HIGH PERFORMANCE STRUCTURAL LIGHTWEIGHT CONCRETE UTILIZING NATURAL PERLITE AGGREGATE AND PERLITE POWDER

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ABSTRACT

HIGH PERFORMANCE STRUCTURAL LIGHTWEIGHT CONCRETE UTILIZING NATURAL PERLITE AGGREGATE AND PERLITE POWDER

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Structural Lightweight Concrete is generally made by using artificial lightweight aggregates such as expanded clay, shale and slate. However, rapidly increasing fuel prices in recent decades and corresponding increase in the production costs of these aggregates have renewed the interest in natural lightweight aggregates such as pumice, scoria, rhyolite and perlite.

This study investigates the mechanical properties and durability characteristics of high-performance lightweight concretes utilizing natural perlite aggregate and perlite powder in comparison to those of high-strength normal weight concrete of similar specific strength (structural efficiency). For this purpose, three concrete mixtures have been designed, namely high-strength lightweight concrete (HSLWC), self-compacting high strength lightweight concrete (SCLWC) and high-strength normal weight concrete (HSNWC). An extensive testing program was conducted on concrete specimens to determine fresh properties such as slump, slump flow, unit weight, air content and setting time; hardened properties such as compressive strength, splitting and flexural tensile strength, modulus of elasticity and thermal coefficient of expansion; durability characteristics such as rapid chloride-ion penetrability, resistance to aggressive chemical solutions and freezing-thawing resistance.

The results have shown that natural perlite aggregate and perlite powder can be satisfactorily utilized in the production of self-compacting lightweight concrete with 28-day compressive strengths up to 50 MPa. It is also shown that perlite aggregate containing high performance lightweight concretes have generally superior or at least similar durability performance to that of high-strength normal weight concrete of similar specific strength.

Keywords: high-performance concrete, self-compacting lightweight concrete, structural lightweight concrete, durability, natural perlite.

ÖZ

HAM PERLİT AGREGASI VE PERLİT TOZU KULLANILARAK YAPILAN YÜKSEK PERFORMANSLI TAŞIYICI HAFİF BETON

Eser, Hasan
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Tez Yöneticisi: Doç. Dr. Lutfullah Turanlı

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Taşıyıcı hafif beton genellikle genleştirilmiş kil, şist ve arduvaz gibi yapay hafif agregalar kullanılarak yapılır. Ancak, son on yıllarda hızla artan yakıt fiyatlarına paralel olarak bu agregaların üretim maliyetlerinin artması, süngertaşı, lav cürufu, riyolit ve perlit gibi doğal hafif agregalara olan ilgiyi arttırmıştır.

Bu çalışma, doğal perlit agregası ve perlit tozu içeren yüksek performanslı taşıyıcı hafif betonlar ile benzer özgül dayanımdaki (yapısal verimlilikteki) yüksek dayanımlı normal ağırlıklı betonların mekanik özelliklerini ve kalıcılıcık karakteristiklerini, karşılaştırmalı olarak incelemektedir. Bu amaçla üç tip beton karışımı hazırlanmıştır: yüksek dayanımlı hafif beton (HSLWC), kendiliğinden yerleşen yüksek dayanımlı hafif beton (SCLWC) ve yüksek dayanımlı normal ağırlıklı beton (HSNWC). Bu betonların, çökme, çökme akışı, birim ağırlık, hava oranı ve priz süresi gibi taze özellikleri; basınç dayanımı, yarma ve eğilme gerilmesi dayanımı, elastisite modülü ve doğrusal ısıl genleşme katsayısı gibi sertleşmiş özellikleri; hızlı klor iyonu geçirgenliği, agresif kimyasal solüsyonlara karşı dayanıklılığı ve donma-çözülme direnci gibi kalıcılık özellikleri kapsamlı bir şekilde incelenmiştir.

Çalışma sonuçları, doğal perlit agregası ve perlit tozu kullanılarak 28-günlük basınç dayanımı 50 MPa'ya ulaşan kendiliğinden yerleşen hafif beton üretilebileceğini göstermiştir. Ayrıca, perlit agregası içeren yüksek performanslı hafif betonların, benzer özgül dayanımdaki yüksek dayanımlı normal ağırlıklı betonlardan, genellikle, üstün ya da en azından yakın kalıcılık performansı gösterdiği ortaya konulmuştur.

Anahtar kelimeler: yüksek performanslı beton, kendiliğinden yerleşen hafif beton, taşıyıcı hafif beton, kalıcılık, doğal perlit.

To My Family

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LIST OF ABBREVIATIONS

ACI : American Concrete Institute
ASR : Alkali-Silica Reaction
ASTM : American Society for Testing and Materials
EFNARC : The European Federation of Specialist Construction Chemicals and Concrete Systems
EN : European Norms (Standards)
HSLWC : High-Strength Lightweight Concrete
HSNWC : High-Strength Normalweight Concrete
LA : Limestone Aggregate
LWC : Lightweight Concrete
NWC : Normalweight Concrete
PA : Natural Perlite Aggregate
RCPT : Rapid Chloride Permeability Test
SCLWC : Self-Compacting High-Strength Lightweight Concrete
USACE : U.S. Army Corps of Engineers
w/b : water-to-binder ratio
w/c : water-to-cement ratio
w/cm : water-to-cementitious materials ratio

CHAPTER 1

INTRODUCTION

1.1. General

High-performance concrete is broadly defined as the concrete which possess one or more properties superior than that of conventional concrete. These properties may include high workability, high strength, high elastic modulus, low permeability, high durability or volume stability, etc. (Kosmatka, Kerkhoff, & Panarese, 2003). Nevertheless, many of these performance characteristics have one point in common, which is low w/b ratio (generally below 0.45). Therefore, some researchers define high performance concrete as "low w/b ratio concrete with optimized aggregate-binder ratio to control its volume stability and which receives adequate water curing" (Aitcin, 2008, p. 333). This definition looks very much like a recipe for durable concrete. Indeed, durability is a key factor for high performance and more important than strength alone to provide a longer service life.

Referring back to broad definition of high performance concrete, high strength concrete and self-compacting concrete can be classified as high-performance concrete since they have superior properties over conventional concretes. For example, the former has high strength and the latter has high workability. Similarly, self-compacting high strength lightweight concrete, which combines high workability, high strength and low density is also an example of high performance concrete.

Structural lightweight concrete is generally made by using artificial lightweight aggregates and usually requires higher binder contents than its normal weight counterparts in order to reach structural strength levels. In last few decades, rapidly increasing fuel prices caused the production costs of cement and artificial lightweight aggregates to increase, both of which are burnt in large kilns in production phase. Resultantly, the pursuit of lowering production costs has renewed the interest in utilization of natural lightweight aggregates and pozzolans in lightweight concrete production.

Currently, there are many studies on structural lightweight concrete majority of which are focusing on those with artificial lightweight aggregates. However, only a limited number of studies exist with a focus on natural lightweight aggregates and even less with a focus on natural perlite aggregate. Besides, there is no recorded study on self-compacting high strength lightweight concrete with natural perlite aggregate and perlite powder. The literature also lacks the investigation of mechanical properties and durability characteristics of structural lightweight concretes in comparison to those of normal weight concretes of similar specific strength (a.k.a. structural or strength efficiency). In many cases, it is the specific strength of concrete rather than strength itself which determines its suitability for a particular application. Therefore, a comparison of concrete properties at similar specific strength is more logical than a comparison at similar strength.

1.2. Objectives and Scope

There are several objectives of this thesis. First is to design a high strength lightweight concrete with natural perlite aggregate by using reasonable cement contents. Second is to design a self-compacting high strength lightweight concrete with natural perlite aggregate and perlite powder. Finally the third is to compare mechanical properties and durability characteristics of these high performance lightweight concretes with those of high-strength normal weight concrete at similar specific strength.

Within the scope of this thesis, an extensive testing program was conducted on concrete specimens to determine fresh properties such as slump, slump flow, unit weight, air content and setting time; hardened properties such as compressive strength, splitting and flexural tensile strength, modulus of elasticity and coefficient of thermal expansion; durability characteristics such as rapid chloride-ion penetrability, resistance to some aggressive chemical solutions (sulphuric acid, magnesium sulfate, sodium bicarbonate) and freezing-thawing resistance.

This thesis consists of six chapters. Chapter 1 introduces the research topic and objectives of this study. Chapter 2 provides background information about structural lightweight concrete and presents a literature review of recent studies on high-strength lightweight concretes and self-compacting lightweight concretes. Chapter 3 presents experimental program, briefly summarizes testing procedures and related issues. The detailed properties of materials used for the designed concretes are also provided in this chapter. Chapter 4 presents the experimental results and discusses findings in detail. Chapter 5 concludes the thesis by highlighting the findings of the research and finally Chapter 6 includes suggestions about further research topics.

CHAPTER 2

BACKGROUND AND LITERATURE REVIEW

2.1. History of Structural Lightweight Concrete

2.1.1. Ancient Applications

Known history of the lightweight concrete starts more than 2 thousand years ago in the Roman Empire. The most significant examples of that time were the Port of Cosa, the Pantheon Dome and the Coliseum (ACI Committee 213, 2003).

The Port of Cosa was built on the west coast of Italy, in 273 B.C. The designers of the port were aware of the fact that lightweight aggregates were more convenient to use in marine structures. Instead of using locally available aggregates (beach sand and gravel) for the construction, the builders have brought natural lightweight aggregates (pumice and scoria) from the volcanic resources located at 40 km away. The harbor consists of four piers, which had resisted the forces of nature except the surface abrasion for almost 2 thousand years and it is now abandoned only due to siltation (ACI Committee 213, 2003).

The construction of the Pantheon was completed in 27 B.C. It has a doom with a diameter of 43.3 m which was the highest record for almost 2 thousand years. The builders used the lightweight aggregates of varying densities in descending order from the base to the top of the doom. In other words, higher density aggregates were used near the base where the stresses are higher; and lower density aggregates were used near the top where the stresses are lower. When the doom was first constructed, it had a metal cover which was soon removed to be used for another structure. Until it was covered with a lead roof recently, it had been exposed to the forces of nature for hundreds of years (Holm & Bremner, 2000). Even today, the Pantheon is still in use for spiritual purposes (ACI Committee 213, 2003).

The Coliseum, which is an ancient amphi-theater of massive size with a 50 thousand seating capacity, was constructed in 75 to 80 A.D. The foundation of the Coliseum was made of a lightweight concrete utilizing crushed volcanic lava as aggregate. Similarly, the aggregates used in its walls were made of porous, crushed bricks. The spaces and the vaults between the walls were made of porous-tufa cut stone (ACI Committee 213, 2003).

2.1.2. Modern Applications

The use of lightweight aggregates after the Romans was limited. This was changed when manufactured lightweight aggregates became commercially available in 20th century (ACI Committee 213, 2003). In 1918, Stephen J. Hyde, a ceramic engineer, patented the process of producing lightweight aggregates through heating and expanding shale, clay or slate in a rotary kiln. At first, the expanded aggregates were used in the construction of concrete ships for U.S. fleet. Later, expanded aggregates started to be used for civilian construction sector.

The first commercial plant for expanded aggregate production was founded in Kansas, in 1920. In 1923, Dan Servey initiated the first production of lightweight concrete masonry units. In 1929, the use of lightweight concrete jumped to high rise construction. 14 additional story were added to the existing 14-story building of South Western Bell Telephone Office by using lightweight concrete (Holm & Bremner, 2000).

Starting from the second half of the 20th century, many multistory buildings were constructed by using structural lightweight concrete. Examples are 42-story Prudential Life Building (Chicago) with lightweight concrete floors and 18-story Statler Hilton Hotel (Dallas) with all lightweight concrete frame and flat plate floors (ACI Committee 213, 2003).

Today, the applications of structural lightweight concrete extended not only to high-rise buildings but also bridges and marine structures. Stolmen Bridge and Heidrun Tension Leg Platform are significant examples of recent applications.

Stolmen Bridge (Figure 2.1) was built in Norway in 1998. It has a main span length of 301 m and total length of 467 m, which is world record (2000) for free-cantilever concrete bridges. The 184 m portion in the middle of the main span was constructed with high-strength lightweight concrete. In the construction of the Stolmen Bridge, 1600 m^3 lightweight concrete was used. The 28-day mean compressive cube strength was 70.4 MPa and the mean density of 28-day water cured specimens was 1940 kg/m^3 (ESCSI, 2010).

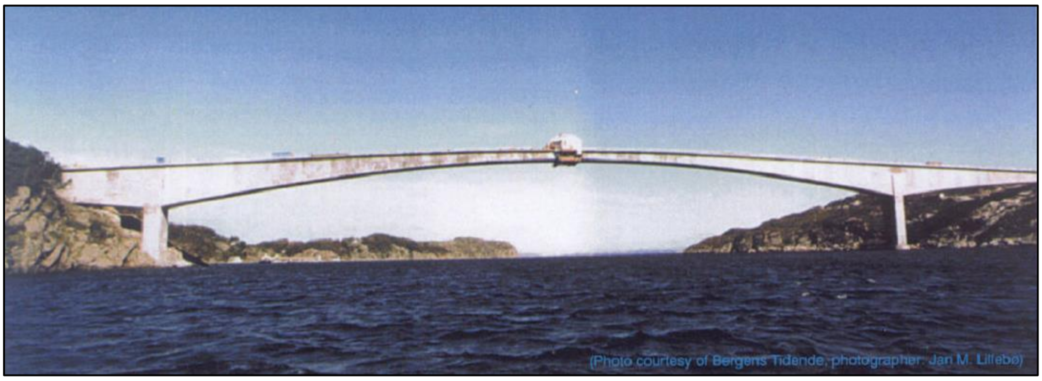

Figure 2.1. Stolmen Bridge, Norway (ESCSI, 2010)

Figure 2.2. Heidrun Tension Leg Platform, North Sea (ESCSI, 2010)

Heidrun Tension Leg Platform (Figure 2.2) was built in 1995 at Heidrun field of the North Sea, where water depth is 345 m. It is the largest floating concrete structure carrying the largest deck load recorded (2000). In the construction of the Heidrun Tension Leg Platform, 65700 m^3 lightweight concrete was used. Job specifications required a compressive strength more than 70 MPa and a maximum density of 1950 kg/m^3 for cast in place concrete and 2000 kg/m^3 for slipformed concrete (ESCSI, 2010).

2.2. Properties of Lightweight Aggregates

2.2.1. Definition

Aggregates with an oven-dry particle density less 2000 kg/m^3 or an oven-dry loose bulk density less than 1200 kg/m^3 are called as lightweight aggregates according to EN206-1:2000. ASTM C330 also defines a maximum limit for the bulk density, which is 1120 kg/m^3 and 880 kg/m^3 for fine and coarse lightweight aggregate, respectively.

2.2.2. Classification

Lightweight aggregates are divided into two categories according to their sources:

1. Natural Lightweight Aggregates
2. Manufactured(Synthetic) Lightweight Aggregates

Natural lightweight aggregates are obtained by processing volcanic rocks. Pumice, scoria, tuff and perlite are some of the examples which fall in this category. Pumice is a light colored porous glass with elongated voids. Scoria is a dark colored porous glass with spherical voids. Tuff is a porous glass formation of consolidated volcanic ash. Perlite is a porous glass with a high silica content. It generally contains 2-5% water (Mehta & Monteiro, 2006).

Synthetic lightweight aggregates are expanded forms of materials such as clay, shale, slate, perlite and vermiculite, produced by heat treatment, generally around 1000^oC. The materials are either reduced to desired size before calcination or crushed after the calcination process. The expansion results from the entrapment of gases, which are generated during heat treatment, inside the processed material. The use of initially pelletized materials in heat treatment process produces spherical aggregate particles with a semi-impervious coating having 12-30% lower water absorption capacity than the particles produced of unpelletized material. Therefore, coated particles are preferable from workability point of view, however they are more expensive than uncoated ones (Neville & Brooks, 2010).

An interesting point to discuss is that perlite can expand up to 20 times (Aşık, 2006) and vermiculite can expand up to 30 times (Neville, 2003) of its uncalcined volume when heat treated. Resultantly, expanded perlite and vermiculite have a very low density and strength and used for insulation purposes. On the other hand, lightweight aggregates which have relatively higher densities such as expanded shale, clay and slate are used for structural lightweight concrete.

In Figure 2.3, lightweight aggregate spectrum showing unit weight of various lightweight aggregates and corresponding unit weight of concretes is given.

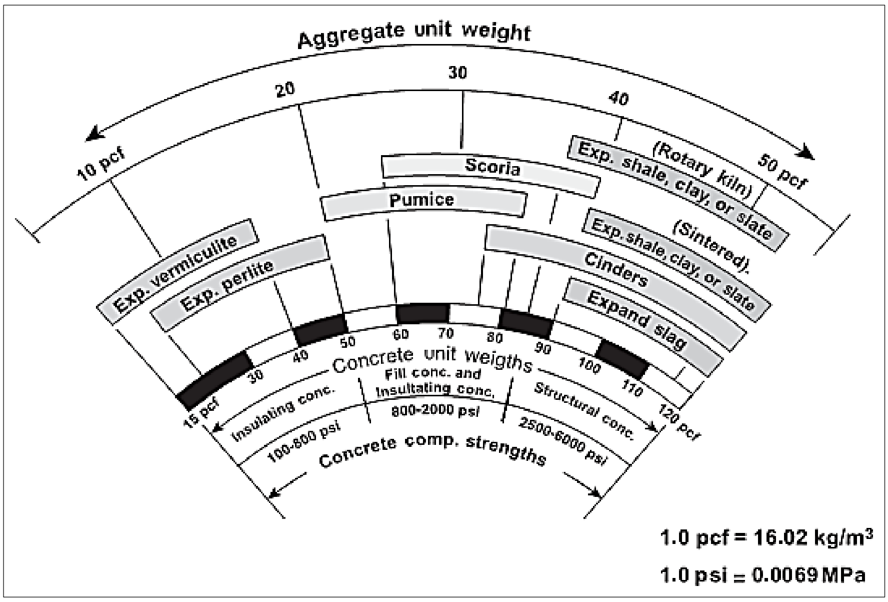

Figure 2.3. Lightweight aggregate spectrum (Mehta & Monteiro, 2006)

2.2.3. Internal Structure of Lightweight Aggregates

Lightweight aggregates have cellular or porous internal structure and resultantly have low specific gravity. In structural lightweight aggregates, these pores are uniformly distributed in relatively crackless vitreous material and its size varies between 5 and 300 μm. Surface pores are permeable and easily fill by exposure to moisture in a few hours. Interior pores are less permeable. Saturation of the interior pores progresses very slowly and can take months. A certain fraction of interior pores are disconnected, thus remains unsaturated for many years (Holm & Bremner, 2000).

2.2.4. Particle Shape and Surface Texture

The particle shape and surface texture of aggregates may significantly vary with the source of the aggregate and method of production (Holm & Bremner, 2000). The particles may have cubical, rounded, angular or irregular shape. The surface texture of the particles may be smooth with fine pores or rough and irregular with large pores. Workability, binder content, water requirement, fine-to-coarse aggregate ratio are directly affected by the particle shape and surface texture of the aggregates.

2.2.5. Specific Gravity

As previously mentioned, lightweight aggregates are lighter than normal weight aggregates due to porous internal structure. The specific gravity of lightweight aggregates is practically about 1/3 to 2/3 of normal weight aggregates. Contrary to normal weight aggregates, fine particles of lightweight aggregates have higher specific gravity than coarse particles from the same source. This is mainly due to elimination of larger pores during crushing (Neville & Brooks, 2010). The amount of difference between the specific gravity of fine and coarse particles varies with the method of production (ACI Committee 213, 2003).

2.2.6. Bulk Density

Bulk density of lightweight aggregates are measured in dry-loose form and it is fundamentally proportional to specific gravity for same grading and particle shape. For different particle shapes, for example, dry-loose bulk density of angular and rounded particles of same specific gravity may show 80 kg/m^3 or more difference (ACI Committee 213, 2003). In Table 2.1, maximum limits for dry-loose bulk density of structural lightweight aggregates are given.

Table 2.1. Maximum dry loose bulk density requirements of lightweight aggregates for structural concrete (ASTM C330)

Size Designation	Maximum Dry Loose Bulk Density (kg/m^3)
Fine aggregate	1120
Coarse aggregate	880
Combined fine and coarse aggregate	1040

2.2.7. Grading

The fact that specific gravity of lightweight aggregates increases with the decreasing particle size contrary to normal weight aggregates, necessitates modification on the grading requirements stated in ASTM C33, in order to fit same volumetric distribution of materials retained on each sieve. This modified gradation is given in ASTM C330, as shown in Table 2.2. The manufacturers generally stock aggregates in a number of standard sizes such as coarse, intermediate and fine, rather than sieve by sieve categorization. By combining fractions of commercially available sizes, grading requirements can be met.

Table 2.2. Grading requirements for lightweight aggregate for structural concrete (ASTM C330)

Nominal Size Designation	Percentages (Mass) Passing Sieves Having Square Openings									
	25.0 mm (1 in.)	19.0 mm (¾ in.)	12.5 mm (½ in.)	9.5 mm (⅜ in.)	4.75 mm (No. 4)	2.36 mm (No. 8)	1.18 mm (No. 16)	300 µm (No. 50)	150 µm (No. 100)	75 µm (No. 200)
Fine aggregate:										
4.75 mm to 0	...	...	...	100	85–100	...	40–80	10–35	5–25	...
Coarse aggregate:										
25.0 m to 4.75 mm	95–100	...	25–60	...	0–10	...	...	...	...	0–10
19.0 mm to 4.75 mm	100	90–100	...	10–50	0–15	...	...	...	...	0–10
12.5 mm to 4.75 mm	...	100	90–100	40–80	0–20	0–10	...	...	...	0–10
9.5 mm to 2.36 mm	...	...	100	80–100	5–40	0–20	0–10	...	...	0–10
Combined fine and coarse aggregate:										
12.5 mm to 0	...	100	95–100	...	50–80	...	...	5–20	2–15	0–10
9.5 mm to 0	...	...	100	90–100	65–90	35–65	...	10–25	5–15	0–10

2.2.8. Water Absorption Capacity

The 24 hour water absorption capacity of lightweight aggregates vary between 5 to 25% by dry mass depending on the pore system of the aggregate, whereas it is less than 2% for most of the normal weight aggregates (ACI Committee 213, 2003). For lightweight aggregates of satisfactory quality, absorption capacity is generally under 15% (Neville & Brooks, 2010). Absorption capacity and rate of absorption is especially important for mix design calculations to correctly establish w/c ratio, which controls workability, strength and permeability characteristics of the concrete. The mix design considerations related to water absorption will be discussed in detail in subchapter 2.4.

