CHAPTER -2
GENERAL LITERATURE SURVEY

2.1 Introduction

Cracking of structural elements due to in-plane shear, known as Mode-II fracture, has considerable importance in concrete structures. This area of study has received greater attention in recent years and efforts have been made to develop suitable test specimen geometries for investigating Mode-II fracture of cementitious materials and many attempts have been made to determine the fracture parameters of cementitious materials, such as critical stress intensity factor, fracture energy etc., which characterize the fracture behavior of cementitious materials. Several experimental and theoretical studies have been made to apply the concepts of linear – elastic and elastic –plastic fracture mechanics to determine the fracture parameters of hardened cement paste, mortar, concrete, fiber reinforced mortar and concrete, ferrocement and soil cement etc., However, only limited attention has been paid to characterize the Mode-II fracture of cementitious materials with partial replacements of conventional fine, coarse and cementitious materials with alternate materials such as crushed granite stone as fine aggregate, hematite as coarse aggregate, fly ash or silica fume as cementitious materials in place of ordinary Portland cement in concrete.

In this chapter, an attempt has been made to review briefly the available studies related to Mode-II fracture of cementitious materials. The review covers

(I) The various test specimen geometries proposed for Mode-II fracture of cementitious materials, experimental and analytical studies carried to determine the fracture parameters in Mode-II fracture.
(II) The Mode-II fracture characteristics of concrete with crushed granite stone as fine aggregate in place of conventional fine aggregate, natural river sand.

(III) The Mode-II fracture characteristics of concrete with hematite as coarse aggregate in place of conventional coarse aggregate granite.

(IV) The Mode-II fracture characteristics of concrete with fly ash as a cementitious material in place of ordinary Portland cement.

(V) The Mode-II fracture characteristics of concrete with silica fume as a cementitious material in place of ordinary Portland cement.

2.2.1 Test specimen geometries for Mode – II fracture of cementitious materials

Many investigators have proposed several methods and geometries of Mode-II test specimen for cementitious materials.

Prakash Desayi [6] conducted compression tests on specimens with skew notch geometry as shown in Fig. 2.1 The materials used in the experimentation are cement mortar and concrete.

![Fig. 2.1 – Prism with a skew notch, Prakash Desayi -1977 [6]](image-url)
Barr and Liu [7] used the “compact shear specimen” shown in Fig.2.2. Their studies are related to in-plane shear behavior of concrete and fiber reinforced concrete materials.

Watkins and Liu [8] conducted the finite element analysis on the specimen geometry shown in Fig.2.2 for wide range of a/w ratios and denominated the specimen geometry as “short beam shear specimen”.

Ingraffea and Panthaki [9] studied the shear strength of concrete and mortar beams in Mode – II fracture considering four point shear specimen with loading arrangement as shown in Fig.2.3
Fig.2.3 Four point shear specimen, Ingraffea and Panthaki-1985[9]

Or Ballatore et al. – 1988 [18]

Liu et. al [10] examined the in-plane shear behavior of fiber reinforced concrete using the geometry shown in Fig.2.2 and termed the geometry as “Mode-II fracture specimen”.

Barr et. al [11] found shear strength of fiber reinforced concrete using the test specimen geometries shown in Figs.2.2, 2.4 and 2.5

Fig.2.4 Compact cylindrical shear specimen, Barr, Hasso, and Liu – 1985  [11]

Or Cylindrical specimen, Barr and Evans -1986 [12]

Or Barr et al. – 1987 [15]
Barr and Evans [12] examined in-plane shear behavior of light weight fiber reinforced concrete using the Mode-II fracture specimen shown in Fig.2.2 and the cylinder specimen geometry shown in Fig.2.4. They also conducted size effect study on cylindrical geometry shown in Fig.2.4.

Symmetrically notched, “four point shear test specimen” shown in Fig.2.6 was used by Bazant and Pfeffer [13] to study the shear strength of concrete and mortar beams.
From the experiments they observed that the crack pattern started with inclined tensile micro cracks which only later connect by shearing. By finite element simulations, they showed that crack band propagates sideways when shear force zone is wide and vertically when the shear force zone is narrow. They showed that the ratio of fracture energy for Mode-II to Mode-I is about 24 times for concrete and 25 times for mortar. This is due to the fact that shear fracture energy includes the energy required to break the shear resistance due to interlock of aggregates and other asperities on rough crack surfaces behind the crack front.

Later Barr [14] determined shear strength and toughness – index of fiber reinforced concrete using the test specimen geometries shown in Figs. 2.7 and 2.8

Fig. 2.7 Compact Cube shear test specimen, Barr – 1987 [14]
Barr et al. [15] conducted size effect study on cylindrical geometry shown in Fig. 2.4. The tests were conducted in concrete both with normal and light weight aggregates and with and without fibers.

Barr [16] reviewed the test geometries proposed in earlier studies to evaluate the shear strength and post crack toughness of fiber reinforced concrete materials and also for the specimen geometry shown in Fig. 2.9.
Barr and Hughes [17] tested the cube specimen of geometry shown in Fig.2.10 and studied numerically the effect of variation of loading positions.

Fig.2.10 Effect of variation of loading position, cube specimen, Barr and Hughes, 1988 [17]

Ballatore et al. [18] conducted detailed numerical study on the four point load test specimen geometry with rectangular cross section shown in Fig. 2.6

Swartz et al. [19] used the “4- point bend test specimen” of geometry shown in Fig.2.11 in order to arrive at the Mode-II fracture parameters for concrete specimens.

Fig.2.11 Four point bend specimen, Swartz, Lu, Tang and Refai-1988 [19]
Barr and Derradj [20] conducted numerical and limited experimental study of the Mode – II test specimen geometries shown in Figs.2.12 and 2.13, mainly to develop more compact shear test specimen geometry to study the size effects.

Fig.2.12 Cube specimen                 Fig.2.13 Prismatic specimen

Fig.2.12 and 2.13 Mode-II test specimens studied numerically, Barr and Derradj-1990 [20]

2.2.2 Punch through shear specimen geometries

Watkins [21] conducted finite element analysis and experimental study on soil cement samples using “punch through shear specimen” geometry shown in Fig.2.14
Devies et al. [22] conducted detailed finite element analysis along with limited experiments with soil–cement on punch through type of shear specimen geometry shown in Fig. 2.15

Davies and So [23] later made the horizontal distance between the notches, H, equal to zero and developed the test specimen geometry shown in Fig. 2.16 and conducted series of tests on mortar specimens.
Davies et al. [24] conducted elaborate experiments with cement mortar on punch through shear specimen geometry shown in Fig.2.16

Davies [25] conducted numerical study of punch through shear specimen with central loading as shown in Fig.2.17

Davies, J [26] conducted series of tests on 100 mm symmetrically notched cube geometries similar to ones shown in Fig.2.15 and 2.16 to study the direction of crack propagation and
progressive formation of fracture zone, due to a combination of loads generating a concentrated shear force zone. In addition, he also conducted numerical analysis to the study the shear stress variation at notches.

