TENSILE STRENGTH AND BONDING CHARACTERISTICS
OF SELF-COMPACTING CONCRETE

A Thesis

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LIST OF ABBREVIATIONS

EASEC     East-Asia Structural Engineering Construction
ECC      Engineering Construction and Contracts
ITZ      Interfacial Transition Zone
LNG      Liquid Nitrogen Gas
µm       Micrometer (micron)
RILEM    International Union of Laboratories and Experts in Construction Materials, Systems and Structures
SEM      Scanning Electron Microscope
TC       Technical Committee
W/C      Water to Cement Ratio
WSM      Workability of Fresh Special Concrete Mixes
ABSTRACT

Self-compacting concrete, also referred to as self-consolidating concrete, is able to flow and consolidate under its own weight and is deaerated almost completely while flowing in the formwork. It is cohesive enough to fill the spaces of almost any size and shape without segregation or bleeding. This makes SCC particularly useful wherever placing is difficult, such as in heavily-reinforced concrete members or in complicated workforms.

The objectives of this research were to compare the Splitting Tensile Strength and Compressive Strength values of self-compacting and normal concrete specimens and to examine the bonding between the coarse aggregate and the cement paste using the Scanning Electron Microscope. Cylinder specimens (8” by 4”) were tested for Splitting Tensile and Compressive Strength after 28 days of standard curing, in order find out if self-compacting concrete would show an increase in these strengths and a better bonding between aggregate and cement paste, compared to normal concrete. The mix design used for making the concrete specimens was based on previous research work from literature. The water – cement ratios varied from 0.3 to 0.6 while the rest of the components were kept the same, except the chemical admixtures, which were adjusted for obtaining the self-compactability of the concrete.

All SCC mixtures exhibited greater values in both splitting tensile and compressive strength after being tested, compared to normal concrete. The splitting tensile strength increased by approximately 30%, whilst the compressive strength was around 60% greater. In addition, the SCC tensile strengths after 7 days were almost as high as those obtained after 28 days for normal concrete. This was possible due to the use of mineral and chemical admixtures, which usually improve the bonding between aggregate and cement paste, thus increasing the strength of
concrete. Images taken from concrete samples having water-cement ratios of 0.3, 0.4, and 0.6, using the Scanning Electron Microscope, have shown that the widths of the physical interface microcracks were greater for normal concrete than for self-compacting concrete, which implies that the aggregate-cement bonds were better for SCC than for normal concrete.
CHAPTER 1. INTRODUCTION

Development of self-compacting concrete (SCC) is a desirable achievement in the construction industry in order to overcome problems associated with cast-in-place concrete. Self-compacting concrete is not affected by the skills of workers, the shape and amount of reinforcing bars or the arrangement of a structure and, due to its high-fluidity and resistance to segregation it can be pumped longer distances (Bartos, 2000). The concept of self-compacting concrete was proposed in 1986 by professor Hajime Okamura (1997), but the prototype was first developed in 1988 in Japan, by professor Ozawa (1989) at the University of Tokyo. Self-compacting concrete was developed at that time to improve the durability of concrete structures. Since then, various investigations have been carried out and SCC has been used in practical structures in Japan, mainly by large construction companies. Investigations for establishing a rational mix-design method and self-compactability testing methods have been carried out from the viewpoint of making it a standard concrete. Self-compacting concrete is cast so that no additional inner or outer vibration is necessary for the compaction. It flows like “honey” and has a very smooth surface level after placing. With regard to its composition, self-compacting concrete consists of the same components as conventionally vibrated concrete, which are cement, aggregates, and water, with the addition of chemical and mineral admixtures in different proportions (see Chapter 3). Usually, the chemical admixtures used are high-range water reducers (superplasticizers) and viscosity-modifying agents, which change the rheological properties of concrete. Mineral admixtures are used as an extra fine material, besides cement, and in some cases, they replace cement. In this study, the cement content was partially replaced with mineral admixtures, e.g. fly ash, slag cement, and silica fume, admixtures that improve the flowing and strengthening characteristics of the concrete.
1.1 Historical Development of Self-Compacting Concrete

Self-compacting concrete, in principle, is not new. Special applications such as underwater concreting have always required concrete, which could be placed without the need for compaction (Bartos, 2000). In such circumstances vibration was simply impossible. Early self-compacting concretes relied on very high contents of cement paste and, once superplasticizers became available, they were added in the concrete mixes. The mixes required specialized and well-controlled placing methods in order to avoid segregation, and the high contents of cement paste made them prone to shrinkage. The overall costs were very high and applications remained very limited.

The introduction of “modern” self-leveling concrete or self-compacting concrete (SCC) is associated with the drive towards better quality concrete pursued in Japan around 1983, where the lack of uniform and complete compaction had been identified as the primary factor responsible for poor performance of concrete structures (Dehn et al., 2000). Due to the fact that there were no practical means by which full compaction of concrete on a site was ever to be fully guaranteed, the focus therefore turned onto the elimination of the need to compact, by vibration or any other means. This led to the development of the first practicable SCC by researchers Okamura and Ozawa, around 1986, at the University of Tokyo and the large Japanese contractors (e.g. Kajima Co., Maeda Co., Taisei Group Co., etc.) quickly took up the idea. The contractors used their large in-house research and development facilities to develop their own SCC technologies. Each company developed their own mix designs and trained their own staff to act as technicians for testing on sites their SCC mixes. A very important aspect was that each of the large contractors also developed their own testing devices and test methods (Bartos, 2000). In the early 1990’s there was only a limited public knowledge about SCC, mainly in the Japanese
language. The fundamental and practical know-how was kept secret by the large corporations to maintain commercial advantage. The SCCs were used under trade names, such as the NVC (Non-vibrated concrete) of Kajima Co., SQC (Super quality concrete) of Maeda Co. or the Biocrete (Taisei Co.). Simultaneously with the Japanese developments in the SCC area, research and development continued in mix-design and placing of underwater concrete where new admixtures were producing SCC mixes with performance matching that of the Japanese SCC concrete (e.g. University of Paisley / Scotland, University of Sherbrooke / Canada) (Ferraris, 1999).

1.2 Motive for Development of Self-Compacting Concrete

The motive for development of self-compacting concrete was the social problem on durability of concrete structures that arose around 1983 in Japan. Due to a gradual reduction in the number of skilled workers in Japan's construction industry, a similar reduction in the quality of construction work took place. As a result of this fact, one solution for the achievement of durable concrete structures independent of the quality of construction work was the employment of self-compacting concrete, which could be compacted into every corner of a formwork, purely by means of its own weight (Figure 1.1). Studies to develop self-compacting concrete, including a fundamental study on the workability of concrete, were carried out by researchers Ozawa and Maekawa (Bartos, 2000) at the University of Tokyo.

![Diagram](image_url)

Figure 1.1 Necessity of Self-Compacting Concrete (Ouchi and Hibino, 2000).
During their studies, they found that the main cause of the poor durability performances of Japanese concrete in structures was the inadequate consolidation of the concrete in the casting operations. By developing concrete that self-consolidates, they eliminated the main cause for the poor durability performance of the concrete. By 1988, the concept was developed and ready for the first real-scale tests and at the same time the first prototype of self-compacting concrete was completed using materials already on the market. The prototype performed satisfactorily with regard to drying and hardening shrinkage, heat of hydration, denseness after hardening, and other properties and was named “High Performance Concrete.”

At almost the same time, “High Performance Concrete” was defined as a concrete with high durability due to low water-cement ratio by professor Aitcin (Ouchi et al., 1996). Since then, the term high performance concrete has been used around the world to refer to high durability concrete. Therefore, Okamura (1997) has changed the term for the proposed concrete to “Self-Compacting High Performance Concrete.”

1.3 Construction Issues

Since the development of the prototype of self-compacting concrete in 1988, the use of self-compacting concrete in actual structures has gradually increased. The main reasons for the employment of self-compacting concrete can be summarized as follows:

* To shorten construction period
* To assure compaction in the structure - especially in confined zones where vibrating compaction is difficult
* To eliminate noise due to vibration - effective especially at concrete products plants

By employing self-compacting concrete, the cost of chemical and mineral admixtures is compensated by the elimination of vibrating compaction and work done to level the surface of
the normal concrete (Khayat et al., 1997). However, the total cost for a certain construction cannot always be reduced, because conventional concrete is used in a greater percentage than self-compacting concrete. SCC can greatly improve construction systems previously based on conventional concrete requiring vibrating compaction. Vibration compaction, which can easily cause segregation, has been an obstacle to the rationalization of construction work. Once this obstacle has been eliminated, concrete construction could be rationalized and a new construction system, including formwork, reinforcement, support and structural design, could be developed (Figure 1.2).

![Diagram of Self-Compacting Concrete](image.png)

Figure 1.2  Rational construction system proposed by Ozawa (Ouchi et al., 1996).

1.4 Existing Tests for Fresh SCC Mixes

Fresh SCC must possess at required levels the following key properties:

**Filling ability**: this is the ability of the SCC to flow into all spaces within the formwork under its own weight.

**Passing ability**: this is the ability of the SCC to flow through tight openings such as spaces between steel reinforcing bars, under its own weight.
**Resistance to segregation:** the SCC must meet the required levels of properties A & B whilst its composition remains uniform throughout the process of transport and placing.

Many tests have been used in successful applications of SCC. However, in all the projects the SCC was produced and placed by an experienced contractor whose staff has been trained and acquired experience with interpretation of a different group of tests. In other cases, the construction was preceded by full-scale trials in which a number, often excessive, of specific tests was used (Ouchi et al., 1996). The same tests were later used on the site itself.

Below is a brief summary of the more common tests currently used for assessment of fresh SCC:

**U-type test:** Of the many testing methods used for evaluating self-compactability, the U-type test (Figure 1.3) proposed by the Taisei group is the most appropriate, due to the small amount of concrete used, compared to others (Ferraris, 1999). In this test, the degree of compactability can be indicated by the height that the concrete reaches after flowing through obstacles. Concrete with the filling height of over 300 mm can be judged as self-compacting. Some companies consider the concrete self-compacting if the filling height is more than 85% of the maximum height possible.

![Figure 1.3 U-type test (Ouchi and Hibino, 2000).](image-url)
**Slump Flow test:** The basic equipment used is the same as for the conventional Slump test. The test method differs from the conventional one by the fact that the concrete sample placed into the mold is not rodded and when the slump cone is removed the sample collapses (Ferraris, 1999). The diameter of the spread of the sample is measured, i.e. a horizontal distance is determined as opposed to the vertical distance in the conventional Slump test. The Slump Flow test can give an indication as to the consistency, filling ability and workability of SCC. The SCC is assumed of having a good filling ability and consistency if the diameter of the spread reaches values between 650mm to 800mm.

**L-Box test:** This method uses a test apparatus comprising of a vertical section and a horizontal trough into which the concrete is allowed to flow on the release of a trap door from the vertical section passing through reinforcing bars placed at the intersection of the two areas of the apparatus (Dietz and Ma, 2000). The time that it takes the concrete to flow a distance of 200mm (T-20) and 400mm (T-40) into the horizontal section is measured, as is the height of the concrete at both ends of the apparatus (H1 & H2). The L-Box test can give an indication as to the filling ability and passing ability.

**Orimet test:** The test is based on the principle of an orifice rheometer applied to fresh concrete (Bartos, 2000). The test involves recording of time that it takes for a concrete sample to flow out from a vertical casting pipe through an interchangeable orifice attached at its lower end. The shorter the Flow-Time, the higher is the filling ability of the fresh mix. The Orimet test also shows potential as a means of assessment of resistance to segregation on a site.

**V-funnel test:** Viscosity of the self-compacting concrete is obtained by using a V-funnel apparatus, which has certain dimensions (Figure 1.4), in order for a given amount of concrete to pass through an orifice (Dietz and Ma, 2000). The amount of concrete needed is 12 liters and the
maximum aggregate diameter is 20 mm. The time for the amount of concrete to flow through the orifice is being measured. If the concrete starts moving through the orifice, it means that the stress is higher than the yield stress; therefore, this test measures a value that is related to the viscosity. If the concrete does not move, it shows that the yield stress is greater than the weight of the volume used. An equivalent test using smaller funnels (side of only 5 mm) is used for cement paste as an empirical test to determine the effect of chemical admixtures on the flow of cement pastes.

Figure 1.4 V-funnel (Dietz and Ma, 2000).

**Slump Flow/J-Ring combination test:** This test (Figure 1.5) involves the slump cone being placed inside a 300mm diameter steel ring attached to vertical reinforcing bars at appropriate spacing (the J-Ring itself) (Kosmatka et al., 2002). The number of bars has to be adjusted depending on the maximum size aggregate in the SCC mix. Like in the Slump Flow test, the diameter of the spread and the T-50 time are recorded for the evaluation of SCC viscosity. The Slump Flow/J-Ring combination test is an improvement upon the Slump Flow test on its own as it aims to assess also the passing ability of the fresh mix. In this respect, the SCC has to pass through the reinforcing bars without separation of paste and coarse aggregate.
**Orimet/J-Ring combination test:** This recently developed test involves the J-Ring being placed centrally below the orifice of the Orimet apparatus, allowing the discharged mix to fall into it and flow outwards (Bartos, 2000). The Orimet time is recorded as in the conventional Orimet test, along with the diameter of the concrete spread and the height of the concrete within the J-Ring. The more dynamic flow of concrete in this test simulates better the behaviour of a SCC mix when placed in practice compared with the Slump-Flow variation. The Orimet/J-Ring combination test will be used in the future as a method of assessing filling ability, passing ability and resistance to segregation (Bartos, 2000).

**GTM Segregation test:** This is a very recent test measuring the separation of aggregate in a sample after a period of time and wet sieving. The test has a potential for detection of tendency to segregate (Dehn et al., 2000). It completes the tests (Slump-Flow, L-Box, etc.) carried out to estimate the filling ability in free or shut-in environment (i.e. with some "wall-effect") by specifying the segregation resistance. This test can be used in laboratory when developing a concrete mix, as well as on site, when carrying out suitability tests on the delivered concrete.
1.5 Acceptance Test at Job Site

Since the degree of compaction in a structure mainly depends on the self-compactability of concrete and poor self-compactability cannot be compensated by the construction work, self-compactability must be checked for the whole amount of concrete just before casting at the job site. However, conventional testing methods for self-compactability require sampling and this can be extremely laborious if the self-compactability acceptance test is to be carried out for the whole amount of the concrete. A suitable acceptance test method for self-compactability has been developed by Ouchi et. al (2000), in two steps. In the first step, testing apparatus is installed between agitator truck and pump at the job site, and the whole amount of the concrete is poured into the apparatus. In the second step, if the concrete flows through the apparatus, the concrete is considered as self-compactable for the structure. If the concrete is stopped by the apparatus, the concrete is considered as having insufficient self-compactability and the mix-proportion has to be adjusted. This apparatus was successfully used at the construction site of the Osaka Gas LNG tank, and saved a considerable amount of acceptance test work (Figure 1.6).

Figure 1.6 Rational acceptance test at job site for self-compacting concrete (Ouchi and Hibino, 2000).
1.6 World-wide Current Situation of Self-Compacting Concrete

Self-compacting concrete has already been used in several countries. In Japan, major construction projects included the use of SCC in the late ’90s. Today, in Japan, efforts are being made to free SCC of the “special concrete” label and integrate it into day-to-day concrete industry production (Okamura, 1997). Currently, the percentage of self-compacting concrete in annual product of ready-mixed concrete (RMC), as well as precast concrete (PC), in Japan is around 1.2% and 0.5% of concrete products.

In the United States, the precast industry is also leading SCC technology implementation through the Precast/Prestressed Concrete Institute (PCI) which has done some research on the use of SCC in precast/prestressed concretes starting with 1999 (Bartos, 2000). It is estimated that the daily production of SCC in the precast/prestressed industry in the United States will be 8000 m$^3$ in the first quarter of 2003 (around 1% of the annual ready-mix concrete). Furthermore, several state departments of transportation in the United States (23 according to a recent survey) (Bartos, 2000) are already involved in the study of SCC. With such a high level of interest from the construction industry, as well as manufacturers of this new concrete, the use of SCC should grow at a tremendous rate in the next few years in the United States. However, even if it is made from the same constituents the industry has used for years, the whole process, from mix design to placing practices, including quality control procedures, needs to be reviewed and adapted in order for this new technology to be applied properly.

Research regarding the self-compacting concrete was also carried out in Canada, few years after the concept was introduced in Japan. Institute for Research in Construction, Canadian Precast/Prestressed Concrete Institute, CONMET-ICON, and ISIS are some of the bodies which studied various aspects of the new technology.
The introduction of the SCC in Europe is largely connected with the activities of the international association RILEM, France, particularly of its Technical Committee TC145-WSM on “Workability of Fresh Special Concrete Mixes” (Dhir and Dyer, 1999). The TC145-WSM was founded in 1992 and immediately attracted expert memberships from all over the world. The aim was to look at the production stage of a number of “special” concretes and identify workability parameters and other characteristics of the mixes in their fresh state that governed the reliable and economical achievement of the “special” or “high-performance” parameters the concretes offered. As the importance of the SCC became widely recognized, other European countries, Germany, Sweden, UK, Denmark, Netherlands, Norway, Finland, etc., have decided to keep up with the developments in this area. For example, in Sweden, the SCC market share was at five percent in RMC and PC in 2002, and was expected to double in 2003. Housing and tunneling, as well as bridge construction for the Swedish National Road Administration were the main areas of use for SCC. In the Netherlands and Germany, the precast industry is mainly driving the development of SCC, with an expected eight percent of market share in 2003 in Netherlands.