2.3. Classification of Lightweight Concrete

According to method of production, lightweight concretes are divided into three (Neville & Brooks, 2010):

a) Utilizing aggregates with low specific gravity: lightweight aggregate concrete
b) Introducing large voids within concrete or mortar body: aerated, cellular, foamed or gas concrete
c) Utilizing only coarse aggregates to provide large interstitial voids: no-fines concrete

In all of the methods above, the lightness of the concrete is achieved by introducing voids into system, whether by porous aggregates or voids in mortar, or interstitial voids between coarse aggregates. It may be argued that increase in porosity of a material is accompanied by strength reduction. For example, this could be problematic for structural concrete where high strength is necessary. On the other hand, the reduction in thermal conductivity due to increased porosity is advantageous for insulating concrete, where high strength is not required. Therefore, the performance assessment of lightweight concretes should be based on its area of application.

The scope of this thesis covers only lightweight aggregate concrete, classification of which is discussed in next section.

2.3.1. Classification of Lightweight Aggregate Concrete

ACI Committee 213 (1987) classifies lightweight aggregate concrete under three categories, which are low density concrete, moderate strength concrete and structural concrete:

Low density concrete is made by highly expanded aggregates such as expanded perlite and vermiculite. Its 28 day air dry unit weight is generally less than 800 kg/m^3 and its compressive strength ranges between 0.69 and 6.89 MPa. Owing to its low thermal conductivity, it is used for insulation purposes.

Moderate strength concrete is made by natural lightweight aggregates such as pumice and scoria. Its 28 day air dry unit weight is generally less than 1440 kg/m^3 and its compressive strength ranges between 6.89 and 17.24 MPa. It is mainly used as fill concrete.

Structural Lightweight Concrete is generally made by expanded forms of shale, clay, slate, slag or fly ash aggregate. Its 28 day air dry unit weight is generally between 1140 and 1850 kg/m^3, and its compressive strength should be higher than 17.2 MPa.

According to new definition by ACI Committee 213 (2003), structural lightweight concrete is the one with an air-dry equilibrium density between 1120 and 1920 kg/m^3, and having a 28-day compressive strength higher than 17 MPa. In the case of a 28-day compressive strength of 40 MPa or higher, it is defined as high-strength lightweight concrete.

In the State of Art Report by U.S. Army Corps of Engineers (USACE), another definition for high strength lightweight concrete is given. According to this report, lightweight concretes having compressive strength higher than 35 MPa are defined as high strength provided that maximum equilibrium density should be less than 2000 kg/m^3 (Holm & Bremner, 2000).

The European Standard EN-206-1:2000 also have a definition for lightweight concrete. In this standard, maximum density for lightweight concrete is defined as 2000 kg/m^3 in oven dry condition. This corresponds approximately to 2050 kg/m^3 in air dry condition. In addition, this standard defines high strength limit as 50 MPa.

2.4. Production of Structural Lightweight Concrete

2.4.1. Mix Design Considerations

2.4.1.1. Methods

The similar mix design considerations to that of normal weight concretes are applied to lightweight concretes but paying increased attention to water absorption characteristics of lightweight aggregates (Holm & Bremner, 2000). In many cases, absolute volume method, which is widely used for normal weight concretes, is also applicable for proportioning structural lightweight concrete (ACI Committee 213, 2003). In this method, sum of the absolute volumes of concrete making materials is assumed as equal to volume of the fresh concrete. To apply this method, absorption capacity and specific gravity for each size of the aggregates in as-batched moisture condition must be known.

An alternative to absolute volume method is volumetric method. In this method, a trial mixture is prepared for estimated volumes of cementitious materials and aggregates. Water amount is determined as the required water at a target slump. Then, calculations are made for yield so as to determine actual quantities of materials per unit volume and if necessary, additional trial mixtures are made until satisfactory proportions are achieved.

There are additional points to be considered in the case of designing self-compacting lightweight concrete mixtures. First of all, self-compacting lightweight concretes should have high flowability and high viscosity similar to that of its normal weight counterparts so that coarse aggregates can float in mortar without any segregation. For all self-compacting concretes, this requires a balance between deformability and stability which can be achieved by utilizing generally a high fines content (about 520 to 560 kg/m^3) and a low water content, use of high-range water reducers (typically polycarboxylate ethers) to plasticize the fresh concrete and stabilizers such as polysaccharides against fluctuations in water content (Kosmatka, Kerkhoff, & Panarese, 2003). Due to lower density of lightweight coarse aggregates and thus its tendency to floating, self-compacting lightweight concretes are much likely to segregation. In such cases, replacement of cement with pozzolans of lower specific gravity may contribute resolving this problem by reducing the difference between the densities of mortar and lightweight coarse aggregate. For sanded-lightweight self-compacting concrete, it is also beneficial to replace the normal-weight fine aggregates with lightweight sand.

2.4.1.2 Effect of Absorption Capacity and Rate of Absorption

For lightweight aggregates with high absorption capacity (10-20%), the relationship between strength and w/c ratio cannot be efficiently established for lightweight concrete mix design since it is difficult to determine how much of the mixing water will be absorbed by aggregate and even more difficult when the fact that absorption can continue for several weeks is considered (Mehta & Monteiro, 2006). Therefore, instead of w/c based estimation of compressive strength, cement content at a specified slump value is more logical for mix design purposes.

Another concern is related to rate of absorption. At the time of mixing, if aggregate is in dry condition, it will quickly absorb water and workability of mix will drop correspondingly. This can be solved by mixing the aggregate with at least one-half of the mixing water before the addition of binding medium (Neville & Brooks, 2010). However, this solution have both positive and negative consequences. The absorbed water in the aggregate, which is not immediately available for hydration, will provide continued hydration -internal curing- after external curing period has ended (Holm & Bremner, 2000). On the other hand, it will increase concrete density and reduce thermal insulation (Neville & Brooks, 2010).

2.4.1.3. Effect of Air Entrainment

Likewise normal weight concrete, air entrainment contributes to durability of lightweight concrete by reducing permeability and increasing freezing-thawing resistance. In addition to durability improvements, it also improves workability by reducing water requirement at a specified slump. Thus, it reduces tendency to bleeding and segregation. The reduction in water requirement will also reduce w/c ratio, which resultantly compensate, to an extent, the strength reduction accompanied by air-entrainment. The reduction in strength due to air-entrainment will also be less than that of normal weight concrete because of elastic compatibility between lightweight aggregate and binder phase (Holm & Bremner, 2000). It is a common practice to use air-entrainment in lightweight concrete regardless of durability concerns (ACI Committee 213, 2003).

2.4.2. Mixing, Placing, Finishing and Curing

To achieve the planned volume of fresh concrete and avoid slump loss during transport, prewetting of lightweight aggregates must be applied before adding other constituents into mixer. However, the saturation of lightweight aggregates cannot be fully achieved, unless prewetting is done by means of pressurized water. Therefore, the measured fresh density of lightweight concrete is approximately 100-120 kg/m^3 lower than the theoretical fresh density (Neville, 2003).

Avoiding segregation is the most important concern in handling and placing of concrete. For satisfactory placement of lightweight concrete; workable fresh mixture with a minimum water content, equipment capable of swiftly moving the concrete, proper consolidation and quality workmanship are required (ACI Committee 213, 2003). A well-designed lightweight concrete mixture generally requires less effort for placing and finishing than normal weight concrete. For example, a slump of 50-75 mm could be sufficient to obtain similar workability in normal weight concrete mixture with a slump of 100-125 mm (Mehta & Monteiro, 2006).

Overvibration of lightweight concrete can drive heavier mortar to downward, which is required at the surface for finishing operations. Therefore, excessive vibration or working of lightweight concrete should be avoided. For satisfactory finishing of lightweight concrete floors, finishing operations should start after free surface bleeding water is evaporated. Use of magnesium, aluminum or other quality finishing tools are also recommended (ACI Committee 213, 2003).

Followed by completion of finishing operations, curing should be started immediately. However, until bleeding is stopped, membrane-forming curing compounds should not be used (Holm & Bremner, 2000). Lightweight aggregate concretes have more tolerance to inadequate curing than normal weight concretes, due to internal curing provided by absorbed water in lightweight aggregates. Internal curing is more important for high performance concrete mixtures containing pozzolan, particularly when w/cm is less than 0.45. This is because, relatively impermeable nature of low w/cm mixtures avoids external curing moisture to penetrate into concrete (ACI Committee 213, 2003). This phenomenon has been illustrated in Figure 2.4.

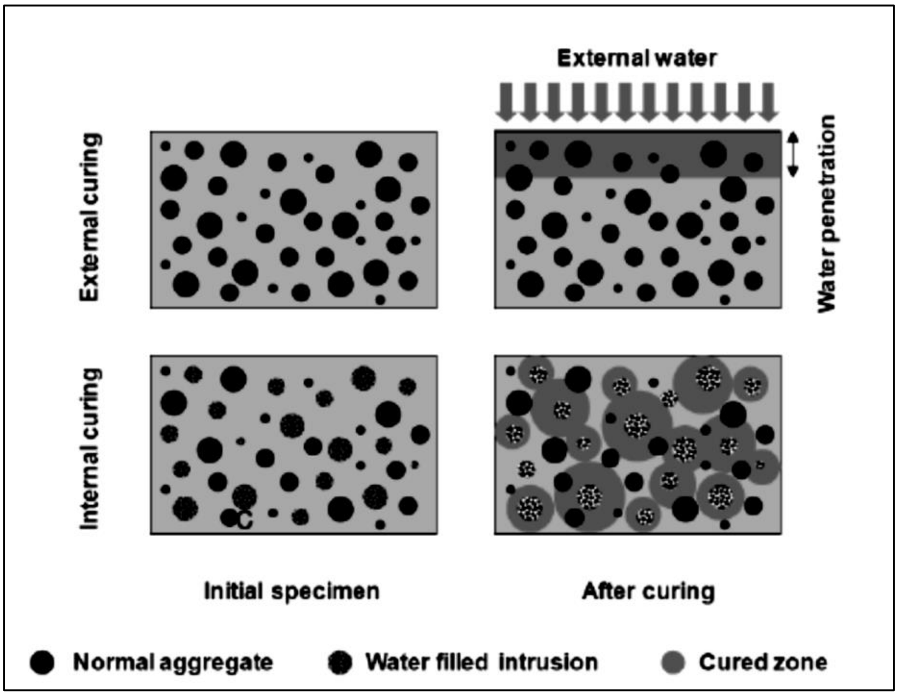

Figure 2.4. The movement of moisture in external and internal curing (Castro, De la Varga, Golias, & Weiss, 2010)

2.5. Engineering Properties of Structural Lightweight Concrete

2.5.1. Compressive Strength

In general, the compressive strength of structural lightweight concrete is affected by similar factors to that of normal weight concretes such as water-cement ratio, cement content, air content, curing etc. The differences are only due to the properties of the lightweight aggregate and its interaction with the binding phase. Therefore, the effect of aggregate related concepts such as elastic compatibility, maximum strength ceiling and contact zone should be comprehended.

2.5.1.1. Elastic Compatibility

Concrete can be considered as a two-phase material, namely the combination of mortar and coarse aggregate. Mortar phase includes fine aggregate, cement, water, admixtures and air. In lightweight aggregate concrete, elastic modulus of these two phases are much closer to each other when compared to normal weight concrete, which results in a relatively more homogeneous stress distribution and reduced stress concentration. Contrary to normal weight concrete, the addition of air-entrainment in structural lightweight concrete will further increase the elastic compatibility of these phases by reducing the stiffness of the mortar phase. This fact explains why the strength reduction accompanied by air-entrainment is generally less significant in lightweight concrete than in normal weight concrete (Holm & Bremner, 2000).

2.5.1.2. Maximum Strength Ceiling

The term "strength ceiling" can be defined as the point at which increase in the content or quality (w/b) of the binder yields to only minor improvements in concrete strength. In other words, at strength ceiling of a concrete, it is the strength of coarse aggregate or quality of the transition zone which will determine the maximum strength of the concrete (Holm & Bremner, 2000). There are two methods of increasing maximum strength ceiling, which are reducing maximum aggregate size and incorporating pozzolans in concrete. Firstly, as the size of the aggregate decreases, the porosity of the aggregate also decreases and resultantly the strength of the aggregate increases. Secondly, using supplementary cementitious materials in concrete results in densification of transition zone through pozzolanic reaction. Therefore, it is a common practice to limit maximum aggregate size and to use pozzolan in high strength lightweight concretes.

2.5.1.3. Contact Zone

The contact zone in lightweight aggregate concretes are improved due to several reasons. First of all, the surface of lightweight aggregate exposed to high temperatures either in production plants or naturally during volcanic activity yields in pozzolanic reactivity at transitional zone. Secondly, surface roughness of lightweight aggregates provide better bonding between cement paste and aggregate phases (Holm & Bremner, 2000). Thirdly, cement paste can penetrate into aggregate's surface pores and further enhances the bond between two phases (Al-Khaiat & Haque, 1999). These physical and chemical interactions between cement paste and aggregate influences the overall strength of lightweight aggregate concrete.

2.5.2. Tensile Strength

In general, tensile strength of concrete is considered as a function of compressive strength. However, this assumption do not take into account neither the strength and surface characteristics of aggregate nor the moisture content of concrete and its distribution. The effect of moisture condition and its distribution is more pronounced while determining the splitting tensile strength of lightweight aggregate concrete. According to ASTM C496, splitting tensile test is applied on lightweight concrete specimens which undergo 7 days of moist curing followed by 21 days of air-drying at 50% relative humidity. On the other hand, specimens of normal weight concrete is tested in moist condition after continuous moist curing. For normal weight concrete, testing specimens in moist condition gives more conservative results than testing in air-dry condition. However, for lightweight concrete, this is vice versa. The reason of this distinction is that moisture loss progresses slowly into the interior zones of lightweight concrete members and creates tensile stresses at the exterior zones, thus reduces the tensile resistance to external loading. Therefore, the tensile strength of lightweight concrete specimens that undergo some drying before testing show better correlation with field behaviour (Holm & Bremner, 2000).

Shear, torsion, anchorage, bond strength, and crack resistance are also related to tensile strength, which is determined by tensile strength of the coarse aggregate and mortar as well as the strength of the bond between these phases (Holm & Bremner, 2000). In normal weight concrete, when mortar matrix cracks, strong and intact normal weight coarse aggregates will continue to provide post-elastic strain capacity and resist splitting. This is almost the same for lightweight concretes with normal strength levels (20-35 MPa), where tensile strength and elastic rigidity of mortar and coarse aggregate phases are relatively close. However, in high strength lightweight concrete, mortar matrix is much stronger than the coarse aggregate. Resultantly, there may be only a slight contribution to post elastic strain capacity by lightweight coarse aggregate. Therefore, the correlations based on tensile strength in design codes which are normally established for normal weight concrete may not be valid for high strength levels of lightweight concrete due to its relatively lower post elastic strain capacity. It will be safer to limit the maximum strength levels for which the ACI 318 requirements govern shear, tension, torsion, development lengths, and seismic parameters to concrete compressive strengths no greater than 35 MPa unless compressive testing programs conducted on concretes with specific combinations of aggregates prove adequate performance at higher strength levels (Holm & Bremner, 2000).

Tensile and shear strength of lightweight concretes may be assumed to vary from 75 to 85 percent that of normal weight concrete, for all-lightweight and sanded lightweight concrete respectively (Holm & Bremner, 2000).

The flexural strength test is another indirect method to determine tensile strength of concrete. Hoff (1992) reported that flexural strength of high strength lightweight concretes was approximately 2/3 and 3/2 of the splitting tensile strength respectively for dry cured and moist cured specimens (as cited in Holm & Bremner, 2000).

2.5.3. Elastic Modulus and Poisson Ratio

Elastic modulus of concrete is governed by the elastic modulus of each constituents and their fractions in the mixture. Elastic modulus of lightweight concrete is lower than that of normal weight concrete, mainly due to lower rigidity of lightweight aggregates. It generally varies from 50 to 75 percent that of normal weight concrete at the same strength. Although there are some formulas suggested to estimate modulus of elasticity, actual results may deviate up to 25 percent due to variations in moisture content, aggregate type, etc. (Holm & Bremner, 2000).

Testing programs by resonance methods (Reichard, 1964) have shown that Poisson's ratio of lightweight concrete to be affected slightly by age, strength and aggregate type and varies from 0.16 to 0.25 (as cited in ACI Committee 213, 2003). Testing programs by static method also yielded in similar values of Poisson's ratio (ACI Committee 213, 2003). Generally, Poisson's ratio is assumed as 0.20 for practical design purposes.

2.5.4. Bond Strength and Development Length

Structural lightweight concrete has lower bond-splitting and post-elastic strain capacity due to lower aggregate strength compared to normal weight concrete (Holm & Bremner, 2000). This difference is more pronounced at higher strength levels. Therefore, design codes generally requires longer development lengths for structural lightweight concretes. For example, ACI 318 suggests to increase development length by a factor of 1.3 for structural lightweight concretes with unspecified splitting tensile strength. However, this increase may not be sufficient where closely spaced and larger diameter prestressing strands are used. Testing programs are advised for use of high strength lightweight concrete in special structures such as long span bridges and major offshore platforms (ACI Committee 213, 2003).

2.5.5. Abrasion Resistance

The strength, hardness and toughness of the cement paste, aggregates and bond between them determines the abrasion resistance of the concrete. Most of the structural lightweight aggregates are formed by solidified glassy material, hardness of which corresponds approximately to quartz on Moh's scale of hardness (hardness number 7). On the other hand, due to porous internal structure, impact resistance of lightweight aggregates are less than that of most of the normal weight aggregates. Structural lightweight concretes used on bridge decks exposed to heavy traffic load including trucks have shown satisfactory performance comparable to that of normal weight concretes, though, it may be necessary to set some limitations in applications where steel-wheeled vehicles are used (Holm & Bremner, 2000).

2.5.6. Shrinkage

Concrete shrinkage in general, is governed by shrinkage characteristics of cement paste, internal restraint provided by aggregate, aggregate volume ratio and ambient humidity and temperature. Aggregate properties such as particle shape and absorption capacity also affect the shrinkage by affecting the water requirement of the mixture.

In general, structural lightweight concrete has slightly higher shrinkage than that of the normal weight concrete with similar cement paste volume, due to lower stiffness of lightweight aggregates. On the other hand, shrinkage strain develops slowly in lightweight concrete and reaching an equilibrium condition takes more time due to internal curing (Holm & Bremner, 2000).

Curing methodology has a significant effect on drying shrinkage of lightweight concrete. Specimens cured with 1 day of steam and 6 days moist curing before exposure in laboratory conditions show approximately 20 percent less shrinkage strain than the standard 7-day moist-cured specimens (Holm & Bremner, 2000).

As stressed by Kulka and Polivka (1978), shrinkage and creep of concrete in real structures is much smaller than laboratory specimens. For example, when designing a lightweight concrete bridge, the shrinkage and creep values were reduced from the laboratory test results approximately 15-20 percent due to the size effect of the member, 10-20 percent due to the ambient humidity, and 10-15 percent due to the reinforcement, which corresponds approximately 50 percent reduction at total (as cited in Mehta & Monteiro, 2006).

2.5.7. Creep and Fatigue

Creep characteristics of a concrete is mainly determined by aggregate characteristics, cement paste volume fraction, curing method, age of concrete at the start of loading and the ratio of applied stress to strength and to a lesser extent, by other factors such as air-entrainment, specimen or member size and ambient humidity (Holm & Bremner, 2000). Creep can either be beneficial for the cases where stress concentrations are reduced by transfer of stress through creep, or detrimental by causing excessive deflections, prestress loss and loss of camber (ACI Committee 213, 2003).

As inferred from Figure 2.5 and 2.6, specifying higher design strength values and preferring steam curing to moist curing are very effective ways of reducing the creep of structural lightweight aggregate concrete.

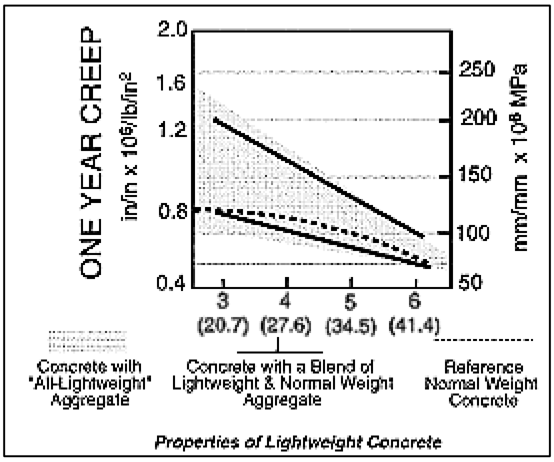

Figure 2.5. Creep of moist-cured lightweight concrete with respect to compressive strength (ACI Committee 213, 2003)

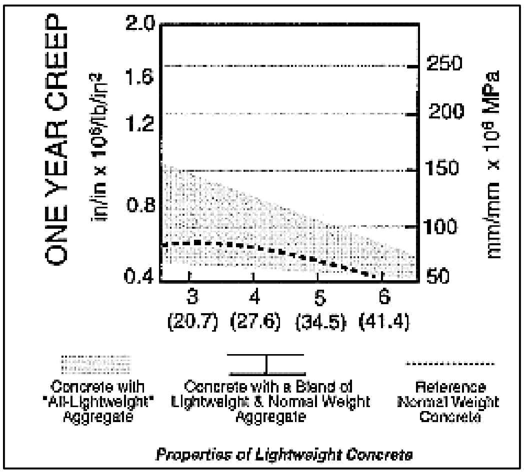

Figure 2.6. Creep of steam-cured lightweight concrete with respect to compressive strength (ACI Committee 213, 2003)

There are several studies that investigated the fatigue behaviour of lightweight concrete. Gray and McLaughlin (1961) have reported that fatigue characteristics of lightweight concrete is similar to that of normal weight concrete and do not vary considerably despite the large variations in strength (as cited in Holm & Bremner, 2000). Hoff (1994), after reviewing many studies conducted throughout North America and Europe, concluded that fatigue performance of high strength lightweight concrete is similar to that of high strength normal weight concrete and frequently provides longer service life under fatigue (as cited in Holm & Bremner, 2000).

2.5.8. Thermal Properties

The coefficient of thermal expansion is governed by expansion characteristic of aggregates, volume proportions of constituents and moisture content of the concrete (Holm & Bremner, 2000). In general, the coefficient of thermal expansion of lightweight concrete is lower than that of normal weight concrete (Neville, 2003). Depending on the type of the aggregate used, the coefficient of thermal expansion of lightweight aggregate concrete varies from 7 $\times10^{-6}$ to 11 $\times10^{-6}$ mm/mm/^oC. In comparison, the coefficient of thermal expansion for normal weight concrete varies from 6 $\times10^{-6}$ to 9×10^{-6} mm/mm/^oC for those with limestone aggregates and 9×10^{-6} to 13 $\times10^{-6}$ mm/mm/^oC for those with siliceous aggregates (ACI Committee 213, 1987).

