

**MECHANICAL BEHAVIOUR OF NORMAL AND HIGH STRENGTH
CONCRETES UNDER COMPRESSION AND BENDING**

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İSTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ

**NORMAL VE YÜKSEK DAYANIMLI BETONLARIN BASINÇ VE EĞİLME
HALLERİNDE MEKANİK DAVRANIŞLARI**

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FOREWORD

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ABBREVIATIONS

LEFM	: Linear elastic Fracture Mechanics
NFM	: Nonlinear Fracture Mechanics
LSC	: Low Strength Concrete
HSC	: Normal Strength Concrete
HPC	: High Performance Concrete
K_{IC}	: Critical value of SIF
SIF	: Stress Intensity Factor
TZ	: Transition zone
LVDT	: Linear variable differential transformer

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LIST OF SYMBOLS

E_c	: Elastic modulus of cement
E_m	: Elastic modulus of mortar
E_t	: Elastic modulus of transition zone
E_a	: Elastic modulus aggregate
σ	: Stress
P	: Peak load
U_r	: Relative absorbed energy
G_f	: Fracture energy
ΣG_f	: Total fracture energy
$G_{f \text{ (peak)}}$	: Fracture energy under elastic region
l_{ch}	: Characteristic length
f_c	: Compressive strength
f_{flex}	: Bending strength
f_{sp}	: Splitting tensile strength
f_t	: Tensile strength
ε	: Strain
ε_u	: Ultimate strain

MECHANICAL BEHAVIOUR OF NORMAL AND HIGH STRENGTH CONCRETES UNDER COMPRESSION AND BENDING

SUMMARY

In this study, mechanical behaviour of low, normal and high strength concretes was investigated for better understanding the fracture behaviour of concrete. Based on uniaxial compression and bending tests, mechanical properties and also fracture properties of concrete such as fracture energy and characteristic length were evaluated. During testing process, except chemical admixtures, sources of all constituents were kept constant. Water / cement ratio and constituents of materials used were variable. In addition, specimens were cured in the same curing conditions.

Experimental studies showed that high strength concretes absorb less relative fracture energy compared to low and normal strength concretes. However, strain capacities of high strength concretes are higher than those of low and normal strength concretes.

According to the experimental results, elastic modulus of the specimens were predicted by means of the available composite models. Predicted and experimentally measured results were compared in order to observe the efficiency of the models.

The following conclusions can be drawn from the results obtained in this thesis. Splitting tensile strength increases with increasing compressive strength of concrete but this increase is not significant in high strength concretes. In other words, as the compressive strength increases the “splitting tensile strength / compressive strength” ratio decreases.

After completion of the splitting tests, the fracture surfaces were examined. In low and normal strength concretes of splitting test specimens, the crack usually does not traverse the aggregate due to their weak matrices, thus inter-granular type of fracture occurs in these concretes. However, in high strength concrete, the cracks usually travel through the aggregate, and fracture tends to be brittle in nature, thus trans-granular type fracture occurs in this concrete.

Characteristic length, which reflects the brittleness of concrete, decreases with increasing compressive strength of concrete.

Significant absorption rate was found for the specimens which have water / binder ratios greater than 0.55.

NORMAL VE YÜKSEK DAYANIMLI BETONLARIN BASINÇ VE EĞİLME HALLERİNDE MEKANİK DAVRANIŞLARI

ÖZET

Bu çalışmada, betonun kırılma davranışını daha iyi anlamak amacıyla düşük, orta ve yüksek mukavemetli betonların mekanik davranışı incelenmiştir. Yapılan basınç ve eğilme deneyleri sonunda mekanik özellikler yanında kırılma enerjileri ve karakteristik boyları gibi kırılma mekaniği parametreleri de incelenmiş ve gerekli değerlendirmeler yapılmıştır. Yapılan deneylerde kullanılan malzemelerin kaynağı, kimyasal katkılar hariç, sabit tutulmuş, su - çimento, agrega ve bunun gibi diğer malzemelerin oranlarında değişiklik yapılarak numuneler elde edilmiştir. Elde edilen numuneler aynı ortamda üretilmiş yine aynı ortamda küre tabi tutulduktan sonra deneyler uygulanmıştır.

Çalışmalar sonucu yüksek mukavemetli betonların daha az bağıl enerji absorbe ettiği görülmüştür. Buna karşın yüksek mukavemetli betonlar diğer dayanımdaki betonlara kıyasla daha fazla şekil değiştirme kapasitesine sahiptir.

Deneyde kullanılan betonların elastisite modülleri mevcut modeller yardımıyla tahmin edilmiş ve bu modellerin ölçülen değerlerle uyumu grafik üzerinde karşılaştırma yapılarak sunulmuştur.

Yarma deneylerinde beton mukavemetindeki artışın yarmada çekme dayanımını arttırdığı görülmüş, fakat, betondaki basınç dayanımındaki artışın, yarma deneylerinde elde edilen çekme dayanımı artışından daha fazla olduğu sonucuna varılmıştır. Diğer bir deyişle, basınç dayanımı arttıkça yarma-çekme dayanımı/basınç dayanımı oranı azalmıştır.

Yarma dayanımı deneylerinde, kırılma sonrası numune yüzeyleri karşılaştırılmış ve düşük mukavemetli betonlarda agregaların tam olarak kırılmadığı gözlemlenmiştir. Bu gözlem doğrultusunda beton mukavemetinin çatlak yöneliminde önemli bir etken olduğu sonucuna varılmıştır. Düşük mukavemetli betonlarda oluşan çatlaklar agregaların etrafında dolaşırken, yüksek mukavemetli betonlarda agregaların içinden geçtiği gözlemlenmiştir.

Betonda gevrekliği belirten karakteristik uzunluk, betonun mukavemetinin artmasıyla birlikte azalmaktadır.

Su / bağlayıcı oranının 0,55' ten yüksek olduğu betonlarda su geçirgenliğinin kaydadeğer biçimde arttığı gözlemlenmiştir.

1. INTRODUCTION

1.1 Background

For specific structures like long span bridges, dams, nuclear reactors, offshore structures and storage tanks, cracking behaviour of concrete is a very important and essential issue. For this purpose, tensile strength and strain capacity of concrete come into question. In order to evaluate these properties, fracture mechanics of concrete can be taken into the consideration. Strains at peak stresses should be calculated to take the control of cracking and determining the risk of cracking and expected failure.

Material behaviour can be evaluated by examining the complete stress – strain curve of concrete. For HSCs, brittleness is worthy of concern in evaluating mix designs and deciding safety factors. Strength, modulus of elasticity, characteristic length, absorbed fracture energy and some physical properties are significantly affected by the water / binder ratio.

Water / binder ratio also affects the water absorption rate of the concrete. Cementitious binders act as filler in the concrete structure and improve the impermeability of the concrete.

Recent research shows that the matrix surrounding the aggregate has quite different stiffness properties than that away from interfacial zone. As a result, it is necessary to consider concrete as three phased structure including aggregates, matrix and transition zone [1].

1.2 Objectives and Scope

A wide range of specimens was evaluated and prepared for the investigation of the fracture behaviour and strain capacity of concrete. Low, normal and high strength concretes were compared in order to understand the relation between internal structure and mechanical behaviour.

Improvement of the bending, splitting tensile strength was compared with the development of the compressive strength. Fracture parameters such as, relative fracture energy up to the compressive strength and characteristic length of concrete were calculated, compared and correlated with the compressive strength of concrete.

The permeability test was performed to analyze the influence of the water / binder ratio on the permeation property of concrete.

2. LITERATURE STUDY

2.1 Physical and Mechanical Properties of Concrete

2.1.1 Tensile strain capacity of concrete

Tensile strength is not considered in conventional design of reinforced concrete structures. In addition to this, research of concrete has been focusing on the improvement of compressive strength during the last two decades.

For better understanding of concrete behaviour, more information about mechanical properties, such as fracture properties and tensile strength must be provided [1].

Giovambattista et al showed that concrete strength improves the tensile strain capacity in bending [2].

Although concrete is weak in tension its tensile behaviour is important in the design of structures like dams, bridges, offshore structures and nuclear reactors. It is also necessary to consider tensile behaviour of concrete in determining the risk of cracking [3].

Tasdemir et al. [3] indicated that “Concrete is much more sensitive to tensile failure than to shear failure and it is impossible to create a pure shear stress distribution in a test specimen without introducing tension.” Additionally, cracking of the concrete is more dependent on the strain capacity rather than the strength .

Tensile strain capacity is affected by the mix composition, curing condition, gauge length, specimen size, loading rate and the presence of a notch [3].

The stress -strain curve of concrete in tension is almost linear up to about 80% of the tensile strength. However, nonlinearity begins due to micro cracking initiation. Behaviour of concrete in tension is schematically shown in Figure 2.1.

Experimental results show that as the volume of coarse aggregate increases, the direct tensile strain capacity of concrete decreases [4]. Results were confirmed in bending [5] and biaxial compression-tension [6].

According to the study of Houghton, it has been found that crushed coarse aggregate and milled sand usage increases the tensile strain capacity of concrete [7].

Tasdemir et al [3] stated that increase in coarse aggregate volume fraction with mean size of aggregate and with w / b ratio do not improve the tensile strain capacity, but the strength does. In normal strength concretes, strength improvement increases the direct strain capacity. Moreover, high strength lightweight concretes show better performance in tensile strain capacity.

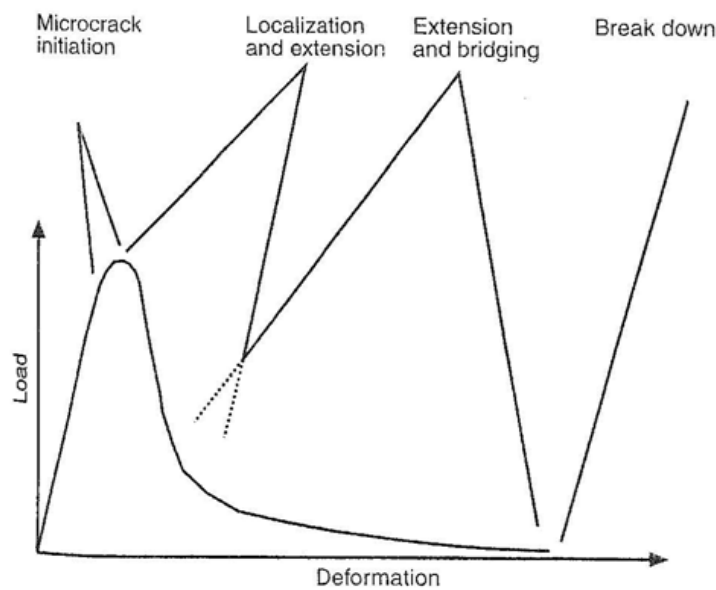


Figure 2.1: Schematic representation of stress-strain curve of concrete in tension [3].

2.1.2 Fracture mechanics approach

Fracture mechanics of concrete is essential for structures like reactors, earthquake resistant buildings, high cost projects and structures, which need climate – environment resistance.

Fracture mechanics examines how, where and in which circumstances the failure starts and the effects that lead the existent crack to propagate (stable or unstable). For this purpose, Linear Elastic Fracture Mechanics (LEFM) was developed. However, LEFM can be applied to hardened cement paste, but for quasi- brittle and heterogeneous materials like concrete, it is not efficient. For this reason, LEFM is modified to Nonlinear Fracture Mechanics (NFM) [8].

Material properties of concrete have a substantial effect on crack initiation and propagation. For example, high strength concretes (HSCs) show more brittle behaviour compared to low and normal strength concretes. On the other hand, after crack initiation, fiber-reinforced concretes show more tough behaviour. Testing laboratory-size specimens or real-sized specimens changes the obtained result thus; modelling various sized materials with different properties can be conducted by utilizing fracture mechanics [8].

If a material contains air void, stress distributions will concentrate around void. Thus, similar process causes stress concentration at the tip of a crack. This situation reduces stress distribution at the bottom and the top surface of crack as seen in Figure 2.2. LEFM supports that stress goes infinite as the crack becomes sharp [7].

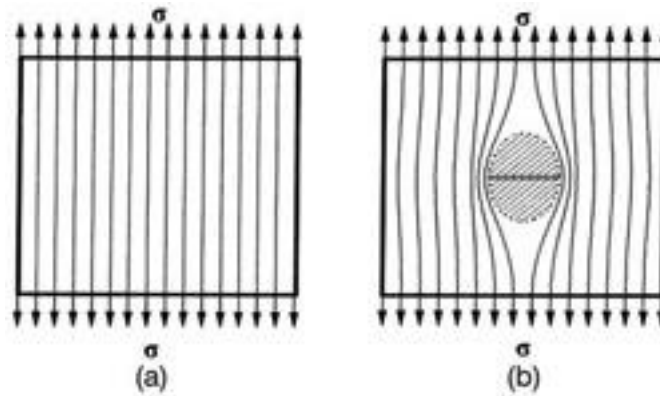


Figure 2.2 : (a) Stress distribution in the material without void in tension. (b) Stress distribution in the material including void [7]

To determine crack propagation and failure type of the brittle materials, fracture toughness (K_{IC}) is sufficient. Fracture toughness of the material can be experimentally calculated by testing specimens with different geometries if they include a certain crack [8].

In contrast, metallic materials have yielding process before failure; so stress distribution at the crack tip will not go infinite and plastic region will occur. (Figure 2.3)

Recent research shows that the mechanical behaviour of concrete is depending on the properties of the interfacial zone, which is the weakest link between aggregate and cement paste. This zone mainly affects the fracture process of concrete [3].

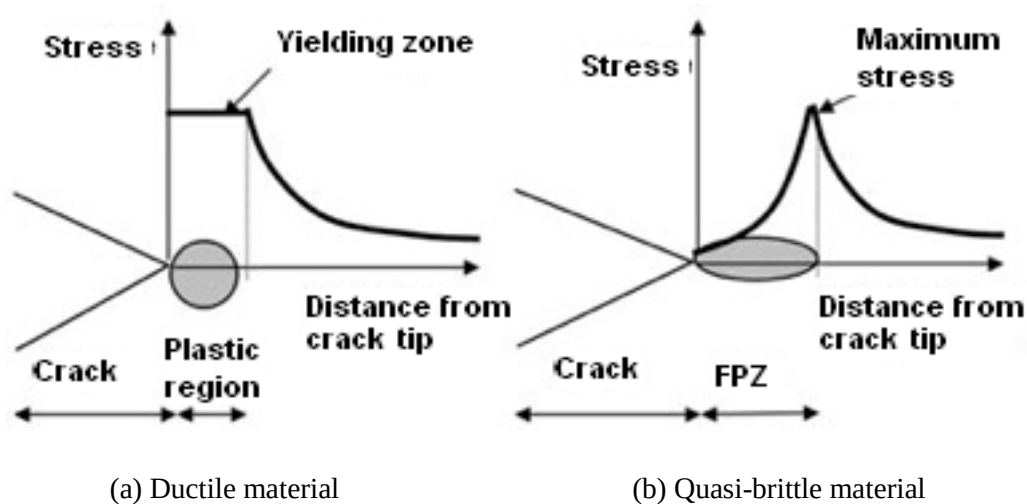


Figure 2.3 : Stress distribution at the crack tip for ductile and quasi-brittle materials

The energy necessary to create unit crack area is defined as fracture energy, which is important in realizing mechanical behaviour of concrete. Additionally, fracture energy, owing to interlocking resistance which develops in the crack zone as deformation increase after peak load, will increase with greater crack tortuosity [9].

