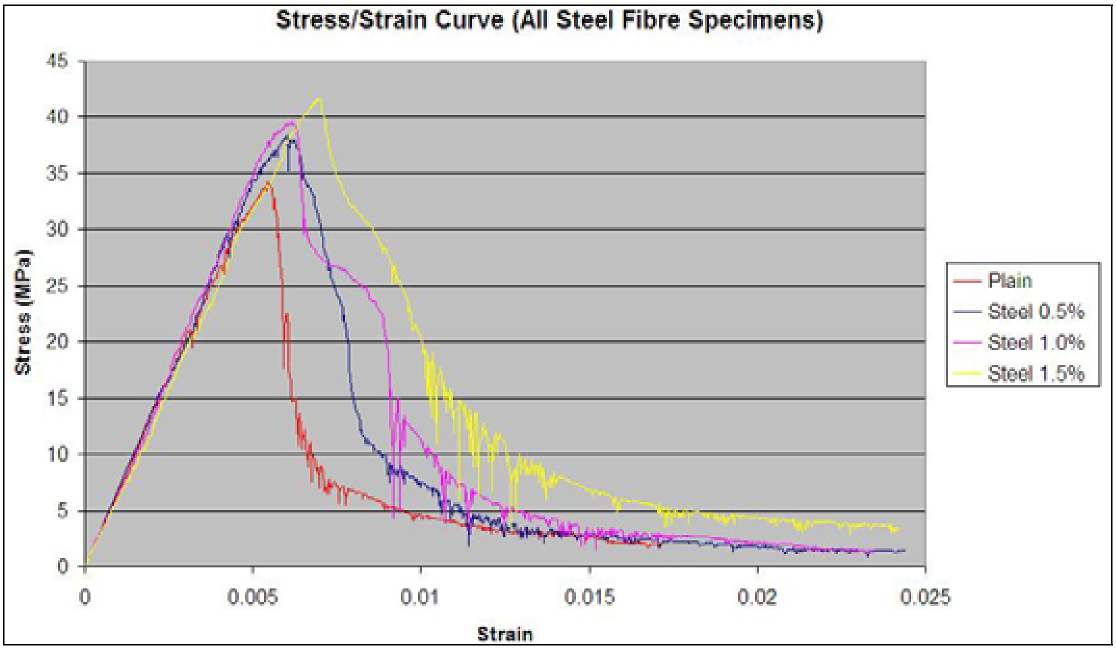


Figure 5.22: Stress/strain relationship curves for concrete containing varying amounts of steel fibres.