It can be inferred that concretes with siliceous aggregates have a tendency to show greater expansion than those with limestone.

Thermal conductivity of structural lightweight concrete is about one half of normal weight concrete due to porous nature of lightweight aggregates. Replacement of normal weight sand with lightweight fine aggregates significantly reduces the thermal conductivity of sanded-lightweight concrete. For example, typical thermal conductivity of concretes with all expanded clay aggregate is about 2/3 of the concretes with expanded clay coarse aggregate with natural sand (Mehta & Monteiro, 2006).

2.5.9. Specific Strength

Specific strength of concrete, which is also referred as structural efficiency or strength efficiency, is the ratio of concrete compressive strength to concrete density. Depending on the moisture state of concrete in service condition, it can be either ratio of compressive strength to saturated density or more commonly as the ratio of compressive strength to air-dry density. In Figure 2.7, historical development of structural efficiency for several concrete types is illustrated.

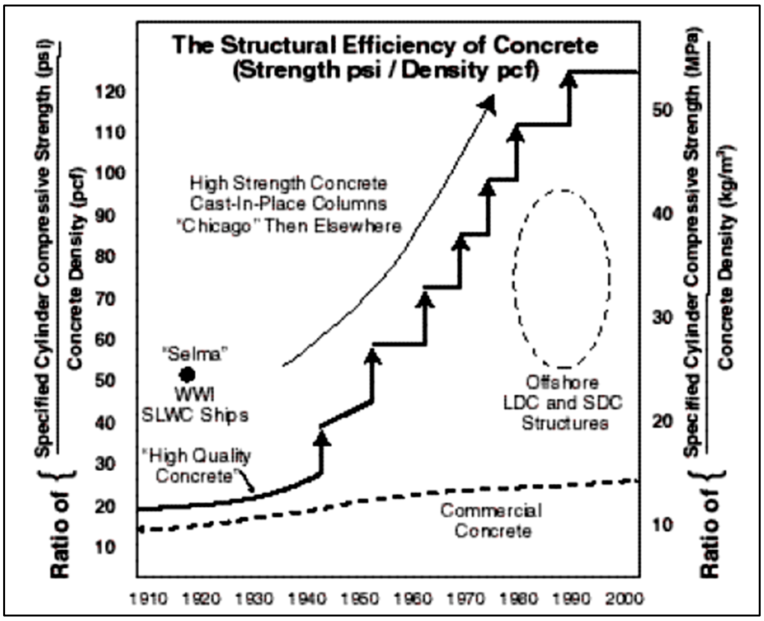

Figure 2.7. Historical development of structural efficiency of concrete (ACI Committee 213, 2003)

As shown in the Figure 2.7, structural lightweight concretes have been highly efficient compared to that of commercial normal weight concrete from past to present. For example, the structural efficiency of lightweight concrete used in USS Selma, which is a ship built in World War I, was only reached almost 40 years after by high strength normal weight concretes. Structural efficiency of concrete has been significantly improved since 1950s, mainly due to use of new generation high range water-reducing admixtures and high-quality pozzolans such as fly ash, metakaolin and silica fume although the first major breakthrough is by use of lightweight aggregates in concrete ships (ACI Committee 213, 2003). Today, structural efficiency of lightweight concrete lies between commercial concrete and high-strength concrete.

2.6. Durability Characteristics of Structural Lightweight Concrete

As with normal weight concrete, durability of the structural lightweight concrete is directly affected by its permeability. In general, concrete permeability is affected by many factors such as w/b ratio, cement type, curing, maturity of concrete, etc. The permeability of concrete as a whole is considerably higher than that of its components, namely the mortar matrix and coarse aggregates. According to Mehta (1986), this is mainly the result of microcracks caused by the elastic mismatch between these components responding differently to temperature changes, service loads and volume changes due to chemical reactions within concrete (as cited in ACI Committee 213, 2003).

Due to similar rigidities of mortar matrix and lightweight coarse aggregate (elastic compatibility), there are reduced number of microcracks observed in contact zone of lightweight concrete compared to that of normal weight concrete, which in turn results in lower permeability. In addition to this, hygrol equilibrium and pozzolanic reaction are two factors also contributing to improvement of the contact zone in lightweight concrete (ACI Committee 213, 2003).

Hygrol equilibrium can be defined as a state at which aggregate surface and mortar matrix have similar water concentration. In normal weight concrete, mixing water accumulates on the surface of the dense aggregate (wall effect) and increases local water-cement ratio, causing porous matrix at the contact zone. In contrast, porous surface of lightweight aggregate allows water transfer and thus avoids accumulation of water on aggregate surface. Therefore, hygrol equilibrium is reached and formation of weak zones caused by differential water concentration are prevented.

The pozzolanic reaction between silica rich surface of lightweight aggregate and calcium hydroxide formed by the hydration of Portland cement increases the density and strength of the interfacial transition zone.

For all these reasons, the contact zone in lightweight concrete is superior to that of normal weight concrete (ACI Committee 213, 2003) and thus less permeable. It should also be remembered that pore system in lightweight aggregates is generally discontinuous, therefore porosity of lightweight aggregates does not influence the permeability of concrete (Neville, 2003).

Since the permeability of the structural lightweight concrete is low, its durability to aggressive chemical solutions is usually quite satisfactory (Mehta & Monteiro, 2006). Sulfate containing groundwater and chlorides in sea water are some examples of the aggressive chemical solutions. Seawater also contains sulfates, however productions of sulfate attack are soluble in sea water due to presence of chlorides. Therefore, sulfates in seawater do not cause deleterious levels of expansion (Holm & Bremner, 2000).

Corrosion of reinforcement in concrete can be either induced by chlorides and carbon dioxide. Presence of these ions lowers the pH of the concrete pore solution, which causes loss of protective layer on steel reinforcement. Due to low permeability of lightweight concrete, chloride penetration into concrete is limited. On the other hand, carbon dioxide in air can diffuse into concrete through the pores in lightweight aggregate (Neville, 2003) and can cause carbonation induced corrosion. For that reason, it is often required to increase the thickness of the cover by an additional 10 mm (Neville & Brooks, 2010).

Another durability concern for concretes is alkali-aggregate reaction. There is no reported case of deleterious alkali-aggregate reaction in lightweight concrete with natural or manufactured lightweight aggregate (Holm & Bremner, 2000). Nevertheless, ACI Committee 213 (2003) recommends testing of natural aggregates against any potential for alkali-aggregate reaction or having a record of satisfactory service history.

Freezing-thawing resistance of structural lightweight concrete is superior to that of normal weight concrete provided that aggregates are unsaturated before mixing (Neville & Brooks, 2010). This performance is generally attributed to the porous structure of lightweight aggregates which act as pressure relief zones for increasing hydraulic pressure as the water freezes (Harrison, Dewar, & Brown, 2001). Air entrainment is especially beneficial when aggregates are close to saturation. Air-entrained lightweight concrete shows similar resistance against freezing-thawing action to that of air-entrained normal weight concrete (Mehta & Monteiro, 2006).

Fire resistance of structural lightweight concrete is also superior to that of normal weight concrete due to lower thermal conductivity, lower coefficient of thermal expansion and inherent thermal stability of aggregates which have already been subjected to very high temperatures during production (ACI Committee 213, 1987). However, this resistance is significantly reduced for lightweight concretes with low permeability in the case of having aggregates with high as-batched water contents (ACI Committee 213, 2003). This is because, upon exposure to fire, the water in these aggregates will vaporize but will not easily leave the concrete due to low permeability and thus increase the steam pressure within and finally results in spalling. In offshore oil platforms where there is risk of intense hydrocarbon fires, this problem clearly requires a solution.

Jensen at al. (1995) reported that even relatively low temperatures between 100 and 300^{o}C may cause significant amount of reduction in compressive strength and elastic modulus of high-strength lightweight concretes and added that spalling depends largely on the moisture content. They also suggested that inclusion of 0.1 to 0.2 percent polypropylene fibers in lightweight concrete mixture results in significant reduction of spalling for lightweight concretes (as cited in Holm & Bremner, 2000). Reduction in spalling is attributed to release of steam pressure through the conduits developed by the melting of the polypropylene fibers (ACI Committee 213, 2003).

2.7. Economy of Structural Lightweight Concrete

The high production costs of expanded lightweight aggregates increase the unit price of structural lightweight concrete. On structural level, however, lightweight aggregate concrete is generally cost efficient and provides significant economic advantages. The relatively high structural efficiency of structural lightweight concrete results in smaller member sizes and reduction in reinforcement requirement due to reduced dead load and corresponding reduction in seismic forces. The transportation cost of precast lightweight concrete members is also significantly less than that of normal weight concrete and this is the reason why the major applications of structural lightweight concrete throughout the world is by using precast elements (Mehta & Monteiro, 2006). Two examples of transportation cost analysis are given in Table 2.3.

Table 2.3. Transportation cost analysis for lightweight and normal weight concrete (ACI Committee 213, 2003)

	Project Example No. 1	Project Example No. 2
Shipping cost per truck load	$1100	$1339
Number of loads required		
Normalweight	431	87
Lightweight	287	66
Reduction in truck loads:	144	21
Transportation savings		
Shipping cost per load	$1100	$1339
Reduction in truck loads	× 144	× 21
Transportation savings:	$158,400	$28,119
Profit impact		
Transportation savings	$158,400	$28,119
Less: premium cost of lightweight concrete	17,245	3799
Transportation cost savings by using lightweight concrete	$141,155	$24,320

*Courtesy of Big River Industries, Inc.

Since the 1970s, rapidly increasing fuel costs has begun to out-market the manufactured lightweight aggregates by increasing production costs and rearoused the interest in natural lightweight aggregates with satisfactory quality (Mehta & Monteiro, 2006). The researches on using natural lightweight aggregates such as rhyolite, perlite and pumice, etc. in structural concrete production are now started to be carried out throughout the world.

2.8. Recent Studies on High Performance Lightweight Concrete

2.8.1. High Strength Lightweight Concrete

Al-Khaiat and Haque (1999) have compared the strength and durability characteristics of structural lightweight concrete and normal weight concrete under various curing conditions. They designed two lightweight concretes utilizing artificial aggregates and one normal weight concrete with crushed quartz aggregates. The nominal compressive strength of lightweight concretes were 35 and 50 MPa (referred as LWC35 and LWC50, respectively). The nominal compressive strength of the normal weight concrete was also 50 MPa (referred as NWC50). All three mixtures had a slump of about 9 cm. Fresh densities were approximately 1800 kg/m^3 and 2350 kg/m^3 for lightweight concretes and normal weight concrete, respectively. Four different curing regimes were applied on specimens, which are 1-day (no curing after removal of molds), 3-day (water curing for 2 days after removal of molds), 7-day (water curing for 6 days after removal of molds) and full curing (water curing till testing). After curing periods had ended, the specimens were moved and stored at an exposure site near sea and their strength parameters and durability characteristics such as water permeability, depth of carbonation, sulfate concentration and chloride penetration were investigated throughout 270 days. The authors underlined that to achieve same strength levels with normal weight concrete, lightweight mixture requires 10-20% more binder content. The results have also shown that LWC50 with 7-day curing regime has shown better strength development at 90-days than its continuously cured counterparts. The authors, by citing a previous work by Bamforth (1987), attributed this to “better densification of the interfacial transition zone due to absorption of aggregates”. As the moisture near the surface of the concrete begins to evaporate, the absorbed water within lightweight aggregates are released and promotes hydration at interior zones where external curing moisture cannot penetrate. Another conclusion which can be inferred from the study was that duration of curing period is more effective on the durability of lightweight concretes than that of normal weight ones. As the curing period extended, water penetration, depth of carbonation and chloride content of lightweight specimens were reduced more than that of normal weight concretes.

Chia and Zhang (2002) compared the water permeability and chloride penetrability of high-strength lightweight concretes with high-strength normal weight concretes. For this purpose, they designed three series of mixtures. In each series, there were two mixtures one of which was lightweight and the other was normal weight concrete. Both have same binder content and w/b ratio while the only difference was the type of coarse aggregates used, namely crushed granite and expanded clay. First series have w/c ratio of 0.55 and 400 kg/m^3 cement, second have w/c ratio of 0.35 and 470 kg/m^3 cement and the third have w/c ratio of 0.35 and 421 kg/m^3 cement and 47 kg/m^3 silica fume. The results have shown that water permeability of lightweight concrete in first series was lower than that of normal weight concrete. However, in second and third series, water permeability of normal and lightweight concrete was similar. This was attributed to the enhanced the quality of the mortar matrix due to reduction in w/b ratio from 0.55 to 0.35. In the light of these results, the authors concluded that that the quality of mortar is more dominant on controlling the water permeability than the type of aggregate used. The results also showed that the chloride permeability was reduced from first series to third series, with increasing mortar matrix quality for both normal and lightweight concrete mixtures and in each series chloride permeability of lightweight and normal weight concretes were similar. The authors concluded that at equal strength levels, lightweight concrete is expected to have higher resistance to water and chloride permeability, considering the lower strength but similar permeability performance of lightweight concretes compared to that of normal weight concretes in each series.

Chi et al. (2003) investigated the effect of aggregate properties and w/b ratio on the compressive strength and elastic modulus of lightweight concrete. Three types of fly-ash lightweight coarse aggregates differing in particle strength were used in the experiment. For each type, three series of concrete were cast with w/c ratios of 0.3, 0.4 and 0.5. Furthermore, each w/c series were divided into subseries with differing coarse-to-total aggregate volume fraction (18, 24, 30 and 36%). The compressive strength of the specimens varied roughly between 25 and 45 MPa while elastic modulus values were between 15 and 25 GPa. The results were predictable. The compressive strength and elastic modulus of the specimens increased with the increasing aggregate particle strength, decreasing w/c ratio and decreasing coarse-to-total aggregate volume fraction. The study has shown that aggregate properties and w/c ratio significantly affects the compressive strength and elastic modulus. The researchers also concluded that when coarse-to-total aggregate volume fraction is

18%, strength and elastic modulus are governed by w/c ratio and the effect of aggregate properties are insignificant.

Kayali and Zhu (2005) compared the chloride induced reinforcement corrosion of high-strength lightweight concrete (LWHS) with moderate strength normal weight concrete (MS) and high-strength normal weight concrete (HS). The 35th day compressive of HS and LWHS was about 70 MPa, whereas MS was only about 30 MPa. The reinforced slabs made of each type of the concretes were exposed to 2% chloride solution for more than 15 months and chloride ion ingress, corrosion potentials, corrosion current density and electrical resistivity were monitored throughout this duration. The results showed that the chloride ion concentration at the level of rebars were the lowest in LWHS and followed respectively by HS and MS. Half-cell potential values of LWHS were more negative than MS and accompanied by insignificant corrosion current, thus attributed to high impermeability and lack of oxygen in the LWHS slabs. The electrical resistivity of HS and LWHS were similarly very high and remained almost unaltered with time. On the other hand, the electrical resistivity of MS was much lower and further decreased with time. The researchers attributed the superior performance of LWHS to its impermeable dense matrix and porous lightweight aggregates which are thought to act as reservoirs for chloride solution.

Mouli and Khelafi (2008) studied the effects of using pozzolan on some mechanical properties of lightweight aggregate concrete. The lightweight aggregates used in the study obtained from the natural deposits in Algeria. The pozzolan used in the study was obtained by grinding same lightweight aggregates to a fineness of 4200 cm^2/g. For assessing the effect of pozzolans, six mixtures with a total binder content of 400 kg/m^3 were designed. First mixture was the control group containing only cement as binder. In other mixtures, cement was replaced with pozzolan by 10, 20, 30, 40 and 50%, respectively. The compressive strength, splitting and flexural tensile strength of the specimens were monitored throughout 1 year. The results showed that specimens with 20% pozzolan showed higher compressive, splitting and flexural tensile strength than reference specimens at all ages starting from 7-days to 365 days. The authors underlined that increase in splitting and flexural tensile strength may increase the service life of concrete by reducing cracking tendency. The specimens containing 30% pozzolan achieved higher compressive strength than that of reference specimens at 90 days. The specimens containing 40% and 50% pozzolan showed lower compressive strength than reference specimens at all ages. Therefore, authors suggested not to use high pozzolan contents unless low heat of hydration or durability is concerned.

Shannag (2011) conducted a study to assess the effect of pozzolan addition on fresh and hardened properties of lightweight concrete utilizing volcanic tuff as lightweight aggregate. For this purpose, binary and ternary lightweight concrete mixtures were prepared with fly ash and silica fume. Total binder and water content of the mixtures were fixed to 400 kg/m^3 and 250 kg/m^3, respectively. The aggregate content and type were identical in all mixtures. The 28th day compressive strength of designed concretes varied roughly from 20 to 45 MPa and air-dry densities were less than 2000 kg/m^3. The results showed that the lightweight concrete with 15% silica fume developed the highest compressive strength and elastic modulus with an increase of 57% and 14% over reference specimen, respectively. This was attributed to the improvement of contact zone by pozzolanic reactivity and filler effect. In the scope of the study, the researcher also plotted and evaluated the complete stress-strain diagram of the specimens under compression. The stress-strain diagrams of the structural lightweight concretes were similar to that of typical normal weight concretes. However, it was stated that the strain capacity of lightweight concrete specimens were comparably higher.

In a study by Kabay and Aköz (2012), the effect of aggregate prewetting methods on properties of lightweight concrete was investigated. For this purpose, the researches designed two series of sanded lightweight mixtures with cement contents of 350 kg/m^3 and 500 kg/m^3. The lightweight aggregates (pumice) were prewetted before batching by three different methods, namely pre-soaking, water-soaking and vacuum-soaking. Firstly, in pre-soaking method, lightweight aggregates and pre-soak water, which corresponds to 1 hour absorption capacity of the aggregates, are introduced in to a mixer and allowed to rest for half an hour. During this resting period, aggregates are mixed three times for "homogenization". This is followed by batching and casting procedures in which aggregates are assumed to absorb water for an additional 30 minutes. Secondly, in water-soaking method, lightweight aggregates are simply immersed in water for 24 hours. Then, these wet aggregates are spread on sieves for drying of surface water. Thirdly, in vacuum-soaking method, aggregates are placed in to a container and the air in container is evacuated by means of a pump until the pressure inside is reduced to -650±10 mm Hg. Then, the container is filled with water and aggregates are allowed to rest for 10 minutes. After the resting period, wet aggregates are spread on sieves for drying of surface moisture. The experimental results have shown that the slump of concretes with water-soaked and vacuum soaked aggregates were close to each other and higher than those with pre-soaked. This was attributed to lower fresh density of concretes with pre-soaked aggregates. This is because the pre-soaked aggregates are relatively less saturated than water-soaked and vacuum soaked aggregates, their density in batch condition is also lower and so as the fresh density of resulting concretes. The study has also shown that the compressive strength of the concretes with water-soaked and vacuum-soaked aggregates were approximately 20% higher than that of concretes with pre-soaked aggregates in series containing 350 kg/m^3 cement. On the other hand, this difference was reduced to 7% in series with 500 kg/m^3 cement. The authors attributed this to reduced lightweight aggregate content in these series. The drying shrinkage of concretes with water-soaked and vacuum soaked aggregates were also lower than those with pre-soaked aggregates. This was attributed to internal curing provided by relatively higher absorbed water content of water-soaked and vacuum-soaked aggregates. The authors concluded that concretes with water-soaked and vacuum-soaked aggregates show better overall performance. It was also underlined that water-soaking is advantageous when economy is concerned while vacuum-soaking is favorable for its considerably shorter application time.

2.8.2. Self-Compacting Lightweight Concrete

Yanai et al. (1999; as cited in Papanicolaou & Kaffetzakis, 2011) have studied on self-compacting lightweight concrete utilizing artificial perlite and coal ash aggregates. The study showed that by adjusting lightweight aggregate content and water-to-powder ratio of the mixtures, self-compacting concretes with good flowability and segregation resistance can be designed. It was concluded that using higher density lightweight aggregates enhances flowing and filling properties of fresh mixtures and increases the strength of hardened concrete. The researchers attributed this result to the reduction of density difference between aggregates and paste.

In an extensive study conducted by Müller and Haist (2004; as cited in Papanicolaou & Kaffetzakis, 2011), self-compacting concretes with densities varying from 1500 kg/m^3 to 2000 kg/m^3 were designed by using expanded clay coarse aggregates. The fresh densities was adjusted by replacement of normal weight sand with expanded clay or bottom ash sand. The authors stressed that to achieve self-compactness, minimization of density difference is not adequate. The rheological performance of the mixtures should also be optimized such that its yield stress and plastic viscosity will be low enough to sustain high flowability and de-aeration, but also high enough to avoid segregation.

Hwang and Hung (2005), as an alternative to ACI 211.2, have developed a new mix design method for proportioning self-compacting lightweight concrete mixtures, which is called as densified mixture design algorithm (DMDA). DMDA is based on the assumption that optimum values of physical properties will be achieved when the constituents are densely packed. To achieve this, DMDA try to reduce interstitial voids between aggregates by arranging the relative proportions of aggregates, cement and pozzolans. The primary purpose of using pozzolans in this method is to reduce interstitial voids rather than its utilization as a cement replacement material. By using DMDA, the researchers managed to produce self-compacting lightweight concretes with slumps and slump flow spreads between 230-270 mm and 550-650 mm, respectively. The 28th compressive strength values varied roughly between 30 and 50 MPa. They also investigated the chloride penetration and electrical resistance of the designed concretes and concluded that the denser packing of the aggregates and resultant reduction in cement paste volume lowers permeability and increases electrical resistance.

Lo et al. (2007) compared the workability characteristics and mechanical properties of normal weight self-compacting concrete (SCC) and lightweight self-compacting concrete with expanded shale aggregates (SCLWC) at same binder content and similar compressive strength. Seven mixtures were prepared for each group (SCC and SCLWC) and the binder content for both groups varied between 500 and 650 kg/m^3. To achieve similar compressive strength at same binder content; w/b ratio and cement replacement percentage of self-compacting lightweight concrete mixtures were respectively set to 0.30 and 30% compared to 0.40 and 50% for its normal weight counterpart. The slump flow and L-box test results were similar for SCLWC and SCC mixtures. The authors suggested that increasing the binder content more than 550 kg/m^3 is not effective for further improving filling and passing abilities of mixtures although the fluidity is increased. The compressive strength results varied from 40 to 60 MPa with increasing binder content. The results have also shown that elastic modulus of SCLWC was approximately 80% of SCC at similar strength and the density of SLWC mixtures was around 75% of SCC.