Today, self-compacting concrete is being studied worldwide, with papers presented at almost every concrete-related conference, but until now - year 2003 - there is no universally adopted standardized test method for evaluation of self-compactability of this concrete. Currently, the use of self-compacting concrete is being rapidly adopted in many countries. The use of self-compacting concrete should overcome concrete placement problems associated with the concrete construction industry. However, there still is a need for conducting more research and development work for the measurement and standardization of the methods for the evaluation of the self-compacting characteristics of SCC.
1.7 Research Objectives

The main objectives set for this research were to compare the Splitting Tensile Strength and Compressive Strength of self-compacting and normal concrete specimens and to examine the bonding between the coarse aggregate and the cement paste for both types of concrete, using Scanning Electron Microscope. The criteria used were based on 28-day compressive and splitting tensile strength of conventional and self-compacting concrete for five water-cement ratios. Also, 7-day tests were carried out for both types of concrete using only one water-cement ratio (0.4).

Examination of aggregate-cement bonding has been undertaken in order to find out if a better bonding exists in self-compacting concrete, compared to normal concrete, due to the use of chemical and mineral admixtures in the former. This was carried out by observing the physical interface, represented by the contact zone between the aggregate particles and the cement paste surrounding them (see Chapter 5). The immediately adjacent zone is called interfacial transition zone (ITZ), in which the structure of the cement paste is quite different from that of the “bulk” paste. This zone, which has a typical 20-50 µm thickness, plays a very important role in the stiffness, strength, and permeability of cementitious materials containing aggregate due to its generally lower density and decreased strength compared to that provided by bulk cement paste. It also affects significantly the physical interface (increases it) due to the accumulations of calcium hydroxide and ettringite around the aggregate. For attaining high strengths, especially in tension, a significant reduction of the interfacial transition zone is desired, because this practically limits the strength of the aggregate-cement paste bond.

1.8 Research Scope

The scope of this research included an examination of:
The effect of water-cement ratio;

* The effect of mineral admixtures; and

* The effect of chemical admixtures

on splitting tensile and compressive strengths of self-compacting concrete.

The mineral admixtures, which have been used for this project, were comprised of fly ash Class C, blast furnace slag Grade 100, and silica fume. The superplasticizer and viscosity-modifying admixture used in the study were Rheobuild 3000 FC and Rheomac 450 UW, respectively. All the admixtures needed for the research are used in real life applications and have been purchased from construction materials already existing on the market.
CHAPTER 2. LITERATURE REVIEW

Present-day self-compacting concrete can be classified as an advanced construction material. As the name suggests, it does not require to be vibrated to achieve full compaction. This offers many benefits and advantages over conventional concrete. These include an improved quality of concrete and reduction of on-site repairs, faster construction times, lower overall costs, facilitation of introduction of automation into concrete construction. An important improvement of health and safety is also achieved through elimination of handling of vibrators and a substantial reduction of environmental noise loading on and around a site. The composition of SCC mixes includes substantial proportions of fine-grained inorganic materials and this gives possibilities for utilization of mineral admixtures, which are currently waste products with no practical applications and are costly to dispose of (St John, 1998).

2.1 Previous Research Work on Self-Compacting Concrete

Self-compacting concrete extends the possibility of use of various mineral by-products in its manufacturing and with the densification of the matrix, mechanical behavior, as measured by compressive, tensile and shear strength, is increased. On the other hand, the use superplasticizers or high range water reducers, improves the stiffening, unwanted air entrainment, and flowing ability of the concrete. Practically, all types of structural constructions are possible with this concrete. The use of SCC not only shortens the construction period but also ensures quality and durability of concrete. This non-vibrated concrete allows faster placement and less finishing time, leading to improved productivity.

In the following, a summary of the articles and papers found in the literature, about the self-compacting concrete and some of the projects carried out with this type of concrete, is presented.
**Hajime Okamura**: A new type of concrete, which can be compacted into every corner of a formwork purely by means of its own weight, was proposed by Okamura (1997). In 1986, he started a research project on the flowing ability and workability of this special type of concrete, later called self-compacting concrete. The self-compactability of this concrete can be largely affected by the characteristics of materials and the mix proportions. In his study, Okamura (1997) has fixed the coarse aggregate content to 50% of the solid volume and the fine aggregate content to 40% of the mortar volume, so that self-compactability could be achieved easily by adjusting the water to cement ratio and superplasticizer dosage only.

A model formwork, comprised of two vertical sections (towers) at each end of a horizontal trough, was used by professor Okamura to observe how well self-compacting concrete could flow through obstacles. Figure 2.1 shows the ends of small pipes mounted across the horizontal trough and used as obstacles. The concrete was placed into a right-hand tower, flowed through the obstacles, and rose in the left-hand tower.

![Small pipes used as obstacles in formwork (Okamura, 1997).](image)
The obstacles were chosen to simulate the confined zones of an actual structure. The concrete in the left-hand tower rose to almost the same level as in the right-hand tower. Similar experiments of this type were carried out over a period of about one year and the applicability of self-compacting concrete for practical structures was verified. This research was started at the suggestion of professor Kokubu (Okamura, 1997) from Kobe University, Japan, one of the advisors of Hajime Okamura. They thought that it would be easy to create this new concrete because antiwashout underwater concrete was already in practical use. Antiwashout underwater concrete is cast underwater and segregation is strictly inhibited by adding a large amount of a viscous agent (antiwashout admixture), which prevents the cement particles from dispersing in the surrounding water. However, it was found that antiwashout underwater concrete was not applicable for structures in open air for two reasons: first, entrapped air bubbles could not be eliminated due to the high viscosity; and second, compaction in the confined areas of reinforcing bars was difficult. Thus, for the achievement of self-compactability, a superplasticizer was indispensable. With a superplasticizer, the paste can be made more flowable with little concomitant decrease in viscosity, compared to the drastic effect of the water, when the cohesion between the aggregate and the paste is weakened (Figure 2.2).

![The Role of Superplasticizer](image)

Figure 2.2 Effect of superplasticizer on viscosity (Okamura, 1997).
The water-cement ratio was taken between 0.4 and 0.6 depending on the properties of the cement. The superplasticizer dosage and the final water-cement ratio were determined so, as to ensure the self-compactability, evaluated subsequently by using the U-type test (Ouchi and Hibino, 2000) described in the previous chapter.

**Kazumasa Ozawa:** After Okamura began his research in 1986, other researchers in Japan have started to investigate self-compacting concrete, looking to improve its characteristics. One of those was Ozawa (1989) who has done some research independently from Okamura, and in the summer of 1988, he succeeded in developing self-compacting concrete for the first time. The year after that, an open experiment on the new type of concrete was held at the University of Tokyo, in front of more than 100 researchers and engineers. As a result, intensive research has begun in many places, especially in the research institutes of large construction companies and at the University of Tokyo.

Ozawa (1989) completed the first prototype of self-compacting concrete using materials already on the market. By using different types of superplasticizers, he studied the workability of concrete and developed a concrete which was very workable. It was suitable for rapid placement and had a very good permeability. The viscosity of the concrete was measured using the V-funnel test (see Chapter 1).

Other experiments carried out by Ozawa (1989) focused on the influence of mineral admixtures, like fly ash and blast furnace slag, on the flowing ability and segregation resistance of self-compacting concrete. He found out that the flowing ability of the concrete improved remarkably when Portland cement was partially replaced with fly ash and blast furnace slag. After trying different proportions of admixtures, he concluded that 10-20% of fly ash and 25-45% of slag cement, by mass, showed the best flowing ability and strength characteristics.
Subramanian and Chattopadhyay: Subramanian and Chattopadhyay (2002) are research and development engineers at the ECC Division of Larsen & Toubro Ltd (L&T), Chennai, India. They have over 10 years of experience on development of self-compacting concrete, underwater concrete with antiwashout admixtures and proportioning of special concrete mixtures. Their research was concentrated on several trials carried out to arrive at an approximate mix proportion of self-compacting concrete, which would give the procedure for the selection of a viscosity modifying agent, a compatible superplasticizer and the determination of their dosages. The Portland cement was partially replaced with fly ash and blast furnace slag, in the same percentages as Ozawa (1989) has done before and the maximum coarse aggregate size did not exceed 1”.

The two researchers were trying to determine different coarse and fine aggregate contents from those developed by Okamura. The coarse aggregate content was varied, along with water-powder (cement, fly ash and slag) ratio, being 50%, 48% and 46% of the solid volume. The U-tube trials were repeated for different water-powder ratios ranging from 0.3 to 0.7 in steps of 0.10. On the basis of these trials, it was discovered that self-compactability could be achieved when the coarse aggregate content was restricted to 46 percent instead of 50 percent tried by Okamura (1997). In the next series of experiments, the coarse aggregate content was fixed at 46 percent and the sand content in the mortar portion was varied from 36 percent to 44 percent on a solid volume basis in steps of 2 percent. Again, the water-powder ratio was varied from 0.3 to 0.7 and based on the U-tube trials a sand content of 42 percent was selected. In order to show the necessity of using a viscosity-modifying agent along with a superplasticizer, to reduce the segregation and bleeding, the mixture proportion developed by the two researchers was used to cast a few trial specimens. In these trials, viscosity-modifying agent was not used. The cast
specimens were heavily reinforced slabs having 2400x600x80 mm and no vibration or any other method of compaction was used. However, careful qualitative observations revealed that the proportions needed to be delicately adjusted within narrow limits to eliminate bleeding as well as settlement of coarse aggregate. It was difficult to obtain a mixture that was at the same time fluid but did not bleed. This led to the conclusion that slight changes in water content, or granulometry of aggregate may result either in a mixture with inadequate flowing ability, or alternatively one with a tendency for coarse aggregate to segregate. Therefore, it became necessary to incorporate a viscosity-modifying agent in the concrete mixture.

Viscosity-modifying agents can be a natural polymer such as guar gum, a semi-synthetic polymer such as hydroxy propyl methyl cellulose, or water-soluble polysaccharides, including those derived from a microbial source such as welan gum. Experiments involving three types of gums were being carried out by the two researchers. One commonly used thickener in cement-based systems, namely hydroxy propyl methyl cellulose (HPMC), a low-priced gum known as guar gum and a special product called welan gum were selected for studying their suitability for use in self-compacting concrete. On a first consideration, all these qualified as viscosity modifying agents. However, some of these substances, with the exception of welan gum, had shortcomings. Guar gum had to be made into a suspension in water after heating to 60°C and stirring for about one hour. This solution lost its suspending power after twelve hours. HPMC was not compatible with the naphthalene formaldehyde superplasticizer and entrained excessive air, causing a reduction in strength (Figure 2.3).

Welan gum is suitable for use in self-compacting concrete because it combines with most types of superplasticizer and has superior suspending power, compare to guar gum and hydroxy propyl methyl cellulose (HPMC).
In order to arrive at an acceptable combination of dosages of welan gum and superplasticizer, Subramanian and Chattopadhyay (2002) ran several tests related to the tendency of the concrete to bleed and its ability to pass the U-tube test. They discovered that with a combination corresponding to 0.1 percent of welan gum and 0.53 percent by weight of water acrylic copolymer type superplasticizer, a satisfactory self-compacting mixture could be obtained.

**Khayat et al.** The use of self-consolidating concrete can facilitate the placement of concrete in congested members and in restricted areas. Given the highly flowable nature of such concrete, care is required to ensure adequate stability. This is especially important in deep structural members and wall elements where concrete can segregate and exhibit bleeding and settlement, which can result in local structural defects that can reduce mechanical properties.

The objective of Khayat’s (1997) et al. research was to evaluate the uniformity of in situ mechanical properties of self-consolidating concrete used to cast experimental wall elements. Eight optimized SCC mixtures with slump flow values greater than 630 mm and a conventional concrete with a slump of 165 mm were investigated. The self-compacting concrete mixtures incorporated various combinations of cementitious materials and chemical admixtures. The
water-cementitious materials ratios ranged from 0.37 to 0.42. Experimental walls measuring 95 cm in length, 20 cm in width, and 150 cm in height were cast. After casting, no consolidation was used for the SCC mixtures, while the medium fluidity conventional concrete received thorough internal vibration. Several cores were obtained in order to evaluate the uniformity of compressive strength and modulus of elasticity along the height of each wall. Khayat (1997) et al. found out that all cores from both types of concrete exhibited little variation in compressive strength and modulus of elasticity in relation to height of the wall, indicating a high degree of strength uniformity. However, compressive strength and modulus of elasticity were greater for SCC samples than those obtained from the medium fluidity conventional concrete.

**Dehn et al.** Dehn (2000) et al. have focused their research work on the time development of SCC compressive and splitting tensile strength and the bond behavior between the reinforcing bars and the self-compacting concrete compared to normal concrete.

In order to ensure a good production of SCC, a mix design should be performed, so that the predefined properties of the fresh and hardened concrete would be reached for sure. All the components should be coordinated so that bleeding and segregation would be prevented. Because of these aspects, their mix design was based on experience from Japan, Netherlands, France, and Sweden. Due to the fact that the load bearing capacity of a reinforced concrete structure is considerably influenced by the bond behavior between the reinforcing bars and the concrete, the following items were taken into account:

* anchorage of the reinforcing bars
* crack width control
* lapped reinforcing bars
For this reason, investigations on the bond behavior between the re-bars and the SCC were necessary, especially considering the time development of the bond strength. These investigations showed, that the main parameters which influence the bond behavior are the surface of the re-bars, the number of load cycles, the mix design, the direction of concreting, as well as the geometry of the (pull-out) test specimens (Figure 2.4). The bond behavior was determined under uniform static loading using pullout specimens having a uniform concrete cover around the reinforcing bar. The bar diameter for the whole test series was 10 mm and the concrete cover around it had a diameter of 10 cm and a length of also 10 cm.

To avoid an unwanted force transfer between the reinforcing bar and the concrete in the unbonded area, the re-bars were encased with a plastic tube and sealed with a highly elastic silicone material. The re-bars were placed concentrically and the concrete was cast parallel to the loading direction. The tests were carried out in an electro mechanic testing machine where the specimens were loaded with a loading rate of 0,0008 mm/sec. The applied force of the machine
was measured corresponding to the slip displacement of the reinforcing bar on the non-loaded side. The increase of the slip path was constantly monitored during the whole testing period.

Experimental results showed higher compressive strengths (36%) and splitting tensile strengths (28%) of the SCC specimens compared to normal concrete specimens. Also, the bond behavior measured at 1, 3, 7 and 28 days after concreting was better for self-compacting concrete than that of normally vibrated concrete.

**Kuroiwa:** Kuroiwa (1993) developed a type of concrete, which contained materials normally found in conventional concrete such as Portland cement, aggregate, water, mineral and chemical admixtures. The chemical admixtures were added in order to improve the deformability and the viscosity of the concrete. The newly developed type of concrete was called super-workable concrete and showed excellent deformability and resistance to segregation. It could also fill completely heavily reinforced formworks without any use of vibrators. After the laboratory tests it was found out that the super-workable concrete had superior properties in the fresh state and excellent durability after hardening. Because of its properties, it was considered that it would be suitable for projects involving heavily reinforced areas and was employed in the construction of a 20-story building. The concrete was placed in the center-core from basement to the third floor. The building had a design of a hybrid structure, in which the reinforced concrete core was surrounded by a steel mantle. The greatest diameter used for the reinforcing bars was 2” and the forms were very congested. Ready mixed concrete plants situated near the construction site produced approximately 2000 m³ of super-workable concrete, which was placed successfully.

**Ferraris et al.:** The slump test is widely used to evaluate the workability of concrete, but in the case of self-compacting concrete, it has serious drawbacks. Other flow characteristics such as viscosity or filling capacity are needed to define the flow in self-compacting concretes. The
The research objectives of Ferraris (1999) et al. were to test flow characteristics of SCC using two concrete rheometers and the widely recognized V-flow and U-flow tests, and to determine the correlation between the two rheometers and the tests.

The main property that defines the self-compacting concrete is high workability in attaining consolidation and specified hardened properties. Workability is defined either qualitatively as the ease of placement or quantitatively by rheological parameters. The most commonly used test to determine workability in practice is the slump cone test. Either the vertical slump distance or the horizontal spread of the concrete can be measured. The most common rheological parameters used to qualify workability are the yield stress and plastic viscosity as defined by the Bingham equation.

The knowledge of the two parameters, yield stress and plastic viscosity, allows a quantitative description of the workability. The Bingham equation is a linear relationship between the shear rate, \( \dot{\gamma} \), and the shear stress, \( \tau \). The viscosity, \( \eta \), is the slope and the intercept is the yield stress, \( \tau_0 \), as shown in the equation (2.1).

\[
\tau = \tau_0 + \eta \dot{\gamma}
\]  
(2.1)

A highly flowable concrete is not necessarily self-compacting, because SCC should not only flow under its own weight, but should also fill the entire form and achieve uniform consolidation without segregation. This characteristic of SCC is called filling capacity.

In their research Ferraris (1999) et al. measured the rheological properties of the concrete mixtures using two rheometers, the IBB and the BTRHEOM instruments.