Bochenek and Prokopski [27] investigated size effect of coarse aggregate on fracture toughness using the test specimen geometry shown in Fig.2.18

![Fig.2.18 – Punch through shear specimen, Bochenek and Prokopski-1989[27]](image)

Brandt, A.M and Proposki [28] conducted Mode-II fracture tests on cement paste, mortar and concrete using the punch through specimen geometry shown in Fig.2.18

Davies, J. [29] conducted Mode-II fracture tests on cement stabilized soil using the 100 mm cube specimen geometry shown in Fig.2.15

Bhaskar Desai, V.[2] conducted experimental studies on cement paste and mortars of 1:0.5, 1:2, 1:4 and 1:6 for arriving the best suited geometry to represent the Mode-II fracture of cementitious materials, by considering Double Centered Notched specimen (DCN), Double Edge Notched specimen (DEN), Notched Column Footing specimen (NCF) and Modified Double
Edge Notched specimen (MDEN). From his experimental studies, he arrived DCN specimen as the best suited geometry to represent predominant Mode – II fracture of cementitious materials and by considering the DCN specimen geometry, he further conducted experimental studies on Mode – II fracture of cement mortar, concrete, no-fines concrete and fibre reinforced cement mortar and concrete along with their stress-strain behavior in shear. Also he has also conducted linear finite element analysis on DCN geometry for the above cementitious materials considered and concluded that the specimen fails due to predominant Mode-II fracture.

The DCN test specimen geometry consists of cubical shape of size 150x150x150 mm with two axi-symmetrically placed notches of width 2mm with varying depth (notch depth – ‘a’) and running through the thickness of the specimen. More details of DCN specimen, including the type of applied loading are shown in Fig.1.4

Instead of considering stress criteria for Mode-II fracture, as has been done by Prakash Desayi et al. [1] and Bhaskar Desai, V.[2], a practical approach was proposed by Reinhardt and Xu [30] by considering energy criteria. They conducted tests on cubes of size 200 x 200 x 200mm, with and without edge notches, for finding Mode-II fracture energy of concrete considering the load verses displacement plots and the ligament area. They noticed that very sharp crack was initiated in case of un-notched specimens. The crack propagation was stable between the critical load and maximum load and then unstable crack propagation occurred after the maximum load. They specified that the difference in ligament heights to be considered for notch sensitivity studies. They observed that un-notched specimens develop ideal fracture pattern for Mode-II fracture failure. Their experimental results showed that the Mode-II fracture energy for normal strength concrete is 2058 N/m and the ratio between Mode-II to Mode-I fracture energy is about 20 to 25 times for normal strength concrete, a result consistent with Bazant and Pfeiffer [13].
Isaksson and Stahle [31] proposed an equation for the Mode – II crack propagation path in brittle solids (rock, concrete and ceramic, etc.,) under remote biaxial compressive stress field. The process of crack growth in the simplified model was assumed to be controlled by the Mode – II stress intensity factor, $K_{II}$ of the main crack. They did comparison of their model with the experiments conducted by earlier researchers and the curvature prescribed by the analytical model showed good agreements macroscopically, but did not describe the more realistic microscopic failure in detail. Since Mode – II cracks on the microscale depend on inhomogenities in the material. Nevertheless, the analyzed model shows qualitatively a very good agreement with the reported experimental observations of the curvature of closed macroscopic Mode – II cracks subjected to overall compression.

Zhu and Tang [5] have carried out a numerical simulation study on shear fracture process of concrete, using mesoscopic mechanical model, in double edge notched (DEN) and double central notched (DCN) specimen loaded in shear. In their simulation study, they assumed the concrete as a three phase composite material consisting of matrices, aggregates and matrix-aggregate interfaces. They applied an elastic finite element analysis program as the basic stress analysis tool while the elastic damage mechanics was used to describe the constitutive law of meso-level element. They considered maximum tensile strain criterion and Mohar – coulomb criterion as damage threshold and obtained heterogeneous stress field distribution of concrete from numerical simulation. The crack propagation process simulated with this model showed good agreement with the experimental observations. From their simulation they identified that the two pre-existing notches in the shear specimens (DEN and DCN) do not propagate the crack simultaneously because of the heterogeneous material properties of concrete. Also in case of DCN specimen, the crack propagation was generally confined to the plane along the direction of
pre-existing crack. The crack propagation and final fracture in both DEN and DCN shear specimens of concrete were predominantly caused by tensile damage at the mesoscopic level due to heterogeneous material properties of concrete and shear fracture was observed at macroscopic level.

2.3 Comments

From the review presented on the studies on Mode –II fracture, the following points are noted.

1. In this case of prism with skew notch Fig.2.1 the failure plane is inclined to the direction of application of load.

2. In the case of the specimens of simple geometries viz., the compact shear specimen (short beam specimen or prism Fig.2.2), Cylindrical specimen (Fig.2.4), Compact cube test specimen (Fig.2.5) compact cube shear test specimen (Fig.2.7), compact cylinder (disk) shear test specimen (Fig.2.8), four point load with rectangular cross section (Fig.2.9), square section (Fig2.10), cube specimen (Fig.2.12) and prismatic specimen (Fig.2.13) casting and testing is easy; however, tensile stresses are induced in conjunction with shear stresses either at the crack tips or in the middle zone or at both places and a pure shear zone of cracking is not achieved.

3. In the case of four point specimens (Fig.2.3, 2.6, and 11) and also in case of punch through shear specimen (Fig.14) the failure is due to mixed mode fracture rather than Mode-II fracture.

4. For the punch through shear specimen geometries (Fig.15, 16, 17 and 18) crack initiation is in Mode-I which immediately changes to Mode-II fracture.
5. From the above review, it may be seen that various researchers concentrated their attention in arriving at the suitable test specimen geometries, and conducted Mode-II fracture tests on cement paste, mortar, concrete and fibre reinforced concretes.

2.4 Concrete with crushed granite stone fine aggregate, partially/completely replacing the conventional fine aggregate, natural river sand

Realizing the importance of the use of crushed stone fine aggregate as replacement for natural river sand, number of experimental investigations have been reported in the literature. A brief review of the literature related to studies dealing with characterizing the properties of concrete with crushed stone as fine aggregate is presented in the following.

Celik and Marar [32] investigated the fresh and hardened properties of concrete by replacing the crushed rock lime stone fine aggregates at 0, 5, 10, 15, 20, 25 and 30 percentages by mass with machine crushed lime stone dust having particle size less than 75 micron (which is generally considered as a lower limit for defining sand particle size in concrete). They [32] noted that BS 882: Part2:1983 [33] allows a dust content up to 15% by mass of crushed rock fine aggregates in concrete manufacturing. Their experimental results indicated that increase in dust content gradually reduced the slump and air content of concrete. The concrete showed minimum water absorption at 15% replacement, maximum compressive, flexural strengths at 10% replacement and maximum impact resistance at 5% replacement. The permeability of concrete gradually reduced as the dust content increased. However, as the dust content increased more than respective optimal replacements, the amount of fines in the concrete increased, so much that there is not enough cement paste to coat all the coarse and fine aggregate particles which leads to decrease in compressive strength.
Donza et al. [34] carried out experimental investigations on fresh and hardened properties of concrete made with natural sand as fine aggregate and concretes made with crushed fine aggregates from different types of rocks such as granite, limestone and dolomite with dust content (<75 µ) between 7 to 16% by weight of total fine aggregate. They noticed that the concrete made with crushed fine aggregates requires high dosage of super plasticizer and also demands increase in cement content in case of crushed dolomite as fine aggregate. The compressive strength and split tensile strength was observed to be high for concrete made with crushed granite as fine aggregate. This study clearly brings out the need for proper selection of crushed rock type for the obtaining the fine aggregate. From the experimental investigations they concluded the following:

- Concrete with granite crushed sand G530 had higher compressive strength at all ages than concrete with river sand S530. The increase in compressive strength could be related to strong paste fine aggregate interface and the intrinsic strength of granite particles.