Uygunoğlu and Topçu (2009), have investigated the thermal expansion of self-compacting concretes with limestone and pumice aggregate at elevated temperatures. According to this study, the coefficient of thermal expansion of self-compacting lightweight concrete containing pumice aggregate has been found to be significantly less than that of its limestone containing normal weight counterpart. This was attributed to reduced internal thermal stresses due to porous structure of pumice aggregate. The authors also suggested use of air-entraining agents in self-compacting mixtures with limestone aggregates to reduce its thermal expansion.

Kim et al. (2010) studied the effect of replacing normal weight coarse aggregates with lightweight aggregates on the fresh and hardened properties of self-compacting concrete. For this purpose, two types of lightweight coarse aggregates with different densities were used in the experiment and nine mixtures with constant w/c ratio and fine-to-total aggregate volume ratio were designed. First mixture was the control group which consists of all-normal weight aggregate. Four groups utilized the lightweight coarse aggregates manufactured from rhyolite powder and the remaining four groups utilized the lightweight aggregates obtained from recycled sludge by 25, 50, 75 and 100 percent of the total coarse aggregate content. The study has shown that as the density of lightweight coarse aggregate decreases, the flowability increases but the mixture becomes more liable to segregate.

Topçu and Uygunoğlu (2010) investigated the effect of aggregate type on hardened properties of self-compacting concrete such as compressive and tensile strength, elastic modulus, abrasion resistance and thermal conductivity. Four types of aggregates used in the study were limestone and natural lightweight aggregates pumice, volcanic tuff and diatomite. Five mixtures were prepared by using each type of the aggregates with different w/b ratio and superplasticizer dosage but with similar powder content about 550-600 kg/m^3. It was reported that self-compacting concrete with diatomite aggregates had shown the highest slump-flow. This was attributed to fact that coarse aggregate content of diatomite containing concretes was the lowest among all other types. It was also stated that regardless of the aggregate type, increase in w/b ratio results in larger slump-flow spreads due to consequent reduction in yield strength of the fresh mixture. Pumice containing self-compacting concrete had the highest compressive strength and lowest unit weight among other lightweight mixtures, although its particle strength was weaker than tuff. The superior performance of pumice containing concrete was attributed to strong interlocking between cement paste and porous surface of the pumice (Figure 2.8). Another conclusion drawn from the study was that both self-compacting normal and self-compacting lightweight concretes have higher thermal conductivity than their ordinary counterparts, which was, by authors, attributed to high powder contents of self-compacting mixtures.

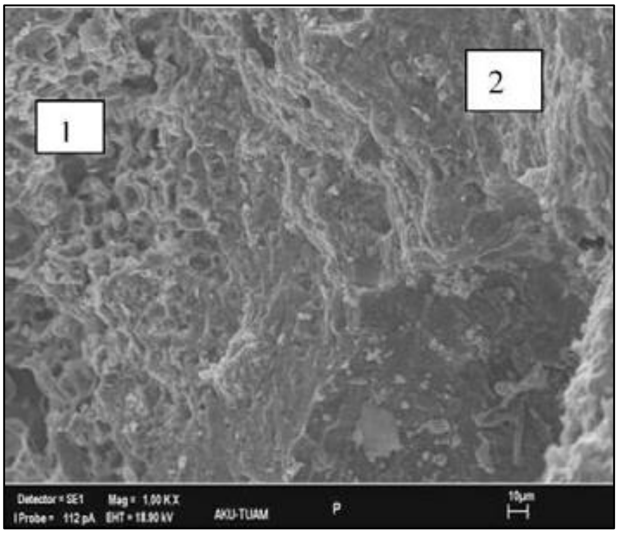

Figure 2.8. Microstructure of self-compacting concrete with natural pumice aggregate (1: aggregate, 2: cement paste matrix) (Topçu & Uygunoğlu, 2010)

In a state of art report by Papanicolaou and Kaffetzakis (2011), the self-compacting lightweight mixtures reported in 16 scientific papers, which are published between 1999 and 2010, were analyzed. The slump-flow ranges and 28^{th} day compressive strengths of these mixtures are statistically shown in Fig 2.9 and 2.10, respectively.

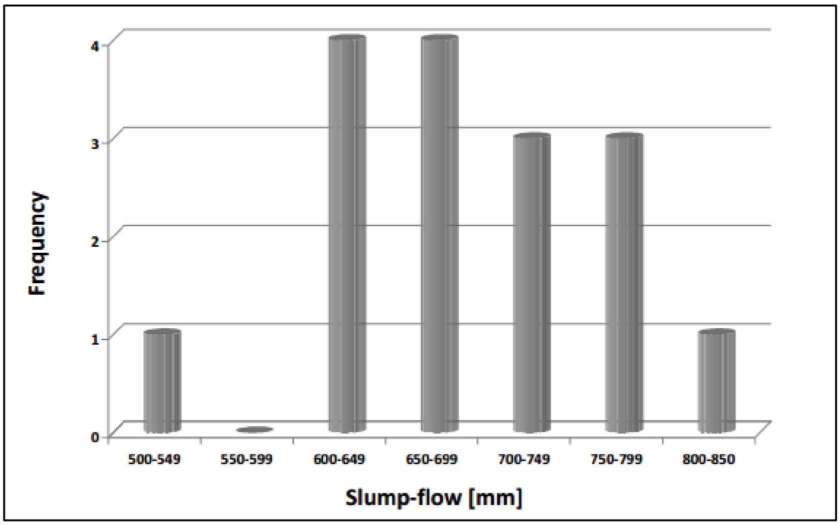

Figure 2.9. Frequency bar chart for slump-flow of self-compacting lightweight mixtures (Papanicolaou & Kaffetzakis, 2011)

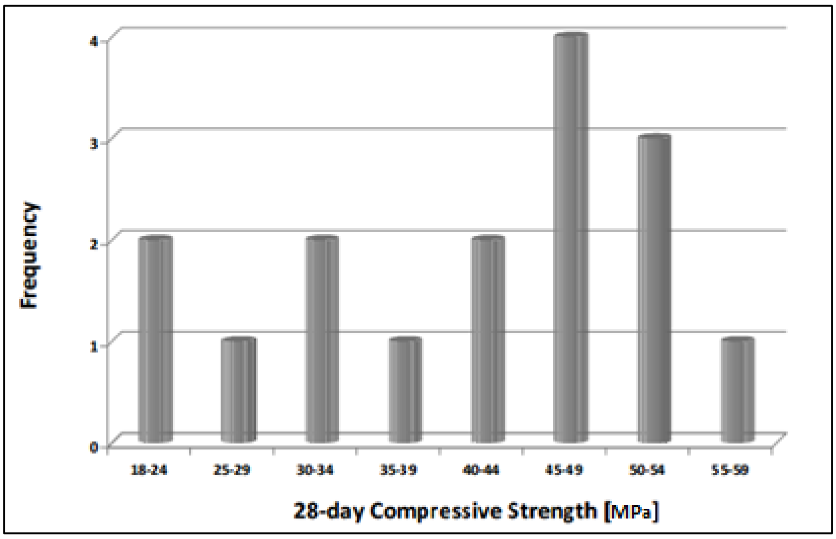

Figure 2.10. Frequency bar chart for 28-day compressive strength of self-compacting lightweight mixtures (Papanicolaou & Kaffetzakis, 2011)

From these charts, it can be concluded that half of the self-compacting lightweight mixtures showed slump flows between 600 and 700 mm and about 60% of these concretes can be considered as high strength lightweight concrete as their compressive strengths are higher than 40 MPa.

Figure 2.11 and 2.12 illustrates the specific strength and its relation to binder content, respectively. By analyzing these charts together, it can be inferred that average specific strength is about 25×10^{-3} MPa / (kg/m^3) and generally achieved with a total binder content of 500-600 kg/m^3.

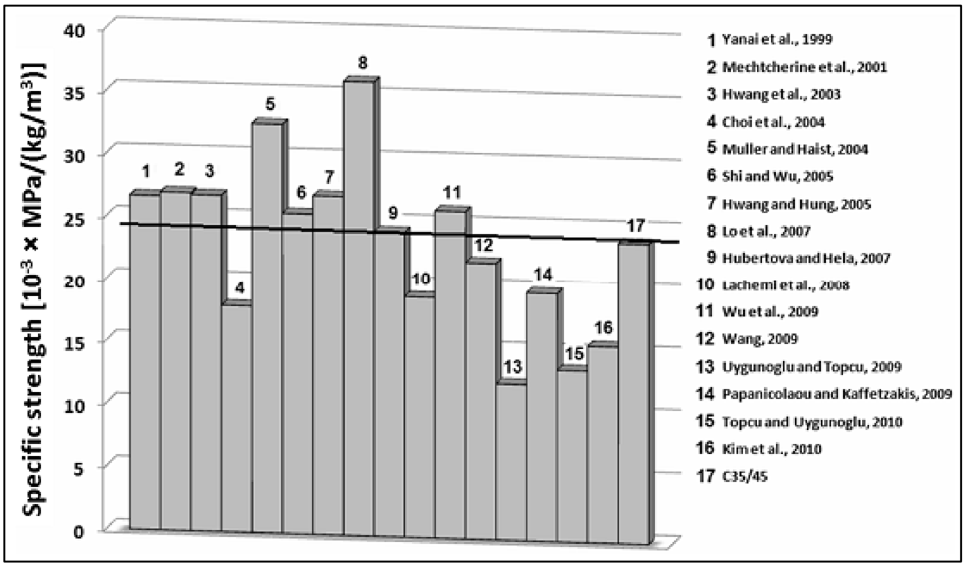

Figure 2.11. Frequency bar chart for specific strength of self-compacting lightweight mixtures (Papanicolaou & Kaffetzakis, 2011)

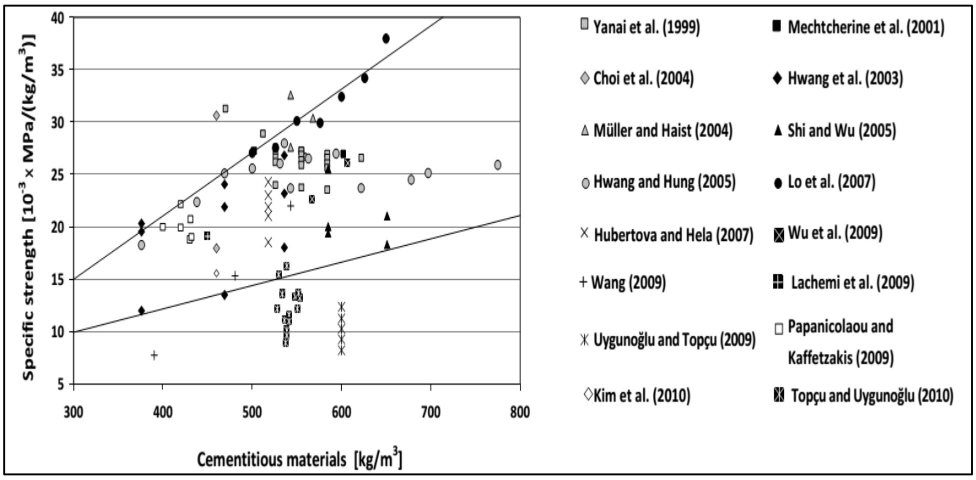

Figure 2.12. Relationship between binder content and specific strength (Papanicolaou & Kaffetzakis, 2011)

CHAPTER 3

EXPERIMENTAL STUDY

3.1. Experimental Program

The aim of the experimental study conducted in this thesis was to design high performance lightweight concretes and to compare their mechanical properties and durability characteristics with normal weight concrete of similar specific strength.

For this purpose, three different concrete mixtures were prepared. These mixtures were designed such that the 28th day specific strength of these concretes would be comparable. The aim here was to set a reference point for the comparison of the test results.

First mix was proportioned as a high strength lightweight concrete utilizing natural perlite as both coarse and fine aggregate. Second mix was proportioned as a self-compacting high-strength lightweight concrete utilizing high volume perlite powder as pozzolan and natural perlite as both coarse and fine aggregate. Third mix was proportioned as a high strength normal weight concrete with limestone aggregate.

In the course of this study, following properties of concrete were investigated:

- Fresh properties such as slump, slump flow, unit weight and setting time;
- Hardened properties such as compressive strength, splitting tensile strength, flexural tensile strength, elastic modulus and thermal coefficient of expansion;
- Durability characteristics such as resistance to rapid chloride ion penetration, aggressive chemical solutions and freezing-thawing cycles.

3.2. Material Properties

3.2.1. Portland Cement

In the experiments throughout this study, CEM I 42.5 R type Portland cement was used. The chemical composition, physical properties and compressive strength of mortar cubes made by using this cement are as shown in Table 3.1.

Table 3.1. Properties of the CEM I 42.5 R type Portland cement

CEM I 42.5 R type PC	
Chemical Composition*, %	
CaO	63.91
SiO_2	20.23
Al_2O_3	5.15
Fe_2O_3	3.22
MgO	1.40
SO_3	2.81
K_2O	0.57
Na_2O	0.15
Cl^-	0.001
LOI	2.45
IR	0.25
Physical Properties*	
Specific Gravity	3.16
Blaine Fineness, cm^2/g	3443
Initial Set, min	120
Final Set, min	175
Compressive Strength** (MPa)	
3-days	32.9
7-days	44.8
28-days	57.0

*As provided by the quality-control department of Limak Çimento, Ankara.
** Specimens were prepared and tested in accordance with ASTM C109.

3.2.2. Perlite Powder

The perlite used in this study was supplied from the natural resources at Erzincan-Mollaköy. The perlite powder was obtained by grinding of natural perlite sand (sized up to 2 mm) in a ball-mill (D=42 cm, L=45 cm) until approximately 80% of the ground material can pass through 45µm sieve, when wet sieved. In each grinding session, 10 kg of perlite sand introduced into mill. Grinding media were small steel balls and cylinders of various sizes. Perlite sand to grinding media ratio was 1:7 by mass. Grinding time varied from 3 hours to 3 hours and 15 minutes. The chemical composition, physical properties and strength activity index of the perlite powder is given in Table 3.2.

Table 3.2. Properties of the perlite powder

Perlite Powder	
Chemical Composition*, %	
SiO_2	70.96
Al_2O_3	13.40
Fe_2O_3	1.16
MgO	0.28
CaO	1.72
Na_2O	3.20
K_2O	4.65
LOI	3.27
Physical Properties	
Specific Gravity	2.38
Blaine Fineness, cm^2/g	4267
Passing 45µm, %	82
Strength Activity Index**, %	
7-days	85.5
28-days	91.3

* As taken from Aşık (2006).

** Strength activity index was determined in accordance with ASTM C311.

Table 3.3 illustrates the compressive strength of the mortar cubes prepared by using perlite powder as pozzolan for different percentages of replacement and throughout 1 year duration.

Table 3.3. Compressive strength of mortar cubes utilizing perlite powder as pozzolan

% replacement	Compressive Strength (MPa)					
	3-days	7-days	28-days	90-days	180-days	360-days
0	32.9	44.8	57.0	61.8	64.6	66.4
20	27.8	38.3	52.0	59.4	62.0	64.2
30	22.7	30.9	45.6	51.8	54.7	58.8
40	16.5	23.3	38.2	43.9	47.8	54.1
50	14.0	20.3	31.2	37.5	45.3	50.4

In Figure 3.2, the change in strength activity index of perlite powder with respect to time and replacement level is shown. The strength activity development for mortar cubes with 20% and 30% replacement almost levels at 90 days. On the other hand, for mortar cubes with 40% and 50% replacement, strength activity development continues even after 180 days. It can also be inferred from the graph that there is a significant drop in the rate of strength activity development for mortar cubes with 50% replacement after 180 days. This may be attributed to deceleration of the pozzolanic reactions due to exhaustion of calcium hydroxide by high volume perlite powder.

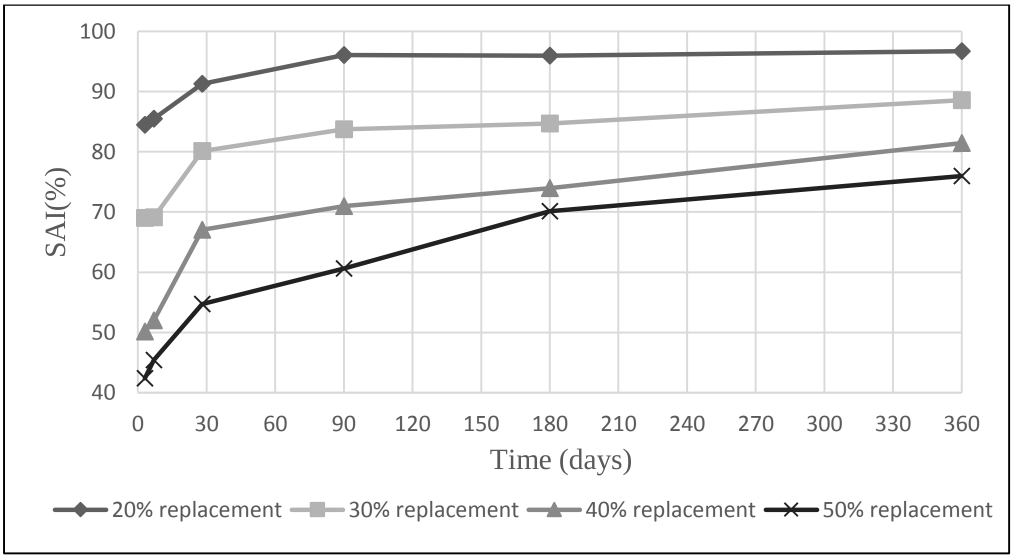

Figure 3.1. Strength activity index of perlite powder versus time

3.2.3. Natural Perlite Aggregate

The perlite aggregate used in the study was brought to laboratory in five commercial sizes (perlite sand in sizes of 0-2 mm, 0-3 mm, 0-4 mm and coarse perlite aggregate in sizes of 4-8 mm, 8-12 mm) as supplied by ER-PER. Physical properties of the perlite aggregate were provided in Table 3.4.

Table 3.4. Physical properties of natural perlite aggregate

Aggregate Size (mm)	0-2	0-3	0-4	4-8	8-12
Dry-Loose Unit Weight (kg/m^3)	1286	1288	1322	1025	1002
Oven Dry Specific Gravity	2.09	2.06	1.99	1.89	1.93
Saturated-Surface Dry Specific Gravity	2.21	2.18	2.15	2.00	2.04
Water Absorption Capacity (%) - 72 hr.	5.45	5.64	7.79	6.14	5.59
No.200 Sieve - % Passing	10.44	11.64	8.75	-	-
Los Angeles Abrasion (%)	-	-	-	49.7	

Gradation curves of perlite aggregate for each commercial size are shown in Figure 3.3. In the production of high performance lightweight concretes, a combined gradation was used, which consists of 0-4 mm, 4-8 mm and 8-12 mm sized aggregates with fractions of 55%, 25% and 20% by mass, respectively. This gradation falls within the limits stated for combined lightweight aggregates in ASTM C330 and its curve can also be seen in Figure 3.3.

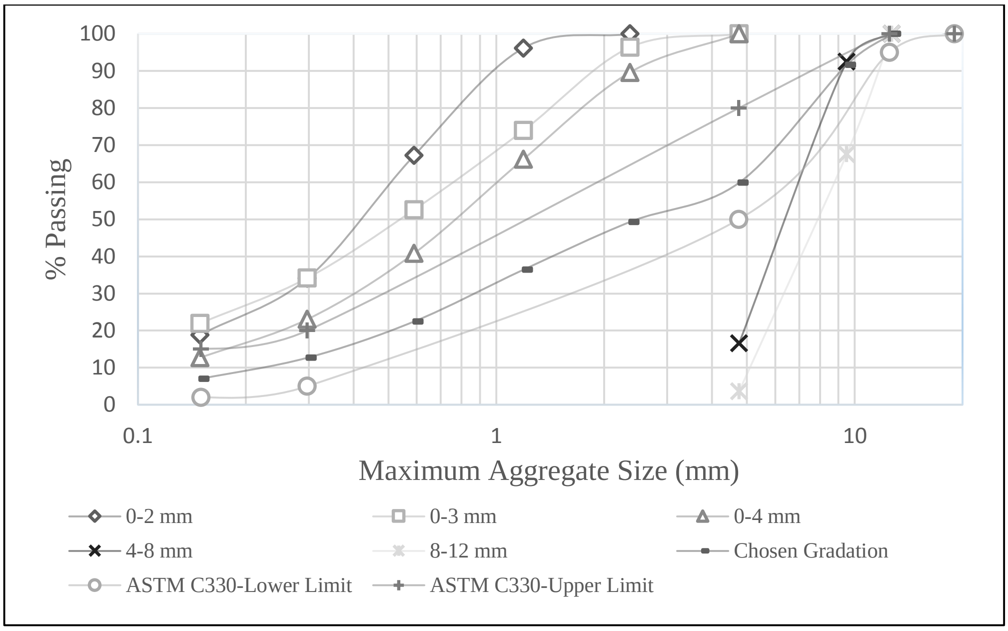

Figure 3.2. Gradation curve of perlite aggregate

Alkali-silica reactivity of the perlite aggregate as determined by ASTM C1260 is shown in Figure 3.4. As it can be seen from the Figure 3.3, corresponding expansion is well under the maximum limit defined by ASTM standards (0.1%).

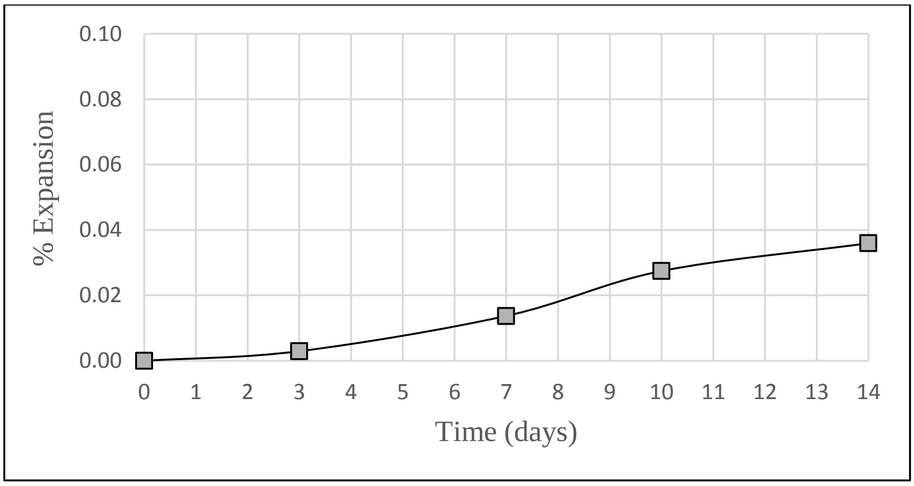

Figure 3.3. ASR expansion of perlite aggregate as determined by ASTM C1260.