The IBB rheometer was developed in Canada and consists of a cylindrical container holding the concrete, with an H-shaped impeller driven through the concrete in a planetary motion. The speed of the impeller rotation was first increased to a maximum rotation rate and
then the rotation rate was decreased in six stages with each stage having at least two complete center shaft revolutions. The torque (N·m) generated by the resistance of the concrete specimen to the impeller rotation was recorded at each stage as well as the impeller rotation rate (revolutions per second) measured by the shaft tachometer.

The BTRHEOM is a parallel plate rheometer, i.e., the concrete is sheared between two plates. The plate at the bottom is stationary and the plate at the top rotates with variable speed, similar to the impeller of the IBB rheometer. The torque generated during rotation is recorded while the rotation rate is first increased and then decreased in stages. The rheological parameters can be calculated using the Bingham equation applied to the torque and rotation rate data of the decreasing speed portion of the test. Due to the simple geometry of the shearing area, it is possible to calculate the results in fundamental units, i.e., Pa for yield stress and Pa·s for viscosity.

The criterion adopted by Ferraris (1999) et al. in the study, in order to evaluate the compactability of the concrete by using the U-flow test, was that if the filling height was more than 70% of the maximum height possible, the concrete would be considered self-compacting. Comparisons of the yield stresses from the two rheometers indicated no correlation. The lack of correlation might have been due to the range of yield stresses measured, which was in the vicinity of zero and sometimes negative. This situation was expected since all the concretes tested were highly flowable and therefore, might have had very small yield stresses. The negative values were due to the method (Bingham or Herschel-Bulkley) used to calculate the yield stress.

The comparisons of the measured viscosities showed a good correlation, which could be approximated by:

\[
\eta_a = 35 + \eta_i ,
\]  

(2.2)
where $\eta_a$ is the viscosity measured with the BTRHEOM and $\eta_i$ is the viscosity measured with
the IBB. The correlation was considered acceptable because of the wide range of viscosity
covered (13 values). It was also considered preliminary due to the lack of variation in the
properties of the materials used, i.e., one type of cement and one type of aggregate.

According to Beaupre (Canada, 1994), a better method to evaluate concrete with
specified flow property is to plot the yield stress versus the viscosity as shown in Figure 2.5.
Concrete mixtures, determined to have a desired property, define an area in the plot called the
“workability box”. Ferraris (1999) et al. plotted the yield stress versus the viscosity as measured
with the BTRHEOM instrument.

![Figure 2.5 Viscosity –Yield stress and the workability box (Ferraris et al., 1999).](image)

The “workability box” defines the range of viscosity and yield stress needed for a self-
compacting concrete. If these results would have been trial batches, the drawing of the box
would have allowed the operator to determine whether a mixture was self-compacting concrete
based on the rheometer results.
After studying the thirteen concrete mixes, Ferraris (1999) et al. found out that the slump flow alone is not enough to determine whether a flowable concrete is a self-compacting concrete. Due to the fact, that the values measured with the two types of rheometers correlate relatively well on viscosity but not on yield stress, further measurements are needed to determine if the correlation holds for other types of aggregate and cementitious materials. Also, based on the data acquired, the viscosity and the yield stress did not correlate with the V-funnel or the U-flow tests.

2.2 Influence of Admixtures on Concrete Properties

In the following are presented several papers, found in the literature, on the effects of mineral and chemical admixtures on the fresh and hardened concrete. The mineral admixtures referred to are blast-furnace slag, fly ash, and silica fume. The chemical admixtures considered are high range water reducer or superplasticizer and viscosity-modifying agent.

2.3 Mineral Admixtures

Mineral admixtures are added to concrete as part of the total cementitious system. They may be used in addition to or as a partial replacement of Portland cement in concrete depending on the properties of the materials and the desired effect on concrete (Mindess et al., 2003). Mineral admixtures are used to improve a particular concrete property such as workability, strength or compactability. The optimum amount to use should be established by testing to determine (1) whether the material is indeed improving the property, and (2) the correct dosage rate, as an overdose or underdose can be harmful or not achieve the desired effect, because they react differently with different cements (Kosmatka et al., 2002).

2.4 Blast Furnace Slag

Blast furnace slag (BFS), also called slag cement, is made by rapidly quenching molten blast-furnace slag and grinding the resulting material into a fine powder. BFS is classified by
ASTM C 989 according to its level of reactivity. Depending on the desired properties, the amount of BFS can be as high as 50 percent by mass, of the total cementitious materials content (Ramachandran, 1981).

In his research, Russell (1997) found out that the use of slag cement lowers concrete permeability, thereby reducing the rate of chloride ion diffusion. Proper proportioning of slag cement can eliminate the need to use low alkali or sulfate-resistant Portland cements. Russell’s results showed that BFS can be used to enhance the strength gain at later ages than 28 days, it replaces 20 to 30 percent by mass of the Portland cement.

Sobolev (1999) studied the effect of adding up to 50% by mass granulated blast-furnace slag in the cementitious material that resulted in the increasing of chemical and thermal resistance. The very low permeability of the concrete obtained, provided high resistance to chemical attack and to freezing and thawing cycles. There was no visible destruction of blast-furnace slag concrete samples after 140 cycles of freezing and thawing at -50ºC, and they also demonstrated high resistance to elevated temperatures.

Ozyildirim (2001) studied three concrete mixtures placed in the jointed plain concrete paving project in Newport News, Virginia. The main goal was to reduce the shrinkage and improve the flexural strength of the concrete. Two of the mixtures that he used contained ground-granulated blast furnace slag and the third contained Class F fly ash. The content of blast-furnace slag in the two mixtures was 30% by mass of the total cementitious material and the maximum water-cement ratio was 0.50.

Flexural strengths at 28 days were similar for fly ash and blast-furnace slag concretes, but after 60 days they were greater for those which contained slag. The shrinkage values of concretes containing slag cement were slightly greater than the values of concretes with fly ash. For
freezing and thawing tests the acceptance criteria at 300 cycles were a weight loss of 7 percent and less, a durability factor of 60 and more, and a surface rating (ASTM C 672) of 3 or less. Blast-furnace slag concretes complied with those requirements, but the fly ash concretes had slightly higher weight loss than required.

Hale (2000) et al. investigated the effects of the cement replacement with 25% by mass blast-furnace slag on fresh and hardened concrete properties. As a result, compressive strengths were increased by approximately 25 percent at 28 days as compared to normal Portland cement mixtures. The use of 25 percent blast-furnace slag led to minor to moderate reductions in slump and slightly lower air contents as compared to conventional mixtures.

Klieger and Isberner (1967) have conducted a comprehensive study on the properties of pastes and concretes made with Portland blast-furnace slag cements (ASTM Type IS). Five commercial IS brands were included in these tests, in addition to a number of Portland cements. These IS type cements were made by intergrinding a mixture of Portland cement clinker and granulated blast-furnace slag or by making an intimate and uniform blend of Portland cement and fine granulated blast-furnace slag. The amount of slag used in the mixtures was between 20% and 350% by mass of the total cementitious material.

The compressive strengths of concretes made with type IS cements were generally lower at early ages than the strengths of concretes made with Type I cement. However, at 3 months, one year, and 3 years, the strengths were generally equal to or greater than those of the Type I cement. Also, at the same compressive strength, values of splitting tensile strength were essentially equal for both types of cement.

2.5 Fly Ash

Gebler and Klieger (1983) studied concretes containing fly ash in order to determine its effect on the air-void stability. 10% to 20% by mass of fly ash was used in the total amount of
cementitious material. The tests undertaken indicated that air contents of concrete containing
Class C fly ash appeared to be more stable than those of concrete containing Class F fly ash. This
occurred primarily because Class C fly ashes have lower organic matter content and carbon
content values. The studies revealed that the higher the organic matter content of a fly ash, the
higher would be the air-entraining admixture requirement for concrete in which the admixture is
used. Practically, all concretes containing fly ash required more air-entraining admixture than
concretes without fly ash and the concretes containing Class C fly ash tended to lose less air than
concretes with Class F fly ash.

Naik and Singh (1997) conducted tests on concretes containing between 15% and 25% by
mass Class F and Class C fly ashes, to evaluate time of setting, bleeding, compressive strength,
drying shrinkage, and abrasion resistance. The effects of moisture and temperature during curing
were also examined. The results of the research showed that concretes containing Class C fly ash
and were moist cured at 73°F (23°C) developed higher early age (1 to 14 days) compressive
strengths than concretes with Class F fly ash. The long-term (90 days and greater) compressive
strength of concretes containing fly ash was not significantly influenced by the class of fly ash.

The air-cured concretes containing Class F fly ash did not develop strengths equivalent to
air-cured normal concretes and air-cured concretes containing Class C fly ash developed
relatively greater compressive strengths than air-cured concretes containing Class F fly ash. For
concretes containing either class of fly ash, compressive strengths at 7 days increased with an
increase in curing temperature. Concretes with fly ash showed less bleeding than conventional
ones. Further, concretes with Class C fly ash showed less bleeding than concretes with Class F
fly ash. Also, drying shrinkage results for concretes containing fly ash were essentially the same
as for conventional concretes, regardless of initial curing temperature. Drying shrinkage results
for concretes with Class F fly ash were, on the average, slightly less than for concretes with Class C fly ash. The abrasion resistance of the concretes was essentially dependent on their compressive strength and time of setting for most of the concretes containing the two types of fly ash was retarded.

Dietz and Ma (2000) in their research, showed a possible application of lignite fly ash (LFA) for the production of Self-Compacting Concrete (SCC). The lignite fly ash has not only some characteristics of potential hydraulic materials, it can also improve the rheological properties of the fresh concrete because of its fineness, which is a primary advantage for SCC. Self-compacting concrete with lignite fly ash shows a good flowing ability and high self-compactability.

Lignite Fly Ashes (LFA) are fine residues of ground lignite burned in the power plant industry. In comparison with fly ashes from coal, LFA contains obviously more free lime and sulfate. The chemical and mineralogical composition of the LFA shows wide variations and because of that, fly ash with a free lime content of approximately 22% was chosen for the project. The choice of this type of ash was due to its availability and its constant quality. Two differently lignite fly ashes were used. One LFA, which was untreated showed a high free lime content and was called Untreated Lignite Fly Ash (U-LFA). The other fly ash, which was treated with water, was called Treated Lignite Fly Ash (T-LFA). In the latter case, the free lime has changed into calcium hydroxide. Cementitious material consisted of 75% cement and 25% fly ash, by mass. It was discovered that, if the cement is replaced from 10% to 25% by U-LFA or T-LFA, the water requirement is reduced. This is favorable for the workability of the fresh concrete. The reduced water requirement indicated that the grains of the cement - LFA mixture were more densely compacted. The volume between the particles, which should be filled with
water, became smaller due to the denser packing. Furthermore, the spherical LFA particles favorably affected the workability of the mixture. It was seen that the U-LFA set very quickly. The setting and hardening of the cement-U-LFA paste was clearly shortened by U-LFA. The higher the U-LFA proportion was, the faster the paste hardened and, because of this, U-LFA was replaced by T-LFA, which shortened the setting times only slightly.

The slump flow and funnel tests showed values within the ranges of other tests previously undertaken. The compressive strengths of hardened concrete specimens decreased with the increasing proportion of U-LFA over 25%, while they remained approximately constant when T-LFA was used in percentages that exceeded 25%. After 28 days, compressive strengths between 50 MPa and 60 MPa and splitting tensile strengths between 4 and 5 MPa were obtained for self-compacting concretes, with w/c ratios ranging from 0.3 to 0.46.

2.6 Silica Fume

In their study, Khaloo and Houseinian (1999) investigated the influence of silica fume on compressive strength and durability of concrete. The percentage of silica fume was between 1% and 15% and the water-cement ratios ranged from 0.3 to 0.6. The coarse and fine aggregates consisted of river gravel and sand with maximum size of 25 mm and 5 mm, respectively.

The test results indicated that 5 to 10 percent by mass replacement of silica fume for cement provided the highest strength for short and long terms. Compressive strength of silica fume concrete at 28 days compared to conventional concrete increased by 20 to 40 percent, for all the variables considered. After 300 cycles, all of the concrete specimens passed ASTM C 666 for freezing and thawing durability test.

Khayat (1997) et al. evaluated the influence of silica fume blended with cement on some properties of fresh and hardened concrete. The properties studied were bleeding, slump loss, time of setting, compressive strength. A total of 26 commonly used concrete mixtures in the Canadian
construction industry were developed. Half of the mixtures were air-entrained and had water-cement ratios ranging from 0.3 to 0.6. The remaining half contained non air-entrained mixtures and the water-cement ratios varied between 0.45 and 0.7.

Studies undertaken revealed that the addition of small percentages of silica fume, usually under 10%, and proper amount of high range water-reducing admixture (superplasticizer) could decrease the viscosity of the paste, thus reducing the water demand and the risk of bleeding. The small particles of silica fume can displace some of the water present among flocculated cement particles and fill some of the voids between the coarser particles, which otherwise can be occupied by some of the mix water. This causes some gain in workability and densification of the fresh paste. Concrete mixtures made with blended silica fume cement exhibited substantially less bleeding than those made with type I Portland cement. In addition, mixtures made with blended silica fume cement showed 15 to 20 mm greater loss of slump than concretes without silica fume.

In general, concrete mixtures made with type I Portland cement and blended silica fume cement exhibited similar initial times of setting that were within one hour apart. When 15% of silica fume was added with a high dosage of superplasticizer, initial and final times of setting were delayed by approximately 1 and 2 hours, respectively. Regarding the compressive strength, Khayat (1997) et al. found out that after three days of curing, no effect of cement type was observed on the development of compressive strength for concretes. However, starting with the seventh day, mixtures containing blended silica fume cement exhibited greater strength than those made with type I Portland cement. After 28 days, in non air-entrained concretes, the use of blended silica fume cement resulted in approximately 20% strength gain compared with mixtures containing only Portland cement. Similar results were obtained in air-entrained concrete.
Ray and Chattopadhyay (1999) carried out studies on the effects of 4, 8, 12, 16% of silica fume by weight of cement on compressive strength and resistance against chemicals (acids and sulfates) of concretes. For testing of resistance against acids and sulfates 50 mm cubic samples were oven dried at 105°C and immersed in 2% HNO₃, 2% H₂SO₄, and 5% Na₂SO₄ solutions for 45 days. The weight and strength losses were noted with reference to a set of undisturbed samples cured in water. Conventional concrete exhibited slight bleeding, but this phenomenon was completely eliminated when silica fume has been added in the mixture. Also, the values of air content decreased with the increase in silica fume content. They dropped from 5.5% for normal concrete to 3.5% for 16% of silica fume replacement. Concretes with a content of 8% silica fume showed the highest compressive strength values after 28 days (45 MPa), followed by concretes having 4, 12, and 16%. Addition of silica fume at all percentages improved the flexural strength, with a significant rise for a 4% content (8.5 MPa).

As regarding the resistance against acids and sulfates, test results showed that immersion in H₂SO₄ has caused maximum loss in weight and strength, followed by HNO₃ and Na₂SO₄. The maximum strength loss of 24% has occurred for mixtures without any silica fume, whereas the minimum loss of 12% occurred for mixtures containing 4% of silica fume.

The influence of silica fume on workability and compressive strength of concretes were the major research objectives for Duval and Kadri (1998). Concretes that have been investigates had low water-cement ratios (0.25 to 0.40). The type I Portland cement was replaced by 10-30% by mass silica fume and superplasticizer was added. It was found that silica fume increased best the compressive strength (25%) and the workability of concretes when its content was between 4 and 8 percent. Duval and Kadri also found out that if silica fume exceeds 15% of the cementitious material, both compressive and tensile strengths are reduced.
2.7 Chemical Admixtures

Chemical admixtures represent those ingredients which can be added to the concrete mixture immediately before or during mixing. The use of chemical admixtures such as water reducers, retarders, high-range water reducers or superplasticizers (SP), and viscosity-modifying admixtures is necessary in order to improve some fundamental characteristics of fresh and hardened concrete. They make more efficient use of the large amount of cementitious material in high strength and self-compacting concretes and help to obtain the lowest practical water to cementing materials ratio.

Chemical admixtures efficiency must be evaluated by comparing strengths of trial batches. Also, compatibility between cement and supplementary cementing materials, as well as water reducers, must be investigated by trial batches. From these, it will be possible to determine the workability, setting time, bleeding, and amount of water reduction for given admixture dosage rates and times of addition. Due to the fact that this research dealt only with superplasticizers and viscosity modifiers, papers found in the literature about these types of chemical admixtures would be presented in the following.

2.8 Superplasticizers

A study of four commercially available superplasticizers used in type I Portland cement concrete mixes was done by Whiting (1979). They represented both melamine- and naphthalene-based formaldehyde condensation products. Hardened concrete specimens were prepared and tested for compressive strength development, drying shrinkage, freeze-thaw resistance, and resistance to deicing scaling. From his research, Whiting found out that high range water reducers were capable of lowering the net water content of concrete mixtures from 10% to 20% when used in dosages recommended by the manufacturers.
Also it was found out that one- and three-day compressive strengths could be substantially increased through use of high range water reducers. Compressive strengths over 10,000 psi (70 MPa) were obtained after 28 days of curing. The drying shrinkage was slightly reduced in the attempt to lower the net water content of the concrete mixtures. Freeze-thaw durability and resistance to deicer scaling of air-entrained concretes containing superplasticizer were equal to or slightly better than air-entrained normal concretes prepared with equal slump and cement content.

