UNIVERSITY OF GAZİANTEP
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CONTRIBUTION OF STEEL FIBER AND METAKAOLIN IN ENHANCING MECHANICAL BEHAVIOR OF HIGH STRENGTH CONCRETES

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Contribution of Steel Fiber and Metakaolin in Enhancing Mechanical Behavior of High Strength Concretes

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Assoc. Prof. Dr. Erhan GÜNEYİSİ

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Arass AKOI

ABSTRACT

Contribution of Steel Fiber and Metakaolin in Enhancing Mechanical Behavior of High Strength Concretes

AKOI, Arass Omer

M.Sc. in Civil Engineering

Supervisor: Assoc. Prof. Dr. Erhan GÜNEYİSİ

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This study reports the results of an experimental study on mechanical properties of plain and metakaolin (MK) concretes with and without steel fiber. To develop the metakaolin included concrete mixtures, Portland cement was partially replaced with MK as 10% by weight of the total binder content. Two types of hook ended steel fibers with length/aspect ratios of 60/80 and 30/40 were used to produce fiber reinforced concretes. Two series of concrete groups were designed with water to binder ratios (w/b) of 0.35 and 0.50. The combined effects of MK and different types of steel reinforcement on the compressive, flexural, splitting, and bonding strength of the concretes were investigated. All tests were conducted at the end of 28 days of curing period. Analyses of variance on the experimental results were carried out and the levels of the significance of the variables on the mechanical characteristics of the concretes were determined. Correlation between the measured parameters was also carried out to better understand the interaction between mechanical properties of the concretes. Moreover, the microstructure of plain and steel fiber reinforced concretes incorporated with MK was observed by scanning electron microscopy (SEM). The results indicated that incorporation of MK and use of different types of steel fibers significantly affected the mechanical behavior of the concretes, irrespective of w/b ratio.

Keywords: Bonding strength; Compressive strength concrete; Metakaolin; Steel fiber; Tensile strength

ÖZ

Yüksek dayanımlı betonların mekanik davranışının çelik lif ve metakaolin katkısıyla iyileştirilmesi

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Temmuz 2012, 74 sayfa

Bu çalışmada, çelik lif içeren ve içermeyen yalın ve metakaolin katkılı betonların mekanik özelliklerinin araştırıldığı deneysel bir çalışma sunulmuştur. Metakaolin (MK) katkılı betonların üretiminde, MK toplam bağlayıcı miktarının %10'u oranında çimento ile yer değiştirilerek kullanılmıştır. Çelik lif donatılı betonların üretiminde, iki tip kanca uçlu 60/80 ve 30/40 uzunluk/narinlik oranına sahip lifler kullanılmıştır. Su/bağlayıcı (s/b) oranı 0.35 ve 0.50 olan iki beton grubu tasarlanmıştır. Metakaolinin ve değişik tipte çelik tel donatıların betonun mekanik özellikleri üzerindeki etkilerini irdelemek amacıyla, basınç dayanımı, yarmada çekme dayanımı, üç noktalı eğilme dayanımı deneylerinin yanı sıra beton ile donatı arasındaki aderans dayanımı deneyleri gerçekleştirilmiştir. Bütün deneyler 28 günlük kür süresi sonunda yapılmıştır. Mekanik özellikler üzerindeki etkili parametrelerin belirlenmesine yönelik olarak istatistiksel bir yöntem olan genelleştirilmiş doğrusal model varyans analizi yapılmıştır. Daha sonra mekanik özellikler arasındaki etkileşimi belirlemek amacıyla korelasyon çalışması yapılmıştır. Ayrıca, yalın ve metakaolin içeren çelik lifli betonların mikro yapıları taramalı elektron mikroskobuyla incelenmiştir. Elde edilen sonuçlara göre betonda metakaolin kullanımının ve betona değişik tipte çelik lif eklenmesinin, s/b oranından bağımsız olarak betonun mekanik davranışı üzerinde önemli düzeyde etkili oldukları görülmüştür.

Anahtar Kelimeler: Aderans dayanımı; Basınç dayanımı; Beton; Çekme dayanımı; Çelik lif; Metakaolin

To My Parents

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LIST OF SYMBOLS

C-S-H	Calicium-silicate-hydrate
MK	Metakaolin
w/b	Water binder ratio
w/C	Water cement ratio
GLM ANOVA	General linear model analysis of variance
CH	Portlandite
ITZ	Inter ferencial transition zone
XRF	X-ray florences
GPa	Gigapascal
ECR	Epoxy coating reinforcement
SEM	Scanning electon microscpy
Fc’	Compressive strength of cocrete
OPC	Ordinary Portland cement
RPC	Reactive powder concrete
HPSFC	High performance steel fiber concrete
FRC	Fiber reinforcment concrete
SFAC	Steel fiber added concrete

SFRC	Steel fiber renforcement concrete
HPC	High performance concrete
Vf	Volume fraction
D	Diameter
L	Length
SF 1	Steel fiber 60 mm
SF 2	Steel fiber 30 mm

CHAPTER 1

INTRODUCTION

1.1 General

Concrete is the most commonly used building material all over the world because of its versatility and availability. Especially reinforced concrete structural elements have been indispensable parts of construction works due to the ease in erection and relatively lower cost than the other structural materials. The proper adherence between reinforcing bars and concrete is the most desired property due to the fact that structural performance of reinforced concrete members depends on the monolithic behavior. The prominent component controlling the competence of the bond is mostly the quality of concrete. Because the reinforcing steel bars are obtained from a fixed manufacturing process and the properties do not significantly fluctuate compared to concrete. However, structural concretes have many different characteristics depending mainly on the amount and type of the ingredients [1]. It is reported that concrete with improved mechanical property has superior adherence with reinforcing steel bars [2].

Apart from its excellent properties, concrete shows a rather low performance when subjected to tensile stress. For this reason, the utilization of fibers to provide enhancement in tensile strength behavior of concrete has attracted the interest of the researchers [3-9]. Mechanical properties of concrete can be improved by exploitation of reinforcement with randomly oriented short separated fibers, which obstruct and/or control initiation and propagation of cracks. Fiber reinforced concrete (FRC) can keep on resisting much amount of loads even at deflections. The characteristics and performance of FRC varies depending on matrix properties as well as the fiber material, fiber concentration, fiber geometry, fiber orientation, and fiber distribution [8].

In order to improve the mechanical properties, particularly compressive strength, use of some pozzolanic materials has been reported by researchers for many years [10-16]. Pozzolans, such as silica fume and fly ash, are the most commonly known mineral admixtures used in production of high-strength concrete. These materials impart additional performance to the concrete through reacting with Portland cement hydration products to form secondary C-S-H gel, the part of the paste mainly responsible for concrete strength [17].

For the last two decades, there has been a growing attraction in the beneficiation of metakaolin (MK) as a supplementary cementing material in concrete to enhance its properties. MK is an ultrafine pozzolana, manufactured by calcination of purified kaolin clay at a temperature ranging from 650 to 900 ^{o}C to drive off the chemically bound water and destroy the crystalline structure [18-19]. Unlike other industrial by-product materials, MK needs a thorough process of manufacturing. It has to be carefully refined to remove inert impurity and ground to particles of micron size. Research has demonstrated that concrete mixtures incorporating high-reactivity MK present comparable performance to the ones with other mineral admixtures in terms of mechanical properties as well as permeability and durability properties [20-28]. Moreover, the use of this material is also environmentaly friendly due to the reduction of CO_2 emission to the atmosphere by decreasing Portland cement consumption.

In this thesis, the combined effect of MK and steel fiber on mechanical properties of concretes was examined through an experimental program. The concretes dealt with this study were produced by two different water/binder (w/b) ratios. For steel fiber reinforced concretes, two different types of steel fiber with length/aspect ratios of 60/80 and 30/40 were used. The steel fibers were added to concrete with 0.25% and 0.75% of the volume of the concrete. The mechanical properties of the concretes were measured through compressive, flexural, and splitting tensile strength testing at the end of 28 days of curing. Moreover, adherence between reinforcing steel bar and concrete were evaluated by means of bonding strength test at the same age. The statistical analysis and calculation of the contributions of the independent factors on mechanical behavior of concretes were realized by general linear model analysis of variance (GLM-ANOVA). Additionally, the relation between mechanical properties and the bonding strength of the concretes were evaluated through correlating the experimental data. Furthermore, the microstruture of different concrete mixtures were studied by scanning electro microscopy (SEM).

1.2 Outline of the Thesis

Chapter 1-Introduction: Aim and objectives of the thesis are introduced.

Chapter 2-Literature review: A literature survey was conducted on steel fiber and metakaolin. The previous studies on the use of MK and steel fiber are investigated.

Chapter 3-Experimental study: Materials, mixtures, casting, curing conditions, and test methods are described.

Chapter 4-Test results and discussions: Indication, evaluation, and discussion of the test results are presented.

Chapter 5-Conclusion: Conclusion of the thesis and recommendation for future studies are given.

CHAPTER 2

LITTERATURE REVIEW

2.1 Metakaolin

Metakaolin is white in color and acts as a pozzolanic material. Recently it has been introduced as a highly active pozzolan for the partial replacement of cement in concrete [28]. Metakaolin improves concrete performance by combining chemically with free lime a by-product of Portland cement hydration to form additional CSH [29]. Metakaolin reacts with portlandite (CH) to form calcium-silicate-hydrate (C-SH) supplementary to that produced by Portland cement hydration. The characteristics of metakaolin have been investigated extensively for the last two decades.

Because of its high degree of whiteness, metakaolin can be used with white cement to get high performance white concrete. It is reported by Gruber that good color matching for architectural applications can be obtained [30]. Metakaolin is ultra-fine and an artificial pozzolan which is produced from the calcination of kaolinite clay at temperatures in the range 700-800 C^{o} [31]. The temperature must be high enough to allow for loss of hydroxyls but below temperatures that cause the formation of vitreous and crystallization of other phases such as mullite. Raw kaolin can be obtained from various quarries having different geological and morphological. characteristics. The Manufacturing procedure covers adjustment of calcinations temperature and the grinding process to increase the homogenity of the final product

[32]. It is manufactured for a specific purpose under carefully controlled conditions since metakaolin is different from other supplementary cementitious materials like silica fume, fly ash and slag which are the wastes of the industrial plants [33]. This reaction becomes important within the interfacial transition zone (ITZ) between aggregate and paste fractions. This region typically contains a high concentration of large, aligned CH crystals, which can lead to localized areas of increased porosity and lower strength [34]. The rates of pozzolanic reaction and CH consumption in metakaolin systems have been shown to be higher than that of silica fume systems, indicating a higher initial reactivity [35]. Because this reaction with CH occurs early and rapidly, metakaolin incorporation may contribute to reduced initial and final set times [36]. In addition, this refinement in the ITZ can result in increased strength in metakaolin concrete [37].

2.1.1 Effect of metakaolin on the mechanical properties of concrete

2.1.1.1 Compressive strength

Metakaolin is the product of processed heat treatment of natural kaolin, is widely studied as an effective pozzolanic material especially for the early strength development and good durability concrete [38]. Researchers studied that pozzolanic reaction and micro filler behavior of metakaolin enhance strength development of the cement-metakaolin mortar. The strength enhancement particularly during 3 days was observed to increase the compressive and flexural strength in the range of 13-18% and 1-16% respectively. An optimum percentage replacement of 20% was found for strength improvement [38]. The importance of the microstructure enhancement was appeared through the very high level of chloride ingress resistance compared to the low level of high strength concrete [38]. In the point of view of the past studies on metakaolin, it has been determined that metakaolin utilized in different amount to replace the Portland cement changes the properties of both fresh and hardened concretes. It increases the compressive strength and reduces the slump for given aggregate binder ratio and water binder ratio. The effect of metakaolin replacement on the air permeability was determined to be secondary.

There have been several studies on the strength development of concrete containing MK. Some studies have shown clearly that with the use of MK considerable improvement in strength, especially at the early ages of curing, can be achieved [39]. The concretes with 5% and 10% of metakaolin revealed improvement in strength at ages up to 365 days. It is investigated that metakaolin-Portland cement concrete showed strengths relatively higher than silica fume Portland cement concrete at the same level of cement replacement by pozzolans.

In the study of Wild and Khatib [40], it was proved that the effect of meakaolin can be observed in three ways, the first is the filler effect that is immediate acceleration of Portland cement which happen at the first 24 hours, and the pozzolanic reaction, that has the maximum influence in the first 7-14 days for all metakaolin replacement levels between 5-30% Although strength gains relative to the control are still present after 90 days, the degree to which strength is improved decline beyond 14 days [41].

The influence of curing temperature on the strength development in concrete containing up to 15% MK was studied by Sabir et al. [41]. It was shown that curing MK concrete at 50 ºC results in increased early strength compared to the strength of specimens cured at 20 ºC. The acceleration in strength development due to the high curing temperature diminishes in the long term (365 days). In term of the strength relative to that of the control concrete cured at 20 ºC, the optimum level of MK replacement for cement in concrete with water binder ratio 0.35 cured at 20 ºC was found to be about 10%. This level of MK was found to be reduced to about found to be about 10%. This level of MK was found to be reduced to about 5% for concrete cured at higher temperature 50 ºC and with higher water binder ratio 0.45.

2.1.1.2 Flexural strength

Velosa [42] reported the results of the flexural strength test of the lime/metakaolin mortars. Table 2.1 shows the chemical properties of the metakaolins used. Figure 2.1 indicates the increase in strength of mortars MK2 and MK3 in relation to lime mortar with no addition. However, results of MK1 were very similar to those of lime mortar, probably due to the fact that K1 was from the initial batch that was produced at a lower temperature. The higher flexural strength exhibited by mortars MK2 and MK3 is probably due to the pozzolanic action of metakaolin as the increase of mechanical behavior of lime mortars due to the addition of pozzolans. It is promoted either by an increase in the compaction of mortar, with a special impact on compressive strength, or by the effect of pozzolanic reaction [42].