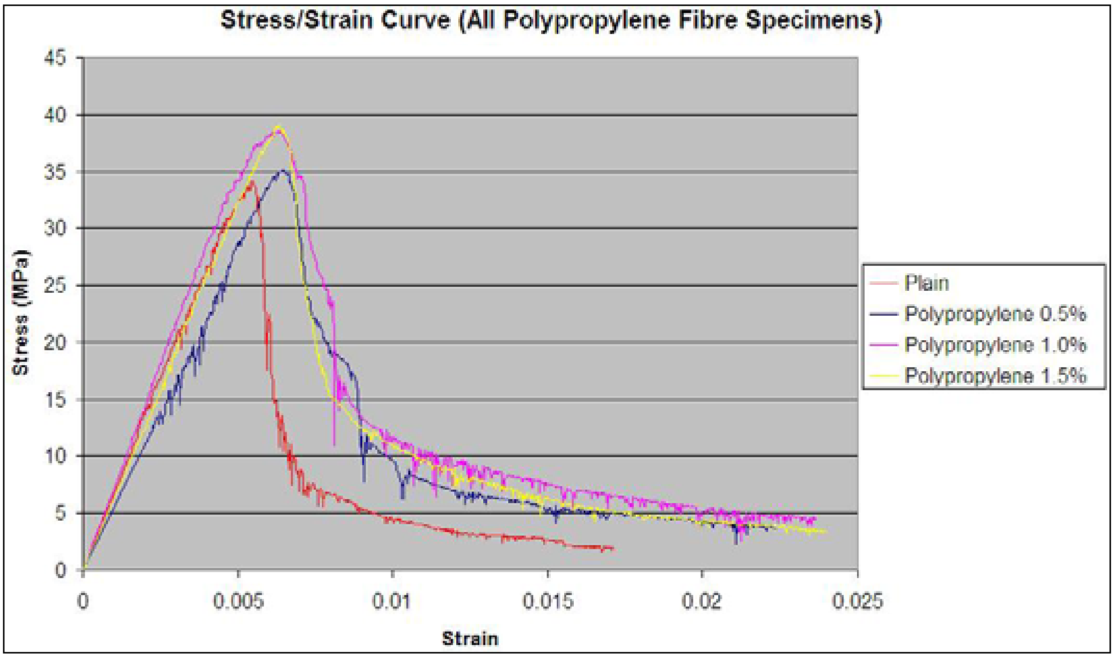


Figure 5.23: Stress/strain relationship curves for concrete containing varying amounts of polypropylene fibres.

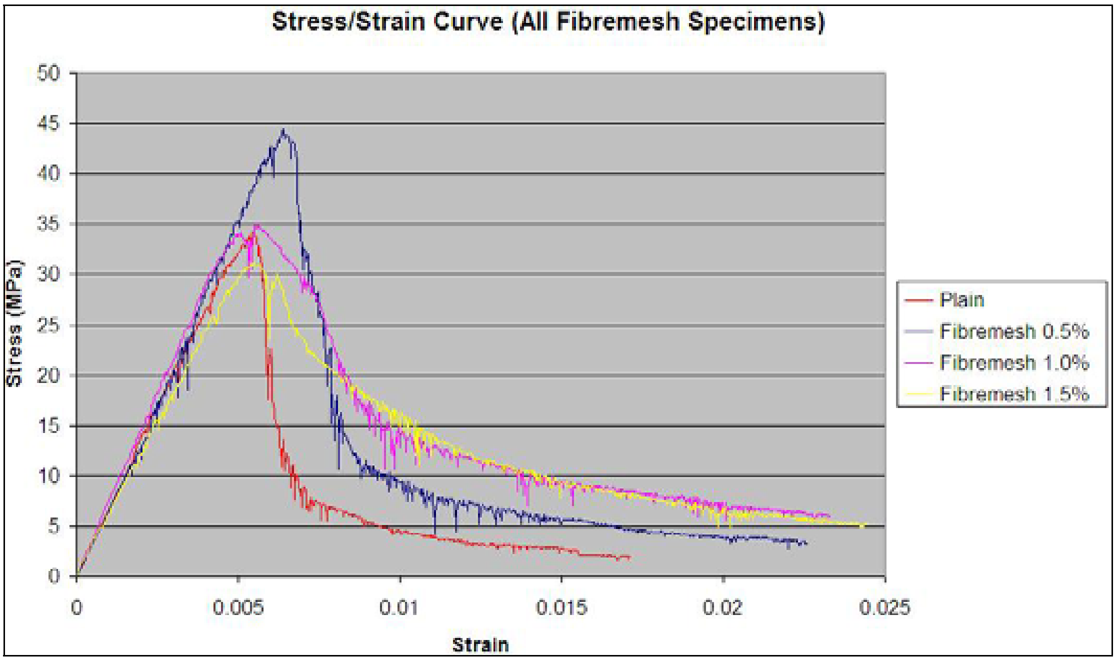


Figure 5.24: Stress/strain relationship curves for concrete containing varying amounts of Fibremesh.

Stress/strain relationship curves of same type of fibre reinforced concrete while increasing the fibre volume dosage rate were shown in figure 5.22, 5.23 and 5.24. It shows that a concrete specimen which is high in stress will tends to deformed more at the pre-cracking stage. Once the first crack appeared (post-cracking stage), the control concrete specimen failed quickly once the ultimate load had been reached and unable to sustain any extra load. However, the experimental test showed that fibre reinforced concrete specimen sustained the load all through the post peak stage. As the concrete specimen cracked and deformed, the fibres tends to hold together and bridged the cracks on the concrete. When the fibre began to give up and pull out, the load bearing capacity of the specimen decreased. The effects and appearance of the test specimen for no fibres and with fibres after the test was shown in figure 5.21 above. It indicates clearly that specimen containing no fibres fall off and failed in shear, while fibre reinforced concrete specimens were still intact.