Ozkul and Dogan (1999) studied the effect of a N-vinyl copolymer superplasticizer on the properties of fresh and hardened concretes. Workability of concrete was measured by slump flow test and in situ tests were undertaken to find out the pumping ability of superplasticized concrete. The coarse aggregate was crushed stone with the maximum size of 25 mm. By using this chemical admixture, which was a little bit different from the conventional ones, the ability of water reduction was increased along with the retention of high workability for a longer time.

In situ test results obtained by Ozkul and Dogan (1999) demonstrated that the superplasticized concrete could be pumped easily from a height of about 13 m and the filling capacity was greater than 85%. The pumping pressure was the same as for normal pumpable concrete and no segregation was observed. For mixtures with water-cement ratios between 0.3 and 0.45, the slump diameters were between 500 mm and 740 mm and the compressive strength varied between 53 MPa and 68 MPa at 28 days of age.

In their work, Roncero (1999) et al. evaluated the influence of two superplasticizers (a conventional melamine based product and a new-generation comb-type polymer) on the shrinkage of concrete exposed to wet and dry conditions. Tests of cylinders with embedded extensometers have been used to measure deformations over a period of more than 250 days.
after casting. In general, it was observed that the incorporation of superplasticizers increased the drying shrinkage of concretes when compared to conventional concretes, whereas it did not have any significant influence on the swelling and autogenous shrinkage under wet conditions. The melamine-based product led to slightly higher shrinkage than the comb-type polymer.

Kasami (1978) et al. have investigated the pumpability of superplasticized concrete under field conditions. In their experiment, about 2000 m³ of normal and lightweight aggregate concrete, involving 14 mixes with and without superplasticizers were pumped horizontally. The pumping distance was 109 m and line diameter 125 mm. The dosage of the naphthalene-based superplasticizer was in the range of 0.03% to 0.04% by weight of cement and concrete mixing was done in ready-mix agitator-type trucks. After the addition of the superplasticizer, the mixer was rapidly agitated for one minute. Following this process, the concrete was pumped at rates of 10, 20, 30, 40, 50, and 60 m³/h. Pump pressure and line pressure were measured at each pumping rate. The tests data indicated that pumping pressure and line pressure loss for normal weight concrete were reduced by about 30%, whereas those for lightweight concrete were reduced by no more than 10%.

The effect of superplasticizer on the balance between flowability and viscosity of paste in self-compacting concrete was investigated by Ouchi (1996) et al. From experimental results, the ratio of V-funnel speed to flow area of cement paste with a fixed amount of superplasticizer was found to be almost constant, independent of the water-cement ratio. A higher amount of superplasticizer resulted in a lower ratio of V-funnel speed to flow area. The ratio was proposed as an index for the effect of superplasticizer on cement paste flowing ability and viscosity from the viewpoint of achieving self-compactability. However, the relationship between high range
water reducer amount and its effect was found to differ depending on the type of cement or chemical admixture.

A rational mix-design method (Figure 2.6) for self-compacting concrete was proposed by Okamura (1997) et al. and the indexes for flowability and viscosity were defined as $\Gamma_m$ and $R_m$, respectively. They were defined as follows:

$$\Gamma_m = \frac{(r_1 r_2 - r_0^2)}{r_0^2}$$  \hspace{1cm} (2.3)

$$R_m = \frac{10}{t}$$  \hspace{1cm} (2.4)

where: $r_1$ and $r_2$ are the measured flow diameters perpendicular on each other;

$r_0$ is the flow cone’s bottom diameter; and

$t$ is the measured time (in seconds) for cement paste to flow through the funnel.

These indexes are practical to use because they are easy to obtain from simple test results. Larger $\Gamma_m$ values indicate higher flowability and smaller $R_m$ values indicate higher viscosity. A cement paste with $\Gamma_m = 5$ and $R_m = 1$ was found to be the most appropriate mixture for achieving self-compacting concrete.
2.9 Viscosity Modifiers

The viscosity modifiers or viscosity modifying admixtures (VMA) were developed in order to improve the rheological properties of cement paste in concretes (Khayat and Guizani, 1997). These admixtures enhance the viscosity of water and eliminate as much as possible the bleeding and segregation phenomena in the fresh concrete. Because not all types of viscosity modifiers have showed satisfactory results, research has concentrated on only two types: welan gum and antiwashout admixtures.

In their research, Takada (1999) et al. investigated the influence of welan gum, a kind of natural polysaccharide-based viscosity agent, on the water-cementitious material ratio. It was found that the viscosity modifier raised the value of the ratio due to its characteristics to make the mixture viscous. Welan gum increased the viscosity of the free water in the fresh concrete by the ability of its polymers’ characteristics to associate each other in water. The tests results showed that a slump flow value of 650 ± 30 mm and a V-funnel time of 11 ± 2 sec were achieved by using 0.01 to 0.02 percent viscosity agent and 0.025 to 0.035 percent superplasticizer from the total cementitious material. The values were considered adequate for a workable self-compacting concrete.

Khayat (1997) et al. evaluated the properties of welan gum in achieving self-compacting concrete for use in congested members and confined areas. The viscosity-modifying admixture (welan gum) was used to ensure adequate stability for concrete cast in deep structural members and wall elements in order to avoid segregation and bleeding which can result in local structural defects that can affect its mechanical properties. All the SCC mixtures had high filling capacities ranging between approximately 60 and 70 percent, indicating excellent deformability without blockage among closely spaced obstacles. No external bleeding was observed on the top surface
of any of the cast wall elements and the settlement values of the self-compacting mixtures measured on 150 cm walls ranged between 1.4 and 2.9 mm. This corresponds to 0.1 and 0.2 percent of the wall heights and is less than that for normal concrete, which was around 0.4 percent.

In order to verify the properties of self-compacting concrete, Dehn (2000) et al. studied the interaction between the superplasticizer and viscosity-modifying agent and the bonding between the reinforcing bars and self-compacting concrete. They found out that the polymer in the viscosity modifier (welan gum) and the polymer in the superplasticizer restrain each other and this phenomena results in a higher segregation resistance and some larger dosage of SP for a particular deformability. It was also seen that depending on the mix design and chemical admixtures dosages the bond behavior in self-compacting concrete was better than the bond in conventional concrete.

Trial mixes with varying dosages of viscosity-modifying admixture (VMA) and high range water reducing admixture (HRWR) to achieve a wide range of flow behavior were investigated by Ferraris (1999) et al. In these mixes, the VMA was incorporated in order to enhance the yield value and viscosity of the fluid concrete, hence reducing bleeding, segregation, and settlement. The enhanced cohesiveness could ensure better suspension of solid particles in the fresh concrete and therefore good deformability and filling capacity was achieved during casting. The HRWR used was a carboxylated copolymer-based mixture and the VMA was a modified cellulose product. Even though the dosages of chemical admixtures have been varied, the slump flow tests undertaken, were not enough to determine whether a flowable concrete was self-compacting or not.
Subramanian and Chattopadhyay (2002) carried out several trials to achieve an approximate mix proportion of self-compacting concrete. At the initial stages of development, mixtures were formulated without incorporating any viscosity-modifying agents. After several trials, it was apparent that these admixtures imparted exceptional stability to the mixture. The viscosity-modifying agent was then required, because slight variations in the amount of water or in the proportions of aggregate and sand would have made the concrete unstable, that is, water or slurry might have separated from the remaining material. However, not all VMAs were suitable for concrete applications, due to the fact that some of them restricted the choice of superplasticizer. The welan gum that was used, a well-known viscosity-modifying agent, was found incompatible with melamine formaldehyde condensate-based type of superplasticizer, but after a few trials, a naphthalene formaldehyde condensate and an acrylic polymer superplasticizers were found to be suitable for application in self-compacting concrete. Another two types of viscosity-modifiers, a hydroxy propyl methyl cellulose (HPMC) and guar gum, were also selected for being used along with the concrete, but they failed to react properly with the HRWR admixture. The guar gum had to be made into a suspension in water after heating it to 60°C and stirring for about one hour, but it lost its suspending power after 12 hours. The HPMC entrained excessive air, causing a reduction in strength. Subramanian and Chattopadhyay (2002) found out that with a combination corresponding to 0.012 percent of welan gum and 0.036 percent acrylic polymer superplasticizer by weight of cementitious material, a satisfactory self-compacting mixture could be obtained.

Investigations regarding the effects of viscosity-modifying admixture (VMA) concentration, placement height, and mode of consolidation on enhancing the stability of mixtures were done by Khayat and Guizani (1997). In a first phase, bleeding and settlement were
determined using 70 cm high columns cast with concrete containing 0.035 and 0.07 percent viscosity modifiers dosages. The water-cement ratios were between 0.50 and 0.70 and the slump values from 140 to 200 mm. In the second phase, bleeding, settlement, and segregation were evaluated for concretes with 200 mm slump, cast in 50, 70, and 110 cm high columns. This time the mixtures had a water-cement ratio of 0.50 and the same amounts of VMA, which was chosen to be welan gum. The superplasticizer used was a liquid sulfonated naphthalene.

The studies showed that the addition of welan gum affected the aqueous phase of the cement paste where chains of the water-soluble polymer could imbibe some of the free water in the system, thus enhancing the viscosity of the cement paste. As a result, less free water can be available for bleeding. The enhanced viscosity of the cement paste can also improve the capacity of the paste to suspend solid particles, process that reduces the sedimentation. Mixtures containing a viscosity modifier exhibited a shear thinning behavior whereas the apparent viscosity decreased with the increase in shear rate. The mixtures incorporated 0.035 and 0.07 percent of viscosity-modifying admixture and between 0.022 and 0.035 superplasticizer, by mass of cementitious materials. All mixtures incorporating VMA exhibited lower rates of increase in bleeding and settlement than those without VMA, regardless of water-cement ratios.

Also, concretes containing 0.035 and 0.07 percent viscosity-modifying admixture had approximately 30 and 50 percent lower segregation coefficients than conventional ones, regardless of the height of casting. As seen from the above investigations, it is important to enhance the stability of fluid concrete used to facilitate the casting in congested or restricted areas. The enhanced cohesiveness of such concrete can reduce structural defects resulting from increased porosity under aggregate and embedded reinforcement. This can lead to improved
tensile strength, impermeability, and bond strength with reinforcement, especially in deep structural sections, which can contribute to the reduction in congestion.

2.10 Tensile Strength

The main purpose of measuring the strength of concrete test specimens is to estimate its strength in the actual structure. This estimation gives an indication of the strength of concrete in a structure, which is dependent on the adequacy of compaction and curing.

The splitting test is rather simple to perform, does not require other equipment than that needed for the compression test, and gives an approximately similar value of the “true” tensile strength of concrete (Neville, 1971).

Some aspects regarding the tensile strength of lightweight concrete and aggregates were studied by Holm (1994). He showed that shear, bond strength between aggregate and cement paste, and crack resistance are related to tensile strength that is, in turn, dependent upon tensile strength of the coarse aggregate and cement paste phases and the degree to which the two phases are securely bonded. Traditionally, tensile strength has been defined as a function of compressive strength, but this is known to be only a first approximation that does not reflect aggregate particle strength, surface characteristics, nor the concrete’s moisture content and distribution.

Pfeifer (1967) studied the effect of natural fine sand, in replacement of the lightweight fines, on seven structural lightweight concrete splitting tensile strengths. The test results showed equal splitting strengths for all continuously moist cured lightweight and normal weight concretes when the compressive strengths were equal. However, the splitting strengths of the lightweight concretes were generally reduced when the cylinders were allowed to dry before testing. The use of fine sand minimized this strength reduction and with the exception of a few lightweight aggregates, the natural sand provided improvements in the workability and
finishability of the plastic concrete. By using 6 x 12 inches cylinders for splitting tensile strength tests, Pfeifer (1967) discovered that splitting tensile strength of air-dried lightweight concrete generally increased with increasing natural sand content.

Studies regarding the effects of curing and drying environments on splitting tensile strength of lightweight and normal weight concretes were carried out by Hanson (1968) in two test series. The first series showed that the duration of the initial moist curing period prior to drying at 50 percent relative humidity had little effect on the splitting tensile strength. While there was a loss of splitting strength for the lightweight concrete early in the drying periods, continued storage in the drying atmosphere led to considerable gain in the splitting strengths.

In the second series, concretes were subjected to drying for 21 days at different levels of relative humidity after initial moist curing for 7 days. Only minor changes of splitting strength were found as the relative humidity varied from 75 to 10 percent.

Investigations involving the splitting tensile strength of very high-strength concrete (f\textsuperscript{c} > 50 MPa) for penetration-resistant structures were carried out by O’Neil (2002) et al. Due to the addition of silica fume, high-range water reducing admixtures and special curing conditions provided, the tensile strength of the concrete was higher than that of conventional concrete.

### 2.11 Bonding between Aggregate and Cement Paste

Physical strength and integrity of the bond achieved at the aggregate-paste interface variously depends upon the character of the aggregate particle surface and the nature of the immediately adjacent hydrated cement, so practically the nature of the interface is controlled by the properties of both the aggregate and the cement paste (Dhir and Dyer, 1999). Aggregates used in concrete have varying degrees of porosity, shape and surface roughness as well as the possibility of adherent dust and dirt (Barksdale, 1991).
The interfacial region of paste surrounding the aggregate has been shown to be different from that of the bulk paste and has been called interfacial transition zone (ITZ) (see Chapter 5).

Subramanian (1999) has shown that the parameters of concrete could be altered for the better by choosing proper materials or by modifying the physical interfaces between the materials. He found that in a 25-50 µm thick zone surrounding the aggregate, a duplex layer of calcium hydroxide - Ca(OH)$_2$ – and a porous shell, having properties different from the bulk cement paste, were present. These factors encourage the deposition of oriented crystals of calcium hydroxide, giving rise to weak planes along which microcracks occur. Some methods found to improve the aggregate paste interface were to reduce the water-cement ratio, to add small amounts of microsilica, and to coat the aggregate faces with a reactive layer. By precoating the aggregate particles with cement-microsilica slurry, the bond strength increased, visible through a compressive strength increase, and the porosity at the transition zone was practically eliminated.

Regarding the aggregate in high-strength concretes, Kosmatka (2002) et al. indicated that careful attention must be given to aggregate size, shape, surface texture, mineralogy, and cleanness. In high-strength concretes, the strength of the aggregate itself and the bond with the cement paste became very important factors. Tests have shown that crushed-stone aggregates produce higher compressive strength in concrete than gravel aggregate using the same size aggregate and the same cementing materials content. It has been assumed that the increase in strength was due to a superior aggregate-paste bond when using rough, angular, crushed material. In addition, coarse aggregates used in high-strength concrete should be clean, that is, free from detrimental coatings of dust and clay, since they might affect the quantity of fines and consequently the water demand and aggregate-paste bond of a concrete mix.
Bijen and Rooij (1999) discovered that the composition of the interfacial transition zone (ITZ) is weak and porous and influences the bulk properties of concrete, such as strength, bonding between aggregate and cement paste, and transportability for water and ions in the pore water. Because of its thickness (around 50 µm) and porosity, a large fraction of the cement matrix belongs to it, fact which affects the bonding between aggregate and the cement paste.

An investigation into the reasons for the existence of the interfacial transition zone has revealed that a phenomenon called syneresis is responsible for the formation of a water-rich layer around the aggregate, which weakens the aggregate-paste bond. Syneresis takes place when a system undergoes rapid flocculation and the result is a loose flock in which most particles tend to be linked to two or three other particles. The structure of these flocks is tenuous and contains a substantial amount of entrapped water. The formation of a great number of contacts leads to a contraction of the dispersed phase, and when its volume decreases the water is spontaneously expelled from the flocks. Bijen and Rooij (1999) have also discovered that the addition of fly ash and silica fume in concrete decreased the thickness of the interfacial transition zone due to the pozzolanic reaction mechanisms.

Detwiler (1988) et al. examined the orientation of calcium hydroxide crystals in the interfacial zone and discovered that the crystals of calcium hydroxide in this zone were strictly oriented. They have seen that the axes of the crystals were perpendicular to the aggregate surface and the degree of such orientation grew currently with time, without being dependent on the sizes and types of the aggregate particles.

Research regarding the influence of the surface condition of aggregate on the cement-aggregate bond and compressive strength of concrete was carried out by Perry and Gillott (1977). They utilized ground glass balls with diameters of 15 mm and various degrees of
roughness as aggregate particles. The research have demonstrated that the gradually increased roughness degree of the aggregate particles constrained the process of formation and propagation of microcracks at the interfacial transition zone, and thus promoted improvement of the compressive strength and cement-aggregate bond of the concrete.

However, it was seen that, in general, the strength of concrete made with aggregate carefully selected was governed by the strength either of the cement paste or of the bond between the paste and the aggregate particles.

2.12 Examples of Self-Compacting Concrete Applications

Since the development of the prototype of self-compacting concrete in 1988, the use of this type of concrete in actual structures has gradually increased. Due to its special properties, self-compacting concrete has been chosen to partially replace the conventional concrete in a few construction projects of major importance, in Japan and Canada. The following are some examples of construction applications, which used self-compacting concrete.