2.1.3 Deformation of concrete

Oladapo's study [10] stated that there are three possible causes of cracking;

- Interfacial tensions occur due to the differences in the moduli of elasticity of the aggregates and mortar, which may cause bond cracks.
- Cracks in the mortar
- Rupture of the aggregate particles

Hsu et al [11] indicates that bond cracks can be found at the interface between coarse aggregates and mortar even if specimens were subjected to low loads or situated in unloaded conditions.

These types of crack do not cause structural failure but determine the shape of the curvature of the stress – strain curve [11].

For concretes prepared from similar aggregates, relative mortar strength takes responsibility for the critical load.

Crack propagation and consequent failure are also dependent on the strength of the mortar. Thus, higher peak load and larger strain capacity load are usually provided by the higher tensile strength.

Early stage of cracking occurs in the interfacial zone between aggregate and mortar. However, it is not the only cause of failure [10].

Micro cracks develop at the interfaces, as the load increases; bond cracks penetrate into the mortar [8].

Bond cracks and mortar cracks are linked at high loads, and generate crack net, which are consisting of continuous crack patterns. Accordingly, the limit tensile strength is reached and finally failure begins.

2.2 A Composite Modelling Approach

2.2.1 Material modelling

Tasdemir et al. [1] focused on the three phase composite model approach and conducted several studies. These models were generated for evaluating the strain values at peak stresses in concrete.

Wittmann [12] states the idea of three levels for modelling concrete: micro-, meso- and macro- levels. In order to, understand the behaviour of macro level, calculations and information about meso level must be provided.

Recent researches [13-15] focused on two principal aspects; (i) the micro-structural features of the interfacial region and their effects on concrete properties, and (ii) Models of the effects of interfaces on the properties of concrete through the application of continuum mechanics and fracture mechanics. The mechanical behaviour is remarkably influenced by the interfacial zone properties, because the interface between mortar and aggregates is the weakest link of the concrete.

The development of bond cracks between the aggregates and mortar is responsible for the inelastic behaviour of concrete.

Interfaces take the considerable total strain and as the bridging bond cracking initiates, the failure occurs in mortar [16,17].

2.2.2 Three phase composite model

Concrete is consisting of two-phase composite material: coarse aggregate and continuous mortar. The interface between the aggregates and the cement paste plays an important role in the fracture of concrete because of its weakness. Thus, it is necessary to consider the interfacial zone for the mechanical behaviour and fracture of concrete. This approach will be very helpful for better understanding of the material behaviour [1].

2.2.3 Stress distributions around the aggregate

The stress distributions around the aggregate were modelled and obtained by the help of the study of Tasdemir et al [1]. Hard inclusion in the soft matrix and soft inclusion in the hard matrix were compared according to the stress distributions around the aggregates, at the interface and in the matrix.

Stress distributions of hard and soft inclusion cases are represented in Figures 2.4 and 2.5.

The tangential stresses around the aggregate surface are higher in the hard inclusion cases than those of soft inclusion one.

The large differences between the elastic moduli of the aggregate and the matrix cause high amount of tangential, radial and shear stress distributions at the interface.

In specimens with soft inclusion, lower stress distributions were developed at the interfaces, because the variation of the elastic moduli of both aggregate and matrix is not significant enough.

In HSCs, tensile stresses formed at the tip of the aggregate, perpendicular to the applied load, and tangential stresses in the matrix close to the interface are greater than those of NSCs. Thus, crack at the interface is able to penetrate into the aggregate.

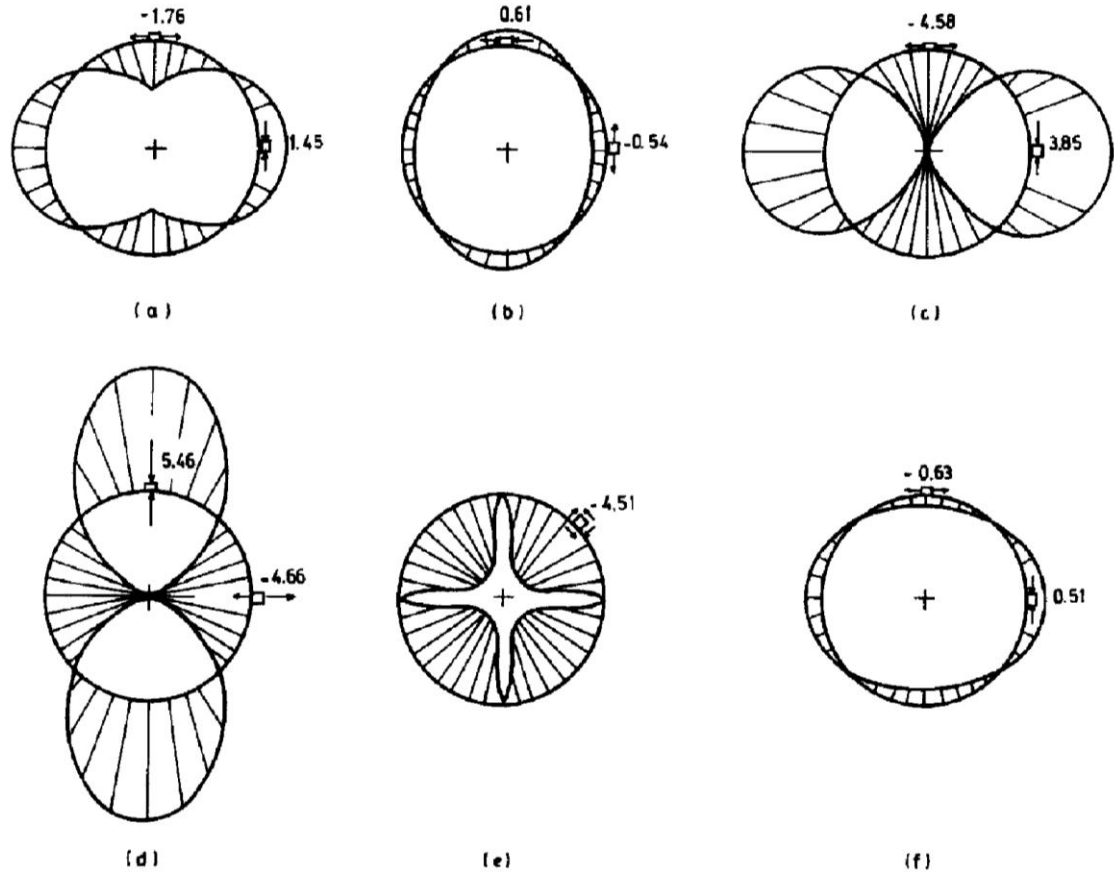


Figure 2.4 : Stress distributions (s ds) in soft matrix and around a hard inclusion: (a) tangential s. ds in the matrix close to the interface; (b) tangential s. ds at the interface; (c) tangential s. ds at the aggregate surface; (d) radial s. ds in the matrix close to the interface, at the interface and on the aggregate surface; (e) shear s. ds in the zones shown in (d); f) tangential s. ds at the interface for HSCs [1]

2.2.4 Influence of the transition zone

Yang [18] investigated the effect of the transition zone with different volume fractions (0.0, 0.1, 0.2, 0.3, 0.4 and 0.5) in three-phase composite model. Additionally, the two-phase model solutions and three phase model solutions were compared with experimental results. Three different thickness ($ht = 0 \mu\text{m}$, $20 \mu\text{m}$ and $40 \mu\text{m}$) of transition zone were observed in the comparison of these two models.

Two-phase model, without transition zone ($ht = 0 \mu\text{m}$), includes only cement paste and fine aggregates. Figure 2.6 illustrates the relationship between volume fraction of aggregate, f_a , and elastic modulus of two-phase composite (mortar).

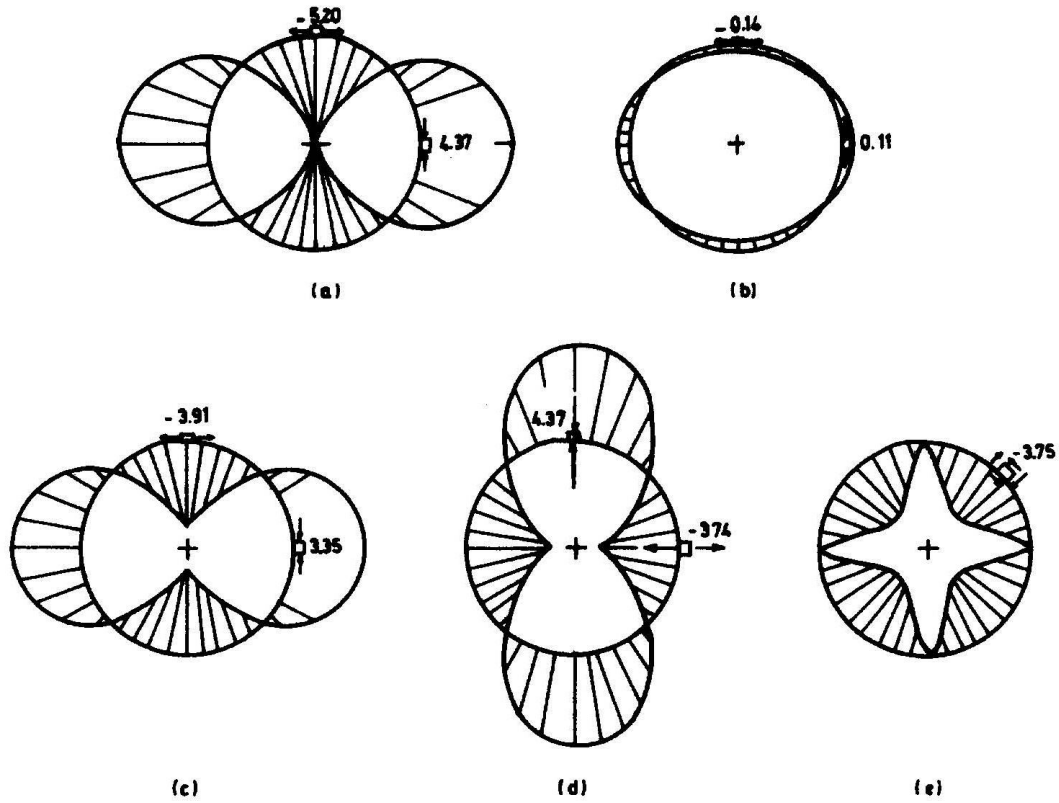


Figure 2.5: Stress distributions (s ds) in hard matrix and around a soft inclusion; (a) tangential s. ds in the matrix close to the interface; (b) tangential s. ds at the interface; (c) tangential s. ds at the aggregate surface; (d) radial s. ds in the matrix close to the interface, at the interface and on the aggregate surface; (e) shear s. ds in the zones shown in (d) [1]

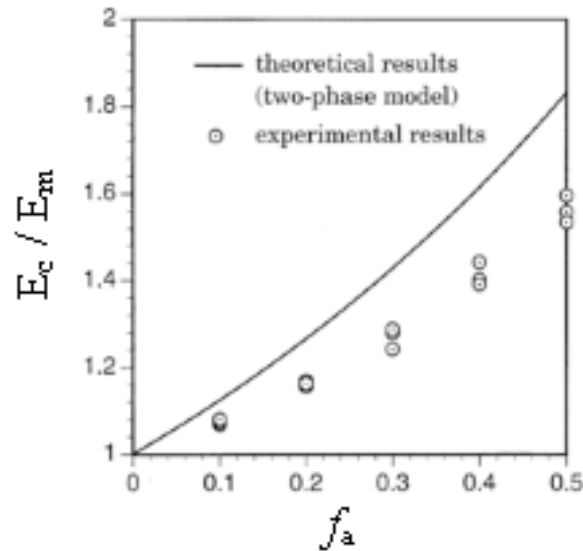


Figure 2.6: Volume fraction of aggregate (f_a) vs. E_c / E_m curves ($h_t = 0 \mu m$) [18]
(E_c : Elastic modulus of cement, E_m : Elastic modulus of mortar)

It was observed that the transition zone had a significant effect on overall elastic modulus of cement – based composite [18].

The effect of transition zone with 20 μm and 40 μm thickness were represented in Figures 2.7 and 2.8. In these figures, each continuous curve represents the elastic modulus of transition zone, E_t .

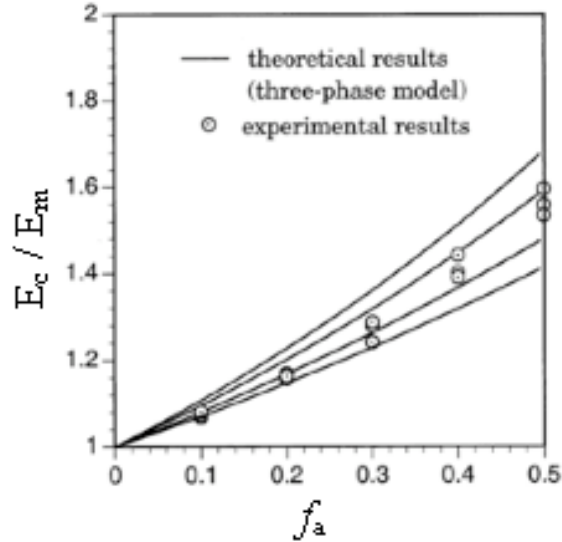


Figure 2.7: Volume fraction of aggregate (f_a) vs. E_c / E_m curves ($h_t = 20 \mu\text{m}$) [18]

In Figure 2.7, test results are within the curves of $E_t = 0.2E_m$ and $E_t = 0.4E_m$. (E_m = elastic modulus of mortar) It shows that the transition zone effect is significant in the composite.