Polypropylene fibre tends to have same decreasing slope in the post-cracking stage even though the fibre volume dosage rate varies. As steel fibre, the rate of decreasing of the slope was almost same, but the deformation was different when more amount of fibres contain in the concrete. However, decreasing slope of Fibremesh was different from the other two fibre reinforced concrete, the rate of decreasing slope reduced when the dosage rate increased.

5.8.2 Post crack behaviour of same fibre volume dosage rate

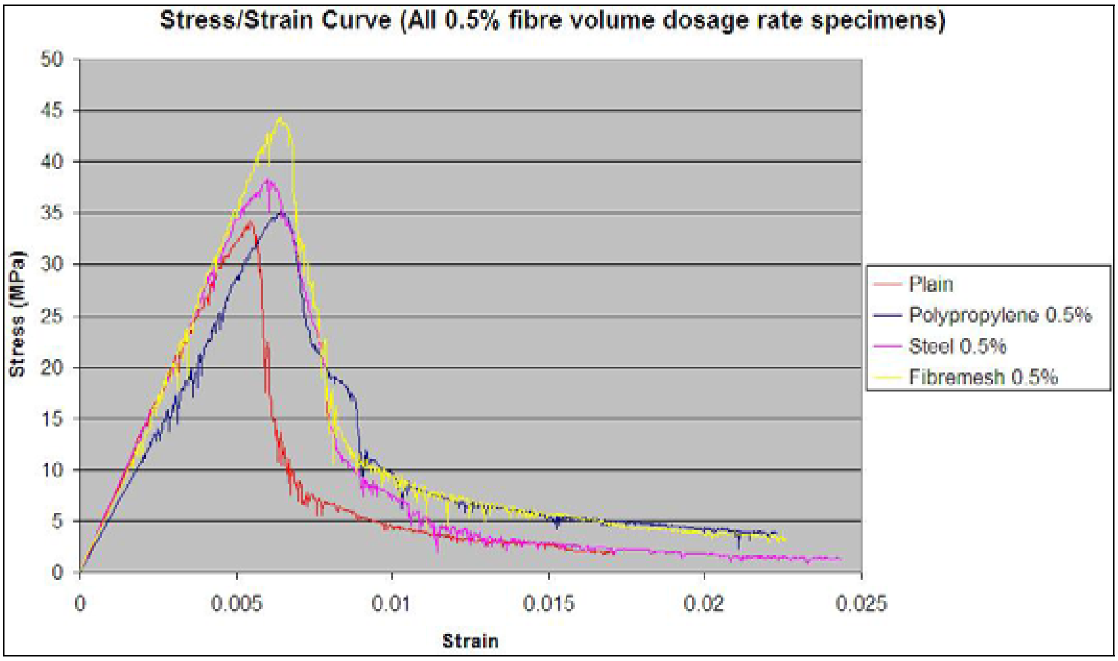


Figure 5.25: Stress/strain relationship curves for concrete containing 0.5% fibre volume dosage rate of different type of fibres.