The Bankers Hall project, which was one of the largest commercial office projects in Calgary, Western Canada, involved the placement of self-compacting concrete in two mat foundations with congested reinforcement (Nmai and Violetta, 1996). The amount of concrete used was approximately 9000 m$^3$ and the mixture was proportioned so that it would have very good flowing characteristics in order to satisfy the pumping and placement requirements, because of the intricate reinforcement.

A very important application of self-compacting concrete was the two anchorages of Akashi-Kaikyo (Straits) Bridge opened in April 1998 in Japan (Ouchi and Hibino, 2000), a suspension bridge with the longest span in the world, approx. 1,991 meters (Figure 2.7). The volume of the cast concrete in the two anchorages was around 290,000 m$^3$. A new construction
system, which made full use of the performance of self-compacting concrete, was introduced for this. The concrete was mixed at the batching plant beside the site and was pumped out of the plant. It was transported 200 meters through pipes to the casting site, where the pipes were arranged in rows of 3 to 5 meters apart. The concrete was cast from gate valves located at 5-meter intervals along the pipes. These valves were automatically controlled so that a surface level of the cast concrete could be maintained. In the final analysis, the use of self-compacting concrete shortened the anchorage construction period by 20%, from 2.5 to 2 years.

Figure 2.7 Anchorage of Akashi-Kaikyo Bridge, Japan (Ouchi and Hibino, 2000).

Self-compacting concrete was also used with success for the wall of a large Liquid Nitrogen Gas (LNG) tank belonging to the Osaka Gas Company, Japan, whose concrete casting was completed in June 1998 (Ouchi and Hibino, 2000). In this case, the volume of the concrete used in the tank amounted 12,000 m³ and was transported from the batching plant using ready-mixed trucks.
A new developed construction system used for casting concrete is the so-called sandwich structure, where concrete is filled into a steel shell (Ozawa, 1989). This sort of structure has already been completed in Kobe, Japan, which could not have been achieved without the development of self-compacting concrete (Figure 2.8).

Figure 2.8  Sandwich structure applied to immersed tunnel in Kobe, Japan (Ozawa, 1989).

Nowadays, self-compacting concrete applications are limited to special cases where it is impossible to use ordinary concretes. In these cases, the quality control relies on several different non-standard, and mostly not fully applicable, tests supplemented by a significant personal expertise of specialist suppliers or contractors. Due to this fact, special measures must be taken in order for the self-compacting concrete to be considered a standard concrete.
CHAPTER 3. SELF-COMPACTING CONCRETE COMPOSITION

Self-compacting concrete (SCC) is a fluid mixture, which is suitable for placing in difficult conditions and in structures with congested reinforcement, without vibration (Bartos, 2000). In principle, a self-compacting or self-consolidating concrete must:

- Have a fluidity that allows self-compaction without external energy,
- Remain homogeneous in a form during and after the placing process, and
- Flow easily through reinforcement

The technology of SCC is based on adding or partially replacing Portland cement with amounts of fine material such as fly ash, blast furnace slag, and silica fume without modifying the water content compared to common concrete. This process changes the rheological behavior of the concrete (Johansen and Hammer, 2002).

Generally, SCC has to have a proper flowability and viscosity, so that the coarse aggregate can float in the mortar without segregating. To achieve a balance between flowability and stability, the total content of particles finer than the 150 µm has to be high, usually about 520 to 560 kg/m$^3$ (880 to 950 lb/yd$^3$) (Kosmatka et al., 2002). Self-compacting concrete is very sensitive to fluctuation in water content; therefore, stabilizers or viscosity-modifying agents such as polysaccharides are used. Figure 3.1 shows an example of mix proportions used in SCC as compared to a regular concrete mix.

In Japan, self-compacting concretes are divided into three different types according to the composition of the mortar:

- Powder type
- Viscosity-modifying agent (stabilizer) type
- Combination type
For the powder type, a high proportion of fines produces the necessary mortar volume, whilst in the stabilizer type, the fines content can be in the range admissible for vibrated concrete. The viscosity required to inhibit segregation will then be adjusted by using a stabilizer (Kosmatka et al., 2002). The combination type is created by adding a small amount of stabilizer to the powder type to balance the moisture fluctuations in the manufacturing process.

However, after completion of proper proportioning, mixing, placing, curing, and consolidation, hardened concrete becomes a strong, durable, and practically impermeable building material that requires no maintenance.

### 3.1 Portland Cement

Portland cement concrete is foremost among the construction materials used in civil engineering projects around the world. The reasons for its often use are varied, but among the more important are the economic and widespread availability of its constituents, its versatility and adaptability, as evidenced by the many types of construction in which it is used, and the minimal maintenance requirements during service (Mindess et al., 2003). Concrete is unique
among major construction materials in that it is generally designed specifically for a particular project using locally available materials (Lay, 1990).

Regarding its composition, concrete is considered a composite material made of coarse granular material (the aggregate or filler) embedded in a hard matrix of material (the cement or binder) that fills the space between the aggregate particles and glues them together (Mindess et al., 2003). Aggregates can be obtained from many different kinds of materials, but the most used materials from the nature are common rocks.

There is a wide variety of cements that are used to some extent in the construction and building industries, or to solve special engineering problems (Bentz, 2001). The chemical compositions of these cements can be quite diverse, but by far the greatest amount of concrete used today is made with Portland cements (Atkins, 2003). In principle, the manufacture of Portland cement is very simple and relies on the use of abundant raw materials. An intimate mixture, usually of limestone and clay, is heated in a kiln to 1400 to 1600°C (2550 to 2900°F), which is the temperature range in which the two materials interact chemically to form the calcium silicates. High-quality cements require raw materials of adequate purity and uniform composition.

Limestone (calcium carbonate) is the most common source of calcium oxide, although other forms of calcium carbonate, such as chalk, shell deposits, and calcareous muds, are used (Mindess et al., 2003). Usually, iron-bearing alumino-silicates are invariably used as the primary source of silica, but clays or silts are preferred since they are already in a finely divided state.

A typical chemical composition of an ordinary Portland cement is given in Table 3.1. It can be noted that the quantities do not add up to 100%, the missing percentages being accounted for by impurities.
When Portland cement is mixed with water, its constituent compounds undergo a series of chemical reactions that are responsible for the eventual hardening of concrete. Reactions with water are designated hydration, and the new solids formed on hydration are collectively referred to as hydration products.

Figure 3.2 shows schematically the sequence of structure formation as hydration proceeds. This involves the replacement of water that separates individual cement grains in the fluid paste (Figure 3.2a) with solid hydration products that form a continuous matrix and bind the residual cement grains together over a period of time, as illustrated in Figure 3.2(b-d).

The calcium silicates provide most of the strength developed by Portland cement. $C_3S$ provides most of the early strength (in the first three to four weeks) and both $C_3S$ and $C_2S$ contribute equally to ultimate strength (Neville, 1993). In commercial cements, the calcium silicates contain small impurities of some oxides present in the clinker, which have a strong effect on the properties of the calcium silicate hydrate (C-S-H). Calcium silicate hydrate is essentially amorphous and develops as a mass of extremely small irregular particles of indefinite morphology. The particles are so small that they can be studied only by scanning electron microscope, and even cannot be completely resolved (Mindess et al., 2003).

<table>
<thead>
<tr>
<th>Chemical Name</th>
<th>Chemical Formula</th>
<th>Shorthand Notation</th>
<th>Weight Percent</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tricalcium silicate</td>
<td>$3CaO \cdot SiO_2$</td>
<td>$C_3S$</td>
<td>55</td>
</tr>
<tr>
<td>Dicalcium silicate</td>
<td>$2CaO \cdot SiO_2$</td>
<td>$C_2S$</td>
<td>18</td>
</tr>
<tr>
<td>Tricalcium aluminate</td>
<td>$3CaO \cdot Al_2O_3$</td>
<td>$C_3A$</td>
<td>10</td>
</tr>
<tr>
<td>Tetracalcium aluminoferrite</td>
<td>$4CaO \cdot Al_2O_3 \cdot Fe_2O_3$</td>
<td>$C_4AF$</td>
<td>8</td>
</tr>
<tr>
<td>Calcium sulfate dihydrate (gypsum)</td>
<td>$CaSO_4 \cdot 2H_2O$</td>
<td>$CSH_2$</td>
<td>6</td>
</tr>
</tbody>
</table>
The hydration reactions of the two calcium silicates are very similar, differing only in the amount of calcium hydroxide formed as seen in the following equations (Mindess et al., 2003).

\[
2C_3S + 11H \rightarrow C_3S_2H_8 + 3CH \quad (3.1)
\]

\[
2C_2S + 9H \rightarrow C_3S_2H_8 + CH \quad (3.2)
\]

Figure 3.2  Microstructural development in Portland cement pastes (Mindess et al., 2003).
C-S-H or C₃S₂H₈ is called calcium silicate hydrate and is the principal hydration product. The formula C₃S₂H₈ is only approximate because the composition of this hydrate is actually variable over quite a wide range.

In Portland cement, the hydration of tricalcium aluminate - C₃A - involves reactions with sulfate ions that are supplied by the dissolution of gypsum, which is added to temper the strong initial reaction of C₃A with water that can lead to flash set. The primary initial reaction of C₃A is as follows:

\[
\text{C}_3\text{A} + 3\text{CSH}_2 + 26\text{H} \rightarrow \text{C}_3\text{A-S}_3\text{H}_{32}
\]

where \( \text{S} \) is equivalent to SO₃ and ettringite is a stable hydration product only while there is an ample supply of sulfate available.

The ferrite phase – C₄AF - forms similar hydration products to C₃A, with or without gypsum, but the reactions are slower and involve less heat. C₄AF seldom hydrates rapidly enough to cause flash set and gypsum retards C₄AF hydration even more drastically than it does C₃A. Usually, changes in the composition of the ferrite phase affect only the rate of hydration and when the iron content is raised, hydration becomes slower.

### 3.2 Aggregates

Generally, aggregates occupy 70% to 80% of the volume of concrete and have an important influence on its properties. They are granular materials, derived for the most part from natural rock (crushed stone, or natural gravels) and sands, although synthetic materials such as slags and expanded clay or shale are used to some extent, mostly in lightweight concretes (Mindess et al., 2003). In addition to their use as economical filler, aggregates generally provide concrete with better dimensional stability and wear resistance. Although aggregate strength can
play sometimes an important role, for example in high-strength concretes, for most applications the strength of concrete and mix design are essentially independent of the composition of aggregates. However, in other instances, a certain kind of rock may be required to attain certain concrete properties, e.g., high density or low coefficient of thermal expansion (Neville, 1993).

In order to obtain a good concrete quality, aggregates should be hard and strong, free of undesirable impurities, and chemically stable (Garber and Hoel, 1988). Soft and porous rock can limit strength and wear resistance, and sometimes it may also break down during mixing and adversely affect workability by increasing the amount of fines. Rocks that tend to fracture easily along specific planes can also limit strength and wear resistance (Neville, 1993). Aggregates should also be free of impurities like silt, clay, dirt, or organic matter. If these materials coat the surfaces of the aggregate, they will isolate the aggregate particles from the surrounding concrete, causing a reduction in strength. Silt, clay and other fine materials will increase the water requirements of the concrete, and the organic matter may interfere with the cement hydration.

3.3 Blast Furnace Slag

Slags are by-products from metallurgical processes, either from production of metals from ore or refinement of impure metals. They are derived from lime-based inorganic fluxes used to extract impurities from metals, which solidify on cooling (St John, 1998). The slags used in concrete come from the blast furnace production of iron from ore and not from metals, and are rich in lime, silica, and alumina.

Reactivities can vary widely from slag to slag and there is not a reliable quantitative relationship between slag composition and its reactivity (Kosmatka et al., 2002). ASTM C 989 classifies slag by its increasing level of reactivity as Grade 80,100, or 120. Blast furnace slag reacts slowly with water, so that it may take several months for a pure slag concrete to reach the
equivalent 28-day strength of a concrete made with Portland cement. This is thought to be due to the presence of impervious coatings of amorphous silica and alumina that form around slag particles early in the hydration process (St John, 1998). Slag needs to be activated by alkaline compounds, which can be either soluble sodium salts, like NaOH, Na$_2$CO$_3$, or NaSiO$_3$, or calcium hydroxide Ca(OH)$_2$, but they are most commonly activated by Portland cement, where Ca(OH)$_2$ formed during hydration is the principal activator. Although only 10-20% of cement is needed for activation, slag-cement blends usually contain much more cement. The slag, which is ground to less than 45 microns, has a surface area fineness of about 400 to 600 m$^2$/kg and its relative density is in the range of 2.85 to 2.95.

Ground granulated blast-furnace slag, when used in general purpose concrete in North America, commonly constitutes between 30% and 45% by mass of the cementing material in the mix (Mindess et al., 2003), even though some slag concretes have a slag component of 70% or more of the cementitious material. It usually decreases water demand by 1% to 10%, depending on dosage, and due to its smooth surface characteristics and fineness, it increases pumpability, workability and reduces bleeding of cast concrete. The use of slag will generally retard the setting time of concrete, fact, which is advantageous during hot weather, because it allows more time to place and finish the concrete, but has a negative effect on the early strength of it. However, the 28-day strength is sometimes improved but, because of the slow pozzolanic reaction, continuous wet curing and favorable temperatures may need to be provided for longer periods than normally required. Because additional calcium silicate hydrate forms when slag is added to the fresh concrete, its permeability will be reduced and the durability will be increased. Ground slag also improves the resistance of concrete to sulfate attack and significantly reduces alkali-silica reactivity.
3.4 Fly Ash

Fly ash is one of the most extensively used by-product materials in the construction field resembling Portland cement (Pfeifer, 1969). It is an inorganic, noncombustible, finely divided residue collected or precipitated from the exhaust gases of any industrial furnace (Halstead, 1986). Most of the fly ash particles are solid spheres and some particles, called cenospheres, are hollow (Figure 3.3) (Kosmatka et al., 2002). Also present are plerospheres, which are spheres containing smaller spheres inside. The particle sizes in fly ash vary from less than 1 µm to more than 100 µm with the typical particle size measuring under 20 µm. Their surface area is typically 300 to 500 m²/kg, although some fly ashes can have surface areas as low as 200 m²/kg and as high as 700 m²/kg. Fly ash is primarily silicate glass containing silica, alumina, iron, and calcium. The relative density or specific gravity of fly ash generally ranges between 1.9 and 2.8 and the color is generally gray or tan (Halstead, 1986).

![Figure 3.3 SEM micrograph of fly ash particles (Kosmatka et al., 2002).](image)

Fly ashes are subdivided into two classes, F and C, which reflect the composition of the inorganic fractions. Class F fly ashes are produced from bituminous and subbituminous coals and
contain as active components aluminosilicate glasses, whereas class C fly ashes derive from the lignitic coals and contain calcium aluminosilicate glasses with the high levels of calcium oxide, comprised in the glassy fraction (Mindess et al., 2003). Although their usage is mainly economic (fly ash is less than one-half the price of cement), the addition of fly ash has many technical benefits.

Many class C ashes when exposed to water will hydrate and harden in less than 45 minutes. In concretes, class F fly ash is often used at dosages of 15% to 25% by mass of cementitious material and class C fly ash is used at dosages of 15% to 40% (Halstead, 1986). Dosage varies with the reactivity of the ash and the desired effects on the concrete (Mindess et al., 2003). Because of their spherical morphology, when using fly ash admixtures as replacement for cement, workability and long-term strengths are achieved in concretes. In such cases, they act like small balls to reduce interparticle friction. Fly ashes are also used in concrete mixes in order to reduce the heat of hydration, permeability, and bleeding. The durability is improved by providing a better sulfate resistance, control of the alkali-silica reaction, decreased chloride diffusion, and reduction of leaching from the reduction in calcium hydroxide (which is the most soluble of the hydration products) and changes in pore structure. However, there are some disadvantages related to the use of fly ash regarding the reduced air-entraining ability and early strength due to the influence of residual carbon from the ash (Gebler and Klieger, 1986).

3.5 Silica Fume

Silicon, ferrosilicon and other silicon alloys are produced by reducing quartz, with coal and iron or other ores, at very high temperatures (2000°C) in electric arc furnaces (St John, 1998). Some silicon gas or fume is produced in the process, which reaches the top of the furnace with other combustion gases, where it becomes oxidized to silica in contact with the air and then
condenses as < 0.1 µm to 1 µm spherical particles of amorphous silica. This material is usually known as silica fume. It is also referred to as microsilica or more properly, condensed silica fume (csf). Silica fume is an ultra fine powder, with individual particle sizes between 50 and 100 times finer than cement, comprising solid spherical glassy particles of amorphous silica (85-96 percent SiO$_2$). However, the spherical particles are usually agglomerated so that the effective particle size is much coarser. Silica fume used in concrete derives from the manufacture of ferrosilicon alloys and is modified by densification, micropelletization or slurrification, to facilitate transportation and handling (St John, 1998). Condensed silica fume has a surface area of about 20,000 m$^2$/kg and a relative density generally in the range of 2.20 to 2.5 (Kosmatka et al., 2002).