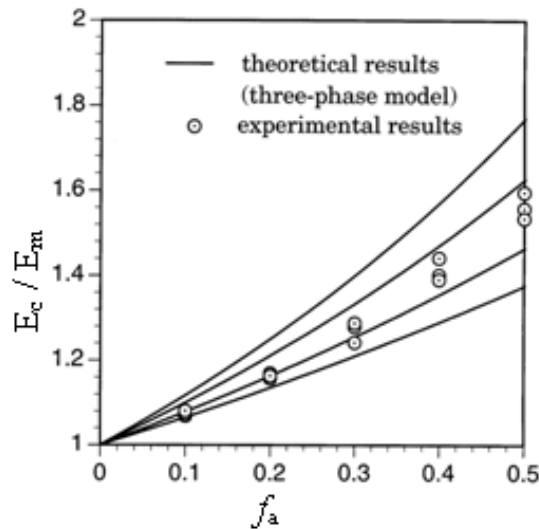


Figure 2.8: Volume fraction of aggregate (f_a) vs. E_c / E_m curves ($h_t = 40 \mu\text{m}$) [18]

In Figure 2.8, the graph correlates the volume fraction of the aggregate with the elastic modulus of the composite for various properties of transition zone. By taking the 40- μm thick interface zone, test results are within the curves of $E_t = 0.5E_m$ and $E_t = 0.7E_m$

In comparison with the experimental data, it was derived that transition zone becomes more effective as the volume fraction of fine aggregates increases.

Analytical and experimental results show that the average modulus of transition zone corresponds to 20 to 40% of modulus of matrix with a transition zone thickness of 20 μm . For a thickness of 40 μm , this ratio is about 50 to 70% of the matrix modulus [18].

Nilsen and Monterino [19] highlighted the importance of the thickness of transition zone and investigated the influence of transition zone on the elastic modulus of concrete.

Nilsen and Monteiro indicated that the mean spacing between aggregate particles is only 75 -100 μm . If the thickness of transition zone for ordinary concrete is about 50 μm , most of the hydrated cement paste exists in the transition zone. In other words, very little bulk hydrated cement paste lies within TZ. It can be concluded that the transition zone has an important effect on concrete properties.

However, in concrete with silica fume, the mean spacing between aggregate and cement paste is reduced to 8 – 10 μm [19], thus transition zone has less influence on silica fume concrete compared to ordinary concrete.

2.3 Testing Procedure

2.3.1 Strain measurement systems

In the study of Jansen et al. [20], cylinders of normal, medium and high strength concretes were tested under variables such as the rate of circumferential strain control, end conditions, strain measuring device, and specimens.

Cylinder specimens tested with the plate-to-plate and LVDT measurements. Instead of surface gages like LVDT, plate-to-plate gages were suggested in evaluating the post-peak; but the larger elastic moduli are measured by the LVDT compared to plate-to-plate measurement and the strain at peak of LVDT is lower than the other.

Jansen et al. [20] stated that “ the failure of the capping compound results in more of a splitting failure than the standard cone-type failure with high strength specimens. This type of failure indicates that the capping compound inadequately confines the ends of the cylinders.”

2.3.2 Notched and unnotched beam specimens

Uniaxial tensile tests are primarily preferred tests in evaluating tensile stresses. Moreover, tensile tests give more reliable results for the uniaxial mechanical behaviour. However, these tests are difficult to conduct. In this study, bending tests were applied instead of uniaxial tensile tests [9].

To obtain the strain-softening branch, a stiff testing machine with the test controlled by feedback from some continuously increasing deformation is necessary [9].

To observe crack propagation in the desired part of the specimens, a notch should be prepared. To prevent the risks due to the geometry, notching the notched specimen is necessary in uniaxial tensile tests [9].

Phillips and Binsheng [9] pointed out that under direct tension, notched and unnotched specimens did not show a significant difference and it is reliable to use both specimens for understanding the mechanical behaviour of concrete. Moreover, stress – deformation relation, fracture energy and failure deformations support this conclusion, and it is found that notch sensitivity is about 0.94 for tensile strength.

Experimental results prove that the use of silica fume increases the peak load of the concrete and makes the softening branch steeper and shorter. Moreover, the characteristic length decreases as the brittleness of concrete increases [15].

In addition, concrete with silica fume influences the crack propagation. Generally, cracks pass through the aggregate and generate transgranular type of fracture. This type of fracture indicates the brittleness of concrete.

In concrete without silica fume, cracks move around the coarse aggregate and spend more energy. On the other hand, the maximum aggregate size has a significant influence on the surface energy and characteristic length in concretes without silica fume [15].

3. EXPERIMENTAL WORKS

3.1 Materials

3.1.1 Portland cement and cementitious binders

Chemical composition of Portland cement , fly ash and silica fume are given in Table 3.1. Oyak cement was used as binder, Tunçbilek fly ash and Elkem micro silica were employed as supplementary cementitious materials. The Portland cement was CEM I 42, 5 N as classified by the TS EN 197-1 standard. Physical properties of Portland cement, fly ash and silica fume are given in Table 3.2.

Table 3.1 : Chemical compositions of Portland cement, fly ash and silica fume

Component %	Cement	Fly ash	Silica fume
SiO ₂	19,54	57,54	>85
Al ₂ O ₃	4,98	18,99	<2,0
Fe ₂ O ₃	5,58	10,34	-
CaO	62,85	3,79	-
MgO	1,33	-	0,42
SO ₃	2,69	0,66	0,13
Na ₂ O	0,18	0,17	0,1
K ₂ O	0,55	1,15	0,54
C ₃ A	3,76	-	-
Cl ⁻	0,0255	0,0062	<0,2

3.1.2 Aggregates

For each concrete mixture, proportions of aggregate were kept constant: 27 % natural sand, 26 % crushed sand, 23 % crushed stone I and 24 % crushed stone II.

Based on these ratios, calculated grading used in low, normal and high strength concretes for each concrete class is given in Table 3.3.

Table 3.2 : Physical properties of Portland cement, fly ash and silica fume

Property		Cement	Fly ash	Silica fume
Density (g/cm ³)		3,21	2,15	-
Specific surface (cm ² /g)		3835	-	210000
Compressive strength (MPa)	(2 days)	25,2	-	-
	(7 days)	41,1	-	-
	(28 days)	55,5	-	-
Loss on ignition	(%)	2,27	0,91	<3
Total alkali	(%)	0,54	0,93	-
Free lime	(%)	2,85	0,02	-
Volume expansion	(mm)	1	1	-

Table 3.3 : Aggregate gradings used in the mixtures

Agg.	Mix %	Sieve size (mm)										
		31,5	22	16	8	4	2	1	0,5	0,25	0,125	0,063
Coarse agg. II	24	100	100	66	2	1	1	1	1	1	1	0.8
Coarse agg. I	23	100	100	100	55	4	1	1	1	1	1	0.9
Crushed sand	26	100	100	100	100	85	55	30	16	6	3	2.5
Natural sand	27	100	100	100	100	100	96	79	52	23	4	2.1
Concr. Agg.	100	100	100	92	66	50	41	30	19	8	2	2
Grading												

3.1.3 Chemical admixtures

In this study, polycarboxylate ether based superplasticizers were used in the mixtures. These superplasticizers were Glenium 51 and Adva 616M. Additionally, air entraining agent (Darex AEA T) was included in order to obtain the target air content.

3.1.4 Fresh concrete

Four different concrete mixtures were prepared. Water/binder ratios of these mixtures were 0.25, 0.40, 0.55 and 0.83.

According to water / binder ratios, concrete mixes are designated as shown in Table 3.4.

Table 3.4 : Mix design designations

Water/ binder ratio	Designation
0,25	M-0.25
0,40	M-0.40
0,55	M-0.55
0,83	M-0.83

Additionally, these mixtures contained fly ash as mineral additive. On the other hand, M-0.25 was a completely different mixture than the others, because M- 0.25 was made with microsilica and Glenium 51.

Experimental values of the constituents of concrete are given in Table 3.5.

Table 3.5 : Concrete composition (kg/m³)

	Cem.	Wtr.	Nat. Snd.	Crsh. Snd.	Crsh. Rock I	Crsh. Rock II	Fly Ash	Slc. Fume	Spr. Plstc.	Air Entr. Agnt.
M-0.25	563	110	417	419	374	390	-	105	10,05	-
M-0.40	315	140	445	447	398	416	98	-	2,55	0,80
M-0.55	273	167	457	461	410	428	85	-	2,22	0,71
M-0.83	240	224	463	466	415	433	75	-	-	1,24

Experimental values of the constituents of concrete obtained by Çilek et al. [21] are given in Table 3.6. Concrete specimens produced by Çilek et al. were tested and evaluated.

Only one type of binder was included into the mixtures. In other words, either fly ash or microsilica was added into the mixtures.

Table 3.6 : Concrete composition obtained by Çilek et al. [21] (kg/m³)

	Cem.	Wtr.	Nat. Snd.	Crsh. Snd.	Crsh. Rock I	Crsh. Rock II	Fly Ash	Slc. Fume	Spr. Plstc.	Air Entr. Agnt.
0.40	320	142	335	477	493	442	100	-	2,59	0,83
0.55	270	165	452	455	405	423	84	-	2,2	0,70
0.80	234	218	452	455	405	423	73	-	-	1,2

The equivalent cement values were calculated as shown below:

According to TS EN 206 -1 standard, efficiency factors for silica fume and fly ash were chosen as 2 and 0.4 respectively.

$$C_{\text{equivalent cement}} = C_{\text{cement}} + 2 C_{\text{microsilica (solid)}} \quad (\text{without fly ash}) \quad (3.1)$$

$$C_{\text{equivalent cement}} = C_{\text{cement}} + 0.4 C_{\text{fly ash}} \quad (\text{without microsilica}) \quad (3.2)$$

Table 3.7. shows some properties of the fresh concretes.

Table 3.7 : Fresh concrete properties

Concrete type	Concrete Temperature (°C)	Slump (mm)	Solid volume fraction (%)	Unit weight (kg/m ³)	Air content (%)
M-0.25	29.0	220	84	2388	3.8
M-0.40	26.8	230	79	2261	6.4
M-0.55	22.4	240	79	2284	5.0
M-0.83	22.4	200	78	2317	0.7

3.2 Experiments

During testing, resources of all constituents were kept constant. M-0.40, M-0.55 and M-0.83 were air-entrained mixtures contained fly ash as mineral additive.

Except M-0.83, Adva 616M was used for all mixtures. There was no need to use chemical admixture for M-0.83 due to its high workability.

On the other hand, M-0.25 was a completely different mixture compared to the others, because M-0.25 was produced with microsilica and a different superplasticizer: Glenium 51.

In order to obtain good concrete from the workability point of view, the placing of a mixture must be followed by curing in suitable environment during the early stages for suitable hardening process. Thus, all specimens were demolded after about 24 hours, and fog curing applied with 98% humidity at 20 °C until 28th day, keeping concrete saturated.

3.2.1 Compressive strength

It is difficult to say which type of specimen, cylinder or cube, is better but in structural members, the situation is similar to the existing behaviour in a test cylinder. For this reason, testing cylinders is more realistic [23]. In this study, three cylinders of 150 mm diameter and 300 mm height were prepared for each water / binder ratio. All cylinders were capped with a sulphur capping compound at least several hours prior to the tests.

Uniaxial stress applied to the specimens prepared according to TS EN 12390 -3 standards. The cylinder specimens were tested using a closed-loop displacement controlled compression test machine of INSTRON 1000RD with the capacity of 5000 kN. Testing machine is shown in Figure 3.1. The loads and signals for displacements were recorded and stored by a computerized data acquisition system.



Figure 3.1: Testing machine (INSTRON 1000RD) with computerized data acquisition system

Stress and strain results were used to calculate modulus of elasticity, normalized stress- strain curves were used to obtain relative fracture energy. Relative fracture energy absorption capacity in compression was determined by calculating the area under the normalized stress- strain curve up to peak stress. In the ascending portion of each curve, both stress and strain axes were normalized, dividing by the compressive strength (f_c') and strain at peak stress (ϵ_u). Thus, based on the normalized curve, the relative absorbed energy, U_r , was calculated using the following equation (3.3).

$$U_r = \frac{\int_0^{\epsilon_u} \sigma(\epsilon) d\epsilon}{f_c' \epsilon_u} \quad (3.3)$$

The relative absorbed energy, U_r , must lie between 0,5 and 1 which reflects perfectly linear elastic or perfectly plastic behaviour of the concrete, respectively.

3.2.2 Splitting tensile test

In splitting tensile test, a concrete cylinder (or disc) is placed with its horizontal axis between the platens of a testing machine, the load is increased until failure by indirect tension in the form of splitting occurs. The acceptance of this test is based on the fact that stress distribution is reasonably uniform along the vertical diameter of the specimen. During the splitting test, the platens of the testing machine should not be allowed to rotate in a plane perpendicular to the axis of the cylinder.

For each type of mixes, three disc shaped specimens of 150 mm in diameter and 60 mm in thickness were used. Specimens was tested after 28 days.

Splitting tensile strength of disc specimens were calculated using the following equation (3.4):

$$\sigma = \frac{2.P}{\pi.D.b} \quad (3.4)$$

where, P represents the peak load, D is the diameter of the disc and b is the thickness of the specimen.

3.2.3 Three point bending test

Although, the results of this testing method are sensitive to the size of specimen, three point bending tests are easy to perform. In order to initiate the crack at the mispan, a notch should be created. Otherwise, the critical crack may develop at any section (generally around the middle section) because of the probability of a weak element being subjected to the critical stress.

The beams prepared for the RILEM fracture energy tests were 500 mm in length and 100x100 mm in cross section. Each beam has notch cut with 40 mm depth. The distance between support points, l , was determined as 400 mm. Bending test set up and schematic for the beam specimens are shown in Figures 3.2 and 3.3. Each beam tested under a loading rate of 0.7 N/sec. Deflections were measured simultaneously by means of a linear variable displacement transducer (LVDT) at midspan. At 28th day, energy graphics were obtained from three point bending tests.



Figure 3.2: Bending testing machine (INSTRON 1195) with computerized data acquisition system

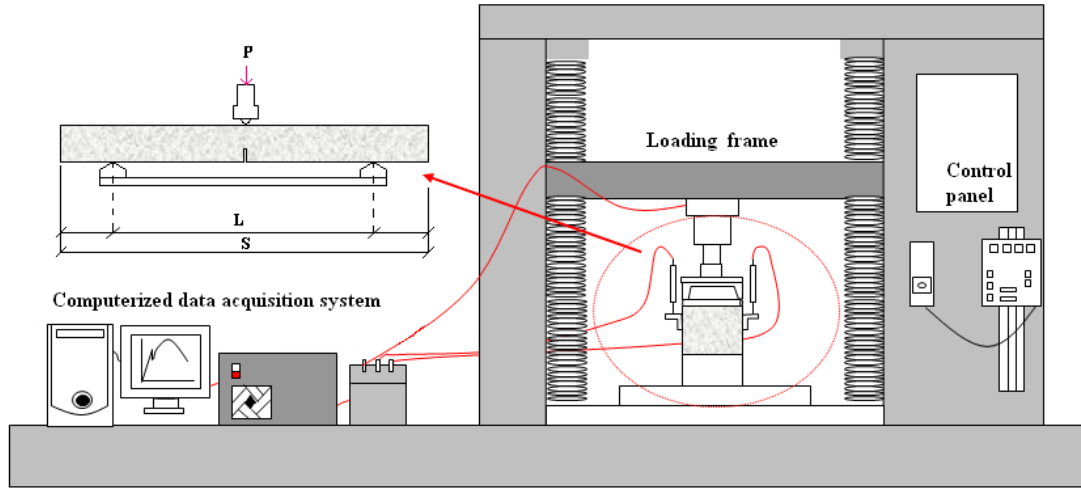


Figure 3.3: Testing schematic of the beam specimens

Fracture energy was calculated as indicated in Equation (3.5), the characteristic length was found as in Equation (3.6).