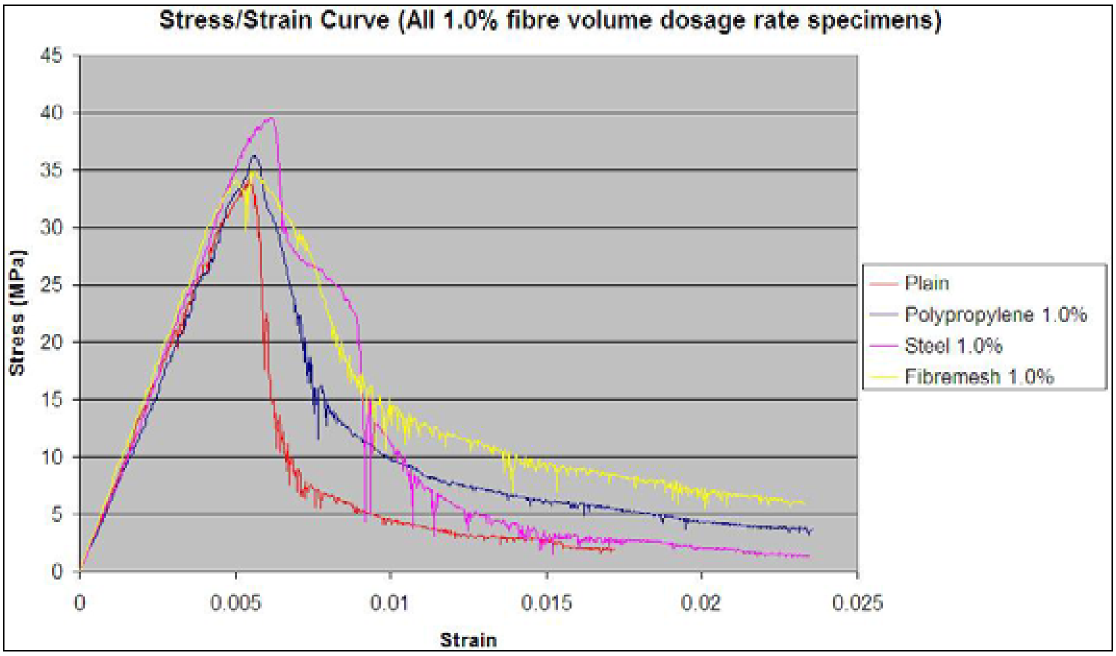


Figure 5.26: Stress/strain relationship curves for concrete containing 1.0% fibre volume dosage rate of different type of fibres.

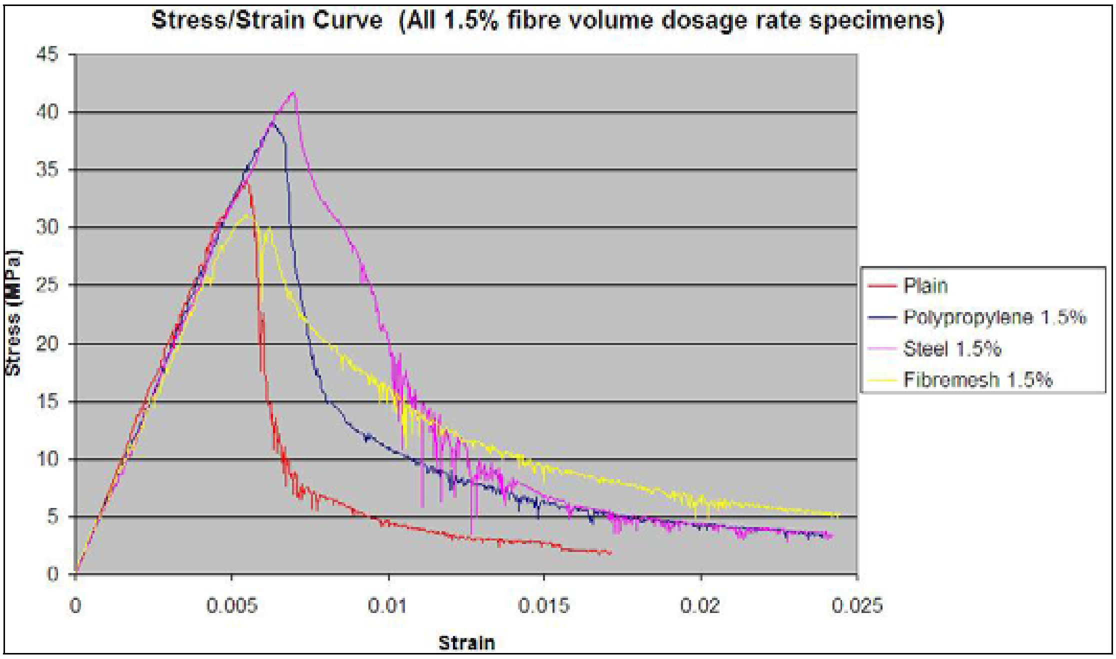


Figure 5.27: Stress/strain relationship curves for concrete containing 1.5% fibre volume dosage rate of different type of fibres.