- Flexural strength of granite crushed sand G530 mix had higher strength values than concrete with river sand S530 mix at equivalent test ages. The improvement of the paste-fine aggregate transition zone could be attributed to the rough texture of granite crushed sand, which increases the mechanical interlocking with cement paste.

- Concrete with granite crushed sand G530 showed lower split tensile strength at 28 days and it had 14% higher than concrete with river sand S530 at one year.

- Modulus of elasticity of river sand concrete S530 was always higher than concrete with granite crushed sand G530.
Topcu and Ugurlu [35] conducted experiments on hardened properties of concrete made with crushed limestone aggregate and concrete made with river aggregates. The aggregates are classified into five different grain groups and they replaced the fine aggregate of size less than 2mm by lime stone mineral filler in percentages of 3,7,10 and 15 by weight of fine aggregate with varying cement dosages of 200, 275 and 350 Kg/m$^3$ of concrete. Their studies indicated that at 7% replacement with lime stone mineral filler the compressive strength and flexural strength of the concrete made with river aggregates was high for all cement dosages whereas for crushed aggregates concrete, the compressive strength was maximum at 7% replacement with a cement dosages of 275 and 350 Kg/m$^3$ of concrete. The maximum impermeability of crushed aggregates concrete and river aggregates concrete occurred at 10% replacement and at a cement dosage of 350 Kg/m$^3$.

Hou-Kit Man and Van Mier [36] conducted experimental study on the size effect on fracture of (numerical) concrete subjected to three point bending. Fracture simulations were performed with a three dimensional lattice model with multi-million lattice beam elements. Their results of comparison of the fracture behavior of concrete with oval shaped and crushed aggregates showed significant difference. Fracture of concrete with oval shaped aggregates occurs along the bond zones and in the cement matrix, where as concrete with crushed aggregates cracks appear through the aggregate.

Menadi et al. [37] investigated the influence of limestone fines as replacement of crushed sand in concrete. Their results showed that concrete containing 15 percent of fines content in crushed sand reduces the water permeability and increases the chloride-ion permeability.
Kou and Poon [38] conducted experimental studies on concretes prepared with the use of river sand, furnace bottom ash and fine recycled aggregate as fine aggregates by designing the concrete mixes with (i) fixed water cement ratio and (ii) fixed slump ranges. Their results suggested that both the furnace bottom ash and fine recycled aggregate can be used as fine aggregates for concrete production.

Ribero et al. [39] compared the Mode - I fracture energy of mortar and concrete produced with crushed rock and pebble aggregates using 0,10,20,30 and 40 percent of aggregates mixed with standard mortar and applying wedge splitting test for stable crack growth. The concrete with crushed rock showed higher crack growth resistance than those with pebbles.

Recently, Lavato et al. [40] using response surface methodology, evaluated the mechanical (compressive strength, tensile strength and elastic modulus) and durability properties (water absorption and carbonation depth) of concretes produced with different percentage replacements of the natural aggregates with recycled coarse and fine aggregates produced from the construction and demolition waste, with different water cement ratios. Their results showed decrease in the compressive strength, tensile strength, elastic modulus, increase in water absorption and increase in the carbonation depth with percentage replacement. They also evaluated the concrete production costs. From economical and technical view they suggested that, concretes with 50 percent substitution of natural aggregates by recycled aggregates, may achieve compressive strengths up to 25MPa, with carbonation depths similar to that of the reference concrete, as long as the water cement ratio was less than 0.60. This study clearly brings out the need to conduct carefully planned experimental investigations to determine the optimal use of wastes to achieve sustainability in concrete construction.
Also different scale modeling for fracture characteristics of materials like concrete has been proposed recently by Van Meir [41], which is based on force – displacement relations at various scales. From the experimental results, Prakash Desayi et al. [1] and Bhaskar Desai [2], found that for a given combination of (a/w) ratio and given mix of concrete, the load – displacement curves (and hence the fracture energy, $G_{II}$) of three nominally similar specimens show variations. Thus, there is a need to characterize the $G_{II}$ on probabilistic basis. A number of studies dealing with probabilistic analysis of Mode-I and mixed mode fracture of concrete are available in literature [42] – [47]. However, similar studies on Mode – II fracture of concrete with varying percentages of crushed granite stone fine aggregate are scanty.

2.5 Concrete with hematite as a coarse aggregate, partially/completely replacing the conventional coarse aggregate, granite metal

Some of the researches in different parts of the world, have conducted experimental investigations on effect of this metallic aggregate on strength and fracture properties of concrete made by replacing the coarse aggregate granite metal with hematite metal in different percentage replacements. But their studies are mainly concentrated on mechanical properties and Mode - I fracture of concrete and Mode – II fracture of different types of rocks.

Keru Wu et al [48] carried out experimental investigation on compressive strength, split tensile strength and Mode-I fracture characteristics of high performance concrete with different replacements of conventional coarse aggregate with hematite aggregate. The percentage replacements considered by them were 0, 25, 50 and 75. From the test results they observed increase in compressive strength, split tensile strength and increase in the Mode – I fracture
energy with the increase in hematite aggregate content. The maximum increase in the compressive strength was reported as 22.1 percent and in splitting tensile strength it was 19.1 percent. The increase in the Mode - I fracture energy was 71 percent.

Sun [49] used shear box test set up to examine the fracture parameters in Mode –II fracture of rocks (Marble, granite, sand stone) and concretes of different types and evaluated the corresponding stress intensity factors.

Bhaskar Desai et al. [50] conducted experimental investigations on concrete with M20 grade (1:1.607:2.972) with partially/completely replacing the conventional granite aggregate by hematite aggregate in 0, 25, 50, 75 and 100 percentage replacements by mass and with constant water cement ratio of 0.5. From their experimental studies, they observed that the compressive strength, split tensile strength and flexural strength gradually increases as the percentage replacement with hematite aggregate increases, the compressive strength of concrete was increased by 17.19%, split tensile strength was increased by 38.07% and the flexural strength was increased by 33.34% for 100 % replacement when compared with 0 % replacement.

2.6 Concrete with fly ash as a cementitious material, partially replacing the ordinary Portland cement in concrete

The increase in the living standards of the people and rapid growth of industrialization in the 21st century has put enormous demands in the electric power consumptions. Due to the fluctuations of water flow in natural rivers and high initial investment costs posed, there is a limited scope for the development of hydroelectric power plants and nuclear power plants, which has ultimately led to the development of thermal power plants in the country. One of the major problem associated with the thermal power plants is the disposal of fly ash obtained from
burning the pulverized coal. This has attracted many researches in the field of concrete technology for the effective utilization of this material in concrete and to prevent its impact on the environment.