Fracture energy

$$G_F = \frac{W_0 + mg\delta}{A_{lig}} \quad (3.5)$$

Characteristic length

$$l_{ch} = \frac{E \cdot G_F}{f_t'^2} \quad (3.6)$$

3.2.4 Initial surface absorption test

Surface absorption tests are intended to examine the characteristics of the outer zone of concrete, which provides protection to reinforcement, that are of greatest interest.

In order to measure the permeability of concrete, initial surface absorption test was carried out according to BS 1881-5. Under a certain head of water, water penetration ratio in an unit area was observed periodically at constant room temperature of 20°C. The rate of water absorption of concrete is determined during a prescribed period, ranging between 10 minutes and 1 hour. Test set up for the water absorption is shown in Figure 3.4. Before testing, specimens were dried at constant room temperature or in oven at a temperature of 105°C until constant weigh. This drying period was determined as approximately 48 hours.

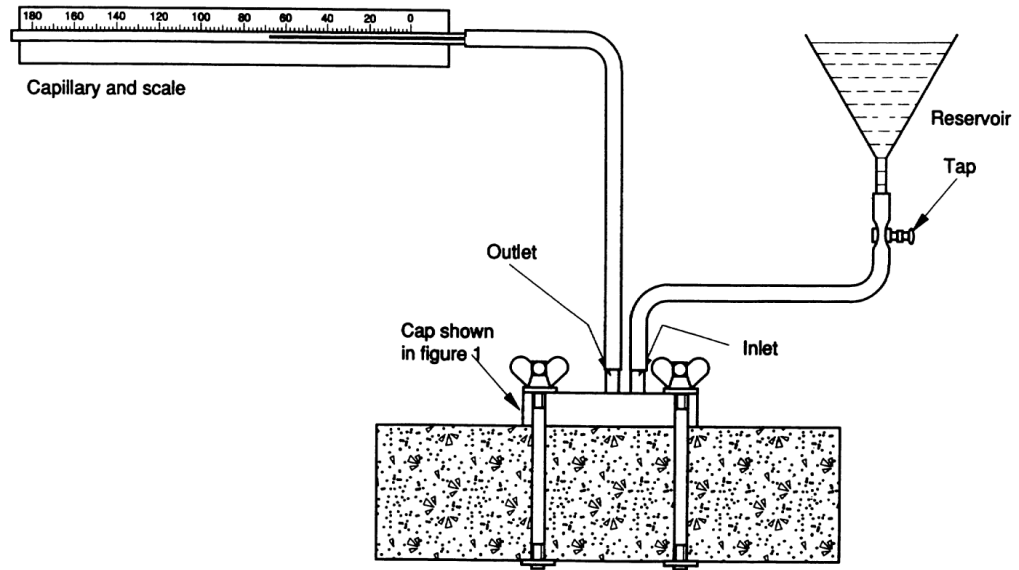


Figure 3.4 : Test set up for initial surface absorption

4. RESULTS AND DISCUSSIONS

4.1 Compressive Strength

The compressive strength test specimens of this study and those produced by Çilek et al. [21] were evaluated. Both studies were concentrated on the same mixtures and water / binder ratios. All mixtures, except M-0.25, were compared with the specimens produced by Çilek et al. Results are given individually and together.

The strength of concrete at a given age is assumed to depend on the water / binder ratio if it is cured in water at a certain room temperature. Increasing water / binder ratio reduces compressive strength and modulus of elasticity. Therefore, concrete specimens with high water / binder ratios have high values of fracture energy, absorption rate and ductility. Stress–strain curves of concretes obtained under uniaxial compression are shown in Figure 4.1.

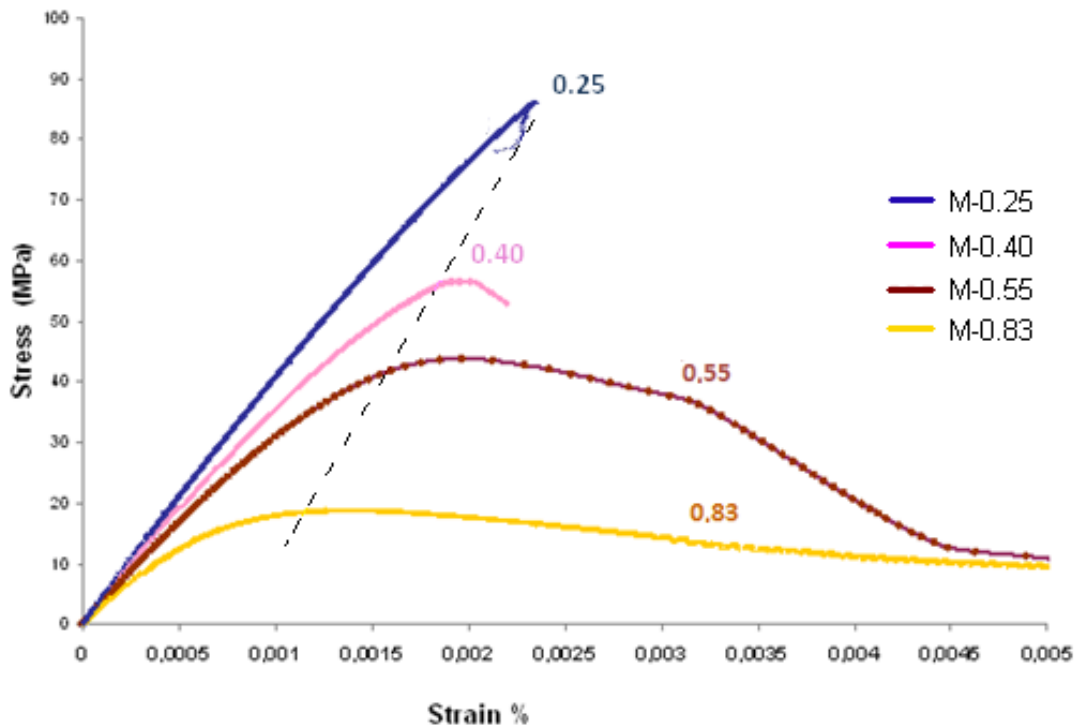


Figure 4.1: Stress –strain curves of concretes tested under uniaxial compression

Compressive strength, modulus of elasticity that calculated from Figure 4.1 and relative fracture energy results are given in Table 4.1.

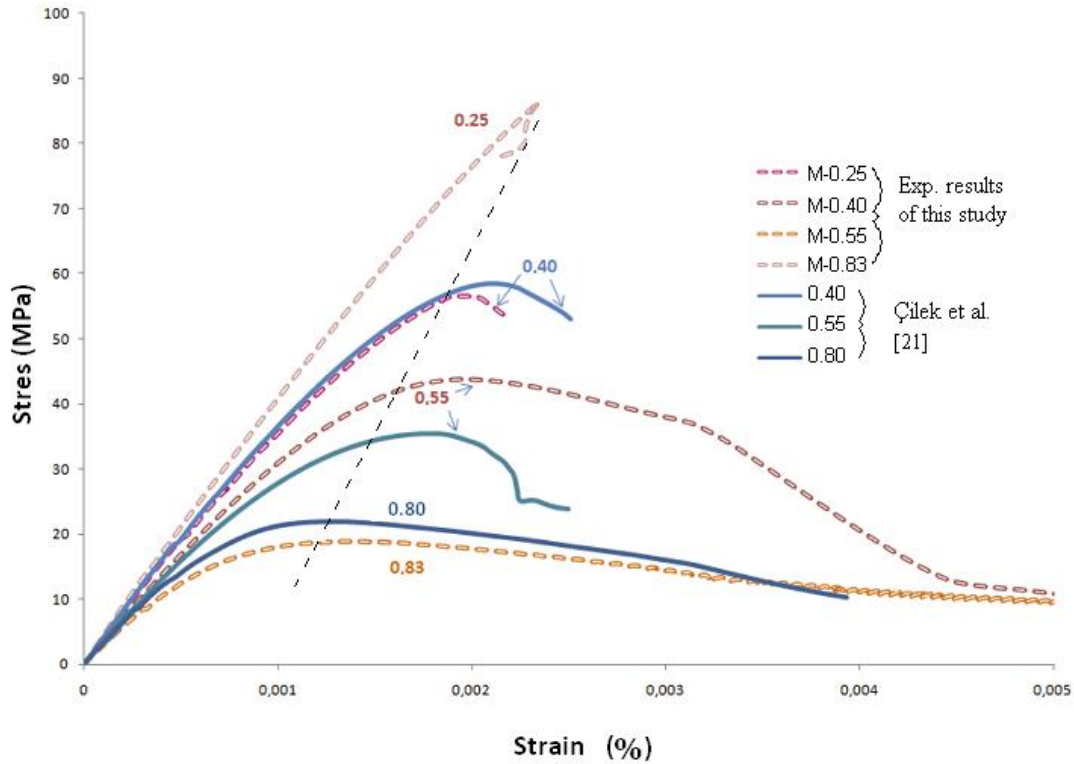


Figure 4.2: Stress- strain curves of concretes produced by Çilek et al [21] and actual test results.

Table 4.1 : Mechanical and fracture properties of concretes

	M-0.25	M-0.40	M-0.55	M-0.83
Compressive Strength (MPa)	86.2	56.5	43.8	18.8
Modulus of Elasticity (GPa)	40.5	38.6	34.1	28.4
Relative Fracture Energy	0.536	0.574	0.633	0.701

The results given in Figures 4.1 and 4.2 and in Table 4.1 were evaluated in this study. Both concrete series were tested and the results obtained are compared in Figure 4.2.

The elastic modulus of each mixture was calculated from the ascending part of the stress-strain curve from 5% to 30% of the composite strength.

Certain basic assumptions were made to analyze concrete as a composite material. Considering the mixture as a two-phase material and defining of a model element, which called unit cell, are the examples of these assumptions.

Paste and aggregate are the main constituents of two-phased models of concrete. These two phases arranged by their shape and placement to obtain unit cell models. As Tanaçan et al. [24] represents, unit cell models for concrete and their mathematical expressions developed to predict the modulus of elasticity (E_c) from the elastic properties of the constituents and their fractional volumes are given in Figure 4.3.

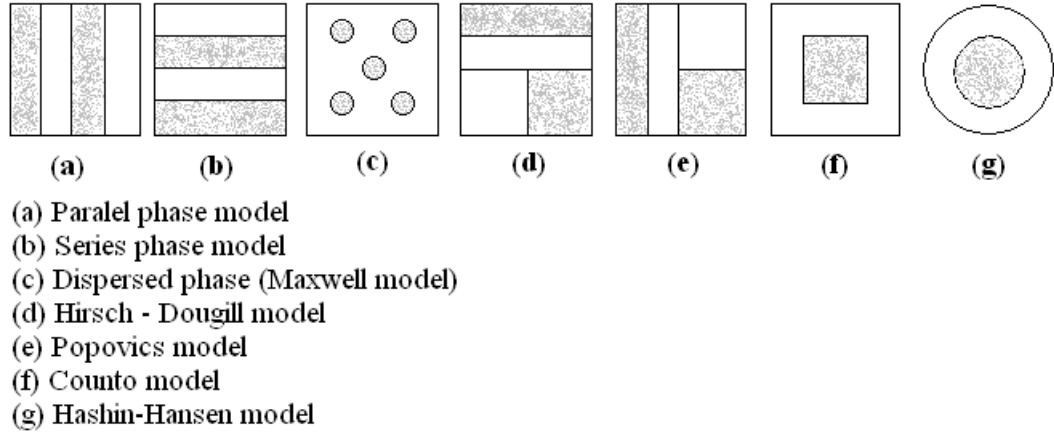


Figure 4.3: Unit cell models of compositions

Parallel phase model :

$$E_c = E_m V_m + E_a V_a \quad (4.1)$$

Series phase model :

$$\frac{1}{E_c} = \frac{V_m}{E_m} + \frac{V_a}{E_a} \quad (4.2)$$

Dispersed phase :

$$E_c = E_m \left[\frac{1 + 2V_a(\alpha - 1)/(\alpha + 2)}{1 - V_m(\alpha - 1)/(\alpha + 2)} \right] \quad (4.3)$$

$$\alpha = E_a / E_m$$

Hirsch-Doughill model :

$$E_c = \frac{1}{2} [E_{c,parallel} + E_{c,serial}] \quad (4.4)$$

Popovics model :

$$\frac{1}{E_c} = \frac{1}{2} \left[\frac{1}{E_{c,parallel}} + \frac{1}{E_{c,serial}} \right] \quad (4.5)$$

Counto model :

$$\frac{1}{E_c} = \frac{1 - \sqrt{V_a}}{E_m} + \left[\frac{1 - \sqrt{V_a}}{\sqrt{V_a}} E_m + E_a \right]^{-1} \quad (4.6)$$

Hansin-Hansen model :

$$E_c = E_m \left[\frac{(E_m + E_a) + (E_a - E_m)V_a}{(E_m + E_a) - (E_a - E_m)V_a} \right] \quad (4.7)$$

Bache and Nepper – Christansen :

$$E_c = E_m^{V_m} E_a^{V_a} \quad (4.8)$$

Estimated and experimental results and their deviation percentages were given in Table 4.2.

Table 4.2 : Measured (E_c) and calculated values of modulus of elasticity

Mix	M-0.25		M-0.40		M-0.55		M-0.83	
	GPa	Dev %	GPa	Dev %	GPa	Dev %	GPa	Dev %
E_c	40.5	-	38.6	-	34.1	-	28.4	-
Paralel	43.7	7.9	42.8	10.9	39.3	15.2	35.2	23.9
Series	37.2	8.2	34.5	10.7	28.8	15.7	21.6	24.0
Dispersed	40.4	0.2	38.5	0.3	33.3	2.3	26.6	6.5
Hirsch-Dougill	40.2	0.8	38.2	1.0	33.2	2.6	26.8	5.8
Popovics	40.4	0.2	38.6	0.1	34.0	0.2	28.4	0.1
Counto	40.3	0.5	38.2	1.0	33.0	3.2	26.2	7.9
Hansin-Hansen	39.3	2.9	37.1	3.9	31.7	7.1	24.6	13.3
Bache-Nepper-Cristansen	39.9	1.4	38.0	1.7	32.9	3.4	26.5	6.7

The most representative predictions were obtained from Popovics and Maxwell models. Lower deviation percentages of these models confirmed this conclusion as seen in Table 4.2. Comparison of theoritical and experimental values are representent in Figure 4.4. It is expected the most succesful prediction should lie on the line with the 45-degree angle. Therefore, Popovics and Maxwell models were in good agreement with the experimental results.