3.2.4. Limestone Aggregate

The limestone aggregate used in the study was brought to laboratory in three commercial sizes (0-5 mm, 5-15 mm, 15-25 mm) as supplied by BAŞTAŞ. Physical properties of the limestone aggregate are provided in Table 3.5.

Gradation curves of limestone aggregate for each commercial size are shown in Figure 3.5. In the production of high strength normal weight concrete, a combined gradation was used, which consists of 0-5 mm, 5-15 mm and 15-25 mm sized aggregates with fractions of 57%, 23% and 20% by mass, respectively. The curve of this gradation can also be seen in Figure 3.5.

Table 3.5. Physical properties of limestone aggregate

Aggregate Size (mm)	0-5	5-15	15-25
Dry-Rodded Unit Weight (kg/m^3)	1880	1549	1534
Oven Dry Specific Gravity	2.59	2.67	2.71
Saturated-Surface Dry Specific Gravity	2.65	2.69	2.72
Water Absorption Capacity (%) - 24 hr.	2.27	0.53	0.33
No.200 Sieve - % Passing	9.57	-	-
Los Angeles Abrasion (%)	-	26	

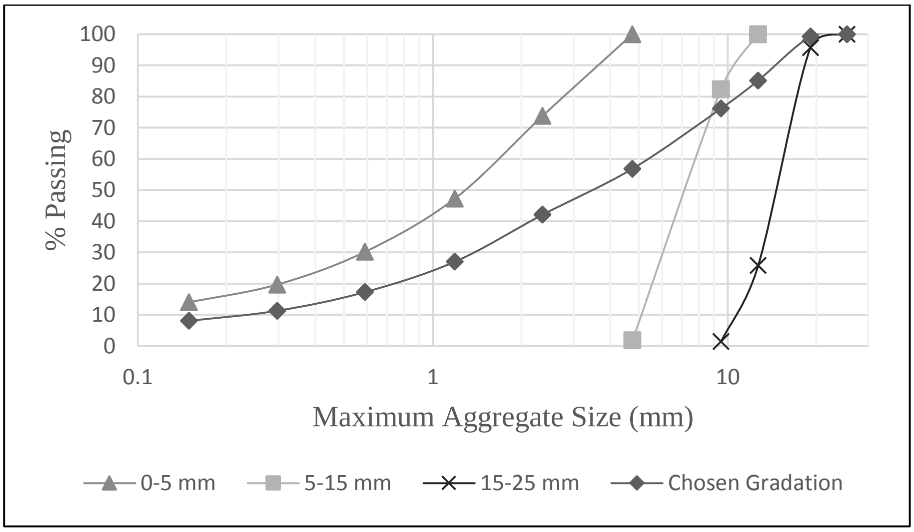

Figure 3.4. Gradation curve of limestone aggregate

3.2.5. Superplasticizer

In the production of all concretes within the scope of this thesis, a high performance superplasticizer (ViscoCrete SF-18) was used at a dosage of 0.4% of the binding medium by mass. The technical properties of this admixture are given in Table 3.6.

Table 3.6. Technical properties of ViscoCrete SF-18 (Sika, 2007)

Chemical Base	Modified polycarboxylate based polymer
Density	1.10±0.02 g/cm^3, 20^{o}C
pH	3-7
Freezing Point	-10^{o}C
Soluble in Water Chloride Ion Content, %	Max. 0.1%

3.3. Experimental Procedures

3.3.1. Preparation of Concrete Specimens

For the mixing of concrete-making materials, the instructions given in ASTM C192, the product manual of the superplasticizer and the absorption characteristics of lightweight aggregates have been taken into account while determining the mixing procedure.

The procedure of mixing have been summarized below:

1. Aggregates and one half of the mixing water were introduced into mixer.
2. Mixer was started to rotate.
3. Aggregates were allowed to absorb water for 3 minutes duration as the mixer continues to rotate, in order to decrease slump loss due to water absorption.
4. Cementitious materials were introduced into mixer.
5. As recommended in the manual of the superplasticizer, ingredients were allowed to mix for an additional 60 seconds before the addition of the remaining half of the mixing water and the superplasticizer dispersed in it.

6. Then, the remaining half of the mixing water and the superplasticizer dispersed in it were introduced into the mixer gradually.
7. Mixing was continued until homogeneity of the fresh concrete was ensured.

After mixing, two types of molds were used for casting. First one is for cylindrical specimens of 200 mm in length and 100 mm in diameter. Second one is for prism specimens of 75x75x320 mm size.

The molds were filled in two equal layers and each layer was compacted. For the compaction of the specimens, concrete vibrator was used; except for the specimens of self-compacting lightweight concrete which already has, as its name implies, self-compacting ability.

After consolidation, excess concrete was struck off and finishing was done by the help of a trowel. Followed by finishing, the specimens were covered with a moist-burlap to avoid evaporation of water. The specimens were removed from the molds 24 hours after casting and stored in potable water at 23±2^{o}C till testing.

3.3.2. Tests on Fresh Concrete

3.3.2.1. Slump Test (ASTM C143)

Workability can be defined as an ability of fresh concrete to transport, place, compact and finish without any harmful segregation. It is a composite property that consists of consistency and cohesiveness. Consistency is, in simple words, the fluidity of a fresh mixture; and cohesiveness is the stability of the fresh mixture, in other words, resistance to bleeding and segregation.

Due to ease of testing, both in the field and in the laboratory, slump test is widely used in measuring the consistency of the fresh concrete. It is also suitable for checking batch-to-batch uniformity of ready-mixed concrete (Mehta & Monteiro, 2006).

To measure the slump in accordance with ASTM C143, a sample of fresh concrete is taken and placed in a slump cone (Figure 3.6) in three approximately equal layers. Each layer is tamped 25 times by using a rod before another layer is introduced into the mold. When the mold is filled, the excess concrete is struck off by rolling motion of the rod. Then, the mold is removed vertically. Afterwards, slump is measured as the vertical distance between the top of the mold and the top of the sample.

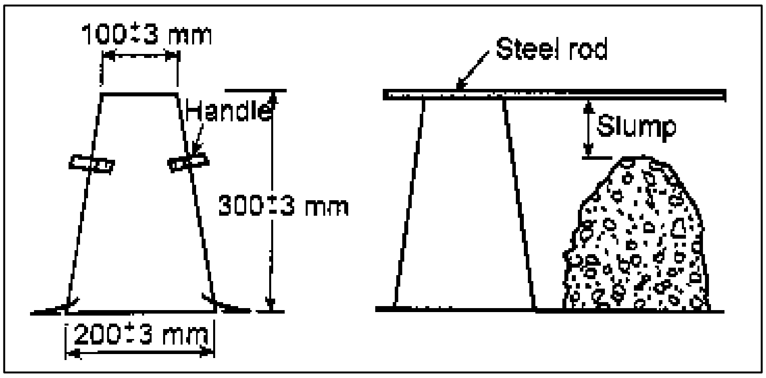

Figure 3.5. Slump cone and measurement of slump (Erdoğan, 2005)

Required slump value can vary from one type of the construction to another. Table 3.7 shows the recommended slumps for various types of construction for structural lightweight aggregate concrete.

Table 3.7. Recommended slumps for various types of construction (ACI Committee 211, 2004)

Types of construction	Slump (mm)	
Beams and reinforced walls	100	25
Building columns	100	25
Floor slabs	75	25

3.3.2.2. Slump-flow (ASTM C1611)

Slump-flow test is a test method to measure the consistency of self-compacting concrete. The procedure of this test is very similar to that of the slump test except that no tamping is applied to the sample in the mold and the mold can be used either upright or inverted. Additionally, test should be performed on a level and nonabsorbent surface moistened with a damp towel. When the mold is removed vertically, fresh concrete is allowed to spread. When flowing is stopped, flow is calculated as the average of the largest spread (d_1) and the spread that is perpendicular to largest spread (d_2).

$$\text{Slump-flow} = (d_1+d_2)/2 \quad \text{(Equation 3.1)}$$

In European Guidelines for Self-Compacting Concrete (EFNARC, 2005), slump-flow classes for a range of applications have been defined. This classification is shown in Table 3.8.

Table 3.8. Slump-flow classes (EFNARC, 2005)

Class	Slump-flow (mm)
SF1	550-650
SF2	660-750
SF3	760-850

SF1 class is favorable in housing slabs, tunnel linings, piles and deep foundations, whereas SF2 class is applicable to walls and columns. SF3 class is appropriate for vertical applications in very congested structures, complex shaped structures and for filling operations under formwork (EFNARC, 2005).

During slump flow test, viscosity can also be evaluated by the T_{500} time. It is the time passing between the removal of mold and the spread of the fresh concrete reaches to 500 mm. It is also useful for checking batch-to-batch uniformity of SCC, together with slump-flow. Viscosity classes defined by T_{500} time are given in Table 3.9.

Table 3.9. Viscosity classes (EFNARC, 2005)

Class	T_{500} (s)
VF1	≤ 2
VF2	> 2

VF1 is suitable for heavily reinforced sections. It has also self-levelling ability and good surface finishing. On the other hand, it may have a tendency to bleeding and segregation. VF2 has improved segregation resistance but likely to have problems with surface finishing (e.g. blow holes) (EFNARC, 2005).

In ASTM C1611, it is stated that stability of self-compacting concrete can be evaluated by visual inspection. Furthermore, a visual stability index (VSI) is given to classify the stability of self-compacting concrete (Table 3.10). In Figure 3.7, fresh concrete spreads corresponding to each VSI value have also been illustrated.

Table 3.10. Visual stability index (ASTM C1611)

VSI Value		Criteria
0	Highly Stable	No evidence of segregation and bleeding
1	Stable	No evidence of segregation, slight bleeding as a sheen
2	Unstable	A slight mortar halo < 10 mm and/or aggregate pile
3	Highly Unstable	Large mortar halo > 10 mm and/or large aggregate pile

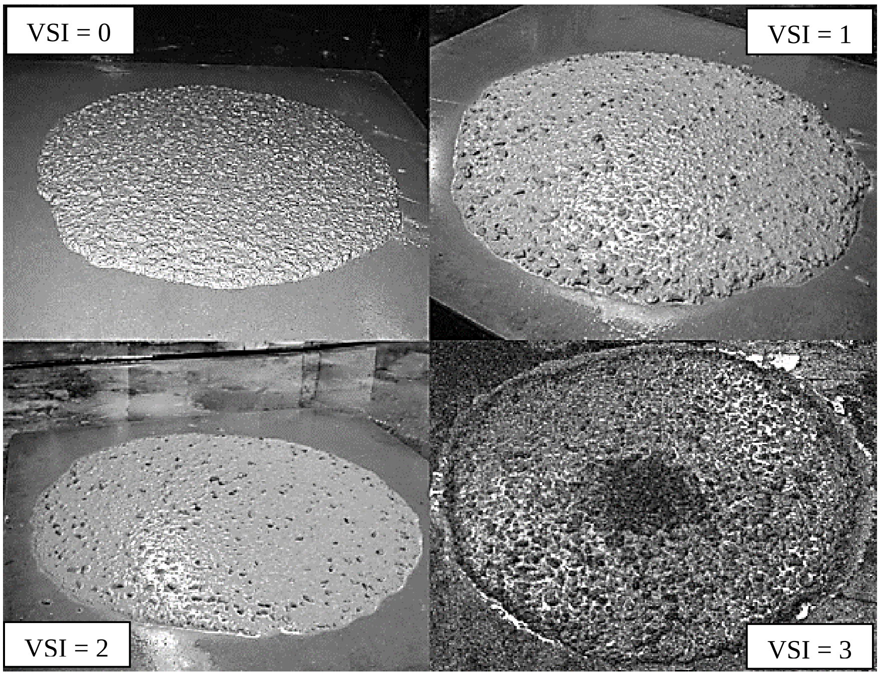

Figure 3.6. Illustration of visual stability index (ASTM C1611)

3.3.2.3. Density (ASTM C138)

In this test, a steel container of a known volume is filled with freshly mixed concrete in three approximately equal layers. Then, the same consolidation practice in slump test is utilized. Each layer is rodded 25 times. During rodding 2^{nd} and 3^{rd} layer, rod is penetrated approximately 2.5 cm into previous layer. After the container is filled, excess concrete is struck off. Then, the container filled with fresh concrete is weighed (M_c). Knowing the self-mass (M_m) and volume of the container (V_m), density of the concrete can be calculated as follows:

$$\text{Density} = (M_c - M_m) / V_m \qquad \text{(Equation 3.2)}$$

3.3.2.4. Air Content (ASTM C231)

To measure the air content of freshly mixed concrete, the pressure method is used in accordance with ASTM C231. Although this method is more suitable for concretes with relatively dense aggregates, it is still being used for concretes with relatively lighter aggregates like natural perlite. To apply this test method, there are two types of air-meters as stated in ASTM C231, namely Type A and Type B. In this study, Type B air-meter (Figure 3.8) was used.

In Type B air-meter, there is a known value of air in air chamber at a known pressure and an unknown volume of air in the fresh concrete, which is placed in measuring bowl by the same consolidation practice in slump test. By using the testing procedures given in ASTM C231, these two air volumes are equalized. The pressure at which this equalization occurs is converted in terms of percentage air and can be read from the pressure gauge.

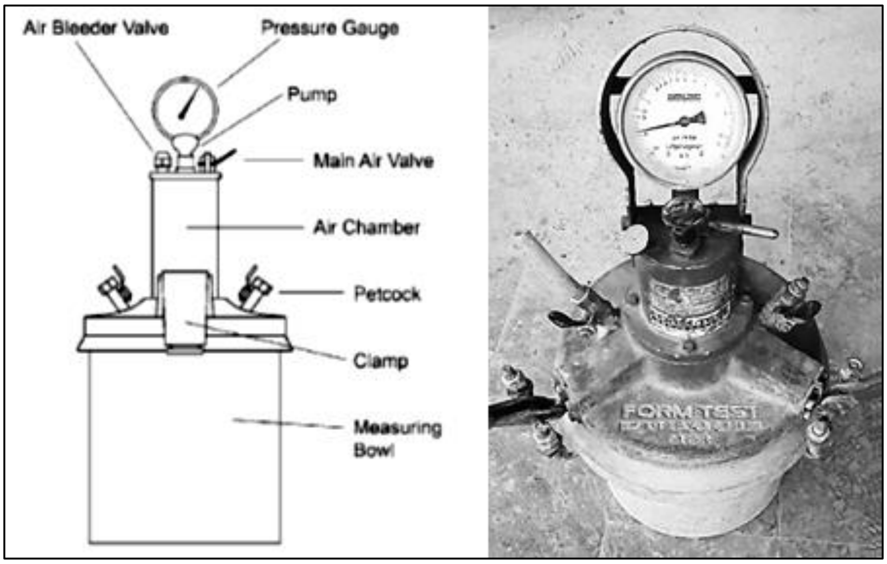

Figure 3.7. Type B air-meter with vertical air chamber (ASTM C231)

3.3.2.5. Setting Time (ASTM C403)

The setting time of a freshly mixed concrete can be determined by testing its penetration resistance in accordance with ASTM C403. For this test, a mortar sample is obtained by sieving freshly mixed concrete through No.4 (4.75 mm) sieve and the mortar is remixed by hand. Then, the remixed mortar is placed in a container. The consolidation of the mortar sample can be either achieved by a vibration table or rocking the container on a rigid surface. Before initial testing, bleed water accumulated on the surface of the sample is removed. For testing surface resistance, a loading apparatus with a penetration needle (Figure 3.9) is used. Initial testing is started after 3-4 hours from the first contact of mixing water with cement. The time passed from the first contact of the mixing water with cement until the surface resistance reaches 3.5 MPa and 27.6 MPa are called as initial setting time and final setting time respectively.

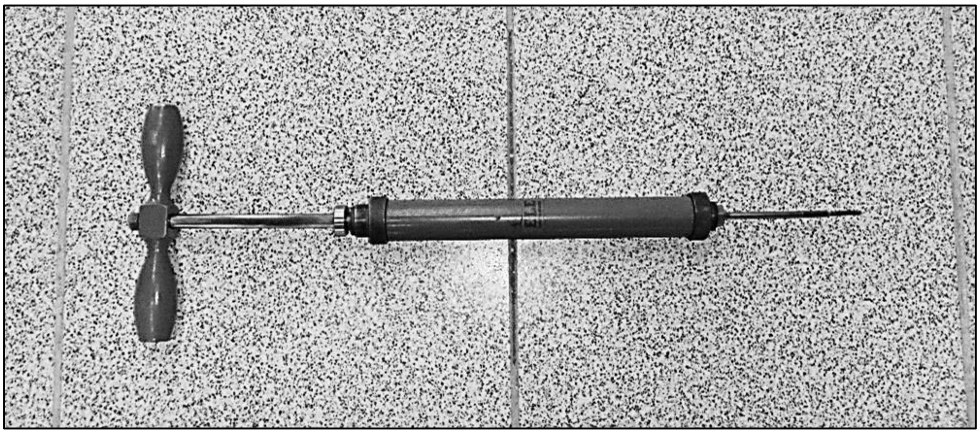

Figure 3.8. Concrete setting time testing apparatus

3.3.3. Tests on Hardened Concrete

3.3.3.1. Compressive Strength (ASTM C39)

The compressive strength of the cylindrical specimens was determined in accordance with ASTM C39. Until the testing day, the specimens were stored in water bath at 23±2^{o}C. After the removal of specimens from the water bath, both ends of the specimens are sawed and capped by a sulphur compound. Then, the specimens were loaded at a constant loading rate of 0.25±0.05 MPa/s. Three specimens of a kind were tested at each testing day and the average of these three results were determined as the compressive strength.

To assess the compressive strength results correctly, there were two considerations taken into account. Firstly, as stated in ASTM C39, the compressive strength results should be multiplied by a correction factor if L/D ratio is less than or equal to 1.75 (Table 3.11). Secondly, in the same standard, the acceptable range of individual cylinder strengths for 20x10cm cylindrical specimens are determined as 9% and 10.6% for 2 cylinders and 3 cylinders, respectively. Any misleading results, which are beyond this range, are omitted.

Table 3.11. Correction factor for compressive strength results (ASTM C39)

L/D	1.75	1.50	1.25	1.00
Correction Factor	0.98	0.96	0.93	0.87

3.3.3.2. Splitting Tensile Strength (ASTM C496)

This test is an indirect method for determining the tensile strength of concrete. In this test, cylindrical specimens are loaded along their length and this resultantly creates tensile stresses on the plane of loading and compression around loading points. This test method and related stress distribution diagram are illustrated in Figure 3.10.

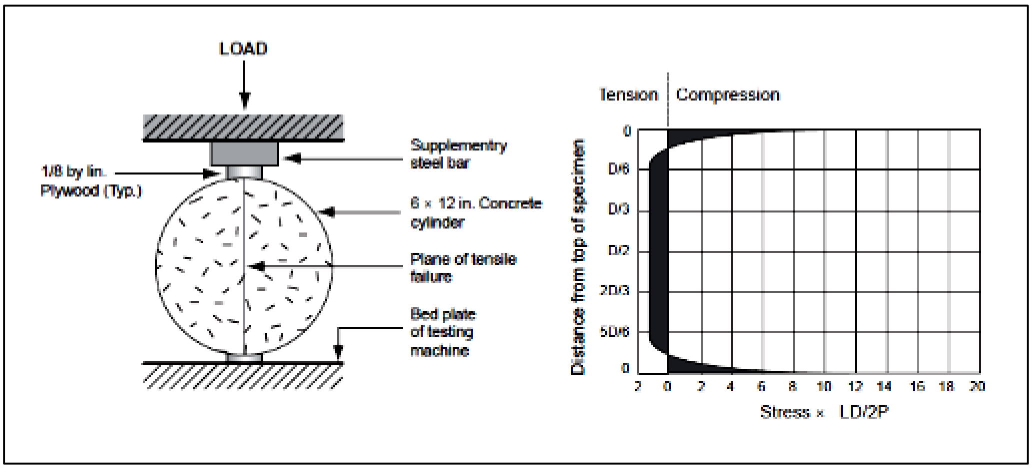

Figure 3.9. Splitting tension test and stress distribution diagram (Mehta & Monteiro, 2006)

Splitting tensile strength can be calculated via following formula.

$\sigma_{st} = 2P/\pi LD$ (Equation 3.3)

P: compressive load at failure
L: length of cylindrical specimen
D: diameter of cylindrical specimen

3.3.3.3. Flexural Strength (ASTM C78)

Another indirect method to determine the tensile strength of concrete is flexural strength test, which is applied on beam specimens. Flexural strength can be either determined by third point loading method as in ASTM C78 or center point loading method as in ASTM C293.

In this study, third point loading method was used since it has some advantages over center point loading. In third point loading, the portion between loading points is exposed only to bending moment and this shear-free moment exists over this entire portion, where the fracture generally occurs. Resultantly, fracture will be due to only tensile stresses induced by bending moment. However, in center point loading, only the loading point has no shear. Therefore, if a specimen does not fracture at loading point –which is generally the case-, the fracture will be due to both shear and bending moment. This can be better comprehended by studying the shear and bending moment diagrams of center point loading (CPL) and third-point loading (TPL), which are given in Figure 3.11.

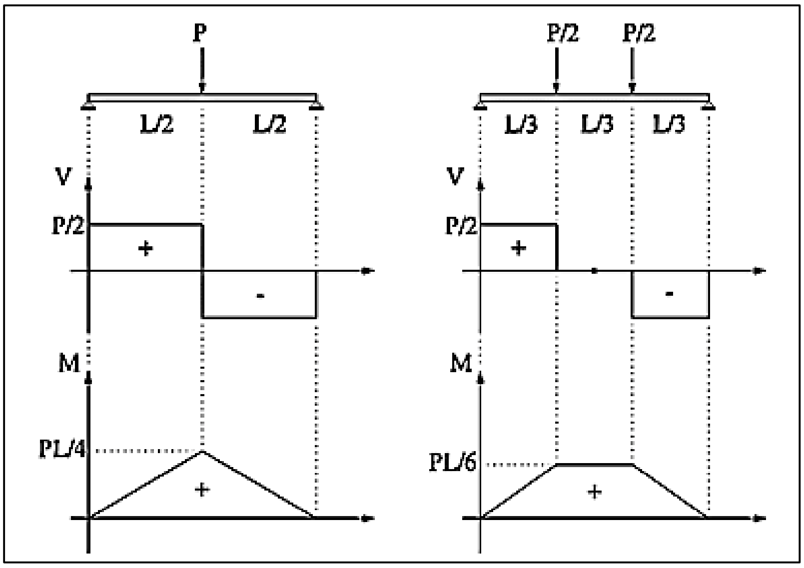

Figure 3.10. Shear and moment diagrams of CPL and TPL

The formula given below is used in calculating flexural strength of a concrete beam exposed to third point loading, if the tested beam fractures within the portion between the loading points.