Table 2.1 XRF results of metakaolin samples [42]

	SiO_2 (%)	Al_2O_3 (%)	Fe_2O_3 (%)	MgO (%)	CaO (%)	Na_2O (%)	K_2O (%)	TiO_2 (%)	LOI (%)
K 1	62.62	28.63	1.07	0.15	0.06	1.57	3.46	0.36	2.00
K 2	59.90	32.29	1.28	0.17	0.04	0.24	2.83	0.36	2.80
K 3	62.48	28.72	1.01	0.13	0.03	2.45	3.55	0.34	1.20

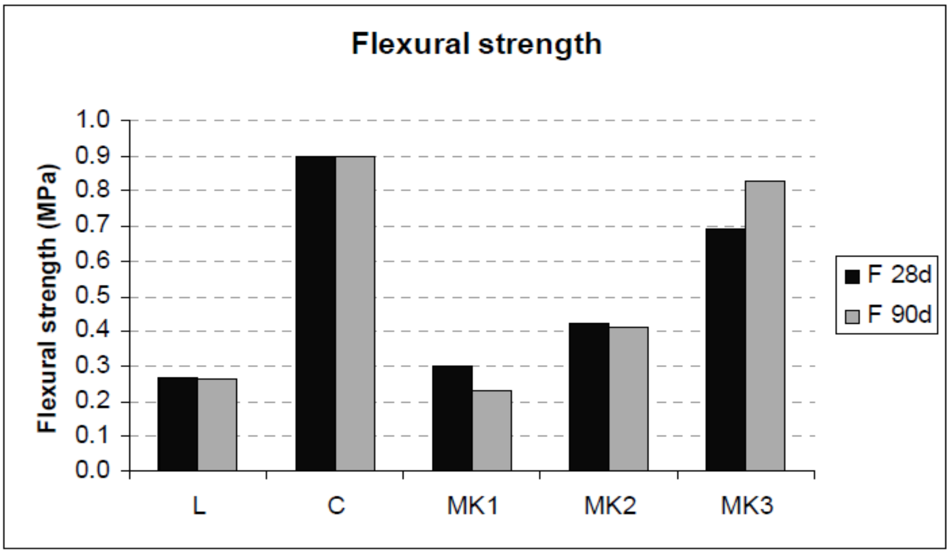

Figure 2.1 Flexural strength of lime/metakaolin mortars (MK) compared with lime mortar (L) and cement mortar (C) [42]

2.1.1.3 Splitting tensile strength

Qian and Li (2001) investigated the tensile strength of concrete incorporating 0, 5, 10, and 15% metakaolin as partial replacement of cement. Metakaolin had specific surface area 12000 m^2/kg, and its average particle diameter was 2.23 μm. 300×100×20 mm samples were tested under direct tension. Tests were conducted at the age of 28 days. Tensile strength test results are presented in Table 2.2. The results showed that tensile strength of concrete increased systematically with increasing metakaolin replacement level. The average tensile strength increases were 7% for 5% metakaolin, 16% for 10% metakaolin, and 28% for 15% metakaolin, and the average ultimate strain increases were 3% (5% metakaolin), 19% (10% metakaolin), and 27% (15% metakaolin). The descending area of over-peak stress was less steep when metakaolin replacement was 5% and 10% whereas with 15% metakaolin it was similar to that for concrete without metakaolin. The modulus of elasticity for these specimens is in the range from 26 to 27 GPa [43].

Table 2.2 Tensile strength of concrete with different metakaolin replacement [43]

Age (days)	Tensile strength (MPa)			
	MK (0%)	MK (5%)	MK (10%)	MK (15%)
28	3.35	3.58	3.88	4.29

2.2 The pozzolanic reaction of metakaolin

The reaction of cement with water is called hydration reaction. It is rare case to hydrate all of the cement. If it occurs, calcium hydroxide (CH) will produce about 28% related to its own weight, but in practice it is about 20%. CH is a hazardous substance in the final product of concrete, because it reacts with acids and destroyed the concrete, but when the pozolanic materials like silica fume, fly ash and metakaolin are used, these react with the calcium hydroxide and the result is the gel, which is a useful substance improving the properties of the concrete such as, durability and strength. In fact metakaolin is a good artificial pozzolan because it mainly reacts well with calcium hydroxide (free lime) and makes water hydrate compounds of Ca and Al silicates. The improvement of metakaolin is principally dependent on many factors like abundance and nature of the kaolin clay in the raw materials, calcinations condition and the fineness of the final product. The final result of the basic reaction between metakaolin and free lime is additional, cementitious aluminum containing C-S-H gel, together with crystalline products, which include calcium aluminate hydrates and alumino-silicate hydrates. The crystalline products formed depend principally on the metakaolinite/calcium hydroxide ratio and the reaction temperature. In addition if carbonate is freely available carbon-aluminates may also be produced. These chemical reactions may be expressed in the equations as follow [59] :

$$AS_2 + 6CH + 9H \rightarrow C_4AH_{13} + 2CSH \quad (2.1)$$

$(AS_2 /CH =0.5)$

$$AS_2 + 5CH + 3H \rightarrow C_3AH_6 + 2CSH \quad (2.2)$$

$(AS_2 /CH =0.6)$

$$AS_2 + 3CH + 6H \rightarrow C_2AH_8 + CSH \quad (2.3)$$

$(AS_2 /CH =1)$

The optimum replacement levels of Portland cement by metakaolin are associated with changes in the nature and proportion of the different reaction products (depending on composition) temperature and reaction time, which are formed in the Portland cement–metakaolin system.

2.3 Bond of the steel reinforcement in concrete

Loads are always applied to the concrete and not applied directly to the steel. Because of the weakness of concrete in tension zone, in reinforced concrete (RC) structures, concrete tends to crack. The tension load is transferred at the cracked place to the steel and between the cracks some of the tension force returns to the concrete. This simple basic stress transfer system shows the behavior of a reinforced concrete element and is the backbone of the reinforced concrete theory. The stress transfer results in bond stresses at the interface between the concrete and the steel. Engineers and researchers have long known the importance of bond to reinforced concrete and studies attempting to understand the bond mechanism and its behavior, dating back to 1877 [44]. The stress transfer (bond) between the concrete and steel allows the two materials to work together and controls the structural behavior of the RC element. Design equations for bond in codes are based on providing adequate bond strength to transfer enough force to the steel that it will yield and provide ductility for a RC member [45]. Bond strength is a measure of the transfer of load between the concrete and steel bar. Bond strength theory is the basis behind current development length equations used in design of reinforced concrete structures. Bond strength is influenced by bar geometries, concrete properties, the presence of confinement around the bar, as well as surface conditions of the bar [44]. Studies as early as 1976 found that the epoxy coating on ECR decreased the bond strength when compared to typical black bar reinforcement [46]. A loss of bond between the concrete and reinforcement could lead to failure of the structure as shown in figure 2.2.

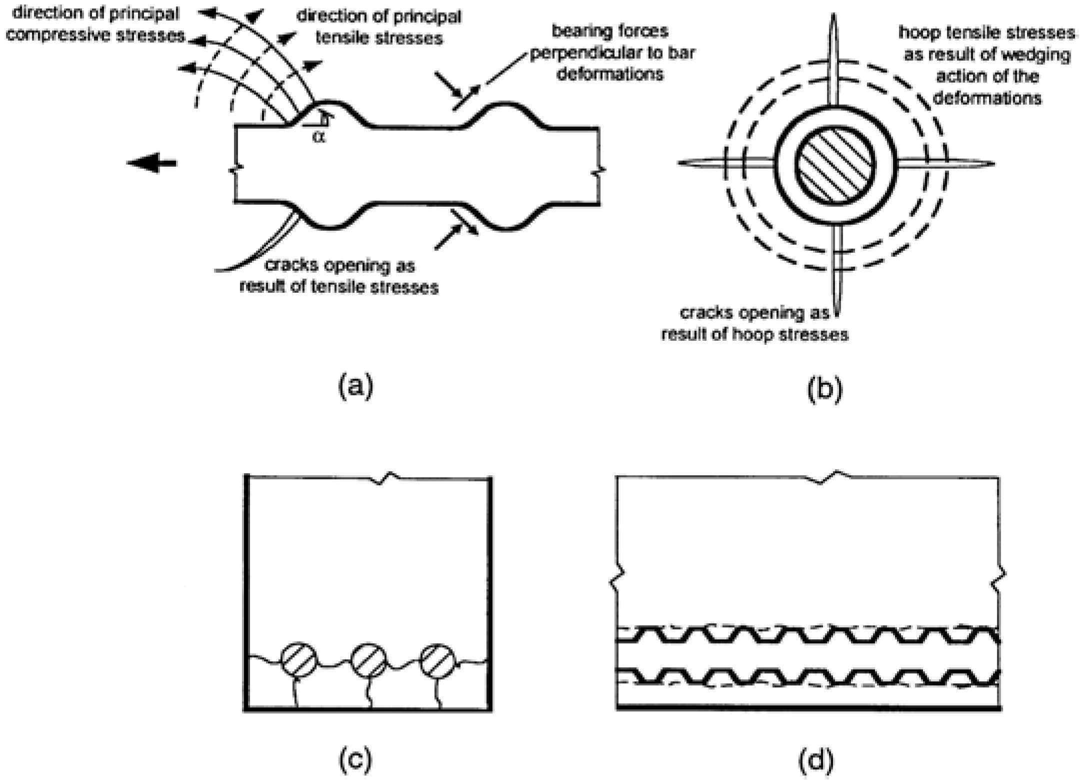

Figure 2.2 Cracking and damage mechanisms in bond: (a) side view of a deformed bar with deformation face angle a showing formation of cracks; (b) end view showing formation of splitting cracks parallel to the bar; (c) end view of a member showing splittingracks between bars and through the concrete cover; and (d) side view of member showing shear crack and/or local concrete crushing due to bar pullout [47]

2.3.1 Load transfer

Bond strength is the transfer of axial force from a reinforcing steel bar to the surrounding concrete results in the development of tangential stress components along the contact surface area [48]. For reinforced concrete to work as composite material, it is necessary for the reinforcing steel to be bonded to the surrounding concrete, if there is a good bond there is little or no slip of the steel relative to concrete and the means by which stress is transferred across the steel-concrete [49]. If there is the deformed steel bar bond strength produce by three mechanisms but if there is the plain bar only two mechanisms work, because mechanical interlocking between the steel bar and concrete does not produce, that is the major reason for their superior bond effectiveness [48].

Experiments have shown that the transfer of stresses between concrete and deformed steel bars occurs by the following mechanisms See Figure 2.3:

• Chemical adhesion between the steel and the concrete,

• Friction between the steel and the concrete, and;

• The mechanical anchorage of the reinforcing steel ribs against the concrete.

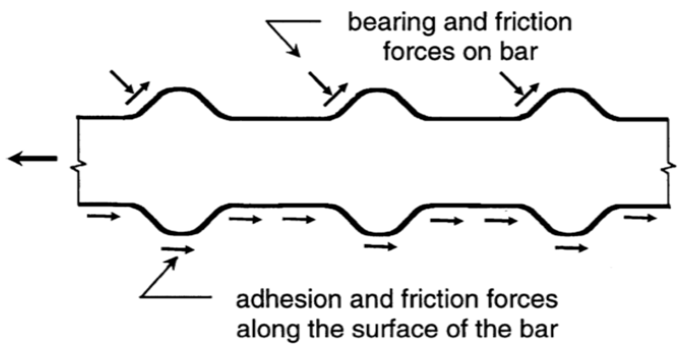

Figure 2.3 Bond transfer mechanism [44]

From the above three factors, first of all adhesion prevent the steel bar form occurring the slip between the steel bar and the surrounding concrete. When the load continues to excess the adhesion resistance, the adhesion resistance fails. Slip begins and the friction forces between the steel bar and concrete and bearing forces at the bar rib are mobilized. The main transfer mechanisms are due to bearing forces and friction forces acting at the steel rib when the load increases the slip also increases, the friction forces on the surface area of the reinforcement bar are reduced [44].

To prevent the balance, the bond forces on the steel bar must be resisted by compressive and shear stresses. These may be resolved into an outward component of the resultant bond force and a shear component parallel to the bar that is the effective bond force see Figure 2.4. The outward component of the bond force is similar to an internal pressure exerted on a thick walled cylinder [44].

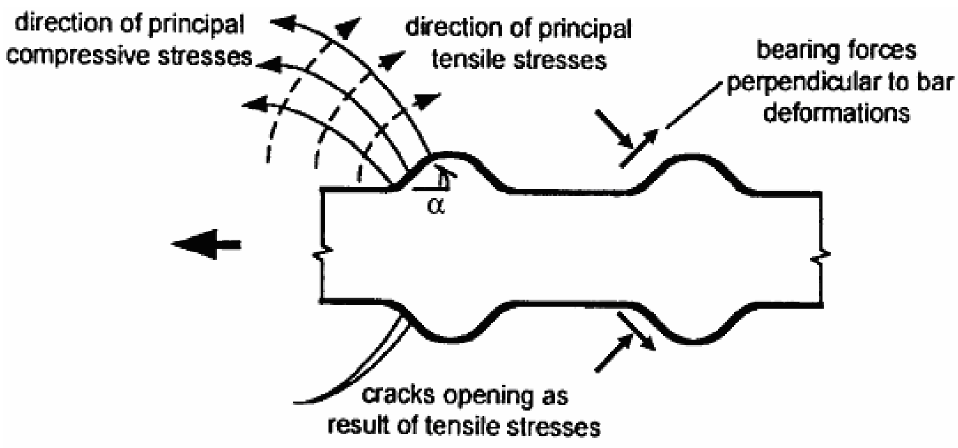

Figure 2.4 Forces and cracks in concrete [44]

Figure 2.5 illustrates the equilibrium conditions for portion of a reinforcing bar of length dx. The bond stress u can be expressed as the change in the stress in the reinforcement over the length dx as follows [50]:

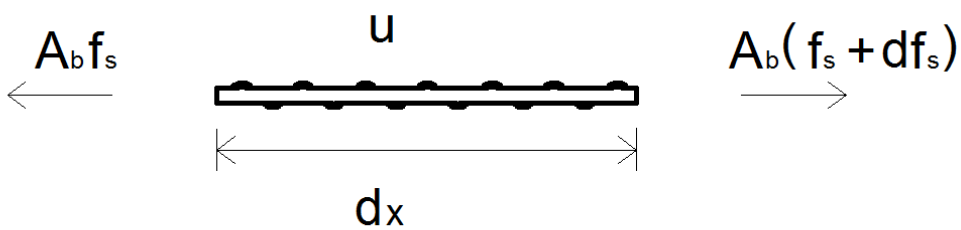

Figure 2.5 Bond stress acting on a reinforcing bar [50]

$$u(\pi db dx) = Ab\,(fs + dfs) - Abfs \qquad \text{(Eqn. 2.4)}$$

and hence

$$u = \frac{A_d fs}{\pi dbdx} = \frac{dbdfs}{4*dx} \qquad \text{(Eqn. 2.5)}$$

Where A*b* is the area of bar, d*b* is the bar diameter, and f*s* is the stress in the bar. For uniform bond, the bond stress can be expressed as:

$$u = \frac{P\ max}{\pi dbLd} \qquad \text{(Eqn.2.6)}$$

Where

P = maximum pullout load, d_b = diameter of the bar And, L_d is the embedded bar length

2.3.2 Bond strength between concrete and rusted steed bar

There are many reports that have been achieved for estimating structural performance of non-corroded reinforced concrete members, particularly focused on bond manner between reinforced bar and concrete. In the study of Congqi et al. [51], it was proved that medium level about 4% of corrosion has no importance effect on the bond strength, but considerable reduction in bond occur when corrosion increase thereafter to a higher level of about 6%. On the other hand according to the study by Kanakubo et al. [52], it was revealed that the manner of the local bond area is related to the diameter of the bar is quarter of the bond length. It is explained that the corrosion of the bar caused the crack in the adjacent concrete. The ratio of the weight rust to bond strength must not be greater than 7%. However, there is no realistic assessment which has been planned for corroded reinforcement concrete structure in the result of lack of experimental data and associated information. A basic theory for bonding systems based on the experimental test results should be proposed to estimate the bond splitting characteristics of corroded reinforced concrete structure. To suggest a basic theory for bond mechanism, the basic characteristics of global bond splitting failures must be explained between corroded steel bar and concrete, focusing on the comparatively long bond region under the condition without confinement.