Similar to figure 5.22 to 5.24, figure 5.25 to 5.27 shows the stress/strain curve for all fibre reinforced concrete specimen in modulus of elasticity test, but these figures compare the variations of the between the post-cracking behaviour of all concrete specimens (with and without fibres) as the fibre volume dosage rate was remaining the same.

From figure 5.25, when the fibre volume dosage rate was 0.5%, it shows that the ultimate stress of Fibremesh was higher than steel fibre and polypropylene fibre, where the ultimate strain of all fibre reinforced concrete was average of 0.005. The decreasing slope of all fibre reinforced concrete was almost similar to each other as the deformation of the specimen increased.

From figure 5.26, the gap of the decreasing slope in post-cracking stage for all fibre reinforced concrete at 1.0% fibre volume dosage rate seems to appear gradually different when the strain of the concrete specimen was increased. It shows that the rate of decreasing slope of Fibremesh was reduced, while rate of decreasing rate for steel fibre and polypropylene fibre were still high.

The difference of rate of decreasing slope in post-cracking stage for all fibre reinforced concrete was clearly have big variations when the fibre volume dosage rate was 1.5%. Fibremesh has the lowest decreasing rate among all fibre reinforced concrete, but it is capable to sustain extra load more than other type of fibre when the deformation of the concrete specimen increased.

Table 5.8: Average ultimate strain values for each mix batches' specimens.

Type of mix batch	Average ultimate strain ($x10^{-6}$)
Control (0%)	5380
0.5% Polypropylene Fibre	6560
1.0% Polypropylene Fibre	6350
1.5% Polypropylene Fibre	6370
0.5% Steel Fibre	6010
1.0% Steel Fibre	6230
1.5% Steel Fibre	7010
0.5% Fibremesh	6570
1.0% Fibremesh	5680
1.5% Fibremesh	5550

AS3600 stated that the ultimate strain for concrete was $3000x10^{-6}$. From the experimental results showed that all of the strain value of the concrete specimens exceeds $5000x10^{-6}$. This experimental test was done in a computerised condition, as the deformation of the concrete specimens was determined from the computer. This modulus of elasticity results was an indication for all concrete specimens. As deformation of the material was not occurred to the specimen itself, but eventually the steel platen of the test machine and sulphur capping on the specimen do have some influence on the deformation. But the computer still recorded these deformations where all of this will effect the deformation measurement of the specimen. Hence, the value for strain of all concrete specimens was not accurate. It is recommended that the modulus of elasticity test for fibre reinforced concrete follow the apparatus and test procedure according to AS1012.17, where the strain measurement core was used to determine the strain manually for all concrete specimens. The other alternative method is applying strain gauges on the concrete specimen while the test was done in computer-controlled test machine and ignores the deformation measurement from the previous computer. Hence, an accurate value of strain for concrete specimen can be obtained, where the strain value is less that ultimate strain value stated in AS3600. In this case, the modulus of elasticity for this test can be factor up and approximately near to the theoretical modulus of elasticity of the concrete (theoretical modulus of elasticity for concrete is 24,000MPa).

5.9 Summary

A summary can be drawn from the test results that the workability of concrete significantly reduced as the fibre dosage rate increases. Results of compressive strength test indicated that the use of fibre in concrete might not efficiently increase in strength. In flexural and indirect tensile test showed specimens with fibres that drastic increase in strength from specimens without fibres. A moderate increase in modulus of elasticity of the fibre reinforced concrete was indicated in modulus of elasticity test. The usage of fibres were fully utilised when it comes to post-cracking stage, as it increase on ductility and toughness of concrete.

CHAPTER 6

Conclusion and Recommendation

This chapter was set out to drawn conclusions on the workability and hardened properties test where it used to assess on the mechanical properties of fibre reinforced concrete. Moreover, the achievement of the project objectives (scopes) set in beginning of the project was also achieved. Lastly, recommendations for further studies were suggested with the usage of the short fibres in concrete to study more on its mechanical properties.