$\sigma_{ft} = PL / bd^2$ (Equation 3.4)

P: load at fracture as shown by the testing machine
L: span length
b: average width of the specimen at the fracture
d: average depth of the specimen at the fracture

3.3.3.4. Elastic Modulus (ASTM C469)

Elastic modulus of concrete is an important property especially for designing structural members. It is used in computing the strain at a known stress value. Physically, elastic modulus can be defined as the resistance of a material to elastic deformation. Mathematically, it is the slope of the linear portion (elastic region) in a stress-strain diagram of a material.

In this study, the elastic modulus of the cylindrical concrete specimens were determined in accordance with ASTM C469. This method assumes, up to 40% of ultimate stress level, the slope of stress-strain diagram of a concrete specimen is linear. Therefore, the specimens are loaded up to this stress level and change in length of the specimens during loading period are monitored by means of a displacement sensor. Then, the longitudinal deformation is converted to longitudinal strain and the stress-strain curve is drawn.

According to ASTM C469, elastic modulus can be calculated by two data points of known stress-strain values. In other words, the standard calculates the chord modulus of elasticity (Equation 3.5). Actually, a more accurate determination of elastic modulus, may be to find the slope of the linear portion of the stress-strain curve obtained by a continuous data acquisition system, rather than a result based on only two data points.

$E = (S_2 – S_1) / (\varepsilon_2 – \varepsilon_1)$ (Equation 3.5)

E: chord modulus of elasticity (MPa)
S_2: stress equal to 40% of ultimate stress (MPa)
S_1: stress at ε_1 strain (MPa)
ε_2: longitudinal strain at S_2 stress
ε_1: longitudinal strain of 0.000050

It should also be noted that specimens are loaded at least twice and the first loading data is not used in elastic modulus calculations. This is because first loading is essentially for seating of gauges (ASTM C469/C469M-10, 2010).

For this study, the specimens were loaded and unloaded 3 times and elastic modulus values were determined from the slope of stress-strain curve derived from the last two loading cycles. In each test, three specimens were used and the elastic modulus values were calculated from the average.

3.3.3.5. Linear Coefficient of Thermal Expansion

To determine the linear coefficient of thermal expansion of concrete specimens, the procedure given below was followed for beam specimens:

1. Specimens were removed from water bath and allowed to dry in an oven at 110±5°C for 24 hours before the testing day.
2. On testing day, specimens were removed from the oven and allowed to cool to room temperature.
3. Then, pins were glued on two consecutive long sides of each specimen. The pins are placed so as to provide a gauge length of 25 cm for measuring the change in length (Figure 3.12).
4. Afterwards, the specimens were put in an oven at 25°C and allowed to remain for 1 hour.
5. Immediately after the removal of the specimens from the oven, length measurements were taken by means of an electronic measuring device (Figure 3.13).
6. Steps 4&5 were repeated for the temperatures of 40, 55 and 70°C.

7. The scatter diagram of the length versus the temperature change was drawn. Then, the best fitting line was drawn, the slope of which gives the change in length per degree ($\Delta L/\Delta T$).
8. The linear coefficient of thermal expansion (change in unit length per degree) was calculated by dividing the slope of the best fitting line by initial gauge length at 25°C (Equation 3.6).

$$\alpha = (\Delta L/L_0)/\Delta T = (\Delta L/\Delta T)/L_0 \qquad \text{(Equation 3.6)}$$

α: linear coefficient of thermal expansion
($\Delta L/\Delta T$): slope of the best fitting line
L_0: average gauge length at 25°C, as determined from the graph

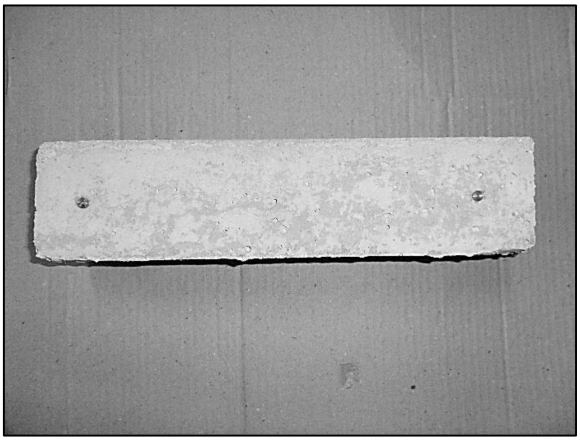

Figure 3.11. Beam specimen for thermal expansion experiment

Figure 3.12. Measurement of thermal expansion of concrete specimens

3.3.4. Durability Tests on Hardened Concrete

3.3.4.1. Rapid Chloride Permeability Test (ASTM C1202)

In this study, rapid chloride permeability test (RCPT) was conducted in accordance with ASTM C1202. For this test, 50±3 mm thick slices of 95-100 mm diameter cylinder specimens are used. Throughout a 6-hour duration, 60V dc is applied to the ends of specimen, one of which is in contact with sodium hydroxide solution and the other is in contact with sodium chloride solution. Amount of electrical current passing through the specimen is recorded. At the end of the test, the total charge passed through the specimen is used in assessing the resistance of concrete specimens to chloride ion penetration. Table 3.12 illustrates the qualitative assessment of chloride ion penetrability.

Table 3.12. Qualitative assessment of chloride ion penetrability (ASTM C1202)

Charge Passed (coulombs)	Chloride Ion Penetrability
>4000	High
2000-4000	Moderate
1000-2000	Low
100-1000	Very Low
<100	Negligible

Before assessing the chloride ion penetrability according to Table 3.12, for the tested specimens having a diameter size other than 95 mm, total charges passed through the specimen should be adjusted by using the following formula.

$Q_s = Q_x \times (95/x)^2$ (Equation 3.7)

Q_s: charge passed through standard specimen (95 mm diameter)
Q_x: charge passed through non-standard specimen
x: diameter of non-standard specimen (mm)

As stated in ASTM C1202, the results of two tests on the concrete samples from the same batch should not differ more than 42% and 51% for single-operator and multi-laboratory precision, respectively. Considering this large amount of variation, although it is not defined as obligatory by the standard, testing chloride penetrability on more than one specimen could provide more reliable results. The chloride penetrability results in the scope of this thesis are based on the average of at least two specimens.

3.3.4.2. Durability in Aggressive Chemical Solutions

To assess the resistance of the concrete specimens to sulfate attack, carbonation and acid attack, magnesium sulfate, sodium bicarbonate and sulphuric acid solutions were prepared respectively and the specimens were immersed in these solutions at the age of 28 days (Figure 3.14). Magnesium sulfate solution was 0.352 M. This molarity value was taken from ASTM C1012. The molarity of sodium bicarbonate solution was also 0.352 M, and it was based on a previous study by Bakharev et al (2001) to model ground water with high concentrations of carbonate ions, in other words, carbonated water. The pH value of sulphuric acid solution was determined as 1.0, which corresponds approximately to 1% sulphuric acid solution. This acid and corresponding pH value was chosen to model acid attack on concrete in sewers. The solutions were stored at room temperature. To provide a steady severity level of deterioration, solution-to-specimens volume ratio was fixed to 2.0 and the solutions were renewed monthly for first six months of exposure. The durability of the specimens in these aggressive chemical solutions was monitored by visual inspection and compressive strength tests till the age of 9 months. The compressive strength tests were conducted after 1, 3, 5 and 8 months of exposure.

Figure 3.13. Storage of concrete specimens in aggressive chemical solutions

3.3.4.3. Freezing-Thawing Resistance (ASTM C666)

Testing procedure of freezing-thawing resistance of the specimens was adapted from ASTM C666, procedure A. The cylindrical concrete specimens at the age of 28 days were placed in a climate cabin after sawing of the both end of the specimens. At the bottom of the specimens, a water level of 1 cm was maintained throughout the experiment. Concrete specimens were subjected to 300 cycles of rapid freezing and thawing and the durability is assessed by testing of the compressive strength of the specimens after 100 and 300 cycles of exposure. According to ASTM C666, during freezing-thawing cycles, temperature of the specimens should be changed from 18^{o}C to -4^{o}C in freezing and -4^{o}C to 18^{o}C in thawing. These temperature values were approximately achieved for the specimens, when the temperature of the climate cabin used in this experiment was lowered from 35^{o}C to -25^{o}C in 2.5 hours in the freezing phase and was raised from -25^{o}C to 35^{o}C in 1.5 hours in the thawing phase.

CHAPTER 4

RESULTS AND DISCUSSION

4.1. Mix Design and Fresh Properties

As stated earlier, within the scope of this thesis, three types of concretes were designed. These are high-strength lightweight concrete (HSLWC), self-compacting high-strength lightweight concrete (SCLWC) and high-strength normal weight concrete (HSNWC). The mix proportions and fresh properties are as shown in Table 4.1.

Table 4.1. Mix proportions and fresh properties

Mix Proportions (kg/m^3)			
Concrete Type	HSLWC	SCLWC	HSNWC
Cement	315	274	308
Perlite Powder	-	274	-
Water	100	127	140
0-4 mm PA / 0-5 mm LA (SSD)	875/0	749/0	0/1135
4-8 mm PA / 5-15 mm LA (SSD)	392/0	335/0	0/450
8-12 mm PA / 15-25 mm LA (SSD)	312/0	267/0	0/391
Superplasticizer	1.26	2.19	1.23
w/cm	0.32	0.23	0.45
Theoretical Fresh Density (kg/m^3)	1996	2029	2425
Fresh Properties			
Measured Fresh Density (kg/m^3)	1915	1950	2373
Air Content (%)	4.2	2.2	2.2
Slump (cm)	4	-	9
Slump Flow (cm)	-	77	-
Slump Flow Class	-	SF3	-
Visual Stability Index (VSI)	-	1-Stable	-
T_{500} (sec)	-	≤ 2	-
Viscosity Class	-	VF1	-
Setting Time			
Initial Set (hr:min)	5:30	4:00	4:30
Final Set (hr:min)	9:30	7:00	7:30

As shown in the table above, HSLWC and HSNWC has similar cement contents. SCLWC has approximately 35-40 kg/m^3 less cement content when compared to HSLWC and HSNWC, however it also contains perlite powder as pozzolan and when its total binder content is considered, it almost doubles the binder content of the other two types of concretes. SCLWC can also be named as self-compacting high volume pozzolan concrete since 50% of its binding medium is pozzolan.

It is hard to define an exact w/c ratio for lightweight concretes, since the absorption capacity of lightweight aggregates are high and the water absorption can continue for several weeks. Nevertheless, using the 3-days absorption capacity data of perlite aggregates, w/cm ratios have been estimated. It was found that HSLWC has w/c ratio of 0.32 and SCLWC has a w/cm ratio of 0.23. These ratios may seem to be relatively low, however to obtain high strength lightweight concretes, it is a necessity. Unlike lightweight aggregate concretes, the determination of w/c ratio for concretes with normal weight aggregates are more accurate and reliable. HSNWC was designed with a w/c ratio of 0.45.

To enhance the workability of the fresh mixtures, a superplasticizer admixture was used at a dosage of 0.4% of the binding medium. Although the slump of the HSLWC was 4 cm, it was within the recommended range defined by ACI Committee 211. During the casting of the concrete in the laboratory, it was also observed to require a fair compacting effort and was placed in to the molds as comfortably as HSNWC, slump of which was 9 cm.

The assessment of the workability of SCLWC was done by several observations on its slump flow. Slump flow of the SCLWC was measured as 77 cm and T_{500} time was under 2 seconds, which corresponds to slump flow class SF3 and viscosity class VF1, respectively. When this two data is considered together, it may be expected to have a tendency to bleeding and segregation. However, when it was evaluated by visual stability index, it was found to be stable (VSI = 1), with only a slight bleeding as a sheen (Figure 4.1).

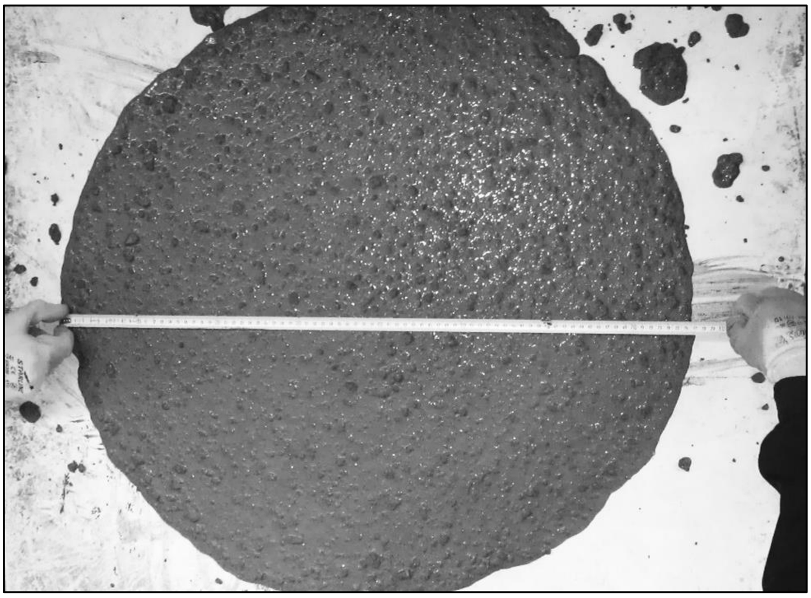

Figure 4.1. Slump flow measurement on SCLWC

Air contents of HSLWC, SCLWC and HSNWC were found to be 4.2, 2.2 and 2.2%, respectively. Since no air-entraining agents was used, these values are due to entrapped air voids. Relatively higher air-content of HSLWC may be attributed to porous nature of perlite aggregate. If so, similar air-content may be expected in SCLWC, which is also made of perlite aggregate. However, this is not the case. Due to self-consolidation property of SCLWC, these entrapped air voids are relatively better eliminated and resulted in a lower air content.

As expected, the measured fresh densities of the concretes were lower than the theoretical fresh densities. This is mainly because the calculations are based on specific gravities of aggregates in SSD condition. In fact, in batching, the aggregates are in dry-state and they cannot totally absorb the water to full absorption capacity during mixing. Resultantly, when the concrete densities are measured, the pores in aggregates are not fully saturated; and some part of the water thought to be absorbed by aggregates, remains in the mixture. Thus, the measured fresh densities are lower than the theoretical ones. The difference between theoretical and measured fresh densities is around 80 kg/m^3 for HSLWC and SCLWC whereas it is 50 kg/m^3 for HSNWC.

Initial and final setting time of the concretes produced are also given in Table 4.1. The shortest setting time is measured for SCLWC, which has the highest binder content and lowest w/b ratio. HSLWC and HSNWC has similar binder contents. Although HSLWC has lower w/c, it has the longer setting time. This can be explained by initially high w/c ratio of HSLWC. The high amount of water calculated to be absorbed by lightweight aggregates, cannot be fully and instantly absorbed. Thus, w/c ratio of HSLWC at fresh state is possibly much higher than calculated and responsible for longer setting time.

4.2. Hardened Properties

4.2.1. Compressive Strength, Density and Specific Strength

As stated previously, the mix proportions are determined such that 28th day specific strengths of all three designed concrete types would be comparable. Since the concrete in structural applications is generally air-dry in service condition, specific strength calculations are based on air-dry density of the specimens. In Table 4.2, compressive strength, specific strength, unit weight of the designed concretes in saturated surface dry (SSD), air-dry (AD) and oven-dry (OD) condition have been provided.

Table 4.2. Compressive strength, density and specific strength

Compressive Strength (MPa)			
Age (days)	HSLWC	SCLWC	HSNWC
7	39.5	36.9	49.7
28	43.7	50.9	54.9
56	47.0	52.6	57.3
90	47.7	55.8	57.6
120	48.0	59.4	57.9
180	52.9	64.6	59.5
270	57.6	66.9	62.7
Density (kg/m^3)			
Moisture Condition	HSLWC	SCLWC	HSNWC
SSD	1939	2017	2416
AD	1881	1983	2376
OD	1849	1928	2327
Specific Strength (MPa/(tons/m^3))			
Moisture Condition	HSLWC	SCLWC	HSNWC
AD	23.2	25.7	23.1

In Figure 4.1, it can be seen that the strength development HSNWC and HSLWC slows down dramatically after 56 days. For HSLWC, however, strength surprisingly re-increases after 120 days. This may be attributed to the pozzolanic activity between the surface of the lightweight aggregate and cement paste. For SCLWC, although the rate of strength development slows down after 28 days, it continues gradually to develop strength even at the age of 180 days, thanks to the pozzolanic activity of perlite powder.

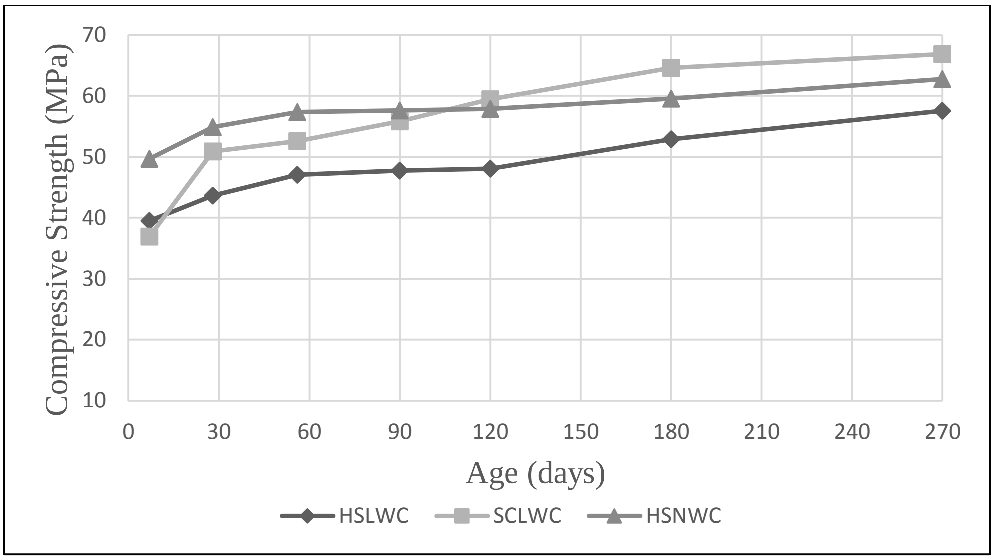

Figure 4.2. The development of compressive strength in the concrete specimens

The densities of the designed concretes were measured in SSD, AD and OD condition (Table 4.2). In classifying concretes with respect to their weights, air-dry equilibrium density is used. HSLWC has an air-dry density of 1881 kg/m^3 which is within the range (1120-1920 kg/m^3) defined by ACI Committee 213 (2003). SCLWC has about 60 kg/m^3 higher air-dry density (1983 kg/m^3) than this definition. However, as indicated by ACI Committee 213 (2003), this definition is not a specification and job specifications may allow higher densities in order to provide a range of strength and density economically. Besides, the oven-dry density of SCLWC is 1928 kg/m^3, which is under 2000 kg/m^3 limit defined by EN206-1:2000. When air-dry densities are compared, HSLWC and SCLWC are 21% and 17% lighter than HSNWC, respectively.

The specific strength values of the designed concretes were found to be 23.2, 25.7 and 23.1 MPa/(tons/m^3) for HSLWC, SCLWC and HSNWC, respectively. Although, the specific strength of SCLWC was found to be relatively higher than planned, it is still comparable with the other two types of concretes.

4.2.2. Splitting and Flexural Tensile Strength

Table 4.3 illustrates the splitting and flexural tensile strength of the designed concretes at the age of 28 and 90 days. It was found that the splitting tensile strength and flexural tensile strength of HSLWC and SCLWC are approximately 1-2 MPa less than those of HSNWC at similar specific strength. It can also be inferred from the results that there is no significant tensile strength development between 28 and 90 days, except for the splitting tensile strength of HSNWC and the flexural tensile strength of HSLWC.

Table 4.3. Splitting and flexural tensile strength

Splitting Tensile Strength (MPa)			
Age (days)	HSLWC	SCLWC	HSNWC
28	3.5	4.2	4.3
90	3.7	4.3	5.2
Flexural Tensile Strength (MPa)			
Age (days)	HSLWC	SCLWC	HSNWC
28	5.1	6.0	8.0
90	5.9	6.1	8.0

4.2.3. Elastic Modulus

The elastic modulus of the designed concretes at the age of 28 and 90 days is given in Table 4.4. There is no significant change between 28th and 90th days. The elastic modulus of HSLWC and SCLWC corresponds approximately to 50% and 60% that of HSNWC, respectively. The lower elastic modulus of lightweight concretes are attributed to lower stiffness of natural perlite aggregates.

Table 4.4. Modulus of elasticity, compressive strength and density

Elastic Modulus (GPa)			
Age (days)	HSLWC	SCLWC	HSNWC
28	22.2	26.0	44.9
90	22.2	25.9	42.5
Compressive Strength, MPa(psi)			
Age (days)	HSLWC	SCLWC	HSNWC
28	43.7(6338)	50.9(7382)	54.9(7962)
90	47.7(6918)	55.8(8093)	57.6(8354)
Density, kg/m^3 (lb/ft^3)			
Moisture Condition	HSLWC	SCLWC	HSNWC
AD	1881(117)	1983(124)	2376(148)

In Figures 4.3 and 4.4, the stress-strain curves of HSLWC, SCLWC and HSNWC up to 40% of corresponding ultimate stress are given. Each line represents the average of 3 specimens.