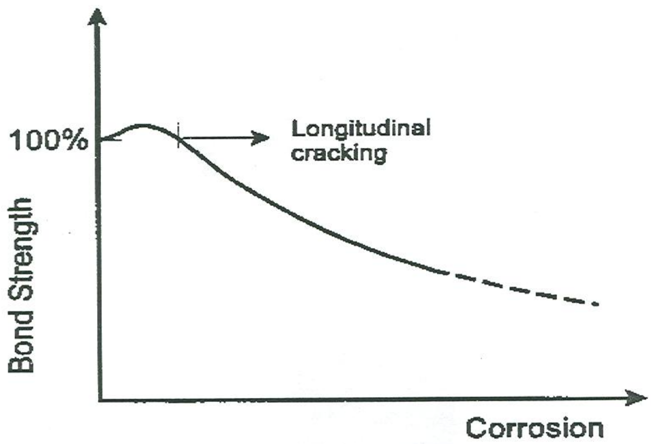

Figure 2.6 Variation of bond strength with corrosion [45]

2.3.3 Factor affecting the bond strength

There are many factors that affect the bond strength of reinforcement in concrete. The individual contributions of the factors are difficult to separate or quantify, however extensive research has gone into attempting to do so [53]. Several important factors are discussed below.

2.3.3.1 Development Length

In order to increase the crack surface area at failure, the bonded length between steel and concrete must be increased, thus the total resistance of the bond to failure increases, the resistance of the bond to failure is proportional to the energy required open a crack. The increase in bond stress however is not proportional to the increase in anchorage length because more of the bond forces are concentrated near the loaded end of the bar and therefore, this end is more effective in resisting bond forces than the far free end [44].

If a reinforcing bar in concrete carries a tensile load that is greater than the bond strength, the bar will pull out of the concrete. To prevent pullout failure an adequate length of bar must be embedded in the concrete. This embedded length is called development length (l_d) in design, and is the length of embedment required to carry a tensile force that causes yielding in the steel. Below is the development length equation used in the current ACI design manual, where l_d is determined in inches.

$$Ld= \left(\frac{3}{40}\frac{f_y}{\lambda\sqrt{f_{c'}}}\ \frac{\phi_t\phi_e\phi_s}{\left(\frac{c_{b+k_{tr}}}{d_b}\right.}\right)d_b \qquad \text{(Eqn. 2.7)}$$

The bar location factor, $Ø_t$, takes into account the reduction in bond due to large amounts of concrete being placed over the bar

2.3.3.2 Bar casting position

In a member the bottom-cast steel bars have better bond strength with concrete than the top-cast steel bars because the density of concrete in the lower part is higher than the top part of the member and the accumulation of bleeding water at the top part and beneath the top bars, water makes the lower contact surface area between the steel bar and concrete, at the end makes lower bond strength [44].

2.3.3.3 Bar size

For the same development length, the lager diameter of the steel bar has the larger bond strength because of the larger surface area and, require larger load to failure, However, in order to attain the same bar stress, larger bars require a larger bonded length since the surface area does not increase as rapidly as the cross sectional area [44].

2.3.3.4 Bar Geometry

It is known that the deformed steel bars have better bond strength than plain steel bars, that is because there is three factors in the case of the deformed bars but two factors in the case of plain bars as there is no mechanical anchorage provided by the concrete keys between the ribs. Figure 2.4 there are more studies to find the influence of the angle and height of the rib on the bond manner Figure 2.4. Results showed that for unconfined bars, the relative rib area (defined as the projected rib area normal to the bar axis divided by the nominal bar perimeter times center to center rib spacing) has no effect on bond strength. For a confined bond critical region, however, the bond strength increases with increasing relative rib area [44].

2.3.3.5 Bar surface condition

If there are dust, mud, oil and non-metallic materials at surface area of the steel bar the bond strength decreases because the surface area available for bond decreases. And also the epoxy coating decreases the bond strength, because the friction between the steel bar and concrete reduces [44].

2.3.3.6 Yield bar

At first there was an idea that the yielding progressed in the bar would decrease the bond strength [54], but then it has been appeared that yielding don’t relate to the bond strength, yielding has basically no detrimental influence on the bond strength, if there is no confining transverse reinforcement, bond strength increases if there is 10% confined by transverse reinforcement. If the bond load can uniformly distributed along the bar surface area with greater slip and bar deformation due to yield, the bond strength increases [55, 56].

2.3.3.7 Compressive and tensile concrete strength

The tensile strength mainly affects on the bond strength. The splitting tensile strength is nearly proportional to ($\sqrt{f_{c'}}$), while $f_{c'}$ is the compressive strength of concrete, bond strength conventionally was expressed in terms of ($\sqrt[\frac{1}{4}]{f_{c'}}$), since the ¼ power better shows the effect of concrete strength on bond strength [44]. This proves that the tensile strength is not the only factor that influences bond strength. In fact, it shows that the bond strength is directly associated to the fracture energy of concrete [44]. However, more studies are needed to confirm this observation. In general, as the concrete strength increases, the bond strength increases but at a slower rate than the concrete strength and the failure mode becomes more brittle [44].

2.2.3.8 Confinement

Confinement is one of the factors that affect the bond strength in the bond critical region. And this confinement depends on the thickness of concrete around the steel bar (cover) and one half the transverse spacing between the bars and steel stirrups.

Adding the fibers to the concrete mix can achieve better confinement [44]. As the confinement increases, the bond stiffness (the slope of the bond stress-slip curve) and the bond strength increase. If the level of confinement is high enough, the failure mode of bond changes from splitting to pullout [44]. Crack surface area increase by increasing concrete cover and bar spacing. Therefore the force required to progress the bond failure increases. And also the bond strength increases. Transverse reinforcement confines the anchored bars by limiting the progression of a splitting crack. Heavily reinforced steel stirrup increase the confinement and prevent a splitting failure from taking place [44]. Adding fibers to the concrete mix not only increases the tensile strength of the concrete but also increases the energy required to open and propagate a crack. Research results have shown that adding fibers to the concrete mix increased the bond strength substantially [44]. Results also showed that adding fibers improved the post-failure behavior of bond resulted in an increase in the residual bond strength and reduced the slip at the peak bond stress.

2.4 Effect of silica fume and fly ash on bond strength

There are numerous studies that focused on the improving the bond strength replacing the ordinary Portland cement by several admixtures like fly ash and silica fume.

Bubshait and Abdulaziz [57] proved that silica fume has good effect on the developing the bond strength between steel bar and concrete, by using the cantilever bond testing, the ultimate pullout load was 21.7 kN. In the case of OPC, but in the case of replacing cement by silica fume the ultimate pullout load was 27.8 kN, that is a good result. Tanyıldızı [58], proved that fly ash and silica fume have attractive influence on the improving bond strength, it is showed experimentally and calculated statistically. These interest behaviors were due to the reaction of the mineral admixture with free lime that is the byproduct of the hydration of OPC, which improves the strength and durability both at normal and high temperature by reducing the free lime content.

2.5 Steel fiber reinforced concrete

The first trial for using the steel fiber in concrete was by Porter in 1910 [60]. However, the first scientific investigation of fiber reinforced concrete (FRC) in United States was done in 1963 [61]. FRC is made utilizing conventional hydraulic cement, fine and coarse aggregates, water, and discrete discontinuous reinforcing fibers. To improve stability workability of FRC, super plasticizers can also be included. In the concrete, fibers are commercially exist and manufactured from steel, plastic, glass and other natural materials. Engineering specifications of fiber address its shape, material, length, diameter and type of cross-section as shown in Figure 2.7. Steel fibers may be described as discrete, short lengths of steel having ratio of its length to diameter (i.e. aspect ratio) in the range of 20 to 100 with any of the several cross section, and that are sufficiently small to be easily and randomly dispersed in fresh concrete mix using conventional mixing procedure [62].

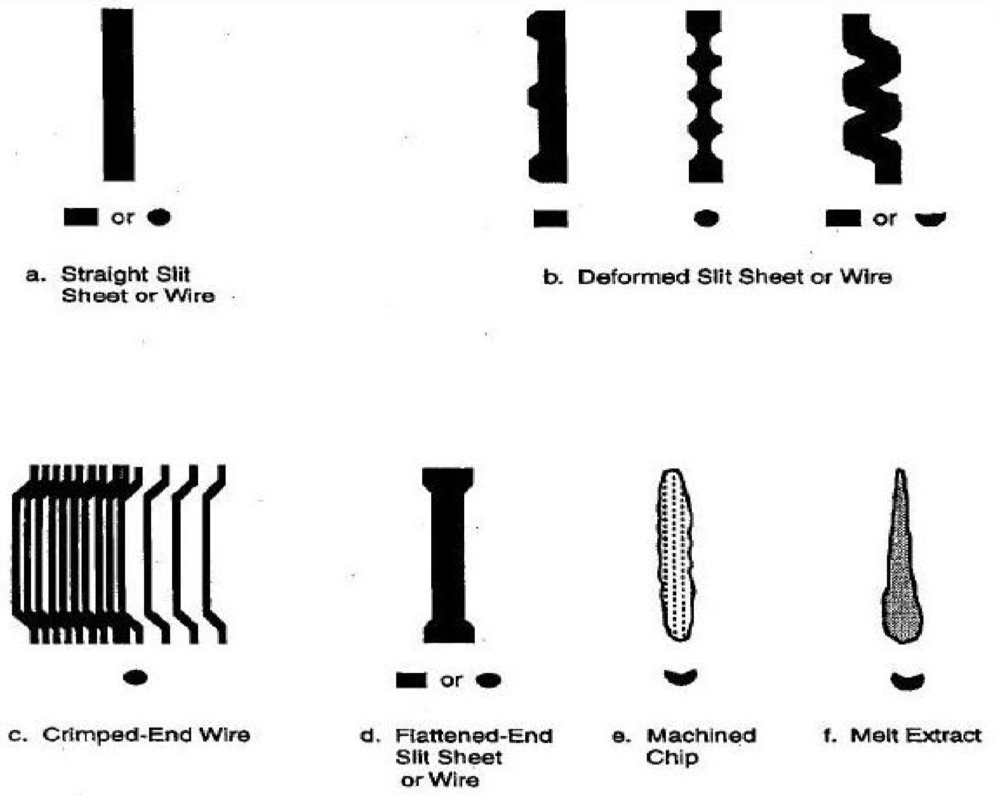

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Figure 2.7 Different shapes of steel fibers [62]

The importance of using of steel fiber in concrete depends on many reasons such as type, shape, length and cross section of fibers, strength and bond characteristics of fiber, fiber content, matrix strength, mix design and mixing of concrete.

Steel fiber enhanced the mechanical properties of plain concrete such as flexural strength, tensile strength, ductility and flexural toughness. Tensile strength can significantly increase when steel fiber was added to the concrete due to the crack arrest effect of the fibers [63]. Also significant increase in the shear strength obtained because beam tests exhibited that the steel fibers changes the brittle shear behavior of plain concrete into a ductile mechanism [64]. Many researches considered that steel fiber enhances the tensile and flexural strengths of the steel fiber concrete members since their strains were higher [65].

The ratio in which the addition of the steel fiber affects the mechanical characteristics of the concrete was dedicated that depend on some parameters such as material and aspect ratio of the fibers, their volume fraction in the mix, and loading rate [64]. Flexural is significant parameter that is mainly estimated to consider the effect of fiber on the post cracking response of the concrete composites. Toughness results were proved to increase with increasing fiber dosage, with an optimum value present for fiber volume fraction [67]. This optimum fiber dosage is dependent on various parameters like concrete type, fiber properties, and mix design.

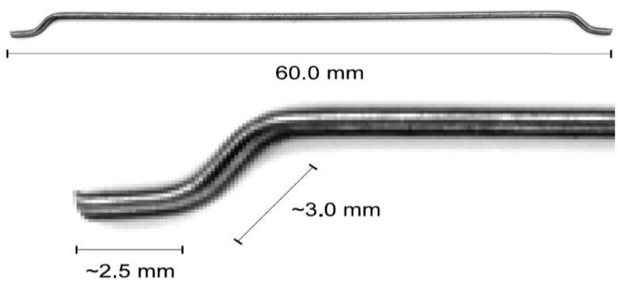


Figure 2.8 Dimensions of the DRAMIX® ZC 60/ 80 fiber and its end part [68]

2.5.1 Types of steel fiber

The fibers can be classified according to either their modulus of elasticity or their origin.

According to the modulus of elasticity fibers can be classified into two main groups, those having the modulus of elasticity higher than the concrete mix (called hard intrusion) such as steel, carbon and glass these types enhance flexural and impact resistance but those having lower modulus of elasticity than the concrete mix (called soft intrusion) such as polypropylene and vegetable fibers these types enhance the impact resistance but do not effect on the flexural strength. According to origin of fibers, they are classified in three categories of metallic, mineral and organic. This method of classification is widely been used. Figure 2.9 shows the classification of fibers [69].