6.1 Achievement of Objectives (Scopes)

The project achievements are as follows:

- In this project, the review and research of current usage to the use of short fibres in the concrete was done into different sectors, such as constructions, industries, previous research and investigation.

- All ten batches of concrete mixes were done, which included the 3 different types of short fibres (steel fibres, polypropylene fibres and Fibremesh) with various fibre volumes (0.5%, 1.0% and 1.5%) and a plain (0%) concrete mix batch.

- Most importantly, this project found that the short fibres have large influence and effects on the mechanical properties of concrete, especially on the workability, indirect tensile strength, flexural strength, ductility, energy absorption capacity and the post-cracking state behaviour. It was found that the short fibres have moderately increased in the compressive strength and modulus of elasticity.

- By the completion of this project, considerable experience was acquired in the handling, application, testing of the specimens and analysing of the test result. In chapter 4, the compaction, levelling and curing of concrete specimens were discussed. The results are discussed and analysed in detail in chapter 5.

6.2 Conclusion

The research project undergo on the use of short fibres in structural concrete alter the mechanical properties of the concrete. It shows that effects of different types, geometry and shape of short fibres may involve and varies workability, compression resistant, tensile strength, flexural behaviour, elasticity and especially the post cracking behaviour such as ductility and energy absorption capacity.

As a result of the research, it was found that the use of fibre in the concrete decreases the workability of the fresh concrete. Evidence of low workability was shown through the results of workability test obtained in standard slump test, compacting factor test and VEBE consistometer test. It was concluded that the increasing percentage volume of fibre added into the concrete would lead the workability decreased. High percentage volume dosage rate above 1.0% showed that the concrete was significantly stiff and difficult to compact. Furthermore, the results obtained indicate that workability of the fresh fibre reinforced concrete eventually influenced by the shape and geometry of the fibres. Smaller shape and mat geometry fibres such as Fibremesh greatly reduced approximately more than 50% of the workability, while long wave cut wire shape and twisted wave shape fibres such as steel fibres and polypropylene fibres slightly reduced the workability of a fresh concrete. This is evident from the results obtained from the workability test, which performed in this research. The decrease of workability of a fresh fibre reinforced concrete could be lesser by adding chemical admixtures such as superplasticiser, silica fume or blast furnace slag. These can reduce the stiffening effect cause by the fibres on the fresh concrete while decreasing the amount of water used in the mix. The recommendation of using low setting admixtures would lead of high workable concrete mix, thus increasing the strength of the fibre reinforced concrete.

Compressive strength test significant shows that different geometry of short fibres affect the ultimate capacity of concrete in compression. It shows that 0.5% fibre volume dosage rate was the optimum dosage applied to concrete. Once the fibre volume dosage rate exceed 0.5% volume dosage rate, Fibremesh and polypropylene fibre reinforced concrete greatly reduce in compressive strength. However, steel fibre reinforced concrete increased it compressive capacity while the dosage rate increased. Again there was two factors affect this strength parameter: the geometry and the type of short fibres applied. In geometry factor, it shows that mat geometry of fibres tends to prevent the cement adhesion and chemical reaction among the concrete ingredients, which causes decreasing of compressive strength when the fibre volume dosage rate increases. This effect shows especially to Fibremesh. Even though the geometry of steel fibre and polypropylene fibre was similar, but the reason causes the increasing and decreasing of strength was based on the modulus of the fibres itself. The modulus of steel fibre is higher than polypropylene fibre, as it has the capability of resist more static load.

In indirect tensile strength test, all fibres show their full ability of increasing the tensile strength while holding the concrete and preventing the splitting itself. These effects are shown in figure 5.9. Furthermore, it shows that the increasing fibre volume dosage rate tends to lead the increasing a higher strength in tensile. However, the properties of the fibre itself were important. The results show that the tensile strength of same type of material such as polypropylene fibre and Fibremesh are same, while steel fibre was other type of material and the tensile strength extreme increase as dosage rate increase. The implication for AS3600 shows that the tensile strength of the concrete was related to compressive strength itself. However, such implication cannot apply to fibre reinforced concrete, as the consideration of fibre was not included.