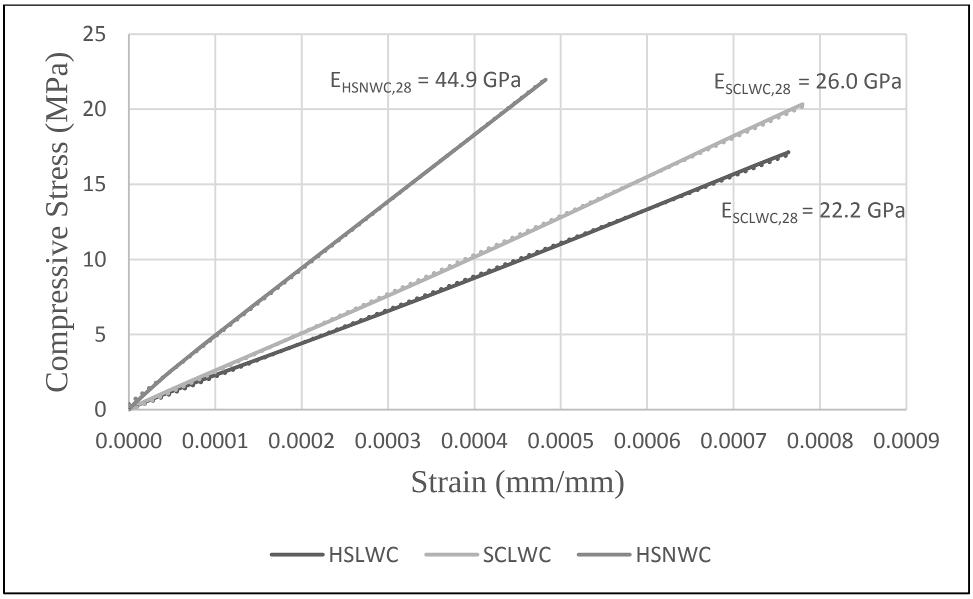

Figure 4.3. Stress-strain curves up to 40% of ultimate stress (28 days)

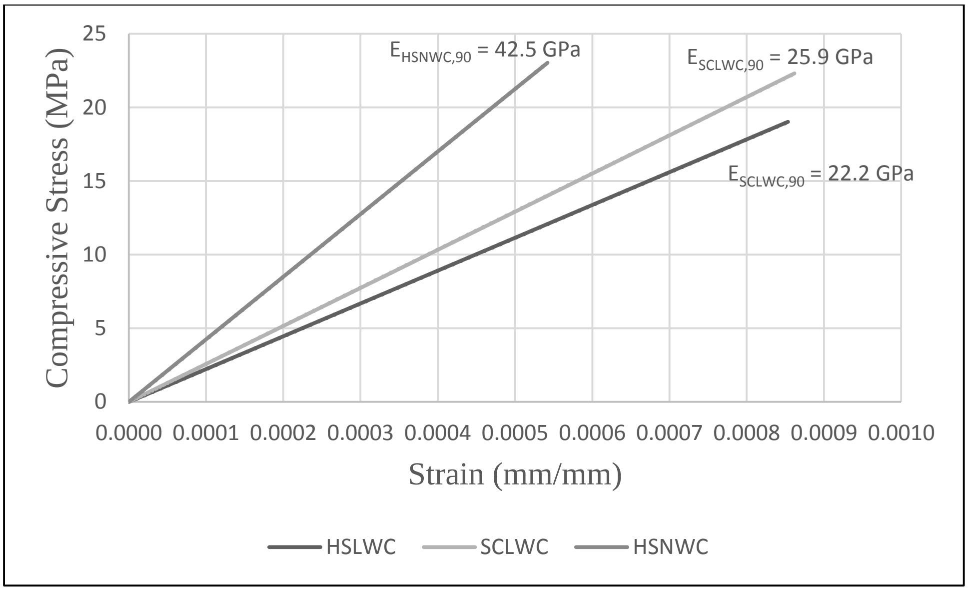

Figure 4.4. Stress-strain curves up to 40% of ultimate stress (90 days)

The experimental results were also compared with ACI 318, Norwegian Standard and CEB-FIP formulas for elastic modulus estimation.

According to ACI 318, elastic modulus of the concretes up to 41 MPa compressive strength and having a unit weight between 90 lb/ft^3 (1442kg/m^3) and 155 lb/ft^3 (2483kg/m^3) can be calculated by using the following formula:

$E_c = 33 \times w_c^{1.5} \times f_c^{0.5}$ (Equation 4.1)
w_c: air-dry unit weight (lb/ft^3)
f_c: 28 days compressive strength (psi)

Though HSLWC and SCLWC have higher compressive strength than 41 MPa, if the formula is applied:

For HSLWC; $E_c = 33 \times 117^{1.5} \times 6338^{0.5}$ = 3324825 psi = 22.9 GPa vs. 22.2 GPa (exp.)
For SCLWC; $E_c = 33 \times 124^{1.5} \times 7382^{0.5}$ = 3915015 psi = 27.0 GPa vs. 26.0 GPa (exp.)

These results show that ACI 318 formula overestimates the modulus of elasticity by only about 3% for HSLWC and SCLWC.

Norwegian Standards have also a formula for elastic modulus of high strength lightweight concretes with a compressive strength between 60 and 100 MPa (Neville, 2003, p. 704):

$E_c = 9.5 \times f_c^{0.3} \times (\rho/2400)^{1.5}$ (Equation 4.2)

f_c: 28 days compressive strength (MPa)
ρ: density of concrete (kg/m^3)

Although HSLWC and SCLWC have lower compressive strength than 60 MPa, if the formula is applied:

For HSLWC; $E_c = 9.5 \times 43.7^{0.3} \times (1881/2400)^{1.5} = 20.5$ GPa vs. 22.2 GPa (exp.)

For SCLWC; $E_c = 9.5 \times 50.9^{0.3} \times (1983/2400)^{1.5} = 23.2$ GPa vs. 26.0 GPa (exp.)

As can be seen from the results, Norwegian Standard formula underestimates the modulus of elasticity by 8 and 11% for HSLWC and SCLWC, respectively.

None of the formulas above is for the strength range of HSLWC and SCLWC. Nevertheless, ACI 318 formula yields to quite accurate estimations, considering the error was only about 3 percent.

CEB-FIP Model Code (1990) formula quite accurately estimates elastic modulus of HSNWC, with an error of 2 percent only. According to this model, for normal-weight aggregates, elastic modulus can be calculated by the following formula:

$E_c = \alpha \times 10^4 \times (f_c)^{1/3}$ (Equation 4.3)

f_c: 28 days compressive strength (MPa)
α, for dense limestone: 1.2 (Mehta & Monteiro, 2006, p. 92).

For HSNWC; $E_c = 1.2 \times 10^4 \times (54.9)^{1/3} = 45607$ MPa = 45.6 GPa vs. 44.9 GPa (exp.)

4.2.4. Linear Coefficient of Thermal Expansion

Table 4.5 shows the change in gauge length of the specimens with the change in the temperature. For each type of the concrete, two specimens of each having two gauges were tested. Using the measurements given in Table 4.5, scatter diagrams (Figure 4.5-4.7) were drawn. Then using Equation 3.6, the linear coefficients of thermal expansion were calculated (Table 4.6).

Table 4.5. The change in gauge length of the specimens in thermal expansion test

T (°C)	Gauge #	Gauge Length (mm)		
		HSLWC	SCLWC	HSNWC
25	1	250.055	250.066	250.053
	2	250.066	250.066	250.058
	3	250.069	250.067	250.059
	4	250.071	250.068	250.060
40	1	250.078	250.087	250.078
	2	250.091	250.090	250.082
	3	250.095	250.094	250.083
	4	250.097	250.095	250.084
55	1	250.106	250.120	250.105
	2	250.122	250.120	250.106
	3	250.122	250.121	250.109
	4	250.126	250.124	250.110
70	1	250.139	250.153	250.134
	2	250.145	250.154	250.137
	3	250.153	250.155	250.137
	4	250.158	250.163	250.138

Table 4.6. The linear coefficients of thermal expansion (mm/mm/°C)

HSLWC	SCLWC	HSNWC
7.45×10^{-6}	7.95×10^{-6}	7.01×10^{-6}

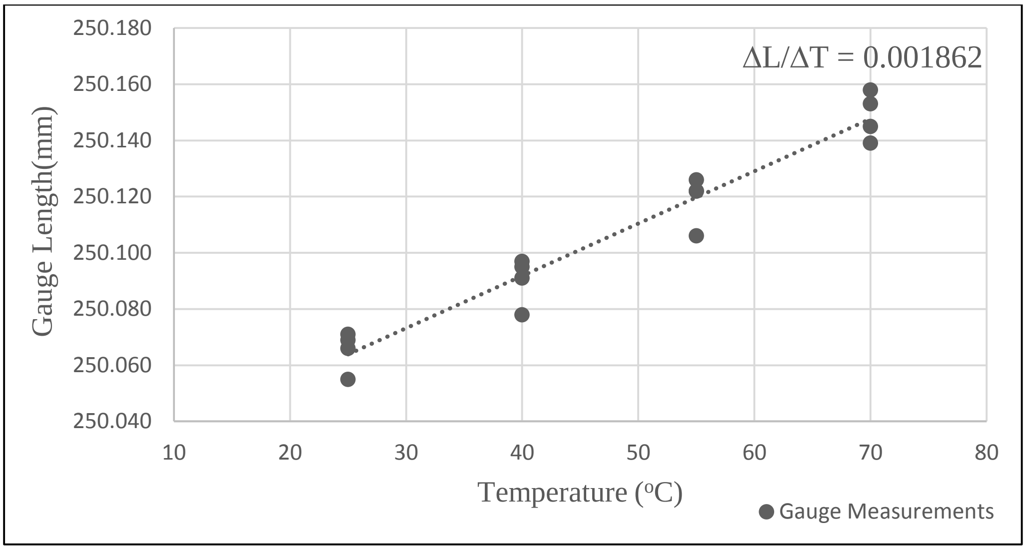

Figure 4.5. Change in gauge length with temperature change (HSLWC)

$(\Delta L/\Delta T) = 0.001862$

$L_0 = 250.063933$

$\alpha = (\Delta L/\Delta T)/L_0 = 0.001862/250.063933 = 7.45 \times 10^{-6}$ mm/mm/ºC for HSLWC.

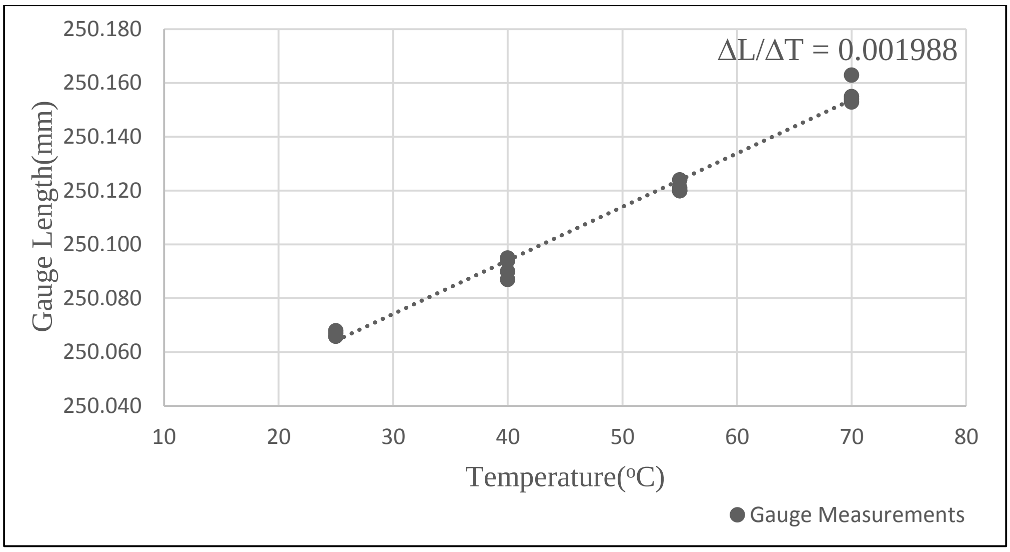

Figure 4.6. Change in gauge length with temperature change (SCLWC)

$(\Delta L/\Delta T) = 0.001988$

$L_0 = 250.064192$

$\alpha = (\Delta L/\Delta T)/L_0 = 0.001988/250.064192 = 7.95 \times 10^{-6}$ mm/mm/^{o}C for SCLWC.

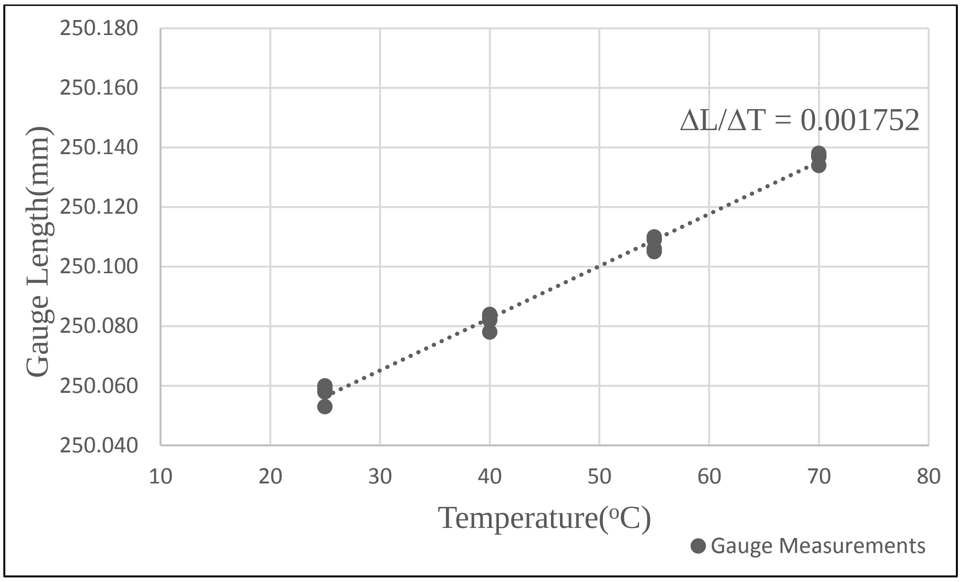

Figure 4.7. Change in gauge length with temperature change (HSNWC)

$(\Delta L/\Delta T) = 0.001752$

$L_0 = 250.056408$

$\alpha = (\Delta L/\Delta T)/L_0 = 0.001752/250.056408 = 7.01 \times 10^{-6}$ mm/mm/^{o}C for HSNWC.

Normally, lightweight concretes are expected to have lower linear coefficient of thermal expansion than its normal weight counterparts, however the results have shown that thermal expansion of HSLWC and SCLWC is slightly higher than HSNWC. This may be attributed to the mineralogy of the aggregates. It is known that siliceous aggregates generally have higher thermal expansion than limestone aggregates. If perlite containing HSLWC and SCLWC are compared with each other, HSLWC has lower linear coefficient of thermal expansion. This may be attributed to higher air content of HSLWC, which results in lower internal thermal stresses.

4.3. Results of Durability Tests

4.3.1. Rapid Chloride Ion Penetrability

The results of rapid chloride ion penetrability tests have been tabulated in Table 4.7. At the age of 28 days, HSLWC has low penetrability and SCLWC has very low penetrability. On the other hand HSNWC has moderate penetrability.

Table 4.7. Chloride ion penetrability of concrete specimens

Charges Passed(coulombs) - Chloride Ion Penetrability						
Age(days)	HSLWC		SCLWC		HSNWC	
28	1486	Low	800	Very Low	3696	Moderate
90	767	Very Low	271	Very Low	3054	Moderate

These results are as expected. The highest binder content and lowest w/cm ratio of SCLWC among the other two, yielded to the lowest penetrability. The effect of pozzolanic activity also have a major part in this result.

Although HSLWC and HSNWC has similar binder content, HSLWC has a lower penetrability because of several reasons such as lower w/c ratio, disconnected porous nature of perlite aggregate and pozzolanic activity in contact zone.

At the age of 90 days, the chloride ion penetrability of HSLWC was reduced by half and penetrability of SCLWC was reduced to one-third. However, there was only about 20% reduction in the penetrability of HSNWC.

In Figures 4.8 and 4.9, total charges passed versus time during the RCPT have been illustrated for 28 days and 90 days, respectively.

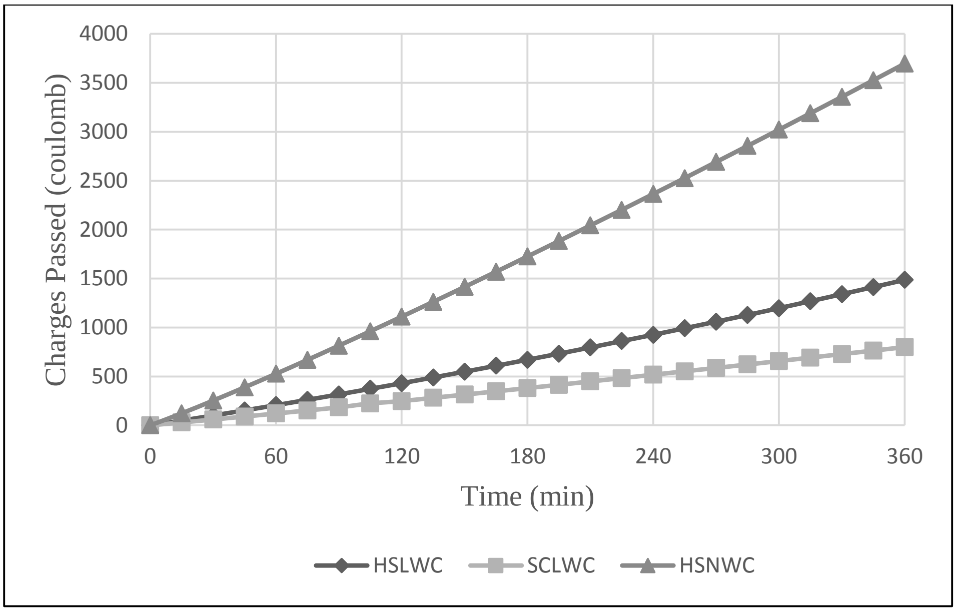

Figure 4.8. RCPT, total charges passed vs. time (28 days)

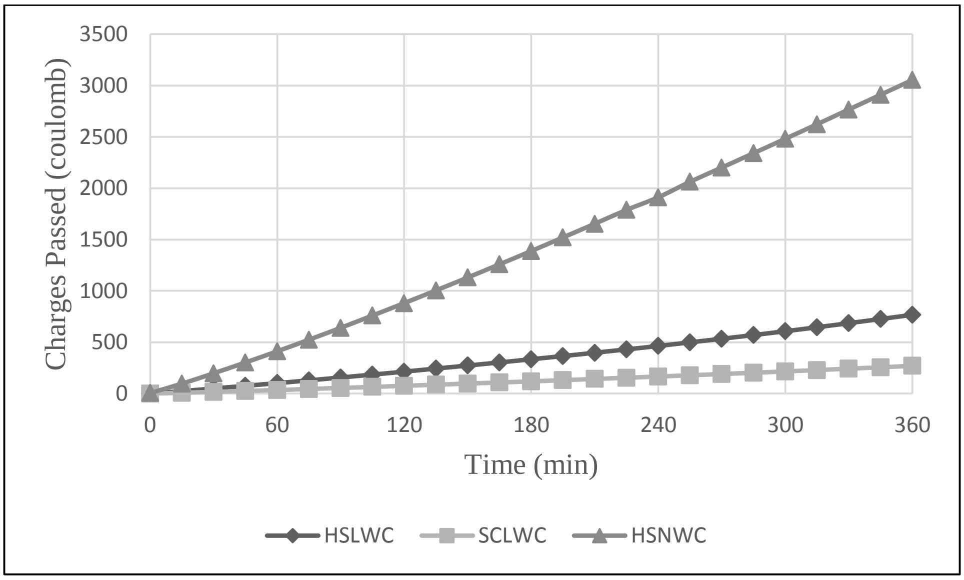

Figure 4.9. RCPT, total charges passed vs. time (90 days)

4.3.2. Specimens in Aggressive Chemical Solutions

4.3.2.1. Magnesium Sulfate Solution

The change in the compressive strength of the specimens stored in 0.352 M $MgSO_4$ solution is as shown in the figures from 4.10 to 4.12, separately for each type of the concrete. The line representing the control group and the line representing the specimens in the magnesium sulfate solution almost coincides in all graphs. This means that HSLWC, SCLWC and HSNWC are equally resistant to sulfate attack when the change in compressive strength is considered.

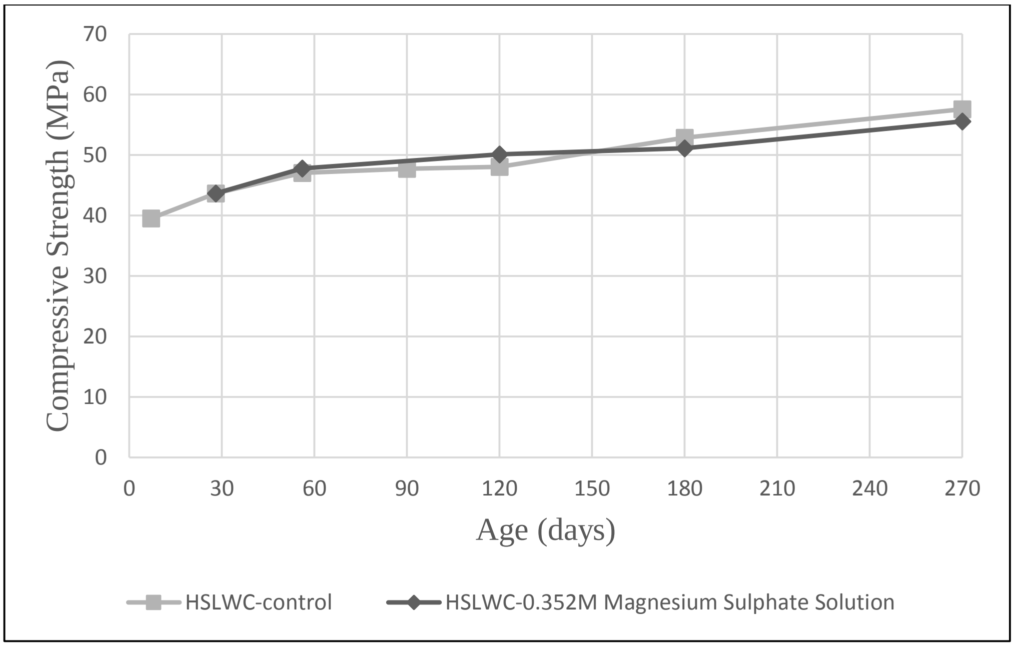

Figure 4.10. Change in compressive strength of HSLWC specimens stored in magnesium sulfate solution

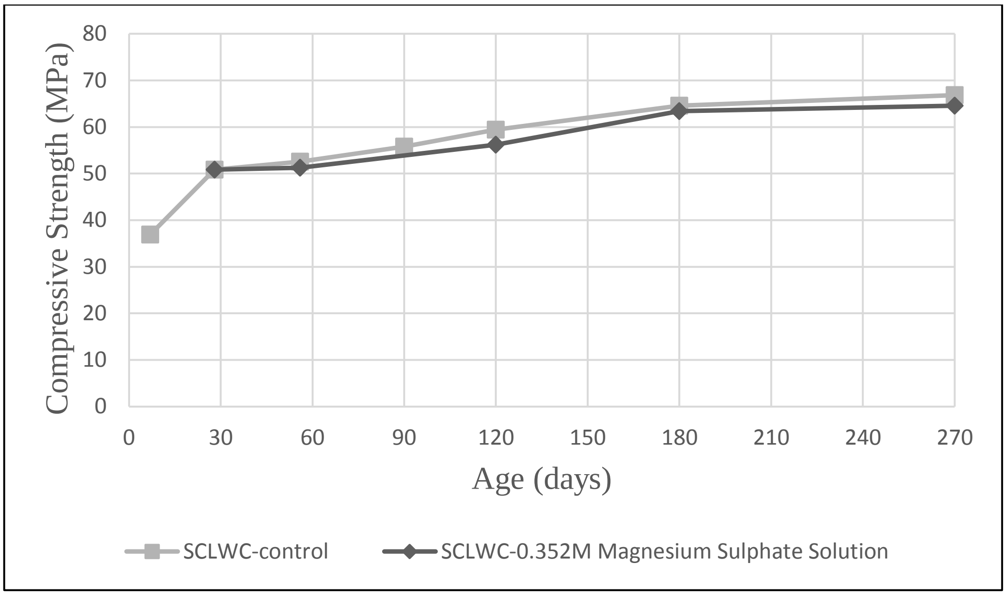

Figure 4.11. Change in compressive strength of SCLWC specimens stored in magnesium sulfate solution

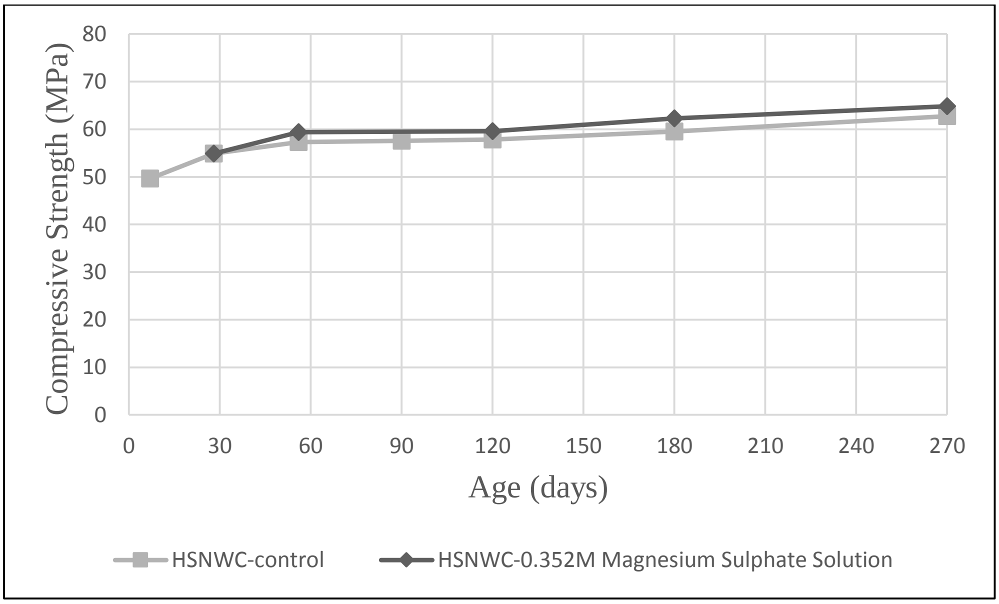

Figure 4.12 Change in compressive strength of HSNWC specimens stored in magnesium sulfate solution

In Figures 4.13 to 4.15, surface deterioration of the specimens stored in magnesium sulfate solution have been illustrated. It can be concluded that surface deterioration of HSLWC and SCLWC is relatively higher than HSNWC.