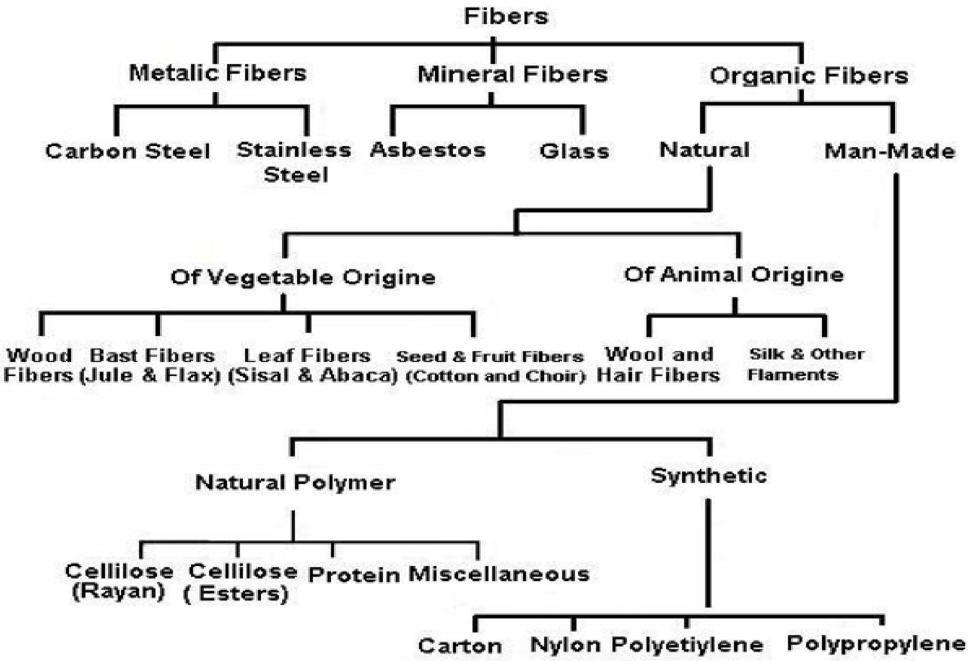


Figure 2.9 Fiber classifications [69]

Figure 2.10 Different types of steel fibers [68]

2.5.2 Application of steel fiber reinforced concrete

Steel-fiber-added concrete (SFAC) has been used at an increasing rate in various applications such as [70]:

1. Airfield pavements and highway: for the new highway pavement construction steel fiber concrete can be used and in the repairing pavement also can be used because of the higher flexural strength of steel fiber concrete that leads to minimize the thickness of the pavement ,and also has a good resistance to impact load and repeated loading. But the number of the joints must be increase in both directions, under conditions of restrained shrinkage, the greater tensile strain capacity of steel-fiber concrete results in lower maximum crack widths than in plain concrete.

2. Hydraulic structures: steel fiber concrete has good resistance to cavitations or erosion damage by high velocity of water flow because of that it can be used in the hydraulic

3. Fiber shotcrete: Fiber shotcrete has been used in rock slope stabilization, tunnel lining and bridge repair. A thin coating of plain shotcrete applied monolithically on top of the fiber shotcrete, may be used to prevent surface staining due to rusting. In addition to usual shotcrete advantages, the fibers are aligned in two dimensions (in a plane) by the mode of application of relatively thin coating. The fiber shotcrete can be used in the protection of structural steel work particularly in the support structure.

4. Refractory concrete: Steel-fiber reinforced refractory concretes have been reported to be more durable than their unreinforced counterpart when exposed to high thermal stress, thermal cyclic, thermal shock or mechanical abuse. The increased service span is probably due to the combination of crack control, enhanced toughness, the spall and abrasion resistance imparted by the steel fibers.

5. Precast application: They include manhole covers, concrete pipes, machine bases and frames. Improved flexural and impact strengths may allow the use of steel-fiber concrete components in rough handling situations.

6. Structural applications: The following possibilities maybe considered for the application of SFRC in structural members.

a) Fiber reinforcement can provide an increased impact resistance to conventionally reinforced beams, and thus, enhanced resistance to local damage and spalling.

b) Fiber reinforcement can inhibit crack growth and crack widening; this may allow the use of high strength steel without excessive crack widths or deformations at service loads.

c) Fiber reinforcement provides ductility to conventionally reinforced concrete structures, and hence, enhances their stability and integrity under earthquake and blast loading.

d) Fiber reinforcement increases the shear strength of concrete. As a consequence punching shear strength of slabs in increased and sudden punching failure maybe transformed in to gradual ductile one [70].

2.5.3 Mechanical properties of steel fiber reinforced concrete

2.5.3.1 Compressive strength

Johnston [71] investigated the compressive strength of concrete developed by 0 to 15%, when 1.5% of fibers by the volume of concrete are added to it. The improvement of the spalling resistance, ductility and toughness are indicated by the descending portion of the slope of the stress-strain curve of FRC as shown in Figure 2.11 [72].

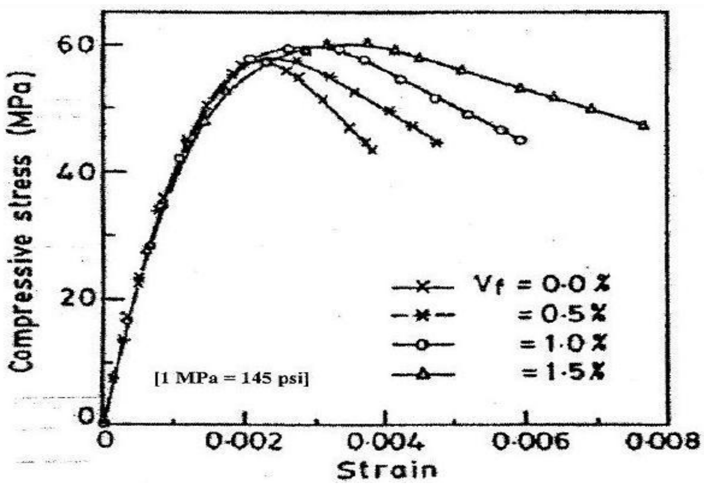


Figure 2.11 Effects of steel fibers content on the compressive stress-strain curve of FRC [72]

According to a research on improving the mechanical properties of concrete [73], 16% of concrete compressive strength increases after 28 days curing, when steel fiber is added, the compressive strengths increases by 12.1%, 29.8% and 20.19% for 0.5%, 1% and 1.5% steel fiber by volume of concrete respectively, as observed in Figure 2.12 [73].

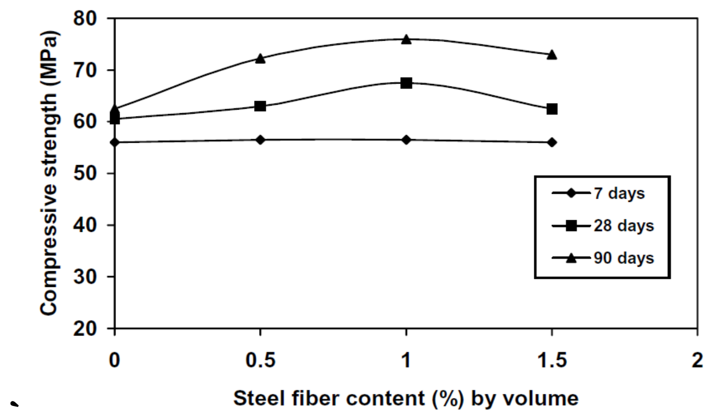


Figure 2.12 Effect of steel fibers on compressive strength at different curing ages [73]

The number of fibers, deformability and bond to the matrix are the main effects on the ability of the fiber to control micro cracking growth [74]. A higher number of fibers in the matrix lead to a higher probability of a micro crack being intercepted by a fiber. If the fiber is stiff enough and it is well bonded to the matrix, it can prevent the micro crack developing. On the other hand, fiber addition causes some perturbation of the matrix, which can result in higher void [75]. Voids can be seen as defects where micro cracking starts. In addition to fiber quantity, perturbation also depends on the ability of the matrix to accommodate fibers, which is an important property of the mortar fraction of the concrete. Therefore the influence of fibers on the compressive strength may be seen as the balance between micro crack bridging and additional voids caused by fiber addition [75].

The compressive strength of concrete was obtained by conducting tests on standard cubes of size (150X150X150) mm size with fibers 0 to 5% with an increment of 0.5% and silica fume of 5%.The compressive strength was determined by carrying out compressive test. Experimental results and results of regression analysis are presented in Figure 2.13 [76]

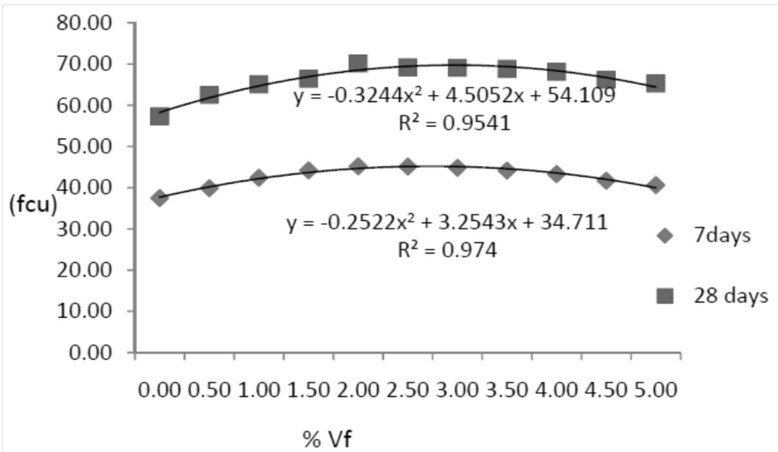


Figure 2.13 Compressive strength of concrete with respect to Vf % and 5 % silica fume for 7 and 28 days [76]

2.5.3.2 Splitting tensile strength

Splitting tensile strength of the high performance concrete increases with increasing steel fiber at all ages of curing, relative to control concrete. Figure 2.14 shows that the higher splitting tensile strength leads to qualify the mechanism of steel fiber in arresting the development of crack.

Shakir et al reported [73] the percentages of increase in splitting tensile strength of high performance steel fiber concrete (HPSFC) relative to reference concrete at 28 days were 66.66%, 90.6% and 122.22% for HPC with 0.5%, 1% and 1.5% for HPC with 0.5%, 1% and 1.5% steel fiber by volume of concrete respectively. The percentage of increase in splitting tensile strength of HPSFC relative to HPC were 19.7%, 36.9%, and 63.8% for HPC with 0.5%, 1%, and 1.5% steel fiber by volume of concrete [73].

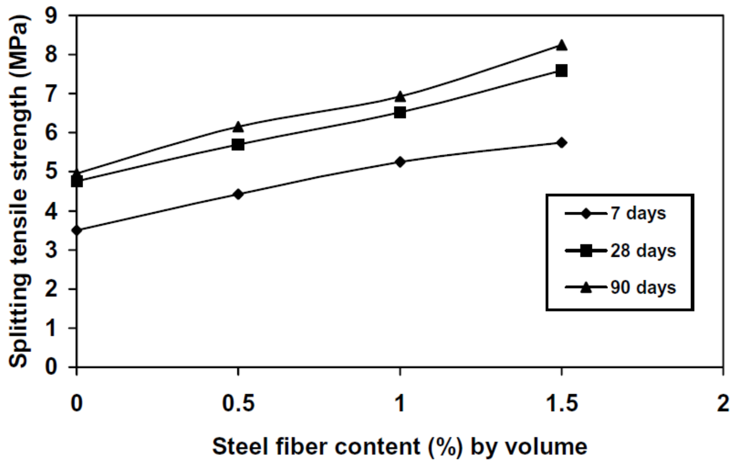


Figure 2.14 Effect of steel fiber on splitting tensile strength at different curing ages [73]

Shakir et al. [73] proved that the splitting tensile strength increases with age and with the increase of steel fiber contents. High performance concrete with l.5% steel fiber showed higher increase in splitting tensile strength. The percentage increases compared to HPC were 64.28%, 63.86% and 66.66% at 7, 28 and 90 days, respectively. Splitting tensile strength has few affected by the silica fume for the specimens with w/c ratio of 0. 35 but is more affected when w/c ratio is 0.55 a bout 9% as the silica fume content increases to 5% which may result from refined pore system achieved by increasing dense hydrated calcium silicate in the [77].

2.5.3.3 Flexural strength

Hannant [78] proved that flexural strength of concrete is more affected by the steel fiber than direct tension and compression by the steel fiber. The flexural strength of FRC is increased by about 55% with a V_f = 2% as reported by Oh et al. [79]. Hannant [78] studied steel fibers of Dramix RC-65/35-BN type in testing 12 different SFRC beams with two different steel fiber dosages of 60 kg/m^3 and 100 kg/ m^3, and conducted that the ratio of the measured ultimate load to theoretical ultimate load turned out to be greater with those SFRC beams having a 60 kg/ m^3 steel fiber dosage [79]. In his studies added 30 and 60 kg/m3 steel fiber were into two grades of concrete the length of the steel fibers were 6 cm and the diameter were 0.75mm with tensile strength 1050 N/mm^2. The results showed the flexural strength increases with increasing amount of steel fiber and also toughness increases for concrete grade 20 by 121% for the 30 kg/m^3 and 135% for the 60 kg/m^3.

The results of the steel fiber concrete for the 28 days show higher improvement of the flexural strength than the control concrete at the same age were 35.6%, 52.02%, and 62.36% for HPC with 0.5%, 1% and 1.5% steel fiber by volume of concrete respectively The increasing in flexural strength for HPSFC relative to HPC were 11.02%, 24.47%, and 32.9% for HPC with 0.5%, 1%, and 1.5% steel fiber by volume of concrete respectively,as seen in Figures 2.15 [73]. the load transfer mechanism of steel fiber in flexural strength test is also shown in figure 2.16 [68].