The pozzolanic reactions take place when silica fume is added to the concrete mixture, and the amorphous silica, which is the major component of the pozzolan, reacts with calcium hydroxide formed from the hydration of the calcium silicates with the resulting product being a calcium silicate hydrate (C-S-H) (Mindess et al., 2003). Usually, after microsilica is being added to the concrete mix, the matrix of microsilica concrete becomes very dense (St John, 1998). This denseness effect has been attributed to the extreme fineness of microsilica, due to the fact that 50,000 to 100,000 microspheres exist for every cement grain, allowing microsilica hydration products to infill the water spaces usually left within the cement hydrates, as showed in Figure 3.4. Silica fume is used in amounts between 5% and 10% by mass of the total cementitious material, in applications where high degree of impermeability and high compressive strength are needed in concrete. In some situations, the water demand of concrete containing silica fume increases with increasing amounts of silica fume, unless a water reducer or plasticizer is used.
Some lean mixes may not experience an increase in water demand when only a small amount (less than 5%) of silica fume is present.

Microsilica is also very effective in reducing both bleeding and segregation. It may contribute to stickiness of a concrete mixture, adjustments, including the use of high-range water reducers, may be required to maintain workability and permit proper compaction and finishing.

The use of silica fume generally aids the pumpability of concrete and is most effective in lean mixtures. Because of its low bleeding characteristics, concrete containing silica fume may exhibit an increase in plastic shrinkage cracking (Mindess et al., 2003). The problem may be avoided by ensuring that such concrete is protected against drying, both during and after finishing. With proper proportioning and material selection, silica fume can improve the
durability of concrete, its resistance to sulfate attack, and the early strength development (Ramachandran, 1984).

### 3.6 Superplasticizers

Superplasticizers (high-range water-reducers) are low molecular-weight, water-soluble polymers designed to achieve high amounts of water reduction (12-30%) in concrete mixtures in order to attain a desired slump (Gagne et al., 2000). These admixtures are used frequently to produce high-strength concrete (> 50 MPa), since workable mixes with water-cement ratios well below 0.40 are possible (Whiting, 1979). They also can be used without water reduction to produce concretes with very high slumps, in the range of 150 to 250 mm (6 to 10 inches). At these high slumps, concrete flows like a liquid (Figure 3.5) and can fill forms efficiently, requiring very little vibration. These highly workable mixtures are called flowing concretes and require slumps to be in excess of 190 mm (8.5 inches).

![Figure 3.5](image)

Figure 3.5 Effect of superplasticizer on cement: (a) Cement and water; (b) Cement, water, and superplasticizer (Ramachandran, 1984).
Water-reducing admixtures are negatively charged organic molecules that adsorb primarily at the solid-water interface, whereas solid particles carry residual charges on their surfaces, which may be positive, negative, or both (Russell, 1983). In cement paste, opposing charges on adjacent particles of cement can exert considerable electrostatic attractions, causing the particles to flocculate (Figure 3.6a). A considerable amount of water is tied up in these agglomerates and adsorbed on the solid surfaces, leaving less water available to reduce the viscosity of the paste and hence that of the concrete. Molecules of the water-reducing admixtures interact to neutralize these surface charges and cause all surfaces to carry uniform charges of like sign (Mindess et al., 2003). Particles now repel each other, rather than attract, and remain fully dispersed in the paste (Figure 3.6b), thus, most of the water is available to reduce the viscosity of the paste and of the concrete. Because superplasticizers have air-detraining properties, an air-entraining agent must be added to the concrete to get a stable air void system before a superplasticizer is added (Gagne et al., 1996).

Figure 3.6 Dispersing action of water-reducing admixtures: (a) flocculated paste; (b) dispersed paste (Mindess et al., 2003).
Some high-range water-reducing admixtures can retard final set by one to almost four hours and if prolonged setting times are not convenient, the admixture can be combined with an accelerating admixture to counteract the retarding tendencies or even to provide some net acceleration of setting. When water-reducing admixtures are used in concrete mixtures, some increases in compressive strength can be anticipated and these increases can be observed in as early as one day if excessive retardation does not occur. It is generally agreed that increases in compressive strength are up to 25% greater than would be anticipated from the decrease in water-cement ratio alone. Probably, this reflects the development of a uniform microstructure when the cement is dispersed (Ozyildirim, 2003). The reduction of the water-cement ratio and the creation of a more uniform pore structure means that the permeability of concrete can be reduced by the use of superplasticizers, along with a general improvement of durability.

3.7 Viscosity-Modifying Admixtures

Viscosity modifiers are high molecular-weight, water-soluble polymers used to raise the viscosity of water. Such compounds increase the cohesiveness of fresh concrete, reducing its tendency to segregate and bleed (Ferraris, 1999). They work by attaching their long molecules to the water molecules, process which inhibits the free displacement of water. These admixtures are helpful in improving the properties of lean concretes with low cement contents, concrete placed under water, and concretes or grouts that are placed by pumping. In the latter case, they reduce pumping pressures through improved lubricating properties, as well as reducing segregation tendencies. When compounds in this category are used to improve the cohesiveness of concrete being placed underwater, they are classified as antiwashout admixtures. Viscosity-modifying admixtures are added in concretes used in places with extreme congestions due to reinforcement.
configurations or unusual geometry forms, where fluid but cohesive concrete is required in order
to resist bleeding and segregation (Dodson, 1990).

The materials commonly used are polyethylene oxides, cellulose ethers, natural gums,
and polyacrylamides or polyvinyl alcohol. Other materials used are finely divided solids such as
clays and lime, but they tend to reduce the strength of the concrete and for this reason are
primarily used in grouts when strength is not of major importance.
CHAPTER 4. SPLITTING TENSILE STRENGTH

The tensile strength of concrete is much lower than the compressive strength, because of the ease with which cracks can propagate under tensile loads, and is usually not considered in design (it is often assumed to be zero) (Neville, 1971). However, it is an important property, since cracking in concrete is most generally due to the tensile stresses that occur under load, or due to environmental changes. The failure of concrete in tension is governed by microcracking, associated particularly with the interfacial region between the aggregate particles and the cement, also called interfacial transition zone (ITZ) (See Chapter 5). The load applied (compressive force) on the cylindrical concrete specimen induces tensile and shear stresses on the aggregate particles inside the specimen, generating the bond failure between the aggregate particles and the cement paste, as shown in Figure 4.1.

![Figure 4.1 Stresses around an aggregate particle (Mindess et al., 2003).](image)
Investigations have shown that very fine cracks at the interfacial region exist even prior to application of the load on concrete, due to the shrinkage (Neville, 1993). These cracks remain stable up to about 30 percent or more of the ultimate load and then begin to increase in length, width, and number. The overall stress under which they develop is sensitive to the water-cement ratio of the paste, and this is considered the stage of slow crack propagation. At 70 to 90 percent of the ultimate strength, the cracks open through the cement paste and fine aggregate and they bridge until a continuous crack pattern is formed. This represents the fast crack propagation stage and, if the load is sustained, failure may take place with time.

Usually, splitting tensile strength test is used to evaluate the shear resistance provided by concrete elements. In North America, there is no standard test procedure to measure the direct tensile strength of concrete, although a number of specimens (none yet standard) are available to measure the fracture properties of concrete under tension. One major advantage of the splitting procedure as used to assess tensile strength lies primarily in the simplicity of the test procedure compared with the many difficulties raised by the pure tensile test. In addition, tensile tests can be carried out on the same specimens as those used to determine the compressive strength, hence the same molds and the same testing facilities. However, the most important advantage is that, when applying the splitting procedure, the tensile strengths are practically independent of either the test specimen or of the test machine sizes, being only a function of the concrete quality alone. Thus, much inconvenience is eliminated, particularly with respect to the scale coefficient, which is involved in direct tensile tests. For this reason, this procedure is considered to reproduce more exactly the real concrete tensile strength.

The tensile strength of concrete is most often evaluated using a split cylinder test, in which a cylindrical specimen is placed on its side and loaded in diametrical compression, so to
induce transverse tension. Practically, the load applied on the cylindrical concrete specimen induces tensile stresses on the plane containing the load and relatively high compressive stresses in the area immediately around it. When the cylinder is compressed by the two plane-parallel faceplates, situated at two diametrically opposite points on the cylinder surface then, along the diameter passing through the two points, the major tensile stresses are developed which, at their limit, reach the fracture strength value, $f_{cs}$:

$$f_{cs} = \frac{2F}{\pi dl},$$

where: $F$ is the fracture compression force acting along the cylinder generatrix, $d$ is the cylinder diameter; $l$ is the cylinder length. The load and stress distribution pattern across the cross section is shown in Fig. 4.2 ($f_{cs}$ is the concrete strength).

![Diagram](image)

Figure 4.2 Splitting tensile stresses on concrete cylindrical specimens (Avram et al., 1981).

If it is assumed that the load is concentrated at the tangent points then, over the cross section, only tensile stresses would be developed. In practice, however, the load is distributed over a finite width owing to material deformations. So, over the cross section, horizontal compressive stresses are developed too, in the close vicinity of the contact point between the press platens and the material. Since the compressive stresses only develop to a small depth in the cross section, it may be assumed that the tensile stresses are distributed evenly along the diameter where the splitting takes place. Fernando Carneiro, from Brazil, worked on this
principle and devised a complete practical procedure for the determination of concrete strength through tensile stresses (Neville, 1993). This procedure was then adopted (in 1947) by the national standard specifications of Brazil and is known now as "the Brazilian test".

Since, along the loaded generatrix, high compressive stresses are developed, cardboard, plywood or leather strips a few millimeters thick are laid between the press platens and the concrete cylinder, to achieve the load distribution. As a tensile crack propagates through concrete, its leading edge often consists of multiple branching microcracks that eventually coalesce into a single macrocrack as the tensile displacement increases (Neville, 1993). Depending on the relative tensile strength of the mortar, the aggregate, and the ITZ, the macroscopic appearance of the failure surface may be smooth or rough. A low ratio of aggregate to ITZ strength will result in a relatively smooth failure surface, but as the relative strength of the aggregate increases, the failure surface becomes progressively more uneven, usually resulting in a higher tensile strength and improved fracture properties. Because compressive strength is the principal material property that is measured for hardened concrete, the relationship between tensile and compressive strength is of particular interest.

In general, as age and strength increase, the ratio of tensile to compressive strength ($f_t'/f_c'$) decreases (Mindess et al., 2003). Also, since crushed coarse aggregate seems to improve tensile strength more than it does compressive strength, the $f_t'/f_c'$ ratio also depends on the type of aggregate (Figure 4.3). It has been found that, compared to moist curing, air curing reduces the tensile strength more than it does compressive strength, probably because of the effect of drying shrinkage cracks. However, incomplete consolidation and air entrainment affect the compressive strength more than they do the tensile strength. Usually, the ratio of splitting tensile strength to compressive strength ranges from about 0.06 to 0.20 (Avram et al., 1981).
Regarding the direct tensile strength, no relationship between the values of tensile strength from the splitting test and those measured in direct tension has been found yet. While it is commonly assumed that splitting tensile strength values are 5 to 12% higher than direct tensile strength values, this is not always true (Mindess et al., 2003). Some recent studies have shown very little difference between the two values of tensile strength, and for some concretes, the direct tensile strengths have been found to be slightly higher than the splitting strengths. It may appear that the ratio of direct tensile strength to splitting tensile strength depends upon the strength level of the concrete and upon the maximum aggregate size. On the other hand, some recent analyses of the splitting tension test have shown that the splitting tensile strength determined according to ASTM C 496 test procedures should not be considered to be a "true" material property, because the results obtained depend upon the specimen size and the width and type of the load bearing strips (Avram et al., 1981).
CHAPTER 5. AGGREGATE-CEMENT BONDING CHARACTERISTICS

Bonding between aggregate and cement paste is an important factor in the strength of concrete, especially the tensile strength, and regarding the fracture properties of concrete. Bond is due, in part, to the interlocking of the aggregate and the paste owing to the roughness of the surface of the former (Neville, 1993). A rougher surface, such as that of crushed particles, results in a better bond, usually obtained with softer, porous, and mineralogical heterogeneous particles. Generally, texture characteristics, which permit no penetration of the surface of the particles, are not conducive to good bond. In addition, bond is affected by other physical and chemical properties of aggregate, related to its mineralogical and chemical composition (Bijen and Rooij, 1999). So, aggregate shape, surface structure and hardness are all factors affecting the strength of the aggregate-matrix bond. However, today little is known about these phenomena, and relying on experience is still necessary in predicting the bond between the aggregate and the surrounding cement paste.

The determination of the quality of bond of aggregate is rather difficult and no accepted tests exist. Generally, when bond is good, a crushed concrete specimen should contain some aggregate particles broken right through, in addition to the more numerous ones pulled out from their cavities. An excess of fractured particles, however, might suggest that the aggregate is too weak (Neville, 1993). Because it depends on the paste strength, as well as on the properties of aggregate surface, bond strength increases with the age of concrete. Thus, providing it is adequate, the bond strength may not be a controlling factor in the strength of concrete. Most often, concrete fracture occurs according to a pattern, which follows the contact surface zone representing the weakest link. This very thin zone surrounding the aggregate consists of a matrix layer and an adjacent aggregate layer, the two layers being separated by a contact surface or
aggregate-matrix interface (physical interface) as shown in Figure 5.1. Between the two phases, i.e. the matrix and the aggregate, physical forces and interactions may exist, generated by the adhesive and interlocking forces, as well as by matrix-aggregate interpenetration subsequent to cement-paste shrinkage.

![Aggregate-matrix interface](image)

Figure 5.1 Aggregate-matrix interface (Avram et al., 1981).

Another kind of interaction force between aggregate and matrix is that of chemical nature. Here, layers of a certain thickness are affected in the aggregate and the matrix and the forces developed are internal cohesive forces inside these layers, due to chemical reactions (Subramanian, 1999). Owing to the new products formed by reactions which take place at the aggregate surface, the interface between aggregate and matrix becomes diffuse and the reaction products are generated on the surface of aggregate crystals. As a rule, forces developed in the aggregate-matrix contact region are both physical (adhesion and interlocking) and chemical (due to reaction products and to the epitaxial growth) in nature.

Physical forces, which predominate in inert aggregates, essentially depend on aggregate topography and roughness (Avram et al., 1981). Thus, concrete prepared with polished aggregates such as feldspar or mica will always break at the aggregate-matrix interface, however strong the matrix is made. In those areas where the maximum value of the force that the aggregate surface can stand is exceeded, microcracks will start to develop. The fine voids
developed over the aggregate surface represent structural breaks in the continuity and are, at the same time, an opportunity for the accumulation of a liquid interface phase. If concrete is submitted to freezing-thawing actions or to aggressive chemical agents, the liquid phase changes its volume, fact which leads to an additional stress on the aggregate-matrix interface and hence to microcracks. Studies showed that here an important part is played by the type of cement used (Neville, 1993). Cements, which subsequent to hydration generate idiomorphic crystals in the contact region, are less resistant to aggressive actions than cements which produce a gel-like mass (Figure 5.2), wherein a fine crystallization process is initiated.

![Figure 5.2 Interfacial region between aggregate and cement paste (Mindess et al., 2003).](image)

Because, most aggregates show certain physical and chemical interaction with the cement matrix, they can be divided into two classes (Avram et al., 1981):

* Those producing a strong contact layer on the matrix surface while the aggregate surface is left practically unchanged and,

* Those producing weaker contact layers both on the aggregate and on the matrix surfaces.
The first class includes acid siliceous rocks such as quartzite and feldspars, whereas the second includes carbonates and basic rocks such as calcite, limestone, marble, dolomite, etc. The explanation of this phenomenon is supplied by the fact that, in the presence of cement, the chemical reactions of the two kinds of rocks are different. As already known, at common temperature all siliceous materials can react with calcium hydroxide, which in the case of cement results from hydration reactions, thus forming hydrated calcium silicate (C-S-H). Thus, the combination reaction of a silicon dioxide layer from the aggregate surface and the production of a compound having higher binding performances are responsible for the higher hardness of the cement layer, even though this is an extremely slow reaction. However, this slow reaction is the reason why, when under load, early concrete is more prone to microcracking than the older ones, such a process being initiated at the matrix-aggregate interface (Neville, 1993). There are also cases when aggregates prove to be inactive and this is due to a layer of crystallized calcium hydroxide deposited over the aggregate surface, thus preventing good cohesion between matrix and aggregate, fact which leads to a weaker bonding between the two materials (Subramanian, 1999).

Carbonates and basic rocks react, too, with cement hydration products, but the layers that build up on the aggregate and matrix surfaces are characterized by a lower hardness. Another important factor that affects the bond strength is the aggregate size (Neville, 1993). It appears that the higher the aggregate size, the lower is its specific surface and, hence, the lower is the bond strength over the aggregate-cement interface. This phenomenon explains why the concrete cracks and microcracks follow the surface of large aggregates. It has also been observed that the aggregate-cement bonding was greatly influenced by the compaction conditions of the concrete, that is, a better compaction always led to a stronger bond (Kosmatka et al., 2002). The matrix
contact layer (Figure 5.1) adjacent to the aggregate-cement interface (physical interface) and represented by a thin zone, typically 20-50 µm, surrounding the aggregate particles, is referred to as the interfacial transition zone (ITZ) (Bijen and Rooij, 1999). In this zone, the structure of the cement paste is quite different from that of the bulk paste in terms of morphology, composition and density. It does not necessarily have a uniform thickness and may be entirely absent at some parts of the surface. Practically, the structure of the ITZ is different from that of the bulk paste in the following respects (Mindess et al., 2003):

* There is less unhydrated cement.
* There is a higher porosity, a lower density and the pores are generally larger than those found in the bulk paste.
* There is less calcium silicate hydrate (C-S-H).
* There are large, oriented crystals of calcium hydroxide (C-H).
* There is generally a greater concentration of ettringite.