Significant research work has been reported on the use of fly ash in concrete related to mechanical properties such as direct compression, split tension and flexure etc., and heat of hydration of cementitious materials containing fly ash as a supplement. There is not much literature is available related to Mode – II fracture of fly ash concrete.

Ferry et al [51] investigated the reaction of fly ash in concrete and gave a conclusion that the glass material in fly ash gets broken down only when the pH value of the pore water is at least about 13.2, and the increase in the alkalinity of the pore water requires that a certain amount of hydration of the Portland cement in the mix has taken place. Moreover, the reaction products of Portland cement were reported to precipitate on the surface of the fly ash particles, which act as nuclei. When the pH of the pore water becomes high enough (caused due to reaction of ordinary Portland cement), the products of the reaction of the fly ash are formed on the fly ash particles and in their vicinity. A consequence of these early reactions is that their products often remain in the shape of the original spheres of fly ash. With the passage of time, further products diffuse away and precipitate within the capillary pore system; which results in a reduction of the capillary porosity and gives a denser concrete.

Bressan et al.[52] investigated the mechanical properties of concrete by replacing 10% in mass of cement with fly ash in the concrete mixture. They tested the compressive strength on 150x300 mm cylinders and flexural strength on 100x100x550 mm beams and they also measured the fracture toughness of concrete in Mode – I using the three point bending test of a notched beam,
according to RILEM 1991. They reported that the compressive strength as 46.0 MPa, flexural strength as 5.2 MPa and the fracture toughness as 1.25 MPa.m$^{1/2}$. From their results they concluded that the Mode – I fracture is due to rupture of the interface paste and aggregate, and the presence of pores.

ACI committee 211 [53] had given guide lines for selecting proportions for high strength concrete with Portland cement and fly ash. According to it, due to variations in the chemical properties of fly ash, the strength gain characteristics of the concrete might be affected. Therefore, it was recommended that at least two different fly ash contents be used for the companion trail mixtures. Also the type of fly ash, fly ash content (recommended values for fly ash replacement of Portland cement are for class F type is 15 to 25 percent by weight and for class C type is 20 to 35 percent by weight), the fly ash weight in total cementitious material content, volume of cementitious material content (volume of cement + volume of fly ash) etc., should be considered.

Schindler A.K and Folliard J.K [54] proposed heat hydration model for cementitious materials. Also they had done experimental work to quantify the hydration development of various cementitious systems, by considering 15, 25, 35 and 45 percentage replacements of cement on volume basis with class C fly ash and also with class F fly ash over a period of seven days. The results of the concrete with class C fly ash tests indicated that with an increase in dosage, the hydration of the total cementitious system got retarded, the ultimate degree of hydration got increased, and the rate of the hydration reaction was unaffected. The results of concrete with different percentage replacement levels of class F fly ash had little impact on the initial hydration process and this fly ash acts as an inert filler at early ages.
Schindler, [55] conducted experimental work under laboratory and field conditions to determine the effect of temperature on different cements with supplementary cementing materials (replacing the cement with 25 percent class C fly ash and replacing the cement with 20 percent Class F fly ash) on the initial and final setting times of concrete mixtures. His results indicated that initial and final setting occurred at approximately the same degree of hydration for a particular mixture (i.e., for the mixes considered with 25 percent class C fly ash and mix with 20 percent Class F fly ash).

Kumar Mehta, [56] in a report, recommended the usage of higher amounts of fly ash in the order of 25 to 30 percent replacement in place of cement, when there is a concern for thermal cracking, alkali silica expansion or sulfate attack in concrete.

Bharatkumar et al. [57] conducted experiments on concrete by partially replacing the cement with fly ash and also with slag for the study of fracture characteristics of high performance concrete in Mode-I. They used class F fly ash from the thermal power plant near Chennai. They used 25 percent of class F fly ash for replacing the cement at a water binder ratios of 0.4 and 0.36 with super plasticizer for a slump of 45 mm. They used 100 mm size cube specimens for the compressive strength evaluation and 200x100 mm size cylindrical specimens for split tensile strength, 100x100x500 mm prisms for flexural strength and 100 mm dia.50 mm thick disc specimens for rapid chloride permeability test. From their test results, they observed that the compressive strength, split tensile strength and flexural strength got decreased. The permeability of fly ash based concrete got decreased. Their three point load (Mode-I) test result on beams of 50x100x500 mm size with variable notch depth ratio, with fly ash based concrete showed 10 to 25% reduction in fracture toughness at the same water binder ratio as that of ordinary concrete. The fracture energy of the fly ash based concrete in Mode – I fracture also got reduced.
Bumrongjaroen and Livingston [58] developed numerical algorithm for finding the optimum replacement of fly ash in place of cement in concrete, by considering the constraint that there is no dilution of C-S-H contribution by the main cementitious reaction and also there is no excess CH at the completion of the hydration reaction of cement. By this constraint on fly ash replacement in place of cement, they developed a figure of merit (FOM) by the numerical algorithm. If the FOM is greater than unity, then the replacement should provide a beneficial effect. If it is less than unity, then fly ash replacement would degrade the performance of concrete.

From the reports of Thomas [59] on performance of fly ash (class F) concrete relative to Portland cement concrete, it was observed that 15% of fly ash replacing the cement in concrete was having same setting time and same early age strength as that of ordinary Portland cement concrete. Also it induced higher long term strength, more chloride resistance, alkali aggregate reaction prevention and sulfate resistance to the concrete with heat reduction. The scaling resistance of fly ash replaced concrete was found to be same as that of ordinary Portland concrete, but the carbonation resistance was lower than that of ordinary Portland cement concrete.

Sofia and Jonnason. [60] developed a numerical model for the effect temperature (heat of hydration) on the strength development for the concrete made with partially replacing the cement at different fly ash to cement ratios of 0.0, 0.06, 0.11, 0.25 and 0.4 and with water binder ratios of 0.4 and 0.55. They proposed equations for the heat of hydration and strength development of concrete.
2.7 Concrete with silica fume as a cementitious material, partially replacing the ordinary Portland cement in concrete

The utilization of the silica fume (which is an industrial by-product from smelting process of ferrosilicon metals) as a cementitious material in concrete by partially replacing the ordinary Portland cement has started from the last few decades. Some of the research studies available in the literature related to use of silica fume as an admixture in concrete are presented below.