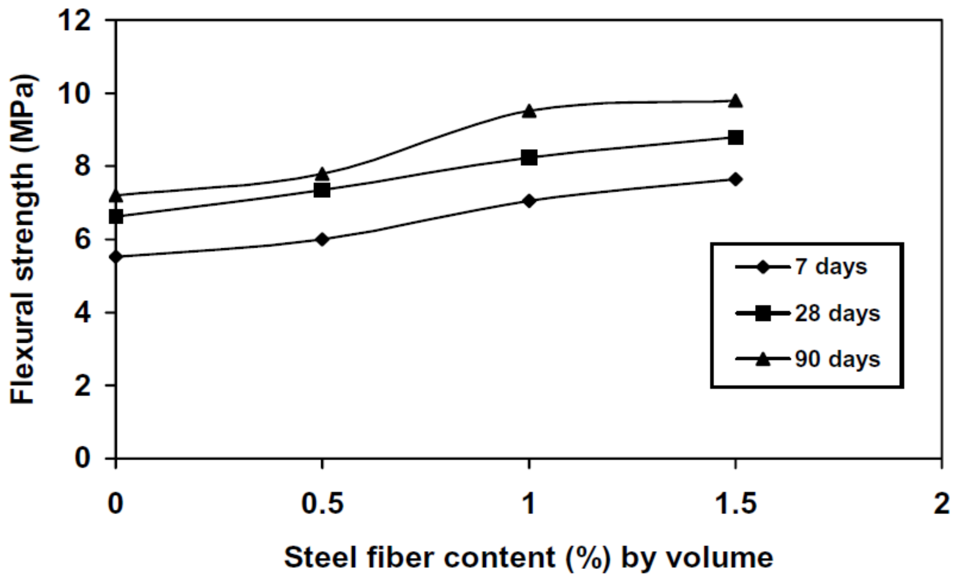


Figure 2.15 Effect of steel fiber content on flexural strength of concrete at different ages [73]

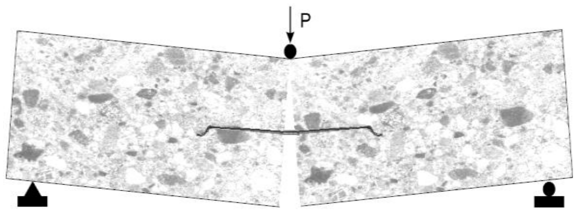


Figure 2.16 Principle of fiber reinforcement [68]

Hoang *et al*. [81] proved that when long steel fiber to reach 2% mixed with short steel fiber with no less than 1% volume of steel fibers in concrete, the workability of fresh concrete didn’t depend on the increase of long steel fiber. Short steel fiber content was high influence on properties of concrete. The increase of short steel fiber reduced workability but enhanced flexural strength. The researches show that adding the steel fiber to both heavy and normal concrete can increase the flexural strength for both of them. The modulus of rupture of heavy concrete also increase with the steel fiber fraction.a 1.5 % of steel fiber fraction raises the tensile strength from 6.61 MPa to 9.47 MPa for normal concrete and from 6.66 MPa to 11.3 and 9.47 MPa for heavy concrete. It makes heavy concrete a higher flexural strength than that of normal concrete. This could be explained by the above results that heavy concrete has a higher compressive strength than that of normal concrete, which leads to a higher flexural strength [82].

2.5.3.4 Bond strength

Yu-Cheng et al. [82] conducted the pull out test bond strength. They reported the result of developing the bond strength by adding the steel fiber compared to those without steel fiber same as splitting tensile strength or modulus of rapture, adding steel fiber can increase tensile strength from 6.14 Mpa to 14.7 Mpa for normal concrete. However, from 6.65 MPa to 16.1 MPa for heavy concrete, obviously, steel fiber could provide the confinement of concrete around the steel bar. Also, it makes heavy concrete has higher bond strength than normal concrete. Furthermore, the type of cracking observed from the fractured surface of all specimens showed that splitting failure is typical for those concretes without fibers and become less clear as more steel fibers were included [82].

CHAPTER 3

EXPERIMENTAL STUDY

3.1. Materials

3.1.1 Cement

CEM I 42.5 R type Portland cement having specific gravity of 3.14 and Blaine fineness of 328 m^2/kg was utilized for preparing the concrete test specimens used in determination of mechanical properties. The chemical composition of the cement is shown in Table 3.1.

3.1.2 Metakaolin

The metakaolin used in this study is a white powder with a Dr Lange whiteness value of 87. It has a specific gravity of about 2.60, and specific surface area (Nitrogen BET Surface Area) of 18000 m^2/kg. Physical and chemical properties of MK used in this study are also given in Table 3.1. The origin of the MK is from Czech Republic.

Table 3.1 Properties of Portland cement and metakaolin

	Item	Portland Cement	Metakaolin
Chemical properties	CaO (%)	62.58	0.5
	SiO_2 (%)	20.25	53
	Al_2O_3 (%)	5.31	43
	Fe_2O_3 (%)	4.04	1.2
	MgO (%)	2.82	0.4
	SO_3(%)	2.73	-
	K_2O	0.92	-
	Na_2O	0.22	-
	LOI (%)	1.02	0.4
Physical properties	Specific gravity	3.14	2.60
	Fineness (m^2/kg)	327*	18000**

* Blaine specific surface area
** BET specific surface area

3.1.3 Aggregate

Fine aggregate was a mix of river sand and crushed sand whereas the coarse aggregate was river gravel with a maximum particle size of 22 mm. Aggregates were obtained from local sources. Properties of the aggregates are presented in Table 3.2. Grading of the aggregate mixture was kept constant for all concretes. Fiure 3.1 shows the radiation of aggreget.

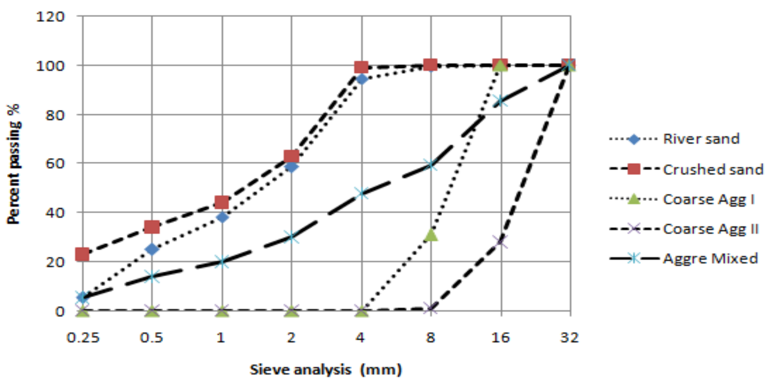


Figure 3.1 Grading of aggregate

Table 3.2 Sieve analysis and physical properties of aggregates

	Sieve size, (mm)	Passing (%)			
		Fine Aggregate		Coarse Aggregate	
		River Sand	Crushed Sand	No I (4-16 mm)	No II (16-22 mm)
Sieve Analysis	31.5	100	100	100	100
	16.0	100	100	100	27.7
	8.0	99.7	100	31.5	0.6
	4.0	94.5	99.2	1.0	0.1
	2.0	58.7	63.3	0.5	0.0
	1.0	38.2	43.7	0.5	0.0
	0.50	24.9	28.4	0.5	0.0
	0.25	5.4	16.4	0.4	0.0
	Fineness modulus	2.87	2.57	5.66	6.72
Physical Properties	Specific gravity	2.79	2.42	2.72	2.73
	Absorption, %	0.55	0.92	0.45	0.42

3.1.4 Super plasticizer

Sulphonated naphthalene formaldehyde based high range water-reducing admixture with specific gravity of 1.19 was employed to achieve slump value of 14±2 cm for the ease of handling, placing, and consolidation in all concrete mixtures. The superplasticizer was adjusted at the time of mixing to achieve the specified slump.

3.1.5 Steel fiber

Two types of commercially available hooked end steel fibers (Dramix 60/80 and Kemerix 30/40) were used for production of steel fiber reinforced concretes. The geometrical properties and aspect ratios of the steel fibers are given in Table 3.3.

Table 3.3 Properties of steel fibers

Designation of the steel fibre	Diameter D (mm)	Length L (mm)	Aspect ratio (L/D)
SF 1	0.75	60	80
SF 2	0.75	30	40

3.1.6 Steel bar

Reinforcing ribbed steel bars having 16 mm diameter and minimum yield strength of 420 MPa were utilized for preparing the reinforced concrete specimens to be used for testing the bonding strength

3.2. Mix proportions

Two series of concrete mixtures with water-to-binder ratios of 0.35 and 0.50 were designed to produce plain and MK incorporated concretes. MK modified concretes were produced by 10% replacement of the cement with MK by the weight. For production of steel fiber (SF) reinforced concretes, each type of SF (SF1 and SF2) were added to the concrete by 0.25% and 0.75% of the total concrete volume. Therefore, 20 different types of concrete mixtures were produced for examining the mechanical properties of the concretes. The details of the concrete mixtures are given in Table 3.4.

The designations of each mix were made according to MK incorporation, type of steel fiber, and volume fraction of steel fiber. For example, 10M 0.75SF1 code stands for the concrete incorporated with 10% MK and 0.75% steel fiber type I (SF1).

Freshly poured concrete specimens were covered with plastic sheet and kept in laboratory at 21±2 ^{o}C for 24 hours. Then, the specimens were demoulded and transferred to a water tank for curing up to 28th day.

Table 3.4 Plain and steel fiber reinforced concretes containing metakaolin (kg/m^3)

Mix ID	w/b ratio	Water	Cement	Metakaolin	Fine Aggregate		Coarse Aggregate		Steel Fiber		SP*
					Natural sand	Crushed sand	No I (4-16 mm)	No II (16-22 mm)	SF 1	SF 2	
Control I	0.35	157.5	450	0	663.1	284.2	568.4	378.9	0	0	11.25
M0-25SF1		157.5	450	0	663.1	284.2	568.4	378.9	19.62	0	12.5
M0-25SF2		157.5	450	0	663.1	284.2	568.4	378.9	0	19.62	16.0
M0-75SF1		157.5	450	0	663.1	284.2	568.4	378.9	58.85	0	13.75
M0-75SF2		157.5	450	0	663.1	284.2	568.4	378.9	0	58.85	19.5
Control II		157.5	405	45	660.0	282.9	565.7	377.2	0	0	10
M10-25SF1		157.5	405	45	660.0	282.9	565.7	377.2	19.62	0	11.25
M10-25SF2		157.5	405	45	660.0	282.9	565.7	377.2	0	19.62	19.5
M10-75SF1		157.5	405	45	660.0	282.9	565.7	377.2	58.85	0	13
M10-75SF2		157.5	405	45	660.0	282.9	565.7	377.2	0	58.85	20
Control I	0.50	175.0	350	0	656.3	281.25	562.5	375.0	0	0	4.75
M0-25SF1		175.0	350	0	656.3	281.25	562.5	375.0	19.62	0	4.75
M0-25SF2		175.0	350	0	656.3	281.25	562.5	375.0	0	19.62	6.0
M0-75SF1		175.0	350	0	656.3	281.25	562.5	375.0	58.85	0	7.5
M0-75SF2		175.0	350	0	656.3	281.25	562.5	375.0	0	58.85	10.25
Control II		175.0	315	35	653.9	280.22	560.4	373.6	0	0	6.25
M10-25SF1		175.0	315	35	653.9	280.22	560.4	373.6	19.62	0	6.75
M10-25SF2		175.0	315	35	653.9	280.22	560.4	373.6	0	19.62	12.5
M10-75SF1		175.0	315	35	653.9	280.22	560.4	373.6	58.85	0	8.5
M10-75SF2		175.0	315	35	653.9	280.22	560.4	373.6	0	58.85	11.0

*SP: Superplasticiser

3.3. Test specimens

The concrete specimens having various dimensions were used for testing. Cubic specimens having 150x150x150 mm were utilized for compressive strength. For three point flexural tensile strength testing, prismatic specimens with 100x100x500 mm dimensions were used to ensure 450 mm span length for testing. Splitting tensile strength of the concrete was measured from cylindrical specimens having Φ150x300 mm dimensions. Bonding strength between concrete and reinforcement were tested on cubic reinforced concrete specimen. In order to have a smooth surface to provide uniform load distribution, the top surface of the pullout specimens were capped with gypsum coating. The details and dimensions of the pullout test specimen are illustrated in Figure 3.2.

For each test, three specimens were used. Each experimental parameter was determined by averaging the results obtained from those specimens. All of the tests were performed at the end of 28 day curing period.

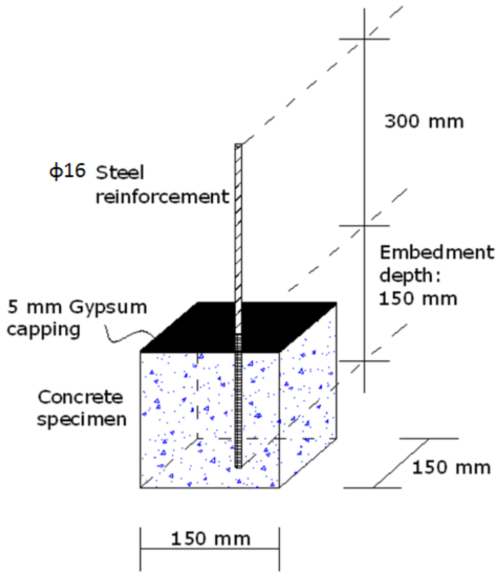


Figure 3.2 Details of the bonding strength test specimen

3.4. Test Methods

The compression test conforming to ASTM C39 [83] was carried out on the specimens by a 3000 KN capacity testing machine. Three-point flexural tensile strength conforming to ASTM C293 [84] was applied to the prismatic specimens through 100 kN capacity bending frame. Splitting tensile strength was carried out according to the specification per ASTM C496 [85]. Bonding strength of the concretes was determined in accordance with RILEM RC6 [86]. According to the standard the bonding strength, τ, is calculated by dividing the tensile force by the surface area of the steel bar embedded in concrete (Eqn 3.1). For this test, specially modified test apparatus was installed to 600 kN capacity universal testing machine.

$$\tau = \frac{F}{\pi \times d \times L} \qquad \text{(Eqn 3.1)}$$

Where F is the tensile load at failure (N), d and L are the diameter (mm) and embedment length (mm) of the reinforcing steel bar, respectively. In this study d and L are 16 mm and 150 mm, respectively.

Scanning electron microscopy (SEM) image analysis was carried out to observe the changes in cement paste matrix due to inclusion of MK. Moreover, the interface between steel fiber and mortar phase of the concrete was also observed for visual assessment of MK incorporation.

CHAPTER 4

TEST RESULTS AND DISCUSSION

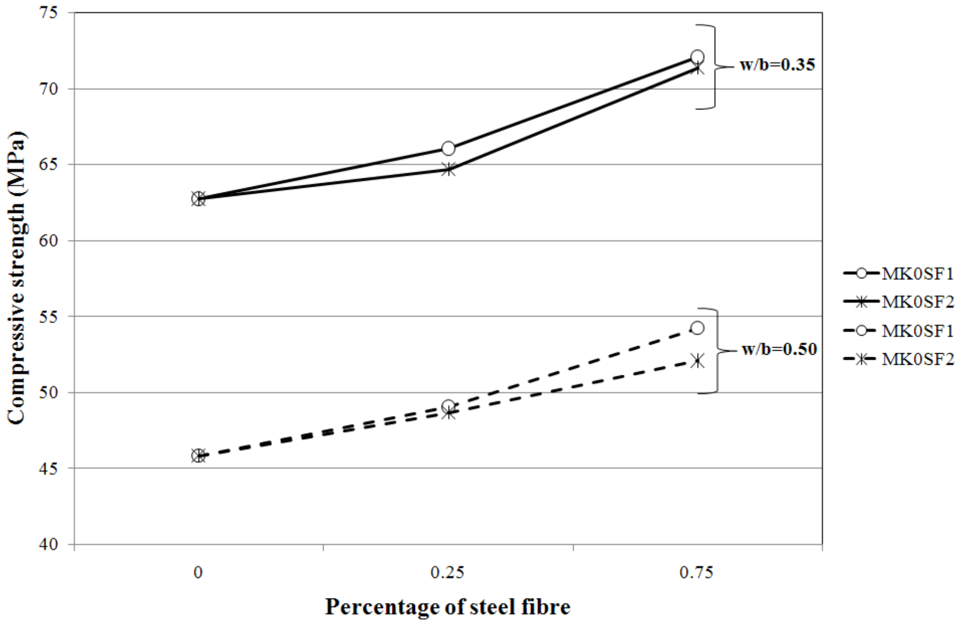
In this thesis, the mechanical properties of concrete are investigated the results are shown in the Table 4.1 each compressive strength, bond strength, flexural strength and splitting tensile strength are discussed in the following paragraphs.