Although the ITZ itself is quite narrow, it occupies a relatively large proportion of the cement paste. If assuming a 40-µm thickness for the ITZ, it has been estimated that this makes up 20-40% of the total volume of the cementitious matrix (Mindess et al., 2003). However, it is generally found that, as the paste-aggregate bond strength increases, the concrete strength also increases, whether in tension, flexure, or compression. These increases range from about 5 to 40%, with improvements in tensile strength being higher than those in compressive strength (Subramanian, 1999).

In normal concretes, the ITZ has less crack resistance than either the aggregate or the hydrated cement paste, and so fracture occurs preferentially in the ITZ (Neville, 1993). If concretes with higher strengths have to be produced, it is necessary to increase the strengths both
of the paste matrix and of the paste-aggregate bond. Unfortunately, reducing the water-cement ratio has a much greater effect on the matrix strength than it does on the strength of the paste-aggregate bond. Thus, the focus has been on improving the strength of the ITZ itself, since fracture usually occurs not right at the physical interface, but about 10-20 µm into the ITZ. In this respect, the most effective way of improving the ITZ is by the addition of chemical and mineral admixtures. Currently, the use of silica fume and fly ash is the most effective way of densifying the ITZ, because they eliminate many of the larger pores in this zone, making its structure more homogeneous (St John, 1998). They also eliminate the growth of calcium hydroxide or transform the calcium hydroxide into C-S-H by the pozzolanic reaction. However, other techniques are now being examined, such as using chemical reagents, like for example, surfactants or water glass in the concrete mix (Mindess et al., 2003). It is considered that such treatments may provide a more economical means of achieving better bonding.

In general, for ordinary concretes, improvements in the ITZ or in the bond strength are unlikely to lead to major changes in concrete behavior (Bijen and Rooij, 1999). The modest increases in strength attributable to better paste-aggregate bonding are largely offset by the increasing brittleness of the resulting materials. However, for high performance concretes, such as self-compacting, high-strength, or high-durability concretes, very good bond strengths resulting from improvements in the properties of the ITZ appear to be more important if the full potential of these special materials is to be attained.
CHAPTER 6. EXPERIMENTAL PROCEDURES

Conventional concrete tends to have a difficulty regarding the adequate placing and consolidation in thin sections or areas of congested reinforcement, which leads to a large volume of entrapped air voids and compromises the strength and durability of the concrete. Using self-compacting concrete (SCC) can eliminate the problem, since it was designed to consolidate under its own weight. Therefore, it is important to verify the mechanical properties of SCC before using it for practical applications.

This research was conducted to find out if self-compacting concrete would show an increase in splitting tensile strength and compressive strength and a better bonding between aggregate and cement paste, in order be used as a replacement for conventional concrete in the construction industry. The experimental program was divided into two phases.

In the first phase eleven sets of cylindrical specimens were made, each containing six cylinders, in order to be tested for compressive and splitting tensile strength after 28 days of standard curing. The water-cement ratios were 0.3, 0.4, 0.45, 0.5, and 0.6. Three normal and three self-compacting concrete specimens were tested for compressive and splitting tensile strength, for each of the five water-cement ratios. In addition, another three normal concrete and three self-compacting concrete specimens were made, with the water-cement ratio of 0.4, in order to be tested only for splitting tensile strength after 7 days of standard curing. All the cast cylinders had 4 in. (101.6 mm) in diameter and 8 in. (203.2 mm) in length and the total number of cast specimens was 66. However, before the actual batching and testing started, a few preliminary mixes were batched and four specimens were cast and tested after three days of curing. These specimens were tested to see if they would yield reasonable results and to ensure that the research was valid and the continuation of any further research was feasible. Also, for
self-compacting concrete, slump flow and U-type tests were carried out (see Chapter 1) in order to evaluate the filling ability and the self-compactability of the concrete.

In the second phase, the physical interface between aggregate and cement paste has been studied. Small samples of concrete with the diameter of approximately 25 mm and thickness of about 4 mm have been cut, in order to be analyzed under Scanning Electron Microscope. The samples were previously polished and then coated with a very thin layer of gold to prevent charge built-up on the surface. They were taken from both types of concrete specimens having 0.3, 0.4, and 0.6 water-cement ratios. In addition, another 6 samples with the dimensions of 70x70x12 mm have been cut from concrete specimens having 0.3, 0.45, and 0.6 water-cement ratios and analyzed under a normal stereo-zoom microscope, in order to compare the number of air voids in the normal and self-compacting concrete.

The materials used in the research were comprised of:

- Portland cement type I
- Water
- River gravel
- Natural sand
- Mineral admixtures: fly ash, blast furnace slag, silica fume
- Chemical admixtures: superplasticizer, viscosity modifying agent

All the materials have been stored in the concrete laboratory in closed containers or bags to ensure that the conditions were kept constant throughout the research period.
6.1 Characteristics of Materials

**Portland cement:** ASTM type I Portland cement is a general-purpose cement suitable for all uses where the special properties of other types are not required. Its uses in concrete include pavements, floors, reinforced concrete buildings, etc. and it has a relative density of 3.15.

**Water:** The water used in the mix design was potable water from the water-supply network system; so, it was free from suspended solids and organic materials, which might have affected the properties of the fresh and hardened concrete.

**River gravel:** The coarse aggregate used in the concrete mixtures was river gravel, taken from the southeastern part of Louisiana and having the maximum size of 19.5 mm. Its absorption value was 1.9% and was determined according to ASTM C 127 “Standard Test Method for Specific Gravity and Absorption of Coarse Aggregate”. Also, sieve analysis was performed on the coarse aggregate according to ASTM C 136 “Standard Test Method for Sieve Analysis of Fine and Coarse Aggregate”. The results presented in Table 6.1 were within the limits of ASTM C 33 “Standard Specification for Concrete Aggregates”.

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Weight Retained (lbs)</th>
<th>Amount Retained (%)</th>
<th>Amount Passed (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>¾” (19mm)</td>
<td>1.4</td>
<td>6.4</td>
<td>93.6</td>
</tr>
<tr>
<td>½” (12.5mm)</td>
<td>10.4</td>
<td>47.3</td>
<td>46.3</td>
</tr>
<tr>
<td>3/8” (9.5mm)</td>
<td>2.3</td>
<td>10.45</td>
<td>85.8</td>
</tr>
<tr>
<td>No. 4 (4.75mm)</td>
<td>6.2</td>
<td>28.2</td>
<td>71.8</td>
</tr>
<tr>
<td>No. 8 (2.36mm)</td>
<td>1.67</td>
<td>7.3</td>
<td>92.7</td>
</tr>
<tr>
<td>Pan</td>
<td>0.03</td>
<td>0.1</td>
<td>99.9</td>
</tr>
</tbody>
</table>

**Natural sand:** The natural sand that has been used to cast the concrete cylinders was very clean and had the maximum size of 1 mm. It was also brought from the south part of Louisiana, which has quite big deposits of different types of sand. Its absorption value was 1.12% and was
determined according to ASTM C 128 “Standard Test Method for Specific Gravity and Absorption of Fine Aggregate”.

**Fly ash**: The experiment included the use of Class C fly ash that complied with the requirements of ASTM C 618 and had the relative density of 2.5.

**Blast furnace slag**: Ground granulated iron blast furnace slag Grade 100 was added in the concrete mixture. Grade 100 indicates that the slag has a moderate activity regarding its interaction with the calcium hydroxide Ca(OH)$_2$ from cement. The slag complied with the requirements of ASTM C 989 and had a specific gravity of 2.89.

**Silica fume**: The silica fume used in the batching process was in the powder form and was complying with the requirements of ASTM C 1240. It contained an average of 95.7% of SiO$_2$ and had the specific gravity of 2.3.

**Superplasticizer**: Rheobuild 3000FC ready-to-use high range water-reducing admixture was the superplasticizer added to the concrete mixture. This type of superplasticizer can be used to produce rheoplastic concrete. Rheoplastic concrete flows easily, maintaining workability for long periods of time without any affect on setting time. Rheobuild 3000FC admixture meets ASTM C494 requirements for Type F, high-range water-reducing admixture and has a recommended dosage range of 4 to 15 ml/1kg of cementitious material for most concrete mixes. Sometimes, function of water-cement ratios, dosages outside the recommended range may be required.

**Viscosity-modifying admixture**: The viscosity modifying admixture used in the study was Rheomac UW 450 anti-washout admixture, a patented ready-to-use, liquid, cellulose-based admixture that was specially developed for underwater concrete applications. Concrete containing Rheomac UW 450 admixture exhibits superior resistance to washout of cement and
fines and is recommended for use at a dosage range of 20 to 130 ml/1 kg of cementitious material. For low water-cement ratios dosages may be decreased.

6.2 Self-compacting Concrete Mix Design

The self-compacting concrete mix design used in the study was based on previous work done in Japan, US, Canada. All the mixes were prepared in 100 lbs batches (for 6 specimens - 4 by 8 inches cylinders and U-type test) using an electrical mixer. The mix proportions for casting the concrete specimens are given in Table 6.2.

The type I Portland cement was replaced by blast furnace slag (25%), fly ash (15%), and silica fume (5%). The water-cement ratios have been varied from 0.3 to 0.6 while the rest of the components was kept the same, except the chemical admixtures, which were adjusted for obtaining the self-compactability of the concrete.

Table 6.2  Self-compacting concrete mix design.

<table>
<thead>
<tr>
<th>Water/Cement ratio</th>
<th>0.3</th>
<th>0.4</th>
<th>0.45</th>
<th>0.5</th>
<th>0.6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water (lbs)</td>
<td>6.6</td>
<td>8.8</td>
<td>9.9</td>
<td>11</td>
<td>13.2</td>
</tr>
<tr>
<td>Cement (lbs)</td>
<td>12.1</td>
<td>12.1</td>
<td>12.1</td>
<td>12.1</td>
<td>12.1</td>
</tr>
<tr>
<td>Slag Cement (lbs)</td>
<td>5.5</td>
<td>5.5</td>
<td>5.5</td>
<td>5.5</td>
<td>5.5</td>
</tr>
<tr>
<td>Fly Ash (lbs)</td>
<td>3.3</td>
<td>3.3</td>
<td>3.3</td>
<td>3.3</td>
<td>3.3</td>
</tr>
<tr>
<td>Silica Fume (lbs)</td>
<td>1.1</td>
<td>1.1</td>
<td>1.1</td>
<td>1.1</td>
<td>1.1</td>
</tr>
<tr>
<td>Fine Agreg.(lbs)</td>
<td>26</td>
<td>26</td>
<td>26</td>
<td>26</td>
<td>26</td>
</tr>
<tr>
<td>Coarse Agreg.(lbs)</td>
<td>46.4</td>
<td>46.4</td>
<td>46.4</td>
<td>46.4</td>
<td>46.4</td>
</tr>
<tr>
<td>HRWR (ml)</td>
<td>340</td>
<td>100</td>
<td>80</td>
<td>50</td>
<td>20</td>
</tr>
<tr>
<td>VMA (ml)</td>
<td>0</td>
<td>15</td>
<td>25</td>
<td>50</td>
<td>100</td>
</tr>
</tbody>
</table>

6.3 Normal Concrete Mix Design

Normal concrete mixes were prepared in 62 lbs batches (for approx. 6 specimens - 4 by 8 inches cylinders) using the electrical mixer. The mix proportions for casting the concrete specimens are given in Table 6.3.
Table 6.3 Normal concrete mix design.

<table>
<thead>
<tr>
<th>Water/Cement ratio</th>
<th>0.3</th>
<th>0.4</th>
<th>0.45</th>
<th>0.5</th>
<th>0.6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water (lbs)</td>
<td>4.1</td>
<td>5.5</td>
<td>6.2</td>
<td>6.8</td>
<td>8.2</td>
</tr>
<tr>
<td>Cement (lbs)</td>
<td>13.6</td>
<td>13.6</td>
<td>13.6</td>
<td>13.6</td>
<td>13.6</td>
</tr>
<tr>
<td>Fine Agreg.(lbs)</td>
<td>16.1</td>
<td>16.1</td>
<td>16.1</td>
<td>16.1</td>
<td>16.1</td>
</tr>
<tr>
<td>Coarse Agreg.(lbs)</td>
<td>28.2</td>
<td>28.2</td>
<td>28.2</td>
<td>28.2</td>
<td>28.2</td>
</tr>
</tbody>
</table>

6.4 Batching Procedure

A total of 11 (eleven) batches based on the above mix designs have been prepared in this research. The procedure used for the batches was as follows:

* predetermined quantities of fine and coarse aggregate were added to the mixer and mixed for 30 seconds;

* predetermined quantities of cement, fly ash, slag cement and silica fume were added to the mixer and mixed together with the aggregates for 1 minute;

* various amounts of water, superplasticizer and viscosity admixture were added and mixed thoroughly;

* different mixtures obtained were used to carry out the slump flow test, the U-type test, and to cast cylindrical specimens.

No vibration or compaction has been applied to the self-compacting concrete specimens, whereas compaction on normal concrete specimens was applied, for approximately 30 seconds, using a tamping rod. All concrete specimens have been cast and cured according to ASTM C 192-95 “Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory”.

Compressive and splitting tensile strengths of concretes were performed according to ASTM C 39-94 “Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens” and ASTM C 496-96 “Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens”, respectively.
CHAPTER 7. RESULTS AND DISCUSSION

7.1 Slump Flow Test

The consistency and workability of self-compacting concrete were evaluated using the slump flow test. Because of its ease of operation and portability, the slump flow test is the most widely used method for evaluating concrete consistency in the laboratory and at construction sites. In this study, the diameter of the concrete flowing out of the slump cone was obtained by calculating the average of two perpendicularly measured diameters for determining the above mentioned properties of concrete. The results from Table 7.1 show that the self-compacting concrete was complying with the requirements found in the literature. Thus, self-compacting concrete was assumed to having a good consistency and workability after gradually adjusting the chemical admixtures in the mix.

Table 7.1 Slump flow test results.

<table>
<thead>
<tr>
<th>W/C Ratio</th>
<th>0.3</th>
<th>0.4</th>
<th>0.45</th>
<th>0.5</th>
<th>0.6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spread diameter (mm)</td>
<td>655</td>
<td>670</td>
<td>685</td>
<td>700</td>
<td>740</td>
</tr>
</tbody>
</table>

7.2 U-type Test

The U-type test was used to assess the self-compactability of concrete. The results presented in Table 7.2 show that the concrete can be considered self-compacting due to the fact that after opening the sliding gate SCC rose in the other half of the U-tube to a height greater than 85% of the maximum possible height, which is 340 mm.

Table 7.2 U-type test results.

<table>
<thead>
<tr>
<th>W/C Ratio</th>
<th>0.3</th>
<th>0.4</th>
<th>0.45</th>
<th>0.5</th>
<th>0.6</th>
</tr>
</thead>
<tbody>
<tr>
<td>U-tube filling height (mm)</td>
<td>300</td>
<td>305</td>
<td>320</td>
<td>320</td>
<td>330</td>
</tr>
</tbody>
</table>
7.3 Concrete Density

Densities of both types of concretes were determined by weighing the cylindrical specimens, after demolding them. The volume of a mold (8” x 4”) is 0.00165 m$^3$ (0.058 ft$^3$). The final densities for each type of concrete have been calculated by averaging the densities of all five water-cement ratios. Results regarding the densities and the weights for both types of concrete are presented in Table 7.3.

<table>
<thead>
<tr>
<th>W/C Ratio</th>
<th>0.3</th>
<th>0.4</th>
<th>0.45</th>
<th>0.5</th>
<th>0.6</th>
</tr>
</thead>
<tbody>
<tr>
<td>NC - Weights (kg)</td>
<td>3.91</td>
<td>3.88</td>
<td>3.86</td>
<td>3.85</td>
<td>3.83</td>
</tr>
<tr>
<td>SCC - Weights (kg)</td>
<td>4.12</td>
<td>4.09</td>
<td>4.07</td>
<td>4.03</td>
<td>3.98</td>
</tr>
<tr>
<td>NC - Density (kg/m$^3$)</td>
<td>2370</td>
<td>2352</td>
<td>2339</td>
<td>2333</td>
<td>2321</td>
</tr>
<tr>
<td>SCC - Density (kg/m$^3$)</td>
<td>2497</td>
<td>2479</td>
<td>2467</td>
<td>2442</td>
<td>2412</td>
</tr>
</tbody>
</table>

* NC-Normal Concrete ; ** SCC-Self-compacting Concrete

7.4 Splitting Tensile Strength

Due to the usage of mineral and chemical admixtures in the concrete mixtures, as explained in the previous chapters, an increase in the tensile strength of the self-compacting concrete could be observed, compared to the strength of conventional concrete. Table A1 from Appendix A presents the splitting tensile strengths for normal and self-compacting concrete specimens, as they were tested after being cured for 28 days. In addition, Table A2 presents the splitting tensile strengths of both types of concrete after 7 days of curing. It can be seen from Figure 7.1 that the strengths of SCC after 7 days are comparable to those obtained after 28 days for NC. This was possible because of the use of silica fume and fly ash, which usually tend to increase the early strength of concrete.
Figure 7.1 Variation of split tensile strength with the w/c ratio.