Yogendran et al.[61] investigated the effect of silica fume in high strength concrete by replacing the cement in percentages of 0, 5, 10, 15, 20, 25 and 30 on a equal weight basis. The materials used for concrete consisted of ASTM Type I cement, coarse aggregate of 14mm maximum size crushed lime stone with ASTM No.7 gradation, river sand as fine aggregate with a fineness modulus of 2.8 and silica fume having specific gravity of 2.02 with an average particle size of 0.1 micron. They adopted the mix proportions after several trial mixes. The aggregate to cementitious material ratios were varied from 3.0 to 3.5. Their experimental work consisted of three test series. In the test series I, they considered the concrete with a constant slump of 50 mm adjusting the quantity of mixing water for each replacement level of cement with silica fume. In the second test series, the water binder ratio was kept constant as 0.34 and the silica fume contents were varied. In the test series III, they considered concrete with water binder ratio of 0.34 and 0.28 and the slump was maintained at 50 mm by varying the dosage of super plasticizer for all replacements with silica fume. All the above three test series were with non-air-entrained concrete. Their test specimens consisted of 300 height and 150mm diameter cylinders for compressive strength, 100x100x350 mm prisms for flexural strength test using third-point loading and 75x75x375 mm size specimens with 15 percent cement replacement at a water binder ratio of 0.34 to study the freeze-thaw resistance. The test specimens were continuously
cured in a fog room at 95 ± 3 percent relative humidity and a temperature of 23 ± 2 °C until testing at 7, 28, 56 and 91 days. Specimens for durability study were cured in the fog room for 14 days before testing. Two specimens were tested for strength at each age. From their experiments in test series I, they observed that water required to maintain a constant slump increased linearly with the increase in percentage of cement replacement. From the variations of the compressive strength at 28 days, it was observed that the compressive strength of concrete with 5 percent replacement was marginally higher than that for control mix and there was a marked reduction in strength at higher replacement levels. But at 56 days age the strength was increased up to 15 percent replacement level. The 28 days flexural strength at constant slump of concrete attained maximum strength at 10 percent replacement level. From the results of the test series II, it was observed that the slump remains unchanged at 5 percent replacement level, the mixes with 10 and 15 percent replacement became very sticky and posed difficulties to achieve full compaction in the specimen moulds. The compressive strength results in the II nd test series at 7, 28 and 56 days showed values slightly higher than those for the control mix. They concluded that the loss in strength at a replacement level of 15 percent was due to improper compaction. The results of the test series III showed loss of workability quickly. The dosage of superpalsticizer required to maintain a constant slump increased linearly from 10 to 30 percent replacement at 0.34 water binder ratio. The compressive strength of mixes with a water binder ratio of 0.34 attained maximum at 15 percent replacement at all ages tested. At water binder ratio of 0.28 and at 5, 10, 15 percent replacement levels, equal or marginally lower compressive strengths, compared to the control mix were observed at 28 and 56 days. At 20 percent replacement, much lower compressive strengths were observed. The flexural strengths of these concretes varied in the same manner as that of compressive strength. Their studies on durability of concrete with 15
percent replacement and 0.28 water binder ratio showed increase in air content by 4.1 percent and reduction in compressive strength by 15 MPa and increased the durability factor by four times.

Rachel et al. [62] conducted experimental investigations on relative contribution of physical and chemical effects of silica fume on mechanical behavior of concrete. For this, to separate the chemical effects of silica fume from the purely physical effects, they introduced carbon black as a mineral admixture in the same dosage as the silica fume in concrete, which was similar to silica fume in terms of particle size characteristics, and was not pozzolanic in nature. They considered the carbon black in the fluffy form with surface area of 25 m²/g. The fine aggregate used for the concrete was locally available quartzitic sand with a fineness modulus of 2.93; the coarse aggregate was a ¾ inch crushed diabase from a local quarry. The water cementitious ratios considered for the concrete mixes were 0.25, 0.35 and 0.50. The concrete mixtures containing silica fume or carbon black used were having the same mix proportions as that of control mix with water cementitious ratios of 0.25, 0.35 and 0.50 except that 10 percent of the Portland cement was replaced by an equal weight of the mineral admixture. To avoid difficulty in obtaining good dispersion with these admixtures, they increased the mixing time and melamine based superplasticizer was used to provide workable consistency. Their test specimens consisted of 100x200 mm size cylinders cured in a fog room at 22°C for a curing period of 7 and 28 days. The compressive strengths of the concrete containing plain cement and concrete with silica fume at the age of 7 days showed very little difference in strength. The compressive strengths of the carbon black concretes showed roughly 10 percent lower strengths than those of the plain cement concrete specimens due to 10 percent less cement content. This observation was made for all the water cementitious ratios considered. From this they concluded that, at the age of 7 days the
pozzolanic reaction effect was very low and the beneficial effects of silica fume on strength at early ages of concrete were significant due to physical mechanism. Also the compressive strength of the control and admixture concretes decreased with the increase in water cementitious ratio. At the age 28 days, the concrete mixes with silica fume content showed higher strength at all water cementitious ratios considered. At the same age of 28 days, the strength of carbon black concrete with 10 percent less cement content was comparable with that of the control concrete. From this study they concluded that, the attainment of higher strength of the concrete with 10 percent replaced silica fume content, was due to the pozzolanic reaction, which caused to attain increase in strength by further improving the packing of hydrated phases. From their test results on modulus of elasticity, the carbon black concretes at 7 and 28 days at all water cementitious ratios considered, showed lesser values than those of control concrete and concrete with silica fume content. For all the mixes considered the Young’s modulus decreased with the increase in the water cementitious ratio. The control concrete showed the least strain at maximum stress, while the carbon black concrete showed higher strains at all the water cementitious ratios considered and at 7 and 28 days. From this they concluded that, the maximum strains in carbon black concrete were due to increased cracking in the transition zone.

Xiaofeng Cong et al.[63] conducted experimental investigations on the role of silica fume in compressive strength of cement paste, mortar and concrete. They considered the mixes of cement paste, mortars and concrete with control mix, mix with super plasticizer and also mix with super plasticizer and silica fume at 15 percent of total binder content. The variations in water binder ratios ranged from 0.30 to 0.39 in the intervals of 0.03 for cement pastes and mortars, 0.33 and 0.39 for concrete. The liquid portion of the superplasticizer was counted as part of the mixing water. The materials used for their investigation consisted of Type I Portland cement, silica fume
in powder form, fine aggregate from Kansas river passing through 4.75 mm sieve and the coarse aggregate of 19mm to 2.36 mm size crushed lime stone, the super plasticizer being the calcium naphthalene sulfonate condensate based superplasticizer. For the mix proportion considered, the slump values of concrete with super plasticizer and with water cement ratio of 0.39 was 190 mm, for the concrete with silica fume and with water binder ratio 0.33 was 140 mm. For the remaining mix proportions of control concrete, concrete with superplasticizer and concrete made with silica fume and super plasticizer was less than 25mm. Their test specimens consisted of 25mm square by 127 mm long for cement paste and mortar and 76mm square by 305 mm long for concrete. To reduce the effects of end conditions during testing, they maintained 3:1 aspect ratio of the specimens by removing equal portions from each end of the cured specimens using a high speed masonry saw. The specimens were tested in a hydraulic triaxial testing machine in compression at 3, 7 and 28 days and the test results were reported on average of two test specimens.