Table 4.1 Test results of the compressive, bond, flexural, and tensile strength

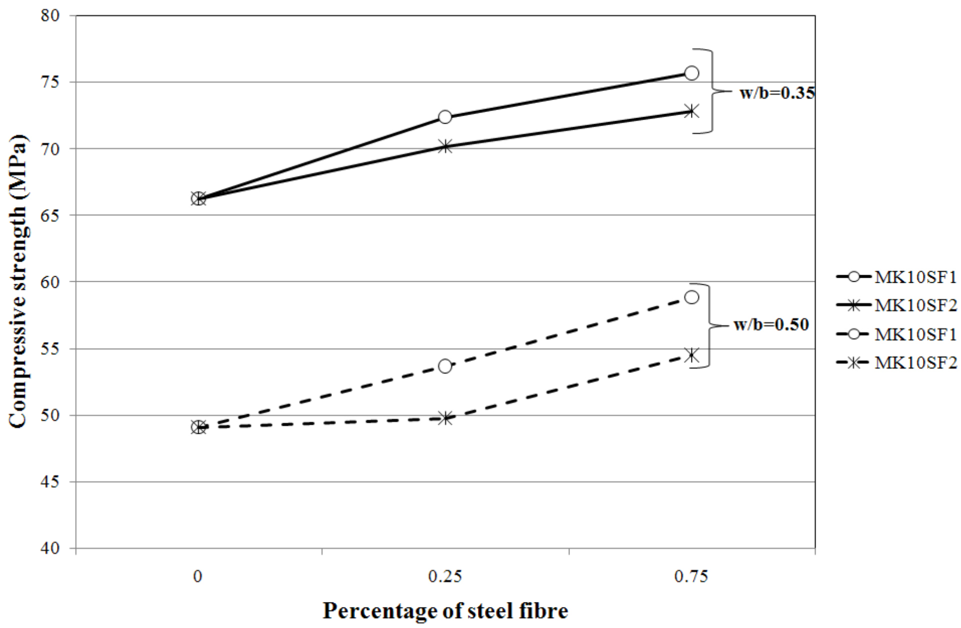
w/b	Mixes	Com. Str. (MPa)	Bond. Str. (MPa)	Tens. Str. (MPa)	Flex. Str. (MPa)
0.35	0MK0SF	62.7	11.7	3.9	5.8
	0MK0.25SF1	66.0	13.1	4.8	7.1
	0MK0.75SF1	72.0	16.0	5.3	7.4
	0MK0.25SF2	64.7	12.3	4.8	7.2
	0MK0.75SF2	71.4	14.1	6.0	7.8
	10MK0SF	62.3	12.6	4.5	7.1
	10MK0.25SF1	72.4	14.0	5.2	7.6
	10MK0.75SF1	75.7	16.9	6.2	7.9
	10MK0.25SF2	70.2	13.3	5.3	7.8
	10MK0.75SF2	72.8	16.4	7.2	8.1
0.50	0MK0SF	45.8	10.1	3.7	5.2
	0MK0.25SF1	49.0	12.6	4.2	6.1
	0MK0.75SF1	54.2	13.3	4.9	6.1
	0MK0.25SF2	48.7	12.2	4.3	6.2
	0MK0.75SF2	52.1	12.7	5.2	6.5
	10MK0SF	49.1	11.2	4.1	6.1
	10MK0.25SF1	53.7	13.0	4.4	6.4
	10MK0.75SF1	58.8	15.0	5.5	6.8
	10MK0.25SF2	49.8	12.9	4.4	7.0
	10MK0.75SF2	54.5	14.3	5.9	7.3

4.1. Compressive strength

Figure 4.1 shows the variation in compressive strength of the plain and MK incorporated concretes with the increase in the amount of fiber reinforcement. Table 4.1. The plain concretes' compressive strength values ranged between 62-72 MPa and 45-54 MPa for w/b ratios of 0.35 and 0.50, while MK incorporated ones had compressive strength values between 66-76 MPa for the former and 49-59 MPa for the latter. The compressive strength results revealed that incorporation of MK had significant contribution on the compressive strength of the concretes. Similar results have been reported by previous authors [9, 22-27]. For example, in the study of Güneyisi et al. [24] concretes incorporated with 5% and 15% replacement level of MK yielded relatively higher strength than that of plain concretes at two different w/b ratios. As it can be seen from the Figure 4.1, increasing the amount of SF resulted in rise of the compressive strength of the concretes without depending on the incorporation of MK and w/b ratio. Nili and Afroughsabet [8] reported that 28 day compressive strengths of plain concrete produced with w/b ratio of 0.46 were 41.30 MPa, 46.35 MPa, and 47.25 MPa for steel fiber volume fractions of 0%, 0.5%, and 1.0%, respectively. Moreover, the influence of aspect ratio was also clearly seen from Figure 4.1. The higher the aspect ratio, the higher the increase in compressive strength was observed, especially for MK incorporated ones. For instance, the plain concretes produced with w/b ratio of 0.35 and steel fiber volume fraction of 0.75% had 71.38 MPa and 72.07 MPa for SF 2 and SF 1, respectively. However MK included concretes with the same parameters had 72.83 MPa and 75.66 MPa for SF 2 and SF 1, respectively.



a)

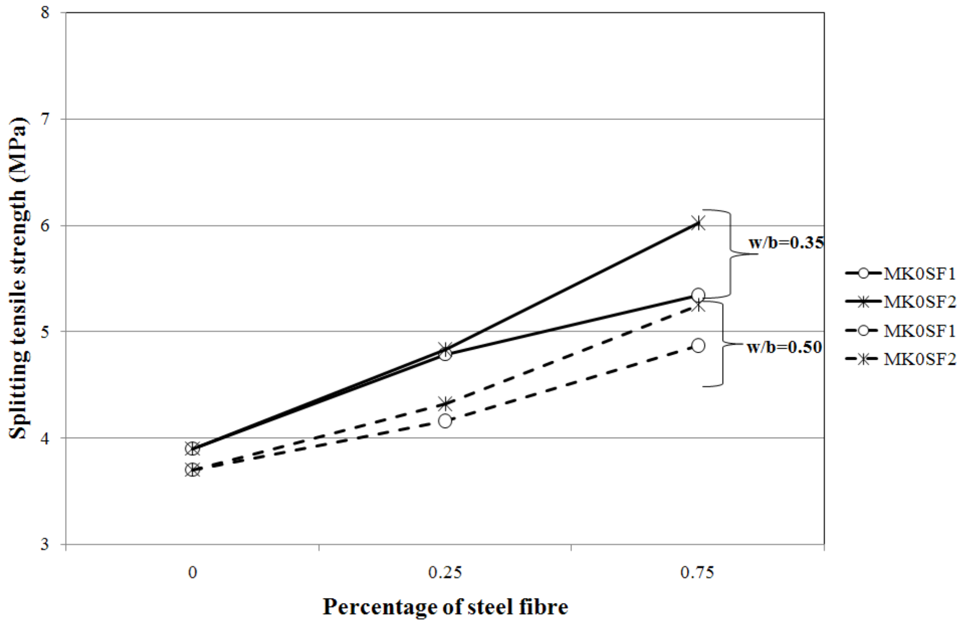


b)

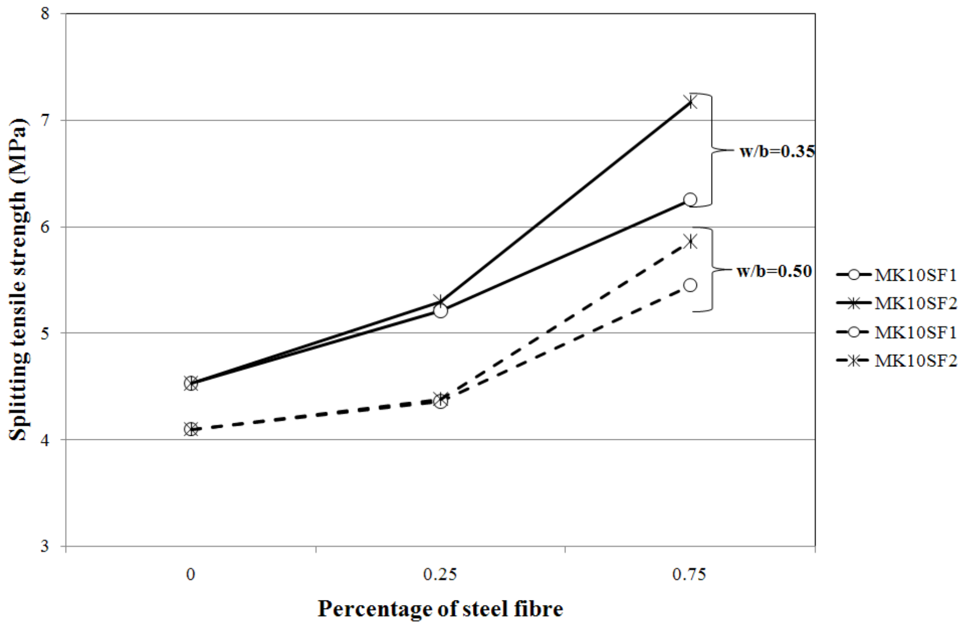
Figure 4.1 Effect of steel reinforcement on the compressive strength of a) plain and b) MK incorporated concretes

4.2. Tensile strength

The tensile strength of plain and MK incorporated concretes were monitored with respect to modulus of rupture and splitting tensile strength. The test results of flexural and splitting tensile strength tests are presented in Table 4.1, Figures 4.2 and 4.3, respectively, to demonstrate the effectiveness of steel fiber reinforcement. The results revealed that steel fiber addition provided increase in flexural strength capacity of plain concretes by 34.0% and 24.6% for w/b ratios of 0.35 and 0.50, respectively. However, MK incorporated ones exhibited 13.5% and 18.0% for those w/b ratios. It was reported that the main and major contribution of the steel fibers was due to the increase of tensile strain capacity of the concrete [87]. The same trend was also observed for splitting tensile strength of the concretes. The maximum splitting tensile strength values of 7.17 and 5.86 MPa were observed for concretes coded 10M0.75SF2 for w/b ratios of 0.35 and 0.50, respectively. Except for modulus of rupture results of MK incorporated concretes, tensile strength values of fiber reinforced concretes appeared to be very close to each other at 0.25% volume fractions. Another noticeable finding from the tensile strength testing is that unlike previous results, the contribution of SF2 was observed to be better than that of SF1. This situation may be attributed to the distribution of the steel reinforcement within the cement matrix. Namely, the shorter the steel fiber, the more homogenous distribution may be achieved. Sanal and Özyurt [88] investigated the effect of orientation of steel fibers on the mechanical performance of the concretes. They reported that, short-cut steel fibers have a tendency to align in the flow direction and greater orientation density in the casting direction resulted in a greater flexural toughness.

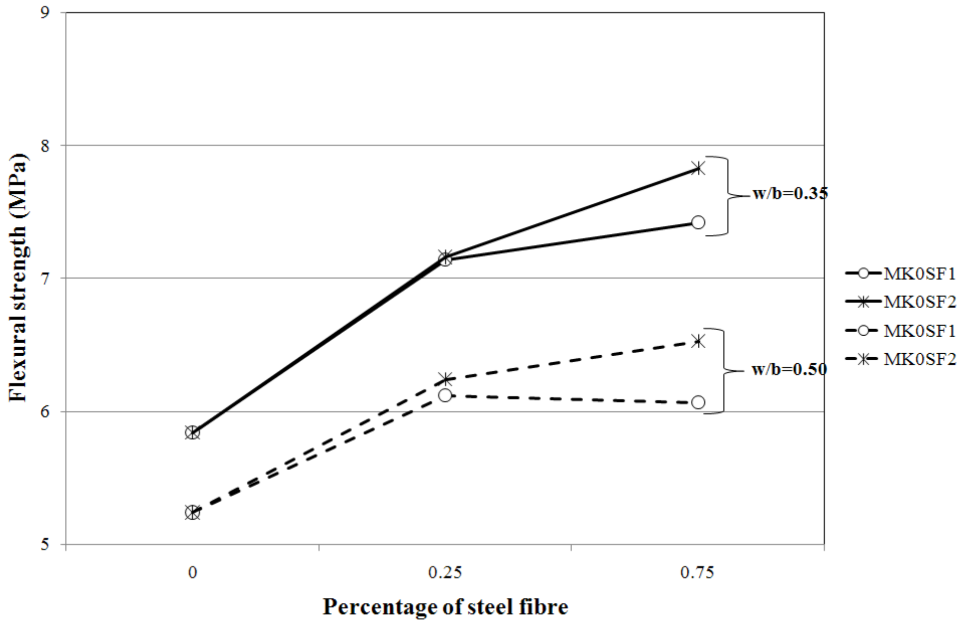


a)

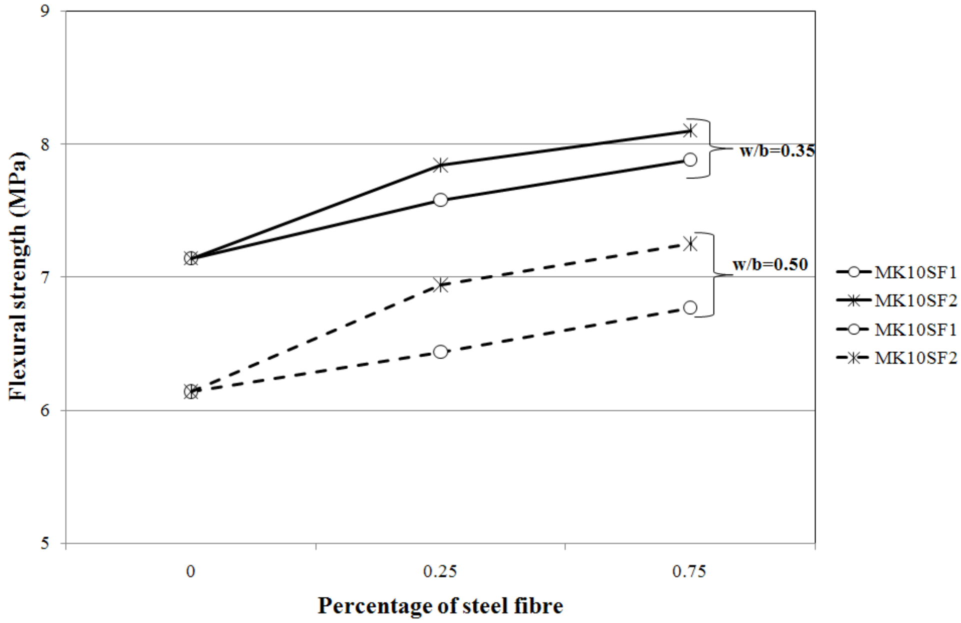


b)