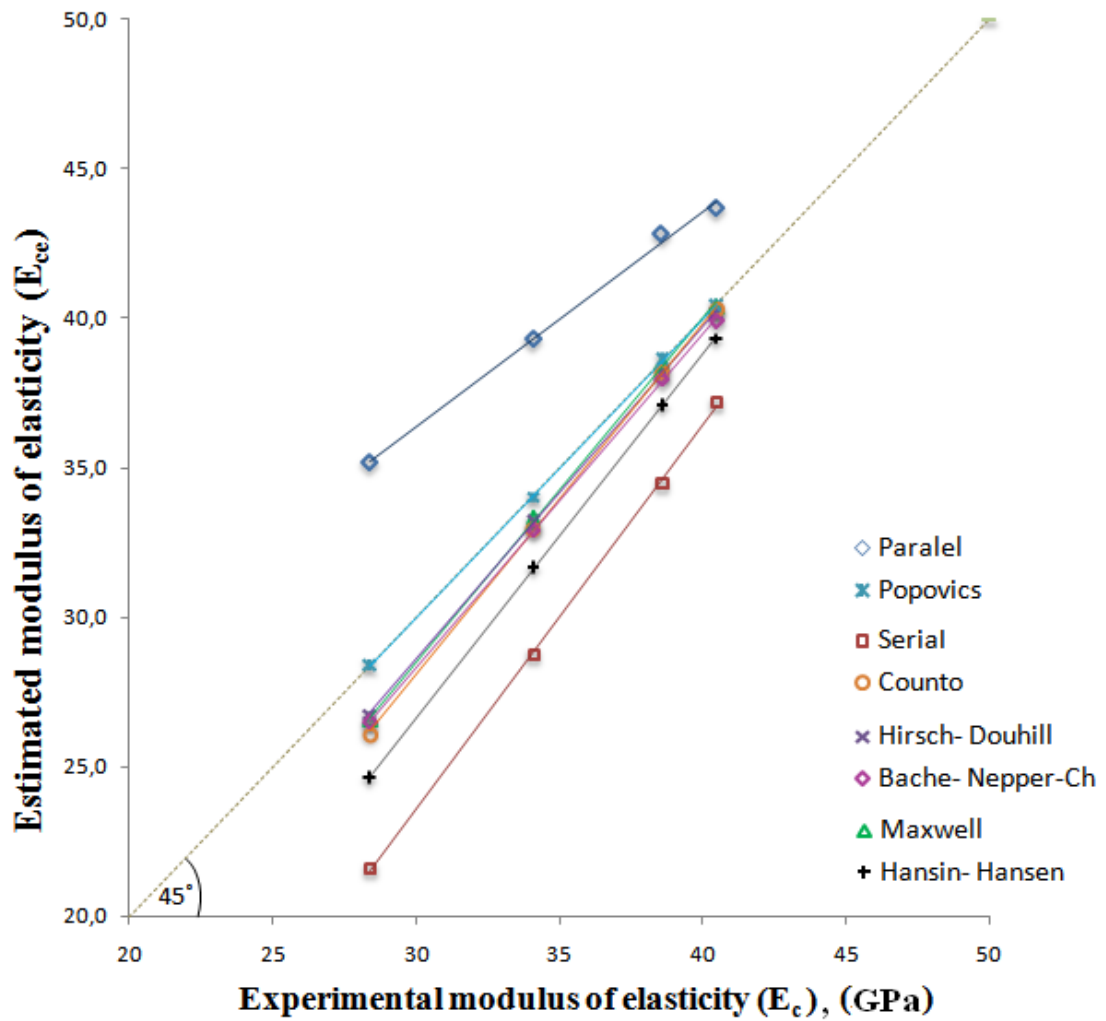


Figure 4.4: Comparison between theoritical and experimental values of modulus of elasticity

High-strength concretes with low water / binder ratios have high axial strain capacities at peak stresses. After peak stresses, the descending branch of the curve decreases suddenly due to the brittleness of concrete. Additionally, as seen in Figure 4.2 these concretes have higher strain values at peak stresses.

The strain at peak stress is dependent on the composition of concrete, curing conditions, shape and size of the specimen, loading rate and age of loading [1]. Strain at peak stress in compression versus compressive strength are given in Figure 4.5.

As seen in Figure 4.5, there is a significant trend showing that as the compressive strength of concrete increases the strain at peak stress also increases, in higher compressive stresses it becomes stable.

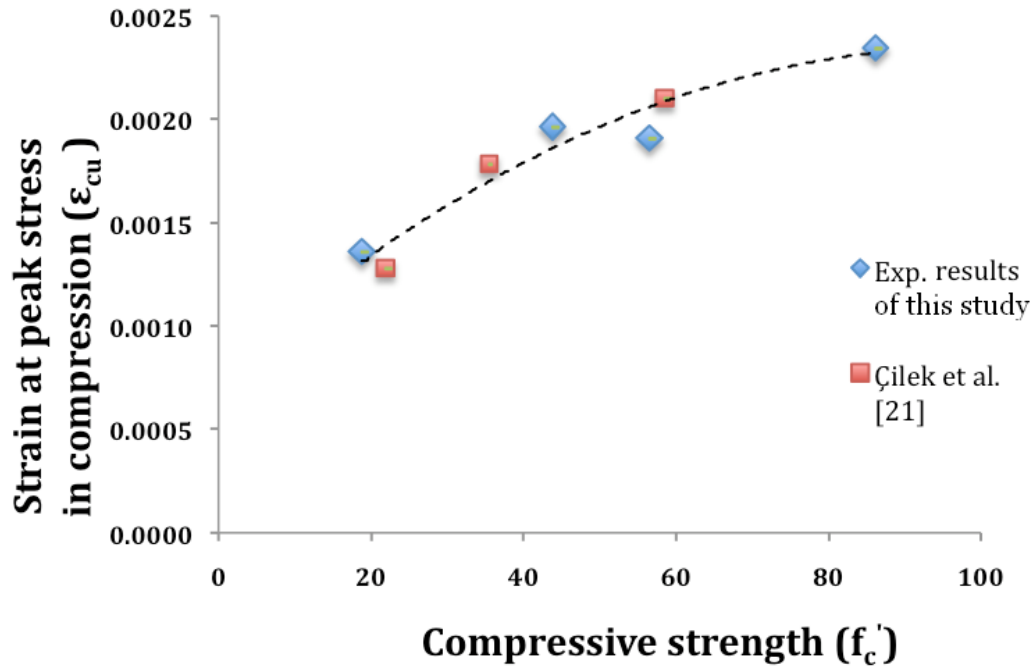


Figure 4.5: Strain at peak stress in compression versus compressive strength

Modulus of elasticity is inversely proportional to the water/ binder ratio. As the compressive strength increases both the modulus of elasticity and the brittleness of concrete increase. The relation between the modulus of elasticity (E) and the compressive strength (f'_c) of specimens are shown in Figure 4.6.

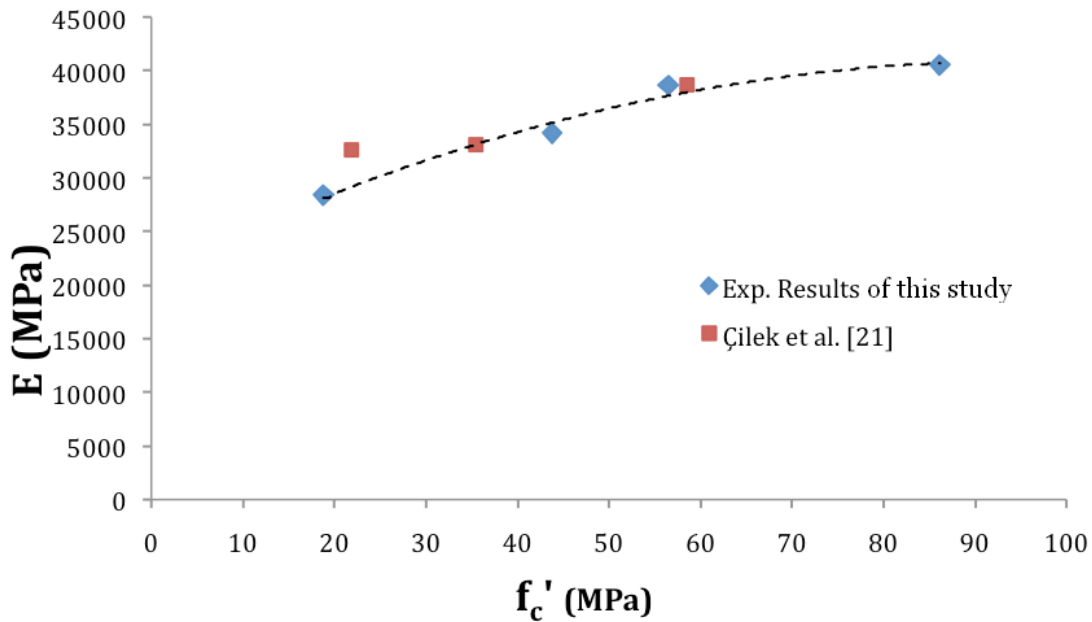


Figure 4.6: Modulus of elasticity versus compressive strength

There is no doubt that the modulus of elasticity increases with an increase in the compressive strength. Nevertheless, the increase in the modulus of elasticity of concrete is progressively lower than the increase in compressive strength.

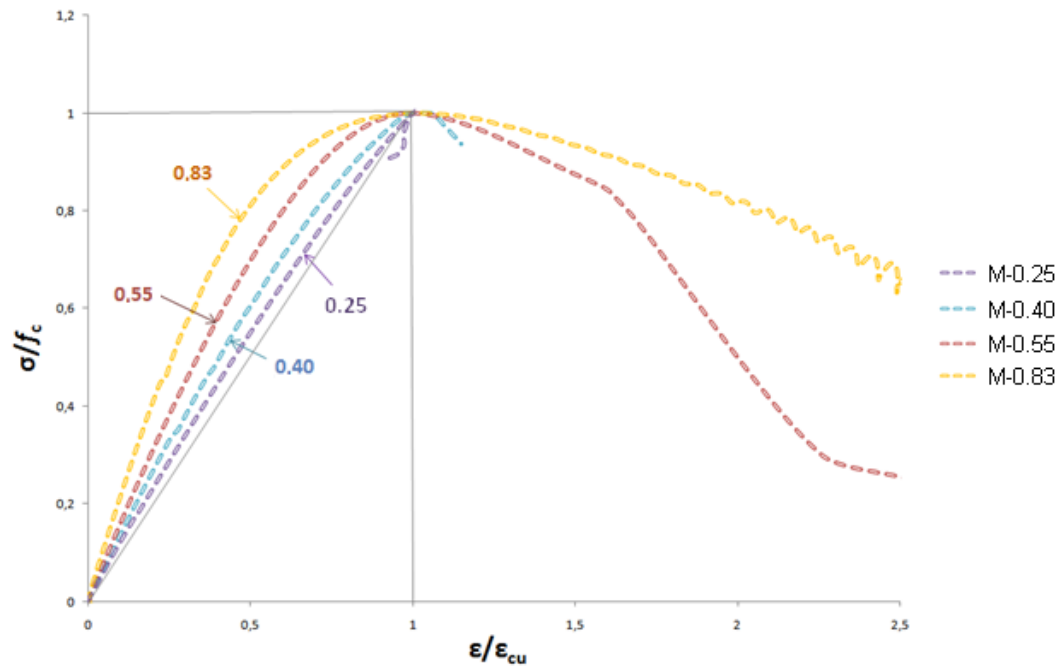


Figure 4.7: Comparison of normalized stress - strain curve

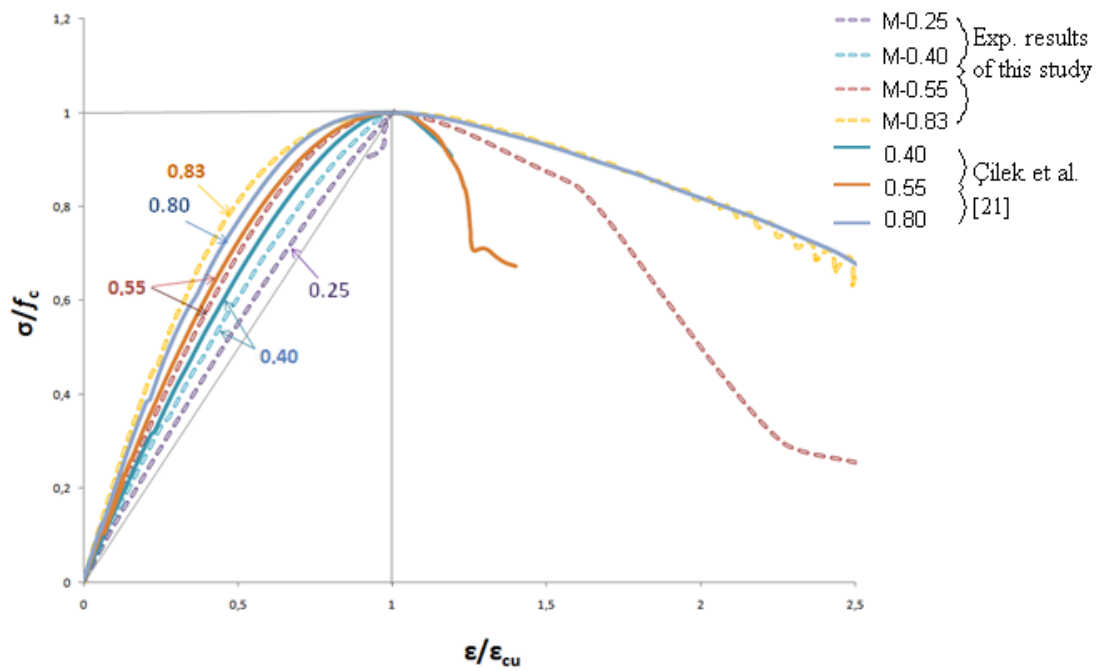


Figure 4.8: Comparison of normalized stress-strain curve
(Note: values on graph represent the water / binder ratios)

Figure 4.9 shows that as the compressive strength increases, the relative absorbed energy up to peak stress decreases significantly. As seen in this figure, the trend obtained is valid for a range of compressive strength from 20 MPa to 90 MPa.