From their experimental observations, it was observed that, the compressive strength of cement paste at 3,7 and 28 days showed higher values of strength for cement paste with silica fume than that of control paste and cement paste with superplasticizer at water binder ratios of 0.33, 0.36 and 0.39 where as for cement mortars the compressive strength at the above ages showed higher values of strength at 7 and 28 days for mortar with silica fume and little lesser values at 3 days age at water binder ratios of 0.33, 0.36 and 0.39. The compressive strength of silica fume concrete had shown lower values at 3 and 7 days age and higher values at 28 days at water binder ratios of 0.33 and 0.39

With the above experimental observations they concluded that the high strength of silica fume paste, mortar and concrete were due to combined effect of super plasticizer and silica fume,
which the function of superplasticizer was to break up flocks of cement particles, which releases water that improves workability and provides for better access of water to the cement particles which results in a greater degree of hydration and higher strength.

The effect of silica fume was found to be bi-functional, first was due to its small particle size than that of cement particles, which enables it to act as filler in the spaces between cement grains, resulting in reducing the size of the individual pores and voids in the matrix, which in turn results in requiring a higher stress to initiate a crack. Secondly due to puzzlonic nature of silica fume.

Hooton [64] had done experimental investigations on mechanical properties of concrete by partially replacing the cement in percentages of 0, 10, 15 and 20 with silica fume and with a mealamine formaldehyde superplasticizer with a water binder ratio of 0.37. Also for comparison, a preslurred silica fume product containing 10 percent silica fume and a superplasticizer was considered. The sand used for making concrete was an Ontario Hydro Laboratory standard Paris sand and the coarse aggregate consisted of standard Dundas crushed lime stone. The mix proportion used was 1:1.82:3.02. The test specimens were 150mm diameter and 300 mm height cylinders for compressive strength, split tensile strength and elastic moduli tests. For drying shrinkage tests 75 x100x 350 mm specimens were cast. All the specimens were moist cured. In majority of the test ages considered (1, 7, 28, 56, 91, 182 days and 1, 2, 3, 4 and 5 years) the compressive strength test results showed higher values for specimens with 10 percent replacement of cement replaced with silica fume in preslurred form. Maximum strength of 88.2 MPa was achieved at 10 percent replacement, at the age of 3 years. The split tensile strength test results at the age of 28 days showed higher value of 6.30 MPa with 10 percent replacement, at 91 days test age, the control mix, mix with 10 percent silica fume replacement and 10 percent silica
fume replacement in presluried form showed approximately same values of 6.80, 6.70 and 6.60 MPa. Also at 182 days test age the control mix and 10 percent silica fume replacement in presluried form showed a value of 7.10 MPa. The test results on elastic modulus showed increase in elastic modulus in each percentage replacement with age and a highest value of 56.6 GPa was observed at the test age of 365 days for concrete with 10 percent silica fume replacement in preslurried form. The density of concrete slightly got decreased from 2499 kg/m$^3$ for control mix to 2466 kg/m$^3$ for mix with 20 percent replacement, where as the density of mix with 10 percent preslurried silica fume was of 2515 kg/m$^3$. From the drying shrinkage tests, the differences in shrinkage were observed to be minor up to 16 weeks of drying where as at 32 and 64 weeks all of the silica fume concretes exhibited 10 to 22 percent higher shrinkages than the control concrete.

Ziad Bayasi and Jing Zhou [65] studied the properties of concrete and mortar by partially replacing the cement with silica fume. For concrete they considered 24 mix proportions with varying the silica fume content ranging from 0, 5, 10, 15, 20 and 30 percent of total binder content by mass, water binder ratios of 0.41 and 0.42, total aggregate to binder ratios varying from 1.0 to 4.0 percent, super plasticizer content varying from 1.0 to 5.0 percent of binder content. For each concrete mix they had cast three 102 x 204 mm size cylindrical specimens for rapid chloride permeability test from which they considered 51mm thick slices with an electric rock saw, three 152 x 305 mm cylindrical specimens for compressive strength and three prismatic specimens of size 102 x 102 x 356 mm size for flexural strength test under third-point loading. For mortar mixes they considered five mix proportions with silica fume content varying form 10, 20, 30, and 40 percent of total binder content, water binder ratio of 0.41 and super plasticizer content of 1 and 5 percent of binder content. For each mortar mix they had cast three
102 x 204 mm size cylindrical specimens for rapid chloride permeability test, two 102 x204 mm cylindrical specimens for compressive strength and three 38 x 38 x 165 mm prismatic specimens for flexural strength over a span of 122mm in centre point loading. They kept all the specimens in their moulds and covered with plastic sheets for 24 hours to prevent rapid moisture evaporation from the surface and plastic shrinkage. After 24 hours they were placed in a curing box at 220$^0$C and 100 percent relative humidity for 7 days. After curing, the specimens were kept at room temperature and humidity until the testing age. The testing age for permeability was 40 days and 35 days for compression and flexural strength tests. From their test results on concrete, it was observed that, at constant superplascer content, the permeability of the concrete had decreased with the increase in silica fume content, and decrease in total aggregate content. In most of the test mix proportions considered, the compressive strength attained optimum values at 15 percent replacement of cement by silica fume, the flexural strength attained optimum values at 15 and 20 percent replacements. It was also observed that the slump of concrete got gradually reduced with the increase in the silica fume content in the mix. In case of mortar also, at constant superplasticizer content the permeability got decreased with the increase in the silica fume content, the compressive strength attained maximum values at 30 and 40 percent replacements, but the flexural strength showed optimum values at 10 percent replacement of cement by silica fume, with 1.0 percent superpalsticizer content and with 40 percent replacement of cement by silica fume having 5.0 percent superplasticizer content.

Zhou et al. [66] conducted experimental investigations on high strength concrete in compression, split tension, flexural strength and fracture energy in Mode – I fracture. In their experimental studies, they used silica fume replacing the cement content by 10 and 15 percent by mass in conjunction with 10mm size gravel, 10mm and 20mm crushed lime stone as coarse
aggregate with water binder ratios of 0.23 and 0.32. The silica fume was considered in slurry form (50% water solution). Their experimental results showed that the cube compressive strength increased with the increase in silica fume content from 10 to 15 percent replacement and decrease in water binder ratio from 0.32 to 0.23 for the concretes with above three types of coarse aggregates. The split tensile strength and flexural strengths also showed the similar trend except that, in case of split tensile strength, concrete with 10 mm size gravel as coarse aggregate with water binder ratio of 0.23 and in case of flexural strength, concrete with 20mm size crushed lime stone as coarse aggregate with water binder ratio 0.23, showed marginal higher values at 10 percent replacement. The fracture energy of concrete in Mode – I with above types coarse aggregates increased with the increase in silica fume content from 10 percent to 15 percent replacement and decrease in water binder ratio form 0.32 to 0.23, except for the concrete with 10mm size crushed lime stone as coarse aggregate with water binder ratio of 0.23, the fracture energy had a marginal higher value at 10 percent replacement.