Figure 4.2 Effect of steel reinforcement on the splitting tensile strength of a) plain and b) MK incorporated concretes



a)



b)

Figure 4.3 Effect of steel reinforcement on the three-point flexural strength of a) plain and b) MK incorporated concretes

4.3. Bonding Strength

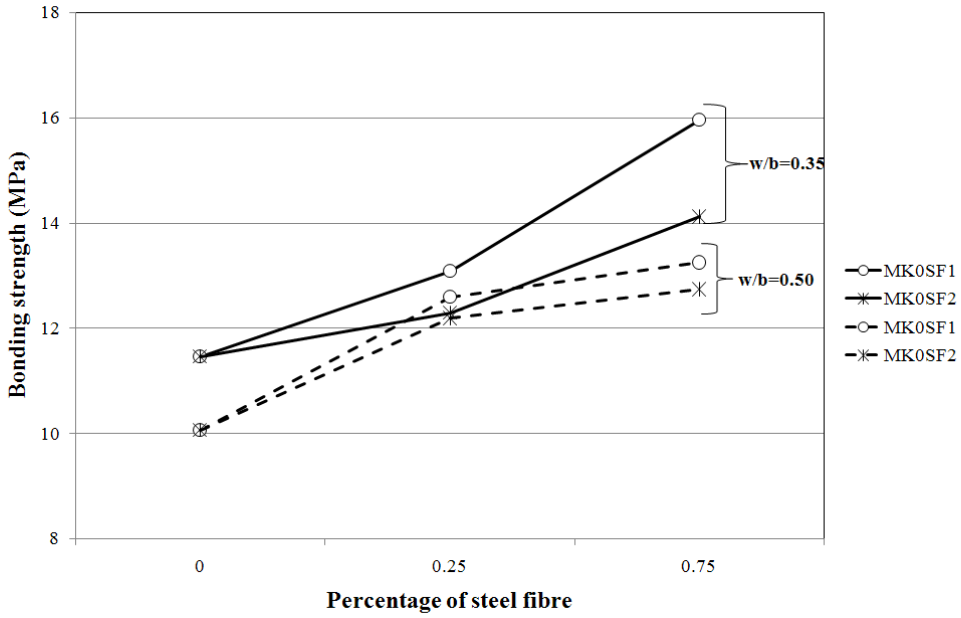
Bonding strength of the concretes versus the amount of the steel fiber reinforcement is showed in Table (4.1) and plotted Figure 4.5. The figure depicted that the increase in the volume fraction of SF resulted in great change in the bonding strength. However, the bonding strength values at 0.25% volume fraction appeared to have close values, regardless the incorporation of MK. Nevertheless, inclusion of MK to the concretes imparted additional performance in terms of bonding strength. For example, at w/b ratio of 0.35, the highest bonding strength for MK modified concretes was observed as 16.9 MPa, while the minimum value for plain concrete was observed as 12.5 MPa. Therefore, 35% enhancement in bonding strength capacity was accomplished by combined incorporation of MK and steel fibers. The similar trend was also observed for the concrete group with w/b ratio of 0.5.Baran et al. [89] stated that steel fibers improve the pull-out resistance of strands by controlling the crack growth inside concrete blocks. They stated that, by this way, the level of confinement at the strand-concrete interface was increased, which resulted in improvements in both friction and mechanical bond components of the resistance. Their results also indicated that more than 30% increase was achieved in pull-out strength due to fiber reinforcement.

Being one of the most popular mineral admixtures MK is known to have comparable contribution to the mechanical and durability performance of concretes as silica fume does [22, 24, 25]. However, the studies regarding the effect of inclusion of MK on the bonding strength between concrete and steel bars has not yet attracted the adequate attention. The previous results presented for silica fume incorporated steel fiber reinforced concretes may highlight the effectiveness of utilization of MK for this purpose. In the study of Chan and Chu [90], the influence of silica fume on the bond properties of steel fiber in matrix of reactive powder concrete (RPC) were studied. They performed pullout tests in their experimental program, with the silica fume content as the primary variable. They indicated that the incorporation of silica fume in RCP matrix greatly enhanced the fiber–matrix bond. Abu-Lebdeh et al [91] also revealed that the quality of matrix has prominent importance on the bonding and tensile strain capacity of steel fibers in high strength concrete. Consequently, owing to its superior enhancement in cement matrix as a result of pore size refinement [28], MK provided improvement in the pullout capacity of the reinforced concretes.

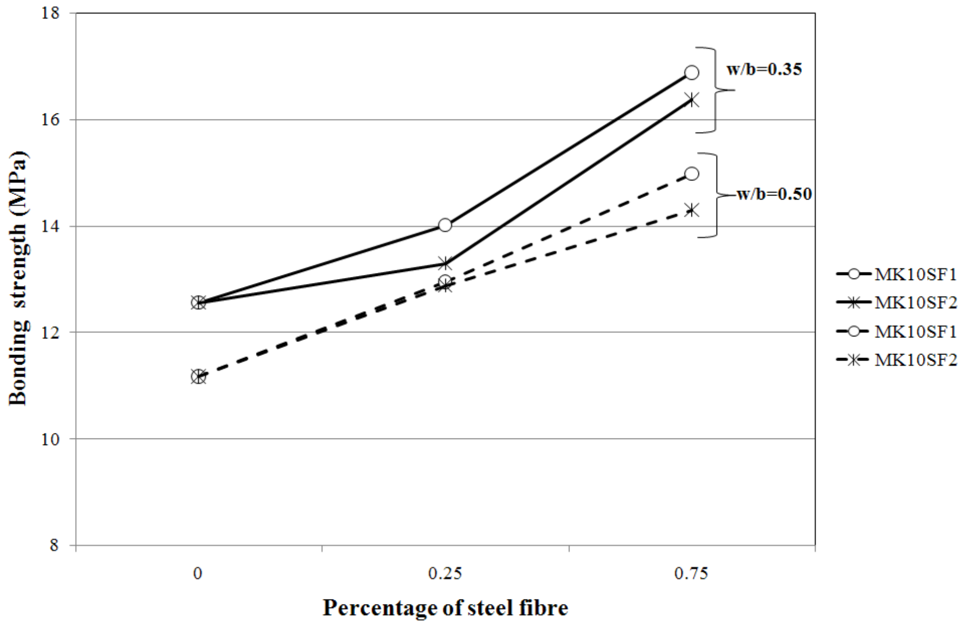
Photographic views of the pullout specimens tested in the current study are given in Figure 4.4. As can be seen, after failure, the reinforcing steel bars were separated from the concretes without steel fiber, whereas steel fiber reinforced concretes did not release the steel bars.

(a) (b)

Figure 4.4 Typical failure patterns of concretes a) without fibre reinforcement and b) with fibre reinforcement



a)

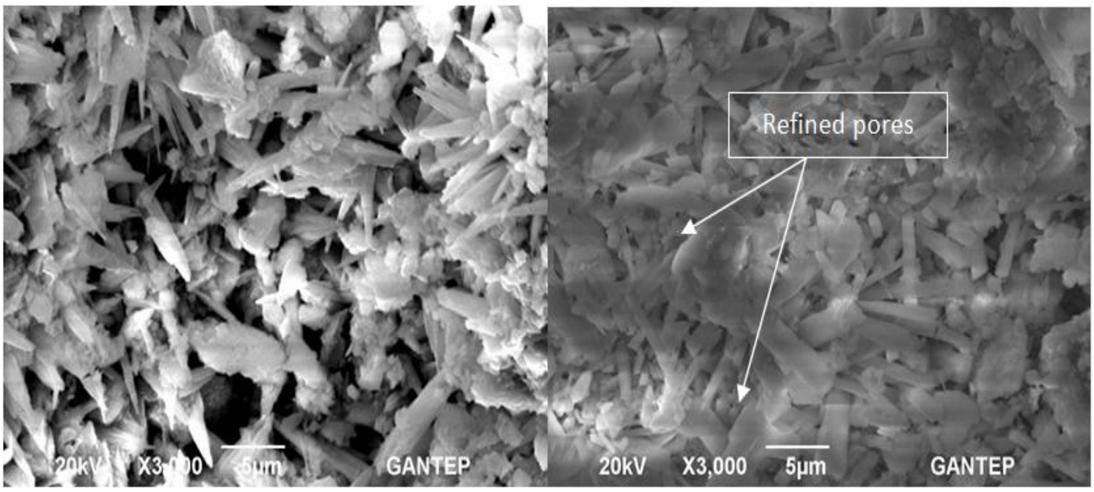


b)

Figure 4.5 Effect of steel reinforcement on the bond strength of a) plain and b) MK incorporated concretes

4.4. SEM image analysis

The change in cement paste matrix of the concrete made with w/b ratio of 0.35 due to the incorporation of MK is demonstrated in Figure 4.6. As seen from Figure 4.6a, there were large pores in plain concrete while a great pore refinement was provided by 10% inclusion of MK into the concrete. Refinement of the pore structure of the concrete led to better mechanical properties mentioned above. When observing Figure 4.7, it was pointed out that better interfacial transition zone (ITZ) between steel fiber and concrete was observed in 10M 0.75SF1 concrete than 0M0.75SF1. As a result of the improvement of adherence between SF and concrete, toughness, ductility, and fracture energy of concrete may be enhanced.



a) b)

Figure 4.6 Refinement in pore structure of the paste matrix for w/b ratio of 0.35 a) plain and b) MK incorporated

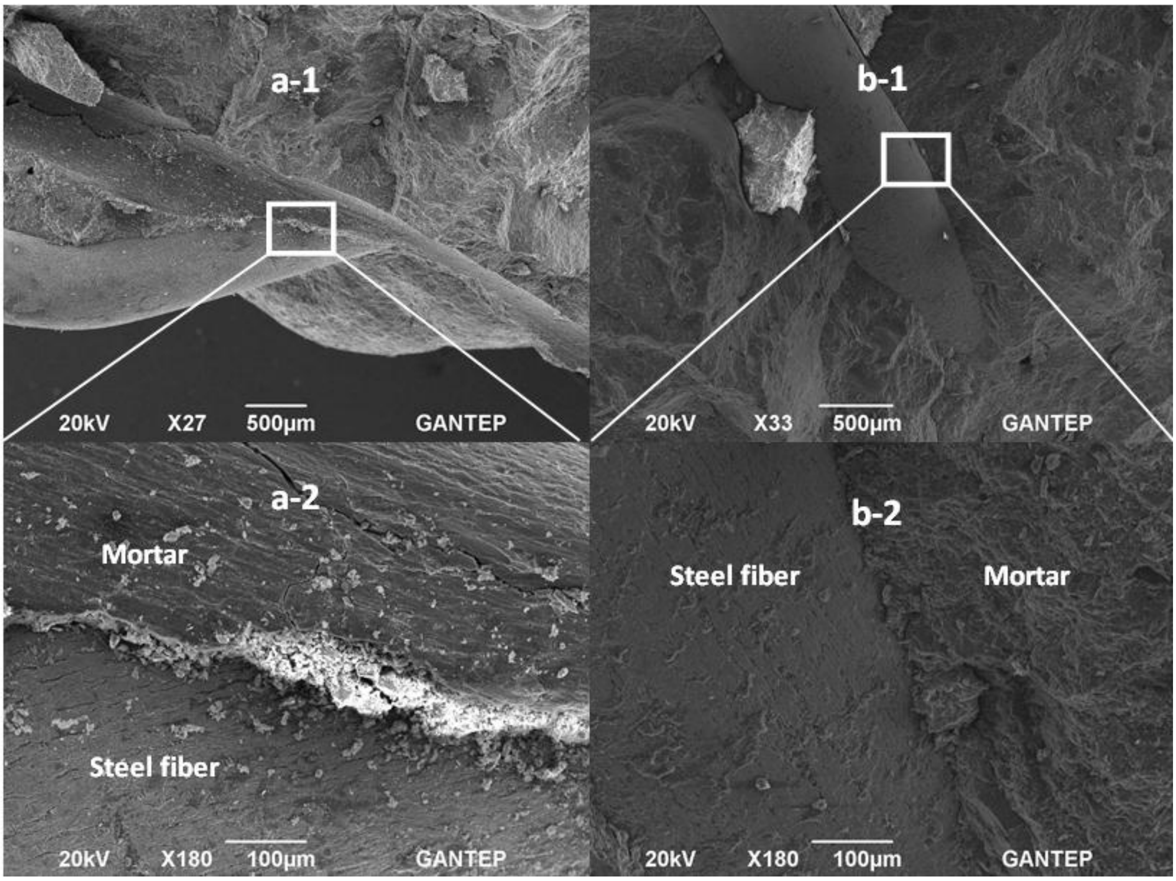


Figure 4.7 Interface between steel fiber and mortar phase of concrete a) 0M0.75SF1 for w/b ratio of 0.35 and b) 10M0.75SF1 for w/b ratio of 0.35

4.5. Statistical evaluation of the test results

A general linear model analysis of variance (GLM-ANOVA) was carried out at a 0.05 level of significance to examine the variation in the tested features of the fiber reinforced concretes in a quantitative manner. For this, compressive strength, splitting tensile strength, flexural strength, and bonding strength of the concretes were assigned as the dependent variables while the type of the steel fibers used, incorporation of MK and w/b ratio were the factors. A statistical analysis was performed to specify the statistically significant (p-level<0.05) factors. The contributions of the factors on the measured test results are also presented in Table 4.2. The column under the percent contribution provides an idea about the degree of effectiveness of the independent factors on the measured response such that the higher the contribution, the effectiveness of the factors to that particular response was higher. Likewise, if the percent contribution is low, the contribution of the factors to that particular response is less. It was observed in Table 4.2 that all of the independent variables had significant effect on the mechanical properties of fiber reinforced concretes. When observing the contribution levels of the factors, it was noticed that the most important parameter in variation of the compressive strength and splitting tensile strength of the fiber reinforced concretes is w/b ratio. However, the influence of using different type of steel reinforcement was observed to be more dominant at bonding strength and flexural strength of the fiber reinforced concretes. Besides, the utilization of MK was also proved to be effective on all of the parameters at moderate levels.