The most remarkable changes in the increasing of strength by the use of short fibres to concrete occur in bending. These changes included the increasing of flexural strength in pre-cracking stage and ductility of the concrete in post-cracking stage. It shows that the increasing amount of short fibres increased the load bearing capacity of the concrete. As ultimate strain of the concrete reaches, the matrix begins to crack. Such cracks were bridge by the effects of the fibres. The load was transfer to the fibre by the effect of pullout and this behaviour continues well in the post-cracking stage, prevents the brittle failure of the concrete. This was shown in figure 5.13 to 5.18. As a result, the concrete member is capable to sustain much higher load at high deformations. It seems that the type of fibres influence the post-cracking behaviour of the concrete. Initially, such energy absorption capacity and toughness of concrete was excellence for steel fibre and Fibremesh, but decreasing slope of load/deformation curve in post-cracking stage was brought. However, polypropylene fibre decays in the beginning when the cracks developed, but it maintains the toughness of the concrete as the deformation increase.

It can be seen that the use of short fibre causes a moderate increase in modulus of elasticity. Figure 5.19 and 5.20 illustrated that such maximum increase of modulus of elasticity were less than 5% from concrete containing no fibres. But remarkable changes in post-cracking stage shows that fibre reinforced concrete largely increases in energy absorption capacity (ductility). Such evidence was shown in the stress/strain diagrams in figure 5.21 to 5.27. The slope of specimens containing no fibres in stress/strain diagram for post-cracking stage was less than the specimens with fibres. Furthermore, different type of fibre reinforced concrete tends to give various effects in the post-cracking stage. Fibremesh has high post peak stress while steel fibre does not exhibit much when the fibre volume dosage was low. However, as the fibre volume dosage rate increases, these two fibres have a change in ultimate stress. Steel fibre is eventually higher than Fibremesh. As for polypropylene fibre, there was no noticeable change. The results show that mat geometry of fibre tends to have more energy absorption capacity when the fibre volume dosage rate was low. But for higher fibre volume dosage rate, long wave cut wire shape and twisted wave shape fibres show a dramatic increase in ductility. Although this is an important property for design of the material, but the moderate increases by applications of fibre can be negligible. Hence, it would not affect the design of a concrete member.

The most drastic effect when the use of fibres in concrete was the increase of ductility in post-cracking stage, which performed both in modulus of elasticity and flexural strength test. Fibres have the ability to prevent cracks and allowing the concrete to carry load onwards after the point of first crack. The results of this research undertaken show that the use of short fibre has varying effects on the mechanical properties to the concrete. Furthermore, different type and geometry of fibres exhibit their true ability and usage to various effects on concrete. Thus, a full determination on the behaviour and mechanical properties of concrete which influences by the effects of fibre were required for further research, more efficient structures can be create at a cost that is viable.

6.3 Recommendation for further studies

Further investigations were highly recommended and should be carried out to understand more mechanical properties of fibre reinforced concrete. Several recommendations for further studies are mentioned below:

- The problem on the workability of the fresh fibre reinforced concrete can be reducing by adding chemical admixture such as superplasticiser, silica fume or blast furnace slag. Hence, with high workable fresh concrete can demote the quick stiffening effects from the fibres.

- More investigations and laboratory tests should be done to study on the mechanical properties of fibre reinforced concrete. Such application of fibres was recommended in testing on concrete slabs, beams and walls or conducting more tests such as abrasion, impact, blasting, shatter, shear or creeping of concrete.

- The combination of short fibres may tend to provide more efficient mechanical properties of structure. Further investigation can be carried out by combination of different types of short fibres into the concrete mix.

- To widen the use of fibre reinforced concrete, different or more complicated geometry of fibre can be used to investigate the effects of short fibres in the concrete through fresh and hardened properties.

- The mechanical properties of fibre reinforced concrete may be different in various temperatures. Test on freeze-thawing conditions were recommended.

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