Apparao and Raghuprasad [67] made experimental investigations on the compressive strength, split tensile strength and Mode-I fracture energy studies of concrete with varying sizes of coarse aggregate from 4.75, 6.3, 2.5 and 20 mm. They considered 43 grade ordinary Portland cement with 10 percent by mass of silica fume replacing the cement. Their experimental results showed that the compressive strength increased with increase in size of the coarse aggregate with and without silica fume content. The fracture energy also increased from 76.6 N/m to 142 N/m in plane concrete as the size of the coarse aggregate increased from 4.75 mm to 20 mm. The fracture energy also increased from 122 N/m to 165 N/m in concrete with the replacement of cement by 10 percent silica fume.
Bhanja and Sengupta [68] did experimental investigations on concrete with varying percentage replacements of silica fume in place of cement (0.5, 10, 15, 20, 25 and 30 percentages) by mass and with varying water binder ratios (0.26, 0.30, 0.34, 0.38 and 0.42). They kept the dosage of superplasticizer as constant and varied the compaction energy to obtain proper compaction. They considered concrete specimens of 150x150x150 mm size for compressive strength, 150 mm diameter and 300 mm height cylindrical specimens for split tensile strength and 100x100x500 mm size beam specimens for flexural strength. From their experiments, they observed that the 28 days compressive strength, split tensile strength and flexural strengths are on higher side for concrete with water binder ratios of 0.26 with 15 percent silica fume replacing the cement. For 0.30 and 0.34 water binder ratios the optimal values of above mechanical properties of concrete have occurred at 20 percent replacement and for 0.38 and 0.42 water binder ratios the optimal values have occurred at 25 percent replacement. From their experimental results they concluded that, at higher water binder ratios higher amounts of hydration will take place and hence higher amounts of silica fume will be required to consume larger amounts of calcium hydroxide produced. (as the pozzolanic reactions do not require any additional water other than that present in calcium hydroxide of hydrated cement, which requires a minimum critical water cement ratio of approximately 0.38 for curing under water). Also at higher water binder ratios, higher degree of porosity will occur in concrete which requires more amount of silica fume required to act as filler for refining the porosities of the transition zone and paste matrix.

Atis et al. [69] studied the influence of dry and wet curing conditions on compressive strength of silica fume concrete by considering the binder contents of 350, 400 and 450 Kg/m³ of concrete by partially replacing the cement with silica fume in the percentages of 0, 10, 15 and 20
by weight, with varying water binder ratios of 0.3, 0.4, 0.5 and 0.6 along with a carboxilic type hyperplasticizing admixture to have a workability of 50 to 60 cm of flow table. For each combinations of variables considered, three sample specimens of cubes of side 150mm were cast for each curing conditions i.e., for 65 percent relative humidity curing and 100 percent relative humidity curing. From their experimental results it was observed that there was reduction in compressive strength with the increase in water binder ratio in both curing conditions for all binder contents and in most of the cases optimum strength was observed at 15 percent replacement level (except in 100 percent relative humidity curing condition, in which two cases with higher compressive strengths were observed at 20 percent replacement level and one case in which optimum strength was observed at 10 percent replacement level). Their results showed that, silica fume concrete was sensitive to curing conditions and needs 100 percent moist environment to react with calcium hydroxide for the development of its binding property.

Poon et al. [70] studied the mechanical and durability of properties of high performance metakaolin and silica fume concretes. In relation to silica fume concrete, they considered 0, 5 and 10 percent replacements of cement with silica fume on weight to weight basis with water binder ratios of 0.3 and 0.5. They used 10 mm maximum size crushed granite as a coarse aggregate for concrete with water binder ratio of 0.3 along with a superplasticizer to attain a slump of not less than 100mm. For the concrete with water binder ratio of 0.5, crushed granite as coarse aggregate of maximum size 20 mm was used. Their test specimens consisted of cubes of size 100x100x100mm, cylinder of size 100 mm diameter and 200 mm height, cured at 27°C temperature in water for curing periods of 3, 7, 28, and 90 days. For the study of durability properties of concrete, they conducted chloride permeability tests following the test procedures of ASTM C1202-94 [71] standards. Their experimental results showed that the chloride
permeability increased with the increase in water binder ratio and decreased with increase in silica fume replacement from 5 percent to 10 percent replacement at all the test ages considered. The average values of test results on the cube compressive strength showed that, at 10 percent replacement of cement with silica fume, the cube compressive strength was increased after 7 days. Their experimental investigations indicated that the replacement of cement with micro silica had an effect in decreasing the interfacial porosity causing decrease in chloride penetrability of concrete along with the increase in the compressive strength.

Ali Behnood and Hasan Ziari [72] conducted experimental investigations on effect of silica fume on the compressive strength of heated and unheated concrete cylindrical specimens of size 102 mm diameter and 204 mm height by replacing the cement with silica fume in 0, 6 and 10 percentages by mass with constant water binder ratio 0.3. Along with this, one control mix with water binder ratio of 0.4 and another mix with 6 percent silica fume replacement with water binder ratio of 0.35 were considered. To maintain a constant slump of 100 to 110 mm modified polycarboxate ether high-range water reducing admixture was used. Three specimens were cast for each mix considered. All the specimens after demoulding were cured in a saturated lime water bath for curing periods of 7 and 28 days. After a curing period of 28 days, specimens were placed in an oven and heated from room temperature (20°C) to 100, 200 and 300°C at an average rate of 3°C/min. Heating of specimens to 600°C was done by an electrically heated furnace with the same heating rate. The remaining time at the target temperature was approximately 3 hours, then the furnace was turned off and specimens were cooled to room temperature. During the heating period, water vapor was allowed to escape freely. From their test results, it was observed that at 7 days and 28 days age, the compressive strength for mixes with 10 percent replacement and 0.3 water binder ratio showed higher values of compressive strength. For all mixes the
compressive strength at elevated temperatures was found to be lower than the corresponding unheated test specimens. But at the elevated temperatures of 100, 200, 300 and 600°C the compressive strength showed higher values at 200°C for all mixes.

El – Hadj Kadri et al.[73] conducted experimental work to identify the effect of silica fume on heat of hydration and compressive strength of high strength concrete. For this study, they considered concrete with ordinary Portland cement of specific surface area of 400 m²/kg, condensed silica fume with 89 percent SiO₂ with a specific surface area of 18200 m²/kg. A high range water reducing admixture having a specific gravity of 1.21 was used. The coarse aggregate consisted of crushed dolomite with two types of particle sizes 5 to 12.5 mm size and 12.5 to 20 mm size. The fine aggregate consisted of a combination of 50 percent rolled sand and 50 percent crushed sand with a combined fineness modulus of 2.56. For the concrete mix proportion, they considered partial replacement of cement with silica fume in 0, 10, 20 and 30 percentages by mass, with varying water binder ratios of 0.25, 0.3, 0.35 and 0.45 for each percentage variations considered. For uniform consistency of concrete, a slump value varying from 160 to 190 mm was adopted with a high range water reducing agent. For measuring the heat of hydration they used semi-adiabatic calorimeter on concrete samples of size 160mm diameter and 320 mm height. From their experimental observations it was observed that the total heat of hydration over a period of 10 days was reduced as the water binder ratio decreased from 0.45 to 0.25 for all percentage replacements considered. Their test results on cube compressive strength at 7, 28, 90 and 180 days showed higher values at 10 percent replacement for the water binder ratios considered and the compressive strength was decreased with the increase in water binder ratio, of the test ages considered. From their experiments it was concluded that the reduction in hydration rate with lower water binder ratios was due to the lack of water available for complete cement
hydration and the compressive strength of silica fume concrete depends on the decrease in water binder ratio than on the replacement of cement by silica fume.