Table 4.2 Statistical analysis of the test result

Dependent Variable	Independent variable	Sequential Sum of Squares	Computed F	P Value	Significance	Contribution (%)
Compressive strength	w/b ratio	1304.11	459.23	0.000	YES	88.2
	MK replacement	54.65	19.24	0.001	YES	3.7
	Type of steel fibre	86.16000	30.34	0.000	YES	5.8
	Error	34.08000	-	-	-	2.3
	Total	1479	-	-	-	-
Bonding strength	w/b ratio	6.4389	13.03	0.004	YES	19.8
	MK replacement	5.5578	11.24	0.006	YES	17.1
	Type of steel fibre	14.5733	29.48	0.000	YES	44.8
	Error	5.9315	-	-	-	18.2
	Total	32.5015	-	-	-	-
Flexural strength	w/b ratio	2.4492	10.68	0.000	YES	24.9
	MK replacement	1.21	61.60	0.005	YES	12.3
	Type of steel fibre	4.9062	17.85	0.000	YES	49.9
	Error	1.2605	-	-	-	12.8
	Total	9.8259	-	-	-	-
Splitting tensile strength	w/b ratio	4.6118	9.47	0.000	YES	68.8
	MK replacement	1.1503	89.43	0.000	YES	17.2
	Type of steel fibre	0.357	30.72	0.002	YES	5.3
	Error	0.581	-	-	-	8.7
	Total	6.7001	-	-	-	-

4.6. Correlating between mechanical properties of the concretes

Correlating the experimental data is one of the most common practices among the researchers for assessment of the findings reported. Theoretically, the main elements controlling the mechanical properties of concrete are the relative volume fractions of paste matrix and aggregate, as well as their quality. As mentioned earlier higher compressive strength reflects improved mechanical behavior. To evaluate the bonding strength between reinforcement and concrete, correlating other mechanical properties with this parameter was carried out. For this, correlation between bonding strength and other mechanical properties for both w/b contents is respectively presented in Figures 4.8-4.10. Based on the facts presented above to specify the possible correlation between the mechanical characteristics of plain and MK concretes with and without reinforcement, the correlation coefficients (R^2) were calculated and presented on those figures as well. The data used for these figures cover the entire test results obtained. Figures 4.8 and 4.9 revealed that the best correlation was achieved by polynomial curve fitting for compressive and flexural tensile strength while for splitting tensile strength exponential curve fitting yielded the highest correlation, without depending on the w/b ratio. Since the scatter of the data for bonding vs. flexural tensile strength was observed to be more irregular than the others, R^2 value determined was relatively lower. However, as a result of the noticeable differences and uniformity for the measured values between bonding and compressive strengths, the strongest correlation was observed to take place between these parameters.

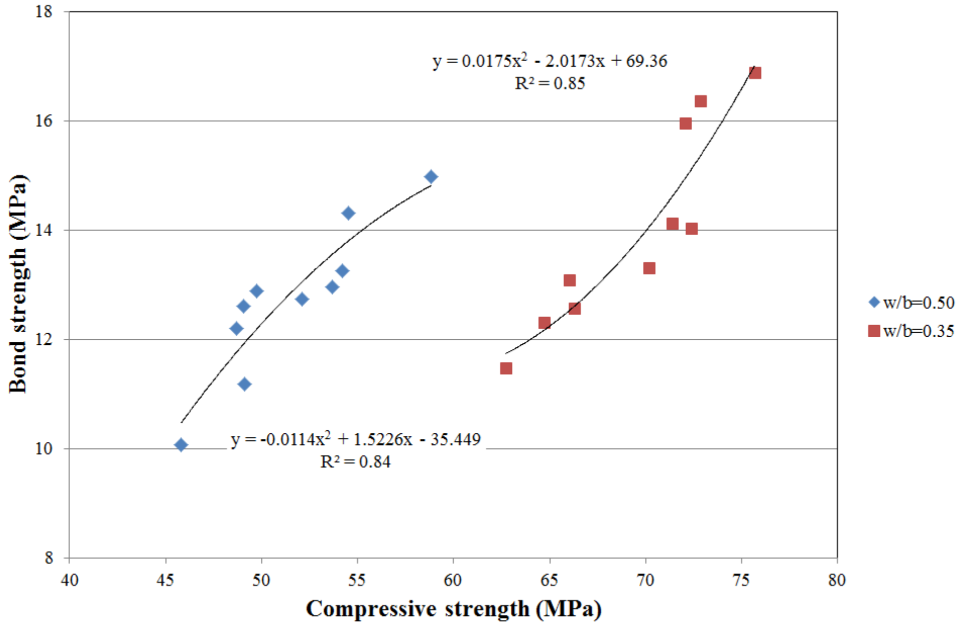


Figure 4.8 Correlation of bond strength vs. compressive strength

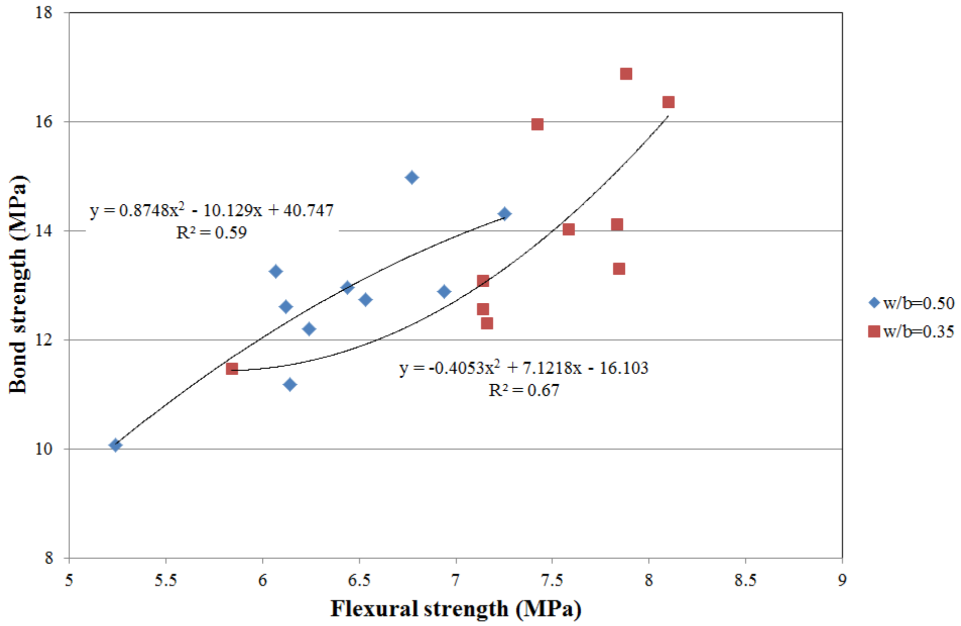


Figure 4.9 Correlation of bond strength vs. modulus of rupture

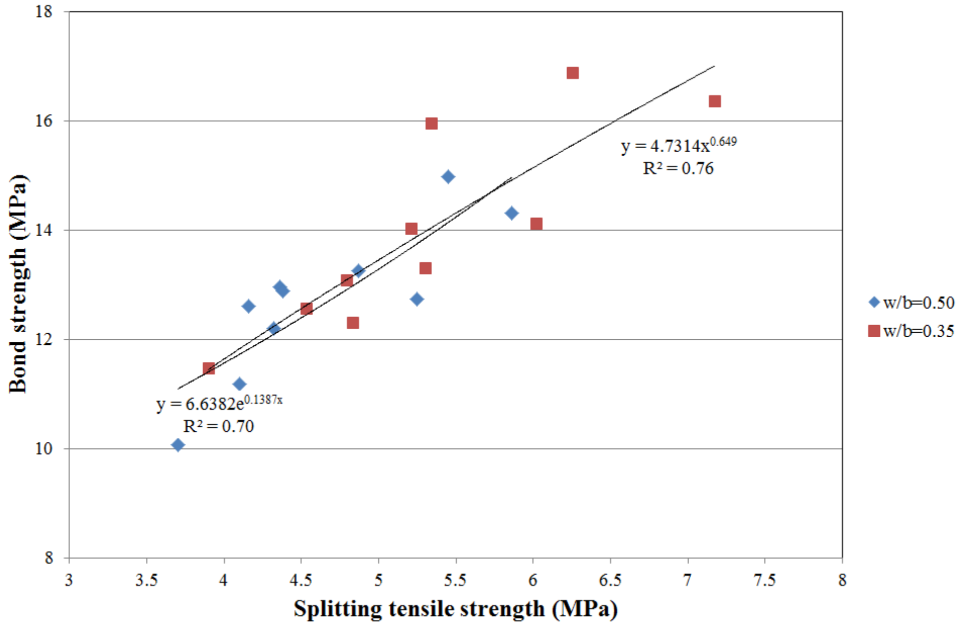


Figure 4.10 Correlation of bond strength vs. splitting tensile strength

CHAPTER 5

CONCLUSION

The following conclusions may be drawn based on the experimental results presented above.

- Use of MK as a replacement material resulted in enhanced mechanical properties of concretes compared to plain ones for both w/b ratios. The highest compressive strength values were measured as 75.7 and 58.8 MPa for concrete groups with w/b ratios of 0.35 and 0.50, respectively. The inclusion of steel fibers also contributed to the compressive strength. The long fibers (SF1) provided higher compressive strength development than SF2 incorporated concretes with increase in volume fraction. The level of improvement was more pronounced for MK concretes than plain ones.

- By incorporation of steel fibers remarkable improvement in bonding and tensile strength capacities of the concretes were observed. The steel fibers with higher length/aspect ratio (SF1) demonstrated higher development in bonding strength while the concretes incorporated with SF2 fibers had better performance in modulus of rupture and splitting tensile strength. This difference in the behavior of steel fiber reinforced concretes may be attributed to the dispersion and orientation of the steel fibers within the concrete.

- 0.25% volume fraction of the steel fibers revealed similar trends in contribution to the mechanical properties of the concretes for both of the SF type used. However, for the higher volume fraction of steel fiber, the behavior was more distinguishable.

  SEM image analysis visually proved that ITZ between steel fiber and concrete was improved as a result of pore refinement of concretes by MK incorporation.

- Statistical analysis revealed that w/b ratio, type of steel fiber and incorporation of MK are all influential factors at varying levels on mechanical properties of the concretes. Especially, for bonding and flexural strength the type of the steel fiber had the greatest effect. Besides, the contribution levels of MK incorporation on the mechanical properties were observed to vary up to 18%.

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APPENDIX A: Photographic Views

Figure A1 Photographic view of used materials (Portland cement, aggregate, Super plasticizer, water).

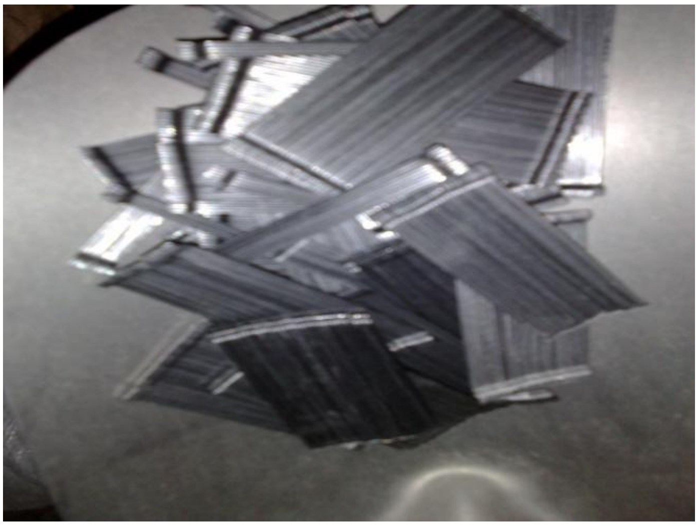

Figure A2 Photographic view of hook ended steel fiber

Figure A3 Photographic view of mixture

Figure A4 Photographic view of slump test-1

Figure A5 Photographic view of slump test-2

Figure A6 Photographic view of samples-1

Figure A7 Photographic view of samples-2

Figure A8 Photographic view of samples-3

Figure A9 Photographic view of the samples during curing

Figure A10 Photographic view of three point flexural testing

Figure A11 Photographic view of flexural specimens

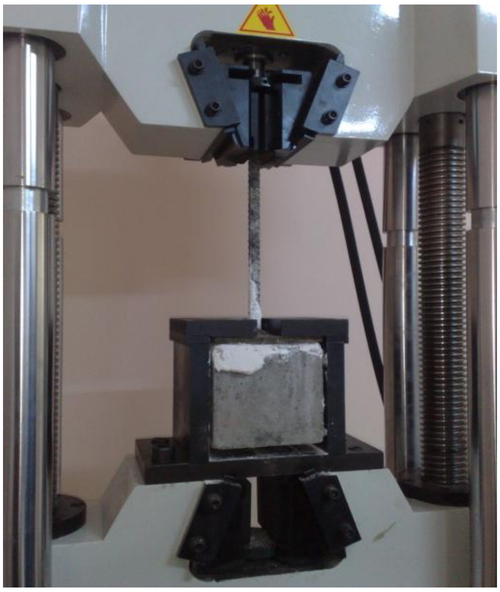

Figure A12 Photographic view of bond strength specimens and pull out device

Figure A13 Photographic view of pullout specimens (splitting failure)

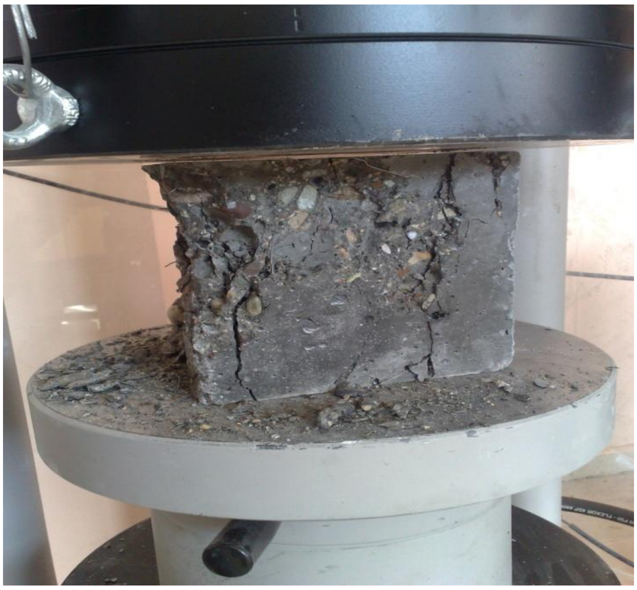

Figure A14 Photographic view of compressive strength specimens