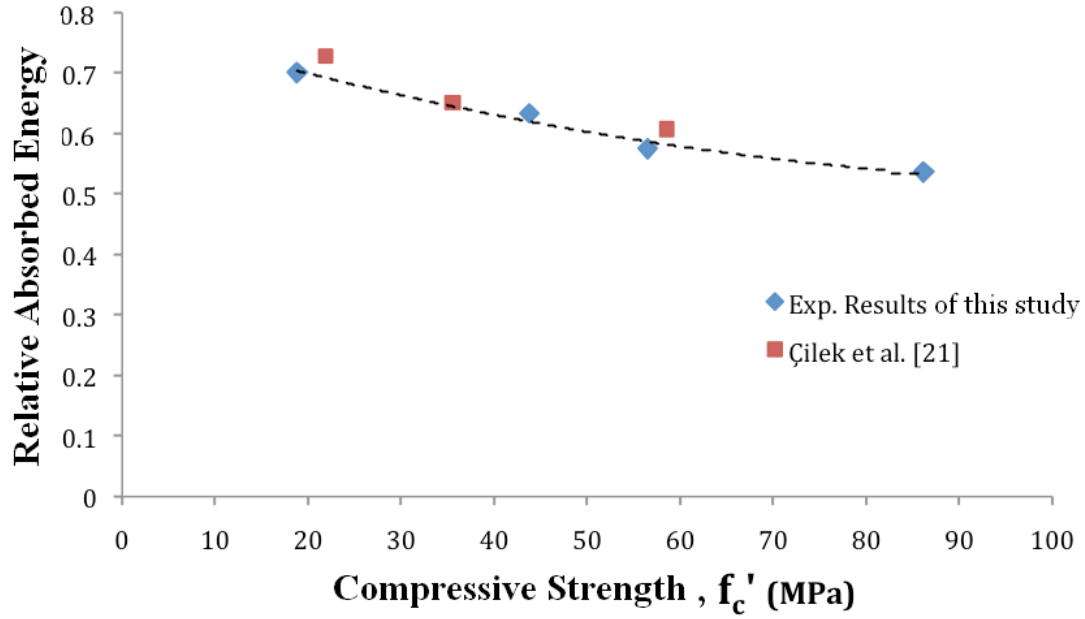


Figure 4.9: Comparison of relative fracture energy with respect to the compressive strength

4.2 Splitting Tensile Test

Although concrete is not normally designed to resist tension, it is necessary to know tensile behaviour in estimating the load under which cracking will develop. There are three types of test for strength in tension: direct tension, flexure (bending) and splitting tension tests. One of them, splitting test, is simple to perform and gives more uniform results than other tension tests [22]. Splitting tensile strength is assumed to be close to the uniaxial tensile strength, that can be 5-12% higher.

Based on the test results obtained, it can be concluded that water / binder ratio increases and the splitting tensile strength of concrete decreases. Observations on the fracture surfaces of disc specimens showed that crack propagation through the aggregates was affected by the concrete strength. It is found that the more strength concrete gains, the more probable that cracks propagate through the aggregate. Figures of specimens verify this result as seen in Figures 4.10 - 4.13.

In Figure 4.10, it was seen trans-granular type of failure occurred even the aggregates were small sized such as crushed stone 1. In addition, due to this type of failure, the fracture surfaces of these concretes are more smooth compared to others. In high strength concretes, branches of the initiated crack were not show any tendency to travel around the aggregate, because the stiffness of the matrix and the aggregates were close to each other. Besides, high density of the interfacial zone also provided basis for this type of failure.

If the cross section of M-0.25 were compared to the those of other concretes, it can be concluded that the dark colour of M-0.25 were also reflected the microsilica content of M-0.25, which was not added into the other mixtures.

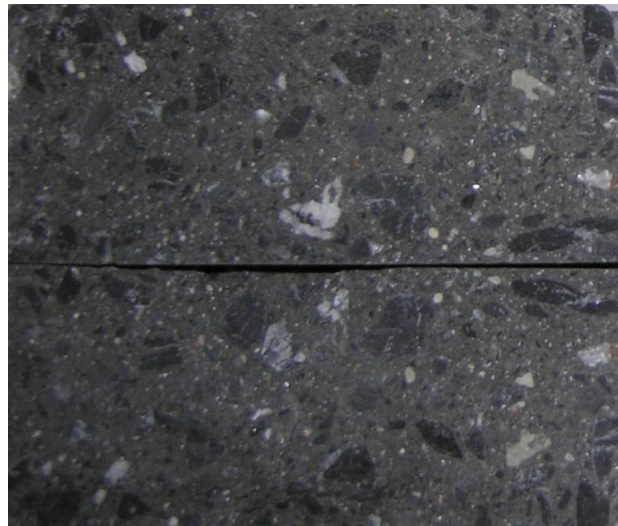


Figure 4.10: Cross section of M-0.25



Figure 4.11: Cross section of M-0.40



Figure 4.12: Cross section of M-0.55



Figure 4.13: Cross section of M-0.83

As seen in Figure 4.13 fracture surfaces are more rough compared to M-0.25.

Aggregate rupture were observed on the surfaces thus, it was seen that intergranular type of fracture occurred on M-0.83, which also reflected the weak bonding quality of low strength concretes.

Splitting tensile strength results are given in Table 4.3.

Table 4.3 : Splitting Tensile Strength

Concrete Mix	Average Strength (MPa)
M-0.25	6.91
M-0.40	6.11
M-0.55	4.17
M-0.83	2.47

Splitting test results obtained were given in Figure 4.14.

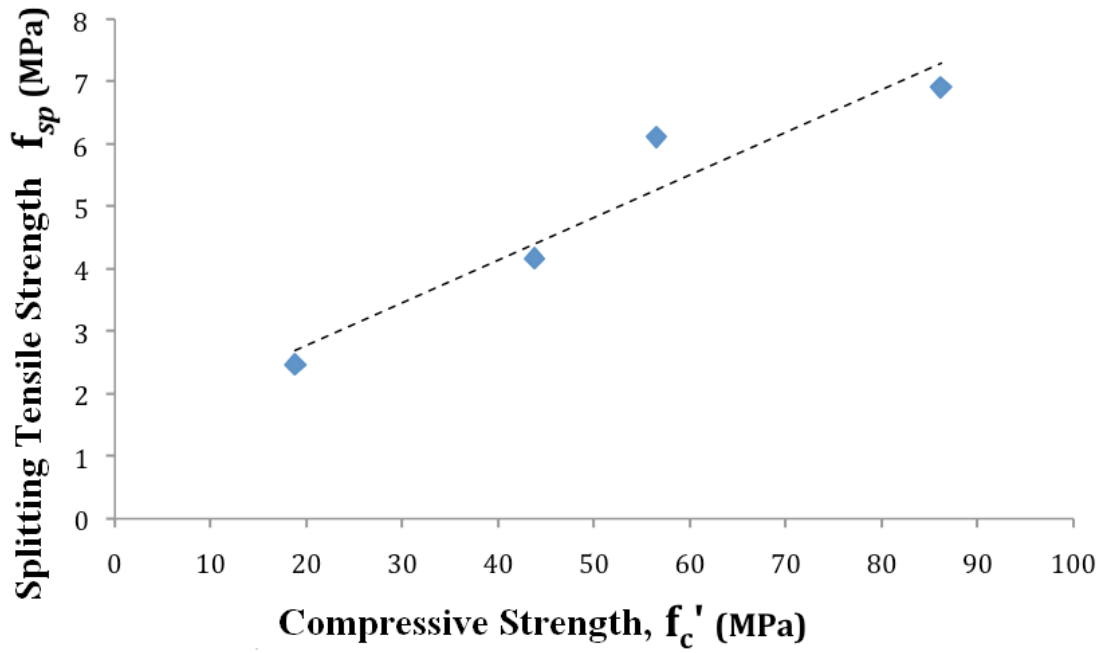


Figure 4.14: Splitting tensile strength (f_{sp}) versus compressive strength (f_c')

Figure 4.14 shows that splitting tensile strength increases significantly as the compressive strength of concrete increases. However, increase in splitting tensile strength is not significant compared to the compressive strength.

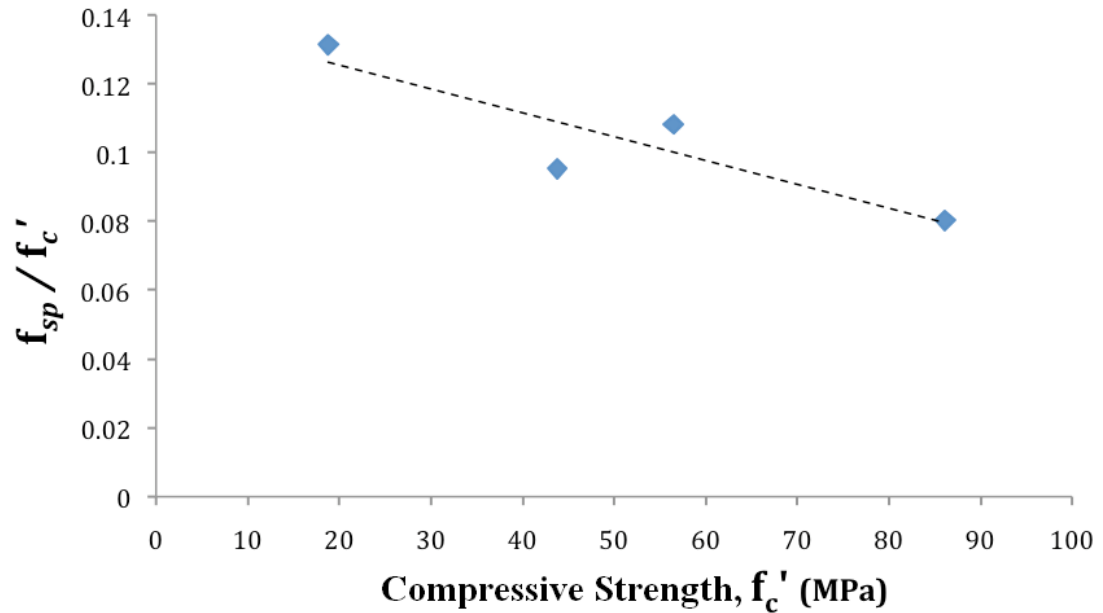


Figure 4.15: (f_{sp} / f_c') versus compressive strength (f_c')

As seen in Figure 4.15, the ratio of “splitting tensile strength / compressive strength” decreases with increasing compressive strength of concrete.

4.3 Three Point Bending Test

Three point bending tests are based on the beam specimens. Even if three point bending test is not as reliable as splitting tensile test [22], it provides necessary data about absorbed energy and other fracture parameters that can not be ignored in examining the mechanical behaviour of concrete.

Four different concrete mixtures were tested and evaluated in this study. In the experiments, it is found that reducing the water / binder ratio increases both the bending strength and the brittleness of concrete. Concrete specimens with high water / binder ratios are able to absorb lower energy rather than those of low water / binder ratios. According to the water / binder ratio, bending behaviour of specimens are given in Figure 4.16. The typical load versus displacement at the midspan curves are shown in Figure 4.16.

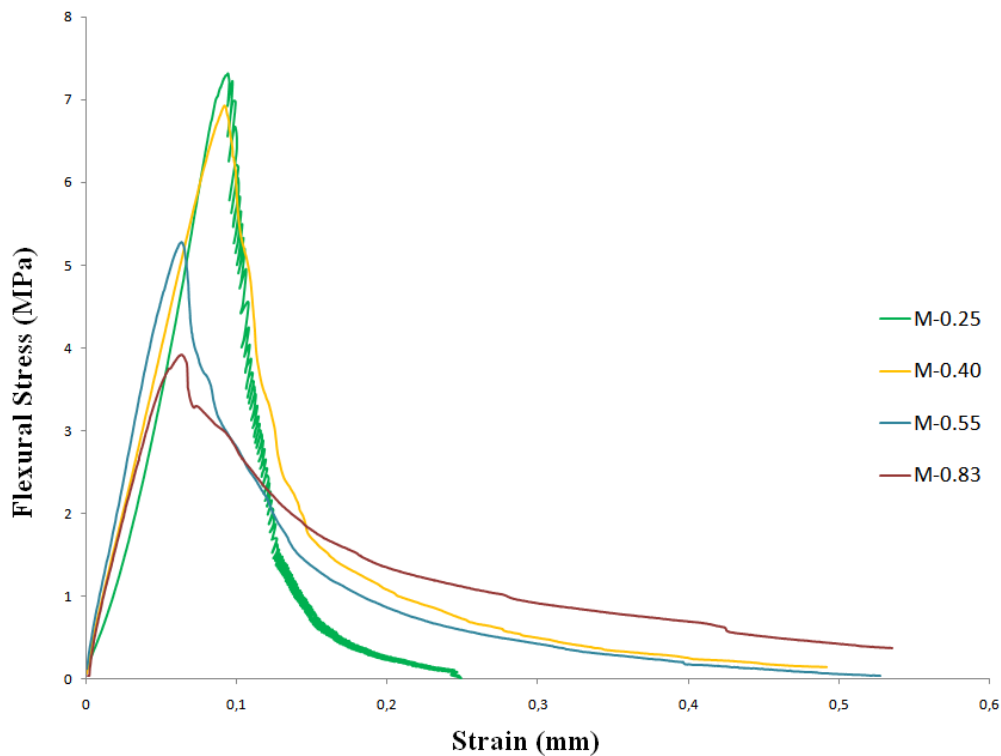


Figure 4.16 : Flexural stress – Strain (mm) diagram

As seen in Figure 4.16, M- 0.25 and M-0.40 have greater peak stresses and steeper gradients of the softening branch. It can be noted that, if the stress –strain curve ended suddenly at the peak, the material would be classified as brittle.

Bond between the aggregate and cement paste is an important factor in the strength of concrete, especially in bending strength. As a result, decreasing water / binder ratio not only improves the tensile strength of the cement paste but also increases the bonding quality of the interfacial zone. In other words, bending strength results also reflect the bonding quality of the interfacial zone.

Neville [23] stated that the flexural strength overestimates the tensile strength due to the shape of actual stress block under loads nearing failure is parabolic, and not triangular. There is a possibility that three point bending test gives a higher value of strength than a direct tension test made on same concrete. Under direct tension, there is a high probability that the weakest element may be subjected to the maximum stress, because the maximum stress also influences the entire volume of the specimen.

Flexural strength results of the experiments are given in Table 4.4.

Table 4.4 : Bending strength results

M-0.25		M-0.40		M-0.55		M-0.83	
Strength (MPa)	Average Strength	Strength (Mpa)	Average Strength	Strength (Mpa)	Average Strength	Strength (Mpa)	Average Strength
5.92		6.05		5.51		3.93	
6.59		6.93		5.30		3.59	
6.38	6.63	5.78	5.79	6.29	5.58	3.52	3.66
7.31		5.37		5.28		3.35	
6.93		4.82		5.50		3.91	

Flexural strength test results versus compressive strength of concrete are shown in Figure 4.17.