Xuan et al. [74] studied the influence of silica fume on the interfacial bond between aggregate and matrix in near – surface layer of concrete by considering the concrete with ordinary Portland cement at 450 kg/m$^3$ and additional silica fume in 0, 6, 9 and 12 percent by mass of cement. Fine sand of maximum size 1 mm and high quality basalt coarse aggregate with maximum size of 8 mm was used in the investigation. They also considered controlled concrete with ordinary Portland cement content at 400 kg/m$^3$. For all the mixes of concrete a constant workability was maintained with super plasticizer. Their experimental results of cube compressive strength on 100x100x100mm cubes showed that with the increase in silica fume content, the compressive strength was gradually increased and at 12 percent addition it attained maximum value of 82 MPa. For interfacial bond strength test, they cast concrete blocks of size 300x300x50mm were cast and cured over a period of 28 days. Their test results showed that the pull force of coarse aggregates was increased with increase in silica fume addition and attained a maximum value at 12 percent addition. This indicated that, silica concentration was higher at interfacial transition zone with decrease in the thickness of interfacial transition zone and higher interfacial bond strength by forming dense hydrated calcium silicate around the aggregates.

Fatih Ozcan et al. [75] had made both experimental and numerical studies to predict the long term compressive strength of silica fume concrete made by replacing the cement content with 0, 10, 15 and 20 percentages by weight of silica fume. For this, they considered ordinary Portland cement, silica fume obtained from Antalya-Etibank Ferro-Chrome Factory in Turkey, with coarse and fine aggregates from natural river, maximum size of coarse aggregates being 16 mm. The variations in water binder ratios were of 0.3, 0.4, 0.5 and 0.6. The mix design was done
according to Turkish standards TS 802. The workability of fresh concrete was maintained in the order of 50 to 60 cm on flow table with the help of carboxilic type hyperplasticizing admixture. Three cube samples of size 150 mm were cast for each variation considered for curing periods of 3, 7, 28, 180 and 500 days at a temperature of 20±2°C. From the experimental results, it was observed that, silica fume in concrete influenced the compressive strength in short term up to 28 days and beyond 28 days the influence of silica fume on compressive strength was diminished. Also, with 10 – 15 percent of replacement of cement by silica fume in concrete had increased the compressive strength between 20 – 50 percent with respect to control concrete. As the water binder ratio was increased, the compressive strength of concrete was observed to decrease for all the mixes considered. They also developed artificial neural network and fuzzy logic models for the compressive strength simulation. The results of simulation also showed that an optimum silica fume replacement existed between 10 and 15 percent. The comparison between artificial neural network model and fuzzy logic model in terms of regression constant R², showed that artificial neural network model provides better results than the fuzzy logic model.

Shannag [76] studied the characteristics of light weight concrete containing the mineral admixture silica fume by considering the replacement of ordinary Portland cement with silica fume in percentages of 0, 5, 10 and 15 percent on weight basis. The light weight coarse aggregate consisted of volcanic tuffs of scoria origin in Al-madina, Soudi Arabia. The fine aggregate consisted of natural silica sand. In his experiments, the total binder content was kept as 400kg/m³ of concrete and maintained a good degree of workability with the help of a superplasticizer. The variations in the slump values ranged from 90 to 160 mm. After a curing period of 28 days the performance of the light weight concrete was evaluated by measuring its density, compressive strength, split tensile strength and stress-strain variation in compression.
The density of concrete was varied from 2050 kg/m$^3$ to 2025 kg/m$^3$ in fresh state, 1946 kg/m$^3$ to 1995 kg/m$^3$ in air dry state (complies with the European specification for structural light weight concrete of air dry density) and 1847 kg/m$^3$ to 1878 kg/m$^3$ in oven dry state. The compressive strength on 100mm size cubes attained a maximum value of 43.2 N/mm$^2$ at 15 percent replacement with 57 percent increase with respect to control concrete mix. The split tensile strength test on 100 mm diameter and 200 mm height cylinders showed a maximum value of 3.47 N/mm$^2$ at 10 percent replacement and the modulus of elasticity was increased by 14 percent with respect to control concrete mix.

Hosam.El_Din et al. [77] identified the effect of elevated temperature on physico-mechanical properties of blended cement concretes by considering the pozzolanic materials in which the silica fume was one among them. For making control concrete, it was considered natural siliceous sand as fine aggregate, crushed dolomite stone with more than 90 percent of its particles had a size of 20mm as coarse aggregate. The ratio of coarse to fine aggregate was kept at 1.80 by weight. Ordinary Portland cement complying with the relevant Egyptian standard specifications was used. To study the effect of silica fume concrete at elevated temperatures, they considered both partial replacement of cement and addition of silica fume at 10 and 20 percents by weight of binder. They maintained the same water content in the mixtures with appropriate level of flowability through a high range water reducing agent with sulphonated naphthalene base at a dosage of 2.0 percent of cementitious materials weight. They cast the specimens of size 150x150x150mm size in six series, out of which two series were used for 7 and 28 compressive strengths and the remaining four series for testing the residual compressive strength at elevated temperatures of 200$^0$C, 400$^0$C, 600$^0$C and 800$^0$C after the curing period of 28 days. Before exposing the specimens to elevated temperatures, they were initially prepared by drying them in
a furnace at $100\pm5^\circ$C, the specimens were then exposed to the predetermined temperatures with $200^\circ$C interval. After 3 hours of exposure to the elevated temperature, the electrical furnace was turned off and the specimens were allowed to cool down for about 2 hours at the furnace environment. The specimens were then brought out the furnace and left in the room environment to cool down. Subsequently after that, the specimens were tested in compression at a constant deformation rate of 0.25 mm/minute. From their experimental results, it was observed that silica fume enhanced the compressive strengths significantly at 7 and 28 days age. The increase in compressive strength at 7 days age over that of control mix was up to 19 percent and 45 percent for the cases of replacement and addition respectively. The increase was up to 50 percent and 75 percent for the 28 day strength. The compressive strength of all mixes increased upon heating to $200^\circ$C over the 28 day strength of the corresponding mixes. This was partly attributed to the evaporation of free water, which leads to friction increase between failure planes. In addition, this level of heat enhances self – autoclaving, which accelerates the rate of hydration of cement phases and pozzolanic reaction. At $400^\circ$C, the compressive strengths of all pozzolanic concrete mixes were slightly affected by the temperature increase. A slight reduction in compressive strength as compared to the 28 day strength was noticed for the control mix as well as the mix with 10 percent silica fume addition. At $600^\circ$C, the reduction in the compressive strength was considerable, due to possible chemical transformations consisting of decomposition of the cementing compound C-S-H with its different phases, dehydration of calcium hydroxide (CH) into free lime, $\alpha – \beta$ quartz transformation. Heating to $800^\circ$C caused a significant reduction in strength of all mixes. The X-ray diffraction analysis revealed the formation of two types of re-crystallized hydrothermally formed new compounds were identified i.e., Gismondine (CAS$_2$H$_4$) and Afwillite (C$_3$S$_2$H$_3$).