The graph also shows that a decrease in the tensile strength takes place as the water-cement ratio is increased. This shows that, regarding the water-cement ratio, the self-compacting concrete behaves like conventional concrete. The values used in the above graph were the minimum and the maximum values (out of three tests) obtained for each water-cement ratio.

The average values (Appendix A) of splitting tensile strengths (out of three tests) for each water-cement ratio are shown in Figure 7.2. Regardless of the water content and the type of concrete, all specimens have had a linear type of fracture and the same relief (surface shape) on the fractured faces. It can be inferred that even if the tensile strength of self-compacting concrete was greater than that of normal concrete (approx. 30%), this did not influence the type of fracture of cylindrical concrete specimens. Also, the proportions of coarse aggregate fractured during tests were different. Taking into account that each fractured face of the cylindrical specimens contained around 60 coarse aggregate particles, the percentage of fractured aggregate for SCC was 15-25% (9-15 aggregate particles), while for normal concrete was around 10% (4-6 aggregate particles), which meant that a better bonding between aggregate and cement paste
exists in SCC compared to normal concrete (Table 7.4). The percentages were obtained by averaging the number of fractured aggregate particles from three specimens. No segregation has been observed in SCC specimens after testing them.

![Splitting Tensile Strength Average - 28 days](image)

Figure 7.2 Variation of splitting tensile strength average with the w/c ratio.

<table>
<thead>
<tr>
<th>W/C Ratio</th>
<th>Normal Concrete</th>
<th>Self-compacting Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No. of fractured aggregate particles</td>
<td>Fractured aggregate (%)</td>
</tr>
<tr>
<td>0.30</td>
<td>15</td>
<td>25</td>
</tr>
<tr>
<td>0.40</td>
<td>12</td>
<td>20</td>
</tr>
<tr>
<td>0.45</td>
<td>11</td>
<td>18</td>
</tr>
<tr>
<td>0.50</td>
<td>10</td>
<td>16.7</td>
</tr>
<tr>
<td>0.60</td>
<td>9</td>
<td>15</td>
</tr>
</tbody>
</table>

Images of the type of fracture and surface shape of the fractured faces of concrete cylinders are presented in appendix A.

### 7.5 Compressive Strength

Self-compacting concrete has also shown an increase in compressive strength, with an average of 65 percent. The values used for plotting the graph in Figure 7.3 were the minimum
and the maximum values (out of three tests) obtained for each water-cement ratio. The average values (Appendix B) of compressive strengths (out of three tests) for each water-cement ratio are shown in Figure 7.4.

![Compressive Strength - 28 days](image)

**Figure 7.3** Variation of compressive strength with the w/c ratio.

![Compressive Strength Average - 28 days](image)

**Figure 7.4** Variation of compressive strength average with the w/c ratio.
It can be said that due to the strengths over 50 MPa obtained for 0.3 and 0.4 water-cement ratios, self-compacting concrete can be considered a high-strength concrete, capable of replacing conventional concrete in the construction field. Regarding the type of fracture, both types of concrete specimens have showed similar patterns, the commonest one being the columnar type due to the rubber caps used during testing. Some pictures of compressive strength types of fracture are presented in Appendix B. No segregation has been observed in SCC specimens after testing them.

7.6 Relationship between Tensile and Compressive Strengths of Self-Compacting Concrete

The variation of splitting and compressive strengths is shown in Figure 7.5. For this study, the values of the ratios for both types of concretes were between 0.06 and 0.10, so they fell within the usual values of the ratio between splitting and compressive strength, in the interval between 0.06 and 0.20 (see Chapter 4).

![Splitting Tensile strength/compressive strength variation](image)

Figure 7.5 Splitting tensile strength versus compressive strength.
In order to estimate better the relationship between the two variables the regression analysis has been used based on the existing data, using Microsoft Excel. The trend lines are similar to those given by different aggregates in Figure 4.3 (Chapter 4).

7.7 Bonding between Coarse Aggregate and Cement Paste

In the last few decades, a lot of research has been done regarding the improvement of the concrete performance. As a result of this research, concretes of higher strength and better durability are being manufactured and used. In order to achieve these types of concretes, their performance parameters can be altered for the better by choosing proper materials or by modifying the physical interfaces between the materials.

Due to the importance of the physical interface in concretes, this study has analyzed the differences between these interfaces with the help of the Scanning Electron Microscope (SEM). Samples, 1” in diameter and 4 mm thick, have been taken (cut) from both types of concretes, for three different water-cement ratios: 0.3, 0.4, and 0.6. They were gold coated on the examined face to prevent charge build-up on the surface and then photographed, five pictures for each w/c ratio, by using the electron microscope. According to the images presented in Appendix C, for 0.3, 0.4, and 0.6 water-cement ratios, self-compacting concrete samples have shown smaller physical interface microcracks than normal concretes, which meant that SCC had better bonds than normal concrete, between aggregate and cement. This explains the increase in tensile and compressive strength for the self-compacting concrete compared to the normal concrete. As mentioned earlier, a better bonding due to the smaller physical interfaces in SCC increased the percentages of fractured aggregate compared to normal concrete. Intervals of the physical interfaces widths from all the images acquired are presented in Table 7.5. Five measurements have been carried out for each water/cement ratio.
Table 7.5 Interfaces microcracks widths intervals for normal and self-compacting concretes

<table>
<thead>
<tr>
<th>W/C Ratio</th>
<th>Normal Concrete Width Interval (µm)</th>
<th>SCC Width Interval (µm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.3</td>
<td>0.62-1.28-1.52-1.56-1.64-1.75</td>
<td>0.01-0.50-0.51-0.62-0.75</td>
</tr>
<tr>
<td>0.4</td>
<td>0.63-2.25-2.75-3.21-3.22-5.63</td>
<td>0.01-0.10-0.12-0.21-1.12</td>
</tr>
<tr>
<td>0.6</td>
<td>1.23-1.75-2.25-3.12-3.38</td>
<td>0.01-0.31-0.37-0.61</td>
</tr>
</tbody>
</table>

Anyhow, the analysis of the concrete specimens under the electron microscope did not show a certain relationship between the water-cement ratios and the physical interface, although an improvement existed when the w/c ratio has decreased. Further investigations have to be carried out in order to determine a good relationship.

7.8 Air Content

Due to the fact that finely divided mineral admixtures, such as fly ash and silica fume, were added to the concrete mix, the air entrainment has been reduced in the self-compacting concrete. As it can be seen from the pictures in Appendix D, normal concrete samples exhibited more air voids (counted with the naked eye) (Table 7.6) than the self-compacting concrete samples, phenomenon which also contributed to an increase in strength of self-compacting concrete. The samples were cut from the middle of the concrete specimens. They had 80x80x12 mm in dimensions and have been taken from both types of concrete, having 0.30, 0.45, and 0.60 water-cement ratios. Encircled zones in the pictures, with a black line, represent the areas having air voids (whiter spots).

Table 7.6 Air voids in normal and self-compacting concrete

<table>
<thead>
<tr>
<th>W/C Ratio</th>
<th>No. of air voids</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Normal concrete</td>
</tr>
<tr>
<td>0.30</td>
<td>120</td>
</tr>
<tr>
<td>0.45</td>
<td>200</td>
</tr>
<tr>
<td>0.60</td>
<td>140</td>
</tr>
</tbody>
</table>
CHAPTER 8. CONCLUSIONS AND RECOMMENDATIONS

Taking into account the findings from this study, previously presented, the following conclusions can be drawn:

It has been verified, by using the slump flow and U-tube tests, that self-compacting concrete (SCC) achieved consistency and self-compactability under its own weight, without any external vibration or compaction. Also, because of the special admixtures used, SCC has achieved a density between 2400 and 2500 kg/m$^3$, which was greater than that of normal concrete, 2370-2321 kg/m$^3$.

Self-compacting concrete can be obtained in such a way, by adding chemical and mineral admixtures, so that its splitting tensile and compressive strengths are higher than those of normal vibrated concrete. An average increase in compressive strength of 60% has been obtained for SCC, whereas 30% was the increase in splitting tensile strength.

Also, due to the use of chemical and mineral admixtures, self-compacting concrete has shown smaller interface microcracks than normal concrete, fact which led to a better bonding between aggregate and cement paste and to an increase in splitting tensile and compressive strengths. A measure of the better bonding was the greater percentage of the fractured aggregate in SCC (20-25%) compared to the 10% for normal concrete.

In addition, self-compacting concrete has two big advantages. One relates to the construction time, which in most of the cases is shorter than the time when normal concrete is used, due to the fact that no time is wasted with the compaction through vibration. The second advantage is related to the placing. As long as SCC does not require compaction, it can be considered environmentally friendly, because if no vibration is applied no noise is made.
Further investigations have to be carried out regarding the self-compacting concrete. One major topic, which has to be studied, is related to the influence of cement type and aggregate shape and surface properties on the bonding between cement paste and coarse aggregate. Also, a thorough investigation has to be carried out in order to obtain an appropriate relationship between the water-cement ratio and the aggregate-cement physical interface.

As SCC technology is now being adopted in many countries throughout the world, in the absence of suitable standardized test methods it is necessary to examine the existing test methods and identify or, when necessary, develop test methods suitable for acceptance as International Standards. Such test methods have to be capable of a rapid and reliable assessment of key properties of fresh SCC on a construction site. At the same time, the testing equipment should be reliable, easily portable and inexpensive. The test procedure should be carried out by a single operator and the test results have to be interpreted with a minimum of training. Also, the results have to define and specify different SCC mixes. One primary application of these test methods would be in verification of compliance on sites and in concrete production plants, if self-compacting concrete could be manufactured in large quantities.

However, unless this work is carried out without delay, there will be a real risk of each country adopting more or less different test methods, making any future harmonization very difficult, so that standardization of tests capable of dealing with the key properties of SCC is urgently required.
BIBLIOGRAPHY


APPENDIX A

SPLITTING TENSILE STRENGTH RESULTS

Splitting tensile strength of the specimens has been calculated using the following formula:

\[ T = \frac{2P}{\pi \cdot ld} \]

where

- \( T \) = splitting tensile strength or stress, psi (MPa)
- \( P \) = maximum applied load indicated by the testing machine, kN
- \( l \) = length of cylindrical specimen, in. (mm)
- \( d \) = diameter of cylindrical specimen, in. (mm)

Table A1. Splitting Tensile Strength – 28 days (ASTM C 496-96)

| W/C Ratio | Max. Load (kN) | Average Stress | | Max. Load (kN) | Average Stress |
|-----------|----------------|----------------||----------------|----------------|
|           | Max. Stress | Average Stress | | Max. Stress | Average Stress |
|           | MPa | PSI | MPa | PSI | MPa | PSI |
| 0.30      | 89.6 | 2.76 | 399.3 | 2.92 | 422.5 | 125.3 | 3.86 | 558.5 | 3.77 | 545 |
| 0.40      | 91 | 2.81 | 406.6 | 2.60 | 376.7 | 105.6 | 3.26 | 471.7 | 3.37 | 488.1 |
| 0.45      | 70.4 | 2.17 | 314 | 2.38 | 343.8 | 96.1 | 2.96 | 428.3 | 3.08 | 446.1 |
| 0.50      | 71.2 | 2.20 | 318.3 | 2.07 | 299 | 89.2 | 2.75 | 397.9 | 2.76 | 399.8 |
| 0.60      | 60.1 | 1.85 | 267.7 | 1.76 | 255.1 | 76.1 | 2.35 | 340 | 2.35 | 340 |

99
### Table A2. Splitting Tensile Strength – 7 days

<table>
<thead>
<tr>
<th>W/C Ratio</th>
<th>Type of Concrete</th>
<th>Normal Concrete</th>
<th>Self-Compacting Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Max. Load (kN)</td>
<td>Average Stress</td>
<td>Max. Load (kN)</td>
</tr>
<tr>
<td></td>
<td>Stress</td>
<td>MPa PSI</td>
<td>Stress</td>
</tr>
<tr>
<td>0.40</td>
<td>74.9</td>
<td>2.31 334.2</td>
<td>94.7</td>
</tr>
<tr>
<td></td>
<td>78.5</td>
<td>2.42 350</td>
<td>108.4</td>
</tr>
<tr>
<td></td>
<td>72.6</td>
<td>2.24 324</td>
<td>96.2</td>
</tr>
</tbody>
</table>

100
Figure A1. Linear fracture of normal concrete cylinder (w/c = 0.40).

Figure A2. Linear fracture of self-compacting concrete cylinder (w/c = 0.40).
Figure A3. Normal concrete surface shape of fractured face (w/c = 0.50).

Figure A4. Self-compacting concrete surface shape of fractured face (w/c = 0.50).
Figure A5. Normal concrete surface shape of fractured face (w/c = 0.60).

Figure A6. Self-compacting concrete surface shape of fractured face (w/c = 0.60).
APPENDIX B

COMPRESSIVE STRENGTH RESULTS

Compressive strength of the specimens has been calculated using the following formula:

\[ T = \frac{4P}{\pi d^2} \]

where

- \( T \) = compressive strength or stress, psi (MPa)
- \( P \) = maximum applied load indicated by the testing machine, kN
- \( d \) = diameter of cylindrical specimen, in. (mm)

Table B1. Compressive strength – 28 days (ASTM C 39-94)

<table>
<thead>
<tr>
<th>W/C Ratio</th>
<th>Max. Load (kN)</th>
<th>Normal Concrete</th>
<th>Self-Compacting Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Max. Stress</td>
<td>Average Stress</td>
<td>Max. Load</td>
</tr>
<tr>
<td></td>
<td>MPa PSI</td>
<td>MPa PSI</td>
<td>MPa PSI</td>
</tr>
<tr>
<td></td>
<td>(MPa)</td>
<td>(MPa PSI)</td>
<td>(MPa)</td>
</tr>
<tr>
<td>0.30</td>
<td>330.3 40.74 5894.6</td>
<td>40.94 5923.9</td>
<td>546.2 67.37 9747.4</td>
</tr>
<tr>
<td></td>
<td>330.9 40.82 5906</td>
<td></td>
<td>525.7 64.84 9381.3</td>
</tr>
<tr>
<td></td>
<td>334.6 41.27 5971.1</td>
<td></td>
<td>527.5 65.06 9413.2</td>
</tr>
<tr>
<td>0.40</td>
<td>270.8 33.40 4832.5</td>
<td>33.43 4837.3</td>
<td>466.2 57.51 8320.8</td>
</tr>
<tr>
<td></td>
<td>266.2 32.84 4751.4</td>
<td></td>
<td>451.4 55.68 8056</td>
</tr>
<tr>
<td></td>
<td>276.1 34.06 4928</td>
<td></td>
<td>431.1 53.17 7692.8</td>
</tr>
<tr>
<td>0.45</td>
<td>228.2 28.15 4072.9</td>
<td>27.61 3995.2</td>
<td>387.3 47.77 6911.6</td>
</tr>
<tr>
<td></td>
<td>219.9 27.12 3923.8</td>
<td></td>
<td>397.4 49.02 7092.4</td>
</tr>
<tr>
<td></td>
<td>223.5 27.57 3988.9</td>
<td></td>
<td>394 48.60 7031.7</td>
</tr>
<tr>
<td>0.50</td>
<td>184.7 22.78 3295.9</td>
<td>23.13 3346.1</td>
<td>292.9 36.13 5227.4</td>
</tr>
<tr>
<td></td>
<td>190.6 23.51 3401.5</td>
<td></td>
<td>291.2 35.92 5197.1</td>
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<td>130.1 16.05 2321.9</td>
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<td>242.1 29.86 4320.3</td>
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Compressive strength test - Types of fracture

Figure B1. Columnar type of fracture.
Figure B2. Cone type of fracture.
Figure B3. Cone and split type of fracture.
Figure B4. Shear type of fracture.
APPENDIX C

SCANNING ELECTRON MICROSCOPE IMAGES

Figure C1. Photomicrograph of normal concrete physical interface (w/c = 0.30).

Figure C2. Photomicrograph of self-compacting concrete physical interface (w/c = 0.30).
Figure C3. Photomicrograph of normal concrete physical interface (w/c = 0.40).

Figure C4. Photomicrograph of self-compacting concrete physical interface (w/c = 0.40).
Figure C5. Photomicrograph of normal concrete physical interface (w/c = 0.60).

Figure C6. Photomicrograph of self-compacting concrete physical interface (w/c = 0.60).
APPENDIX D

AIR VOIDS IN CONCRETE SAMPLES

Figure D1. Stereo microscope image of normal concrete (w/c = 0.30).

Figure D2. Stereo microscope image of self-compacting concrete (w/c = 0.30).
Figure D3. Stereo microscope image of normal concrete (w/c = 0.45).

Figure D4. Stereo microscope image of self-compacting concrete (w/c = 0.45).
Figure D5. Stereo microscope image of normal concrete (w/c = 0.60).

Figure D6. Stereo microscope image of self-compacting concrete (w/c = 0.60).
Cristian Druta was born on December 18, 1967 in Constanta, Romania, and graduated from “Ion Neculce” lyceum in 1987. He attended the College of Engineering at Polytechnic University of Bucharest between 1990 and 1995 and received his bachelor of science degree in mechanical engineering with a specialization in hydraulic and pneumatic machines in June 1995. He then came to United States of America in August 2001, to pursue higher education at Louisiana State University. He started his Master of Science program in Engineering Science, in the Department of Civil and Environmental Engineering with the specialization in materials and successfully completed his degree requirements in May 2003.