Strain at peak stress in flexure versus flexural strength is given in Figure 4.18. As seen in figure, flexural strength increases the strain at peak stress. However, the trend line does not seem so reliable. The elastic modulus of M-0.55 did not match with the others. As a result, to obtain a more accurate test results, more experimental data are needed

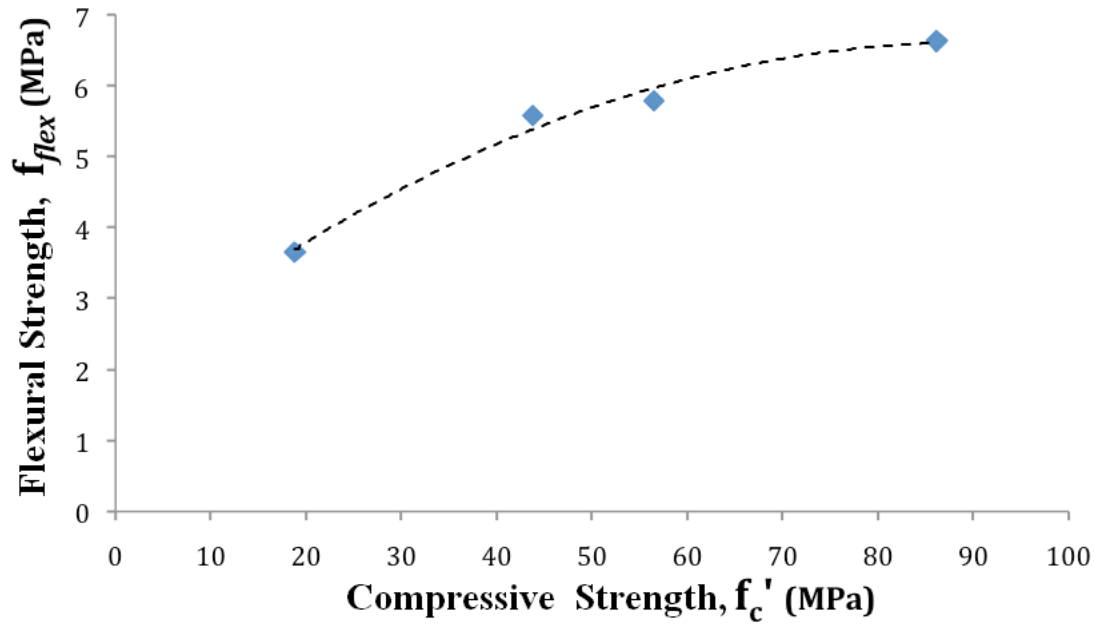


Figure 4.17: Bending Strength (f_{flex}) – compressive strength (f'_c) relation

As seen in Figure 4.17 and Table 4.4 it is clear that flexural strength, f_{flex} , increases with increasing compressive strength of concrete, but at a decreasing rate.

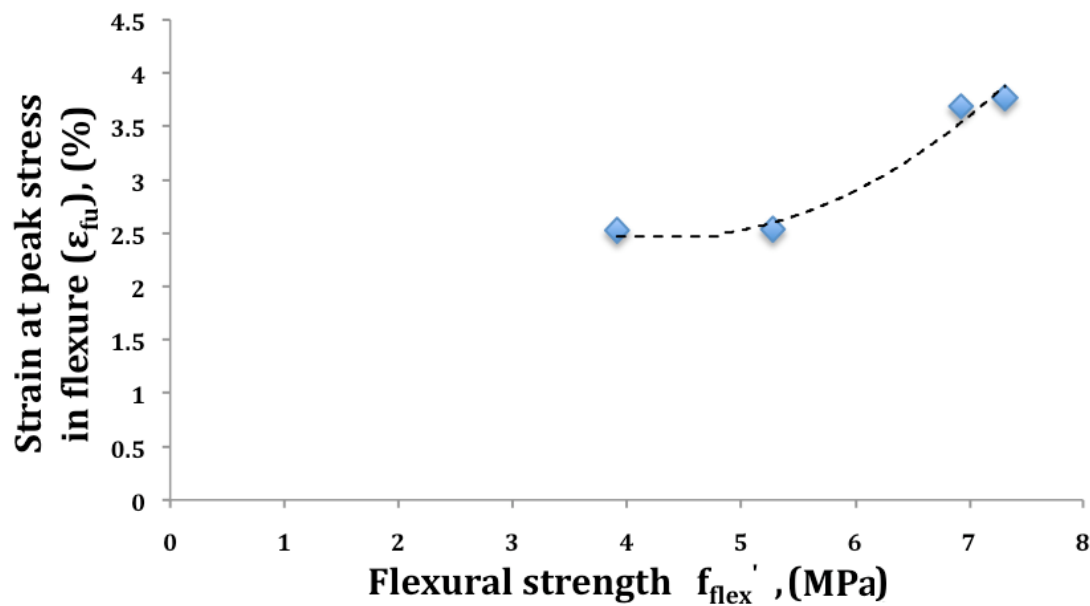


Figure 4.18: Strain at peak stress in flexure (ϵ_{cu}) versus flexural strength (f_{flex}')

Figure 4.19 shows that the ratio of “flexural strength / compressive strength” decreases significantly with increasing compressive strength of concrete.

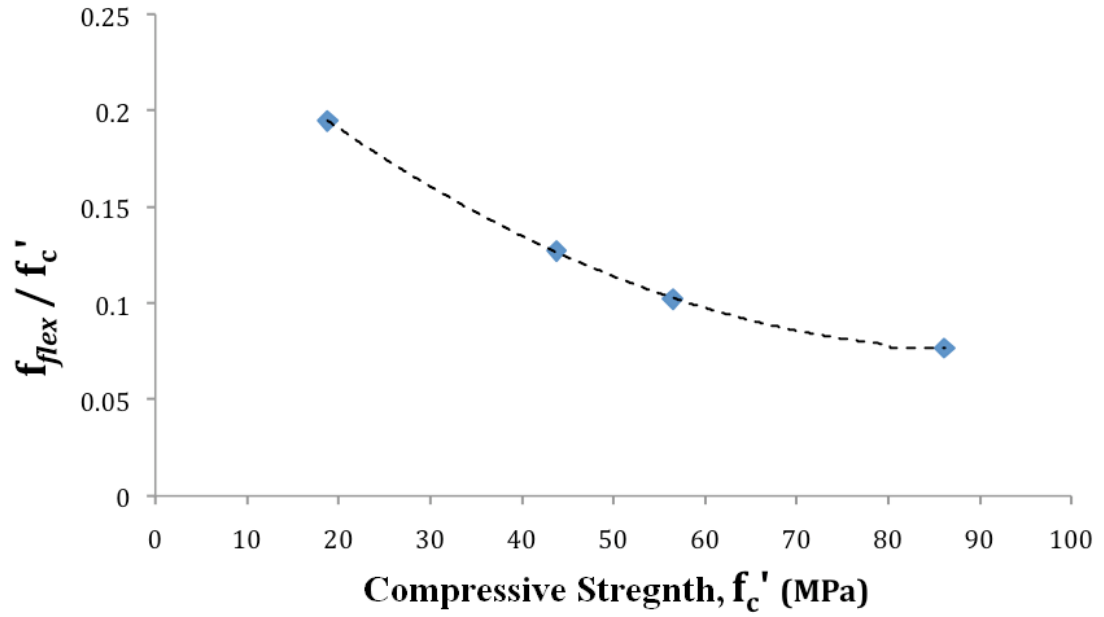


Figure 4.19: (f_{flex} / f_c') versus compressive strength (f_c')

Table 4.5 : Test results of fracture energy (J/m^2)

M-0.25		M-0.40		M-0.55		M-0.83	
Fracture Energy	Average Fracture Energy	Fracture Energy	Average Fracture Energy	Fracture Energy	Average Fracture Energy	Fracture Energy	Average Fracture Energy
60.6		84.3		77.9		85.5	
45.3	54	75.2	75	68.7	75	67.6	77
56.0		64.3		77.0		74.5	
						82.0	

As seen in Figure 4.20, as the compressive strength of concrete increases, at the beginning fracture energy increases significantly. Above 45 MPa, however, fracture energy increases slightly. Similar results were reported by Akkaya et al [7].

The ratio “elastic energy / total fracture energy” can be calculated as $G_{f (peak)} / G_f$, where $G_{f (peak)}$ is the elastic energy up to the peak stress (Figure 4.21) and G_f is the total fracture energy.

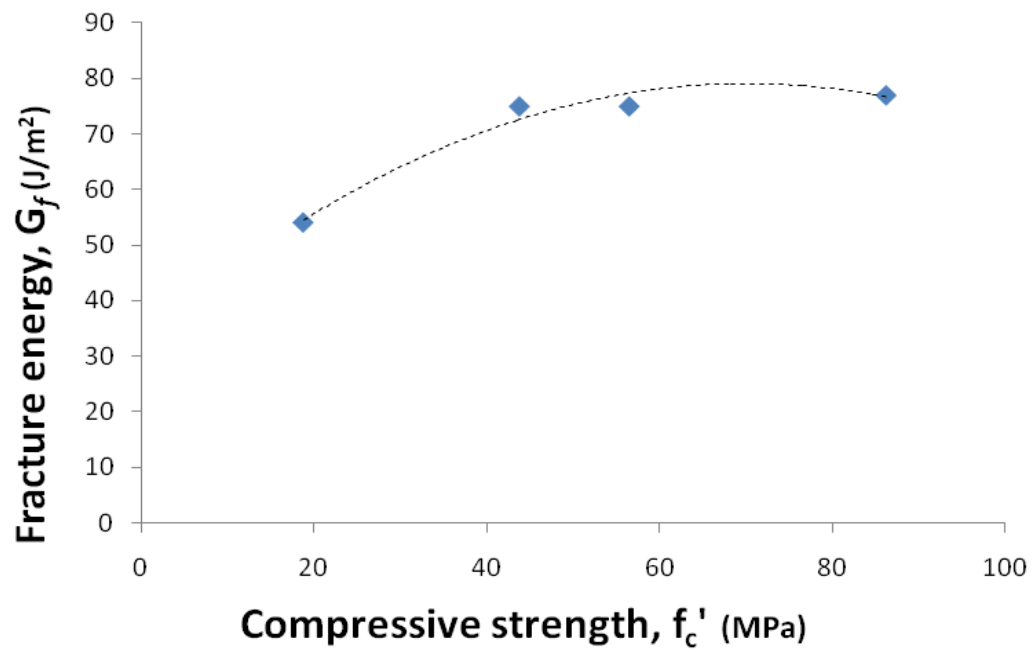


Figure 4.20: Fracture energy versus compressive strength relation

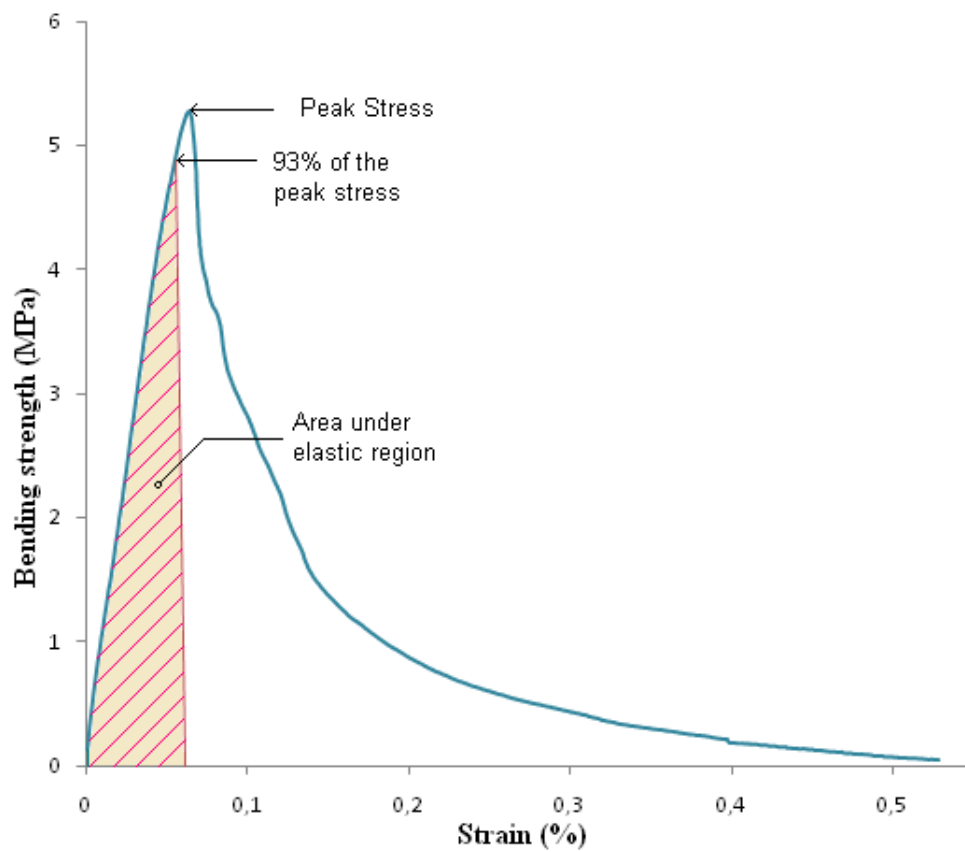


Figure 4.21: Evaluation of $G_{f \text{ (peak)}}$ from bending stress – strain diagram

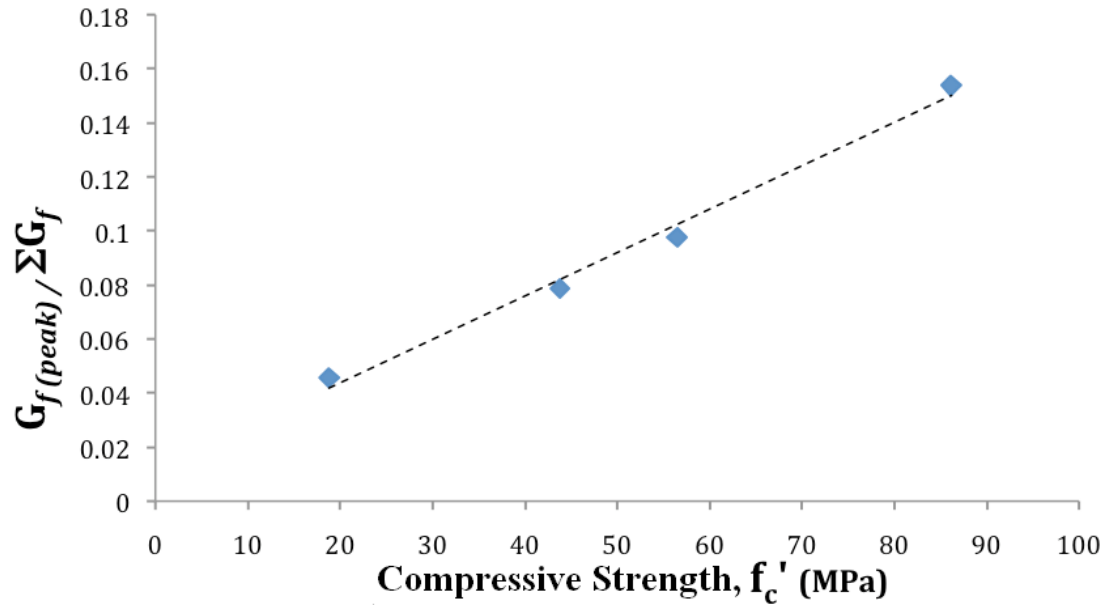


Figure 4.22 : $G_{f(peak)} / G_f$ versus compressive strength (f_c') relation
($G_{f(peak)}$): Fracture energy up to peak stress, G_f : Total fracture energy)

Linear relation obtained from the comparison of the compressive strength and the “ $G_{f(peak)} / \text{Total fracture energy}$ ” ratio is shown in Figure 4.22.

As seen in Figure 4.22, the ratio “ $G_{f(peak)} / G_f$ ” increases as compressive strength of concrete increases.

Characteristic lengths for each experiment were calculated by the help of the equation 3.6. In Table 4.6, characteristic length values are given.

Table 4.6 : Characteristic length values

M-0.25	M-0.40	M-0.55	M-0.83
Characteristic length (l_{ch})	Characteristic length (l_{ch})	Characteristic length (l_{ch})	Characteristic length (l_{ch})
0,042 m	0,068 m	0,095 m	0,128 m

Characteristic length of the mixtures decreases with increasing compressive strength of concrete. This value obtained is a typical indication of brittleness. As seen in Table 4.6 and in Figure 4.23, the characteristic length is a measure of brittleness, because in HSCs, it takes very low values.

Relation between characteristic length and compressive strength were represented in Figure 4.23.

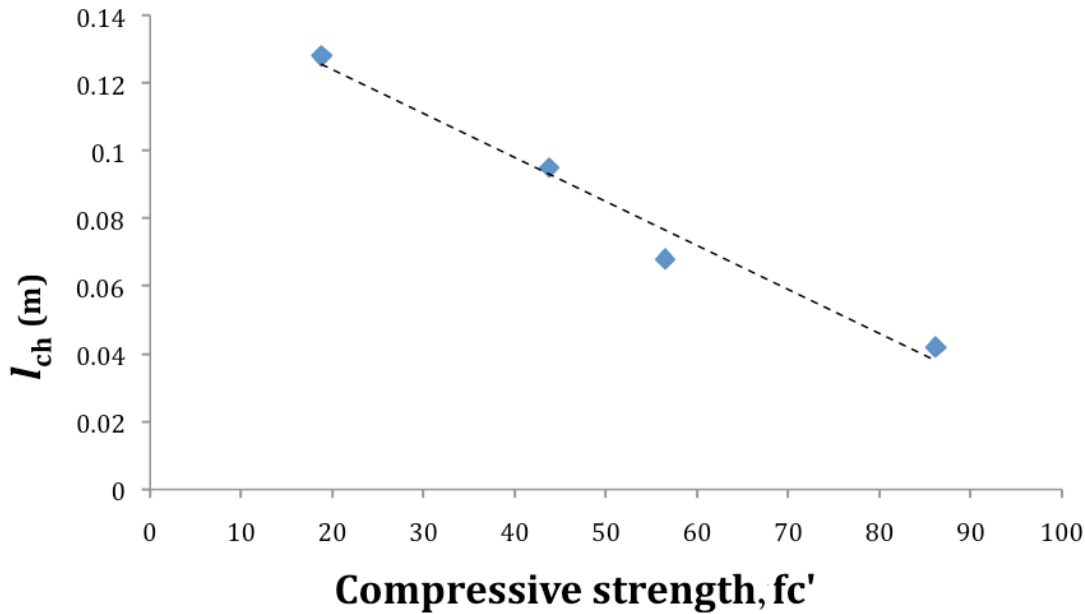


Figure 4.23 – Characteristic length – compressive strength relation

4.4 Initial Surface Water Absorption

The reduction in the permeability of high strength concrete is very low compared to that of normal strength concrete. Concrete, with water / binder ratio greater than 0.55, has significant increase in absorption rate.

Experimental studies show that water absorption rate decreases by the time but, the reduction in low-strength concrete is very high compared to that of high strength concrete. These significant changes in initial water absorption are determined by permeability of the concrete. The relation between concrete type and water absorption rate are given in Figures 4.24 and 4.25.

Test results for both oven and air dried specimens were given in Table 4.7.

Table 4.7 : Initial surface water absorption

Concrete type	Air dry (ml/sn.m ²)			Oven dry (ml/sn.m ²)		
	10 min	30 min	60 min	10 min	30 min	60 min
M - 0.25	0.003	0.001	0.000	0.114	0.058	0.035
M - 0.40	0.007	0.003	0.002	0.212	0.130	0.069
M - 0.55	0.019	0.009	0.008	0.317	0.198	0.139
M - 0.83	0.052	0.030	0.015	0.678	0.463	0.278

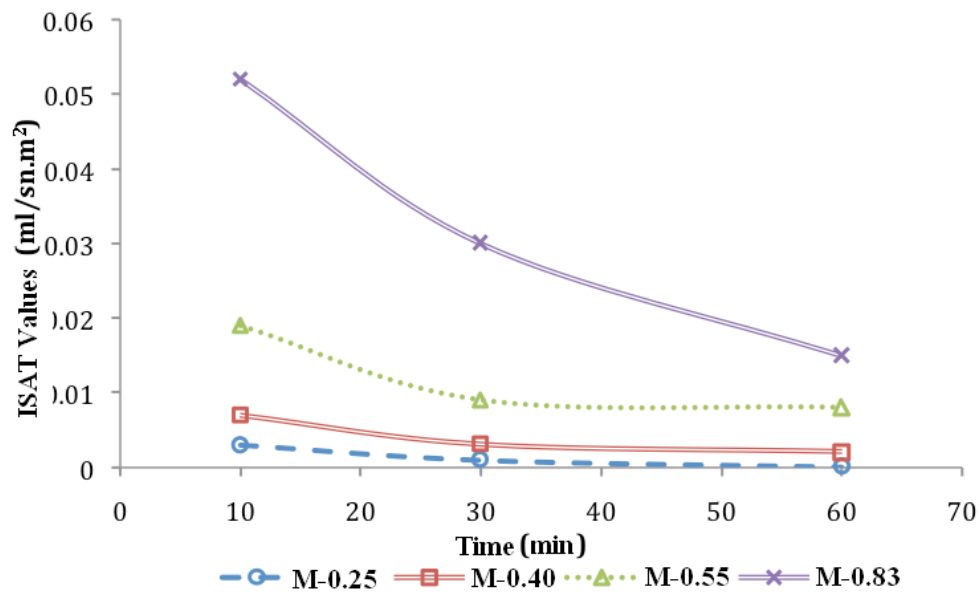


Figure 4.24: Water absorption rate of air dried specimens

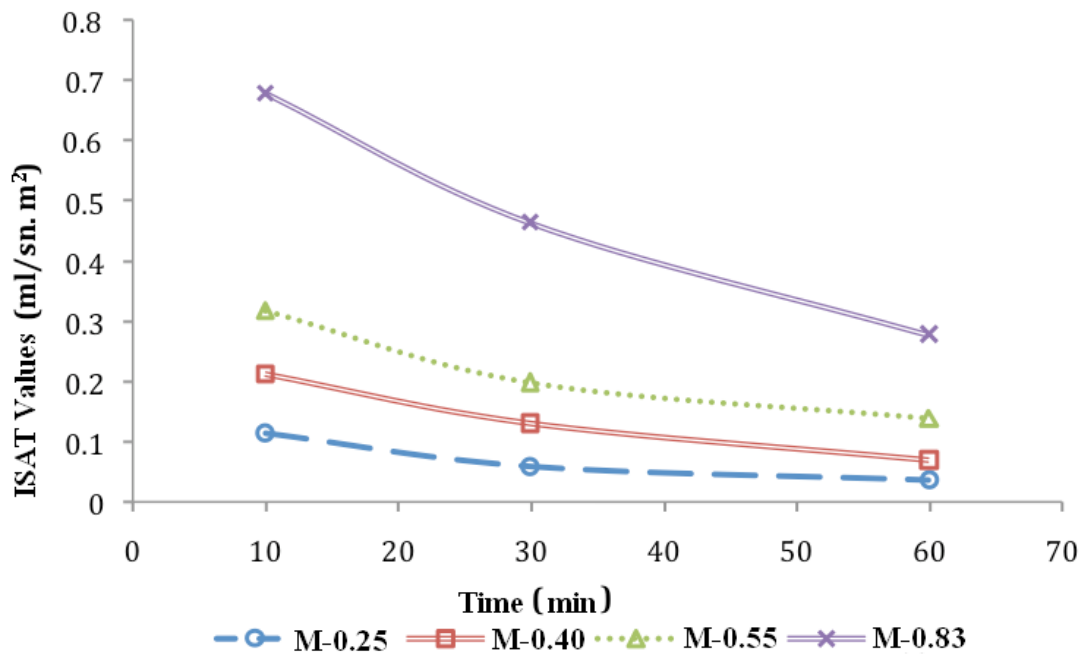


Figure 4.25: Water absorption rate of oven dried specimens

Initial surface water absorption is greatly affected by water / binder ratio of concrete, especially in concretes of low and moderate strengths. However, in high strength concretes the permeability is very low as seen in Figures 4.24 and 4.25.

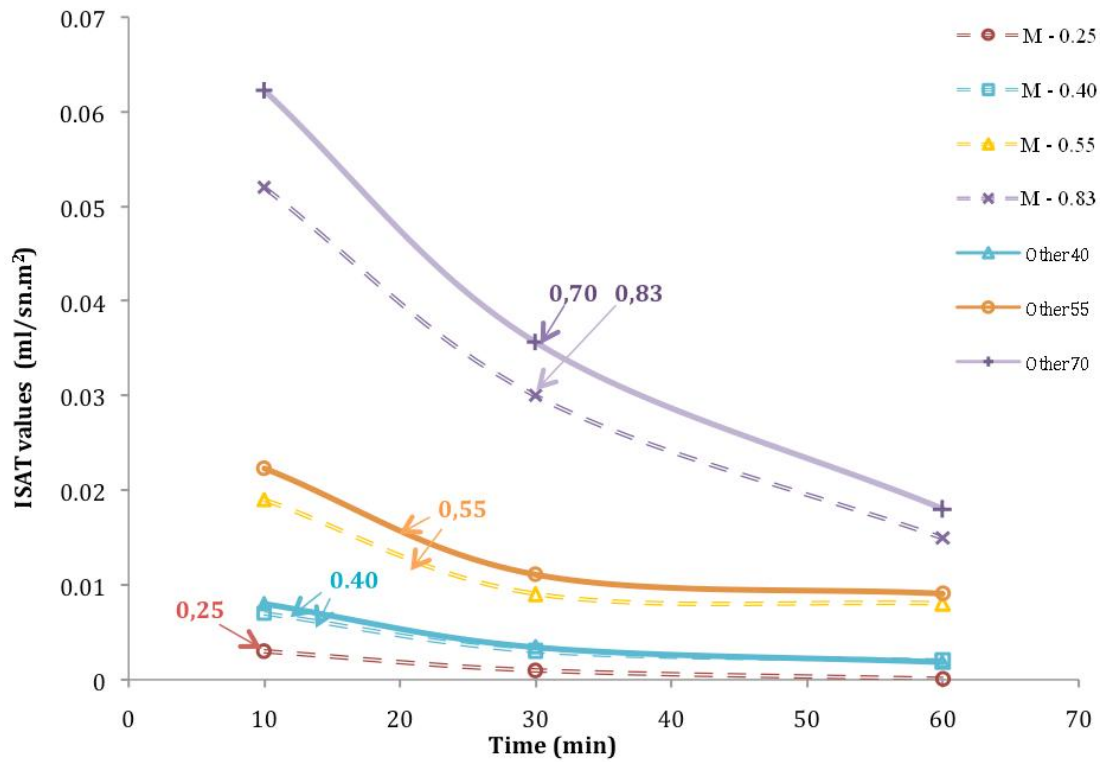


Figure 4.26: Water absorption rate comparison of air dried specimens

Neville [23] pointed out that high performance concrete contains the following ingredients: good quality aggregate; ordinary Portland cement (Type I) at a very high content, 450 - 550 kg/m³, silica fume generally 5 to 15 per cent by mass of total cementitious material and always plasticizer. Dosage of the plasticizer must be high in order to keep w/b ratio around 0.25 and occasionally even 0.20. Additionally, good moist curing is required.

If we compare M-0.25 with these requirements, except considering dosage of silica fume, it can be concluded that M-0.25 can be considered as high performance concrete. However, more durability test results are needed to clarify this assumption.

5. CONCLUSION

Increase in water / binder ratio reduces both the compressive strength and modulus of elasticity. Therefore, concrete specimens with high water / binder ratio have both higher energy absorption capacity and ductility. Although high strength concretes have high strain capacities, the sudden decrease at peak stresses does not allow concrete to absorb high amount of fracture energy.

Elastic modulus predicted by Popovics and Maxwell models were in good agreement with the experimental results. However, the paralel and the serial unit cell models did not give reliable test results.

Relative absorbed energy up to peak stress in compression decreases significantly as the compressive strength of concrete increases which also indicates increased brittleness.

Splitting tensile strength of concrete also increases with increasing compressive strength. However, increase in splitting- tensile strength is not significant compared to the increase of compressive strength. Thus, the ratio of “splitting tensile strength / compressive strength” decreases as the compressive strength of concrete increases.

The examination of fracture surfaces of splitting test specimens shows that the crack usually does not traverse the aggregate in low and normal strength concretes. In these concretes, inter-granular type of fracture occurs because of their weak matrices. However, in high strength concretes, the cracks usually travel through the aggregate thus trans-granular type of fracture occurs in these concretes, thus fracture becomes brittle in nature.

Water / binder ratio affects the development of the bending strength and the brittleness of concrete. Experimental studies prove that lower water / binder ratios are necessary to improve the bending strength of concrete. Furthermore, it is also possible to increase strain capacity of concrete.

In spite of having high bending strength, concrete becomes less ductile and tough material. Characteristic length, which reflects the brittleness of concrete, decreases according to the development of bending strength. For this reason, decrease in the characteristic length will improve the brittleness of the concrete. In addition, the brittleness of concrete shows an inverse relation with the absorbed fracture energy rate. Ductile materials like LSCs are able to absorb more fracture energy than that of HSCs.

Initial surface absorption tests also show the importance of water / binder ratio of concrete. For water / binder ratios, which are greater than 0.55, showed significant increase in the absorption rate.

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