Figure 4.13. Surface deterioration of HSLWC specimens stored in magnesium sulfate solution

Figure 4.14. Surface deterioration of SCLWC specimens stored in magnesium sulfate solution

Figure 4.15. Surface deterioration of HSNWC specimens stored in magnesium sulfate solution

4.3.2.2 Sodium Bicarbonate Solution

Carbonation does not cause harmful deterioration of concrete, however it reduces the pH of concrete pore solution. For reinforced concrete, this causes passive layer to lost, which protects rebars from corrosion. On the other hand, carbonation have several positive consequences such as increase in surface hardness and strength and reduced surface permeability of concrete. This is mainly because, as calcium hydroxide carbonates and turns into calcium carbonate, it occupies larger volume and fills the pores in concrete.

In figures from 4.16 to 4.18, the change in the compressive strength of the specimens stored in 0.352M $NaHCO_3$ solution is as presented, separately for each type of the concrete. There is no significant increase in strength for none of the concretes stored in the solution since all concretes are sufficiently impermeable and interior zones are unaffected. This was shown by phenolphthalein test and shown that pH was still higher than 9.5 so that it still gives pink colour when applied on to concrete. On the other hand, it is visually observed that surface of the specimens in carbonate solution has become less porous and smoother due to surface carbonation. In Figures 4.19 to 4.21, the carbonation on the surface the specimens stored in sodium bicarbonate solution and interior zones tested by phenolphthalein have been illustrated.

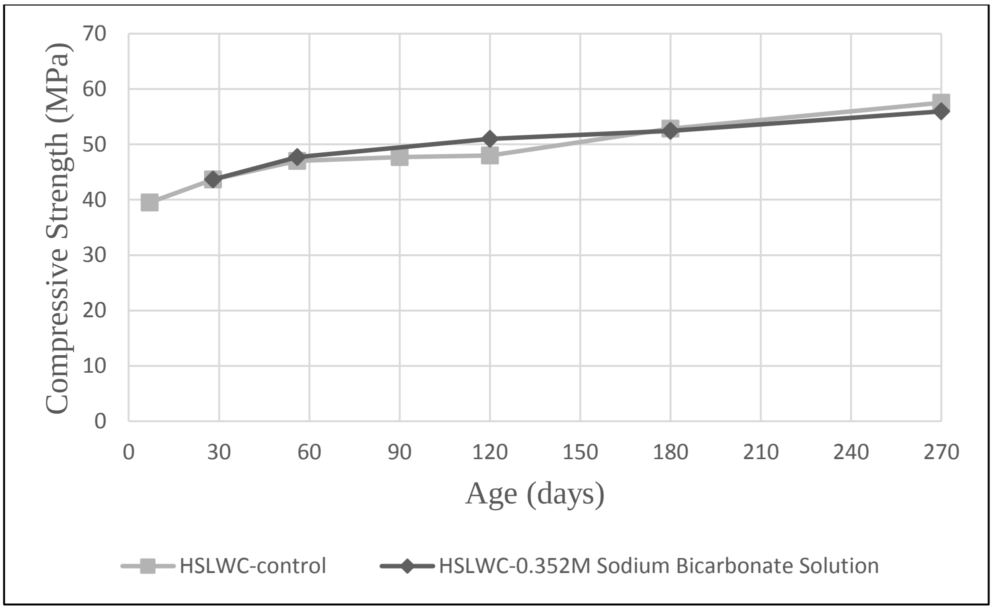

Figure 4.16. Change in compressive strength of HSLWC specimens stored in sodium bicarbonate solution

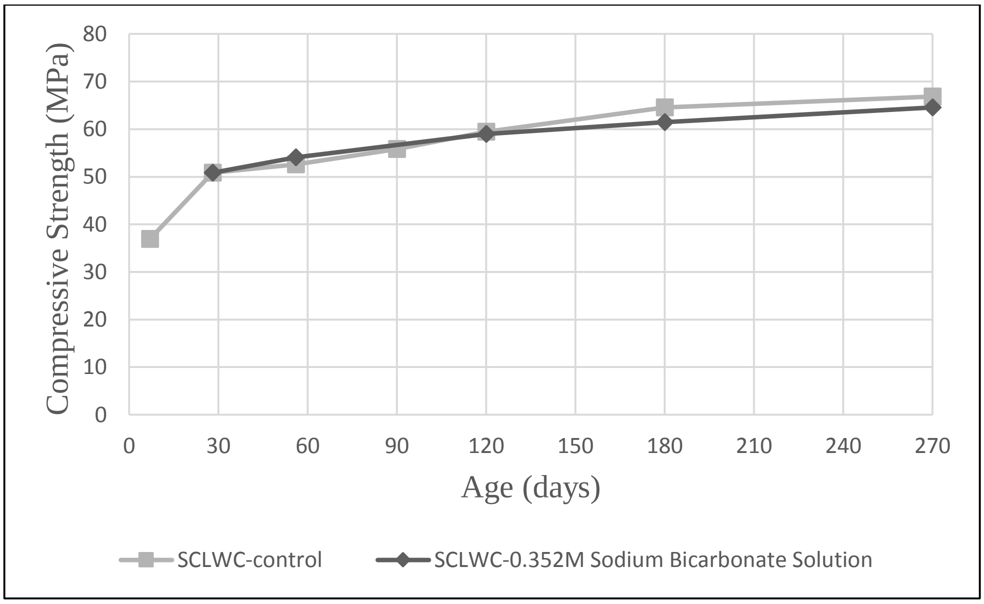

Figure 4.17. Change in compressive strength of SCLWC specimens stored in sodium bicarbonate solution

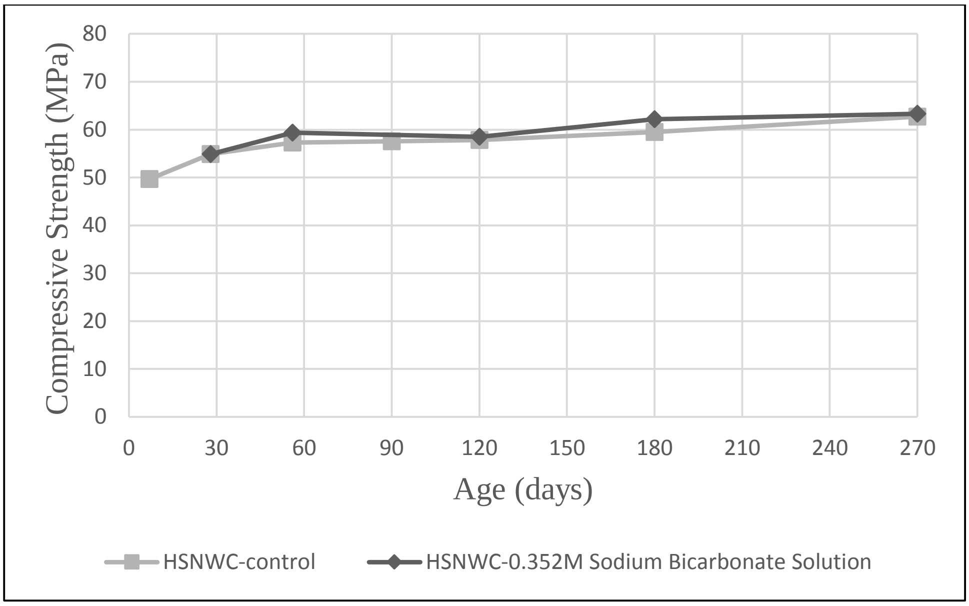

Figure 4.18. Change in compressive strength of HSNWC specimens stored in sodium bicarbonate solution

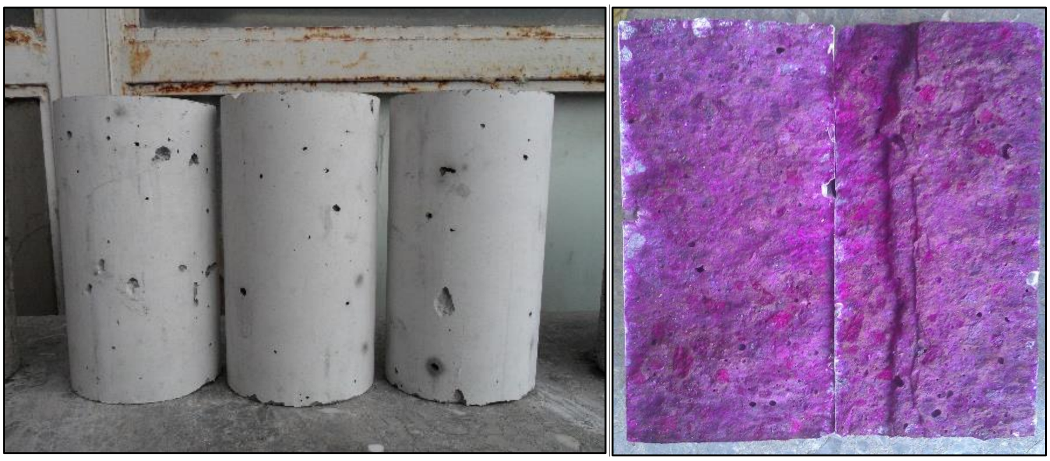

Figure 4.19. Surface deterioration of HSLWC specimens stored in sodium bicarbonate solution and interior of concrete tested by phenolphthalein

Figure 4.20. Surface deterioration of SCLWC specimens stored in sodium bicarbonate solution and interior of concrete tested by phenolphthalein

Figure 4.21. Surface deterioration of HSNWC specimens stored in sodium bicarbonate solution and interior of concrete tested by phenolphthalein

4.3.2.3. Sulphuric Acid Solution

The change in the compressive strength of the specimens stored in 1% H_2SO_4 solution (pH=1) is given in figures from 4.22 to 4.24, separately for each type of the concrete. As it can be seen from the graphs, the line representing the control group and the line representing the specimens in the sulphuric acid solution almost coincides for the first three months of exposure. After that point specimens started to show loss in compressive strength. At the age of 9 months, percentage loss in the compressive strengths compared to control specimens were 12, 14, 0.4 percent, respectively for HSLWC, SCLWC and HSNWC. It can be concluded that HSNWC is more durable to sulphuric acid than HSLWC and SCLWC. Nevertheless, when it is independently evaluated, HSLWC and SCLWC can also be considered durable since their compressive strengths under such aggressive conditions are still higher than the 28-day compressive strengths, namely the compressive strengths at the start of the exposure.

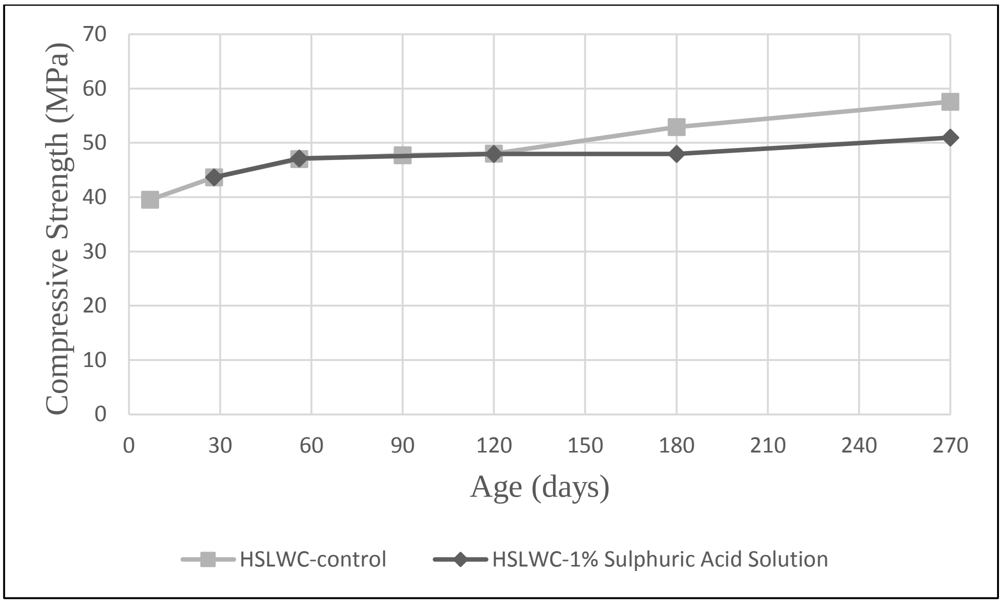

Figure 4.22. Change in compressive strength of HSLWC specimens stored in sulphuric acid solution

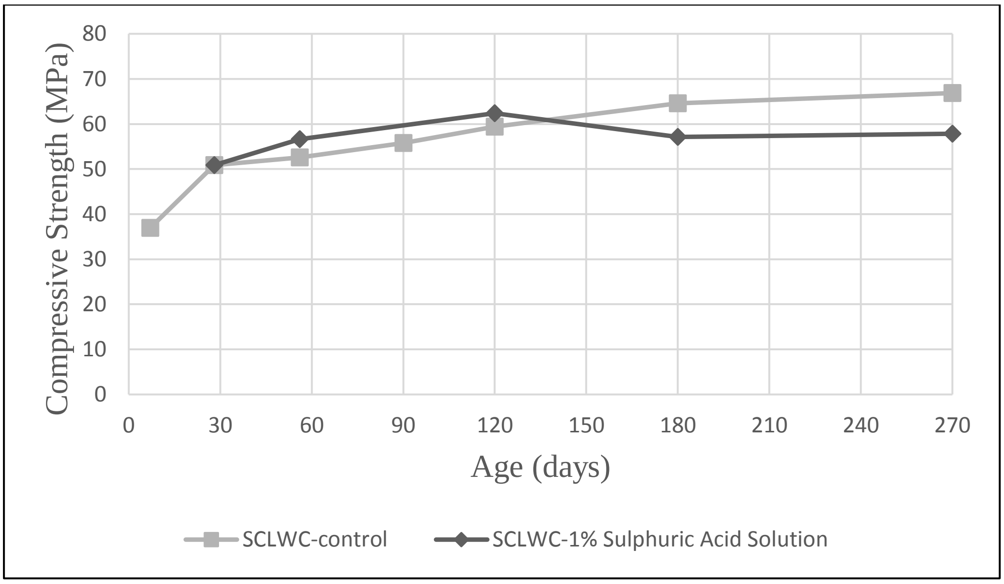

Figure 4.23. Change in compressive strength of SCLWC specimens stored in sulphuric acid solution

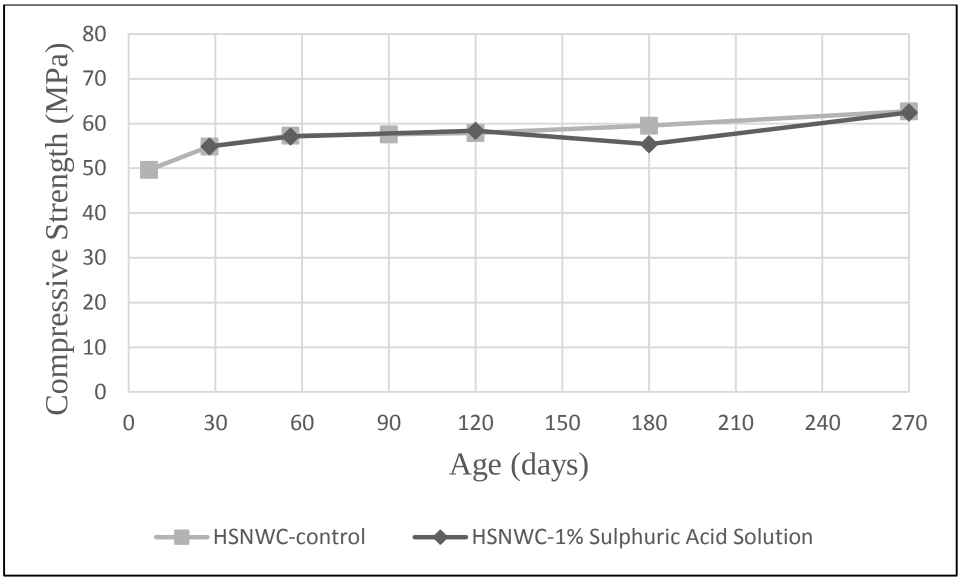

Figure 4.24. Change in compressive strength of HSNWC specimens stored in sulphuric acid solution

In Figures 4.25 to 4.27, surface deterioration of the specimens stored in sulphuric acid solution have been illustrated. It can be seen that the surface of all concretes have been deteriorated. In addition, for HSLWC and SCLWC, a weak cover formation was observed. The thickness of this deteriorated cover had been reached to 4 mm at the end of 8 months of exposure. This corresponds to 15% reduction in load bearing area, which is parallel to strength reduction (12-14%) observed in these specimens.

Figure 4.25. Surface deterioration of HSLWC specimens stored in sulphuric acid solution

Figure 4.26. Surface deterioration of SCLWC specimens stored in sulphuric acid solution

Figure 4.27. Surface deterioration of HSNWC specimens stored in sulphuric acid solution

4.3.3. Specimens Exposed to Freezing-Thawing Cycles

Table 4.8 shows the change in compressive strength of the specimens exposed to freezing-thawing cycles. At the end of F-T cycles, the highest percentage loss in strength was observed for HSLWC and the lowest percentage loss was observed for HSNWC. Although HSLWC has a higher air content of 4.2% compared to 2.2 % air content of SCLWC and HSNWC, as stated earlier, this air content is due to entrapped air since no air-entraining admixtures were used. Therefore, the governing factor here is not the air content but the tensile strength of the specimens which will resist the hydraulic pressure accompanied by freezing of the water in concrete. HSNWC has the highest tensile strength and resultantly highest resistance to freezing-thawing whereas HSLWC has the lowest tensile strength and suffered from the highest strength loss due to freezing and thawing. It should also be noted that all specimens were in fully saturated condition at the start of the freezing-thawing cycles. Therefore, there should be no significant contribution of the pores in lightweight aggregates to act as pressure relief zones. However, as experiment continues, the saturation degree of the specimens decrease with time and aggregate pores start to become effective to act as pressure relief zones. This can be clearly seen if the percent loss in strength at the end of 100 cycles and 300 cycles are compared.

Table 4.8. Compressive strength of the specimens exposed to freezing-thawing cycles

# cycles	HSLWC			SCLWC			HSNWC		
	σ_c (MPa)		% loss in σ_c	σ_c (MPa)		% loss in σ_c	σ_c (MPa)		% loss in σ_c
	Ctrl	F-T		Ctrl	F-T		Ctrl	F-T	
0	43.7	-	-	50.9	-	-	54.9	-	-
100	46.2	43.3	6.3	51.9	49.6	4.4	56.4	53.7	4.8
300	47.5	43.8	7.8	54.6	51.2	6.2	57.5	54.6	5.0

CHAPTER 5

CONCLUSIONS

The conclusions from the experimental study conducted in this thesis can be listed as below:

1. Natural perlite aggregate can be satisfactorily utilized in the production of high-performance lightweight concretes with 28-day compressive strengths up to 50 MPa.
2. Self-compacting high-strength lightweight concretes of good workability characteristics can be produced by using natural perlite aggregate and high volume (50%) perlite powder as pozzolan.
3. To achieve similar specific strength to that of high strength normal weight concrete, high strength lightweight concrete with natural perlite aggregate requires similar cement contents (about 300 kg/m^3).
4. At similar specific strength, high performance lightweight concretes with natural perlite aggregate is about 20% lighter than high-strength normal weight concrete.
5. At similar specific strength, elastic modulus of high performance lightweight concretes with natural perlite aggregate is about 50-60% of high-strength normal weight concrete with limestone aggregate and this was attributed to lower stiffness of lightweight aggregates. It has also shown that ACI 318 formula for elastic modulus estimation is applicable to structural lightweight concretes with natural perlite aggregates.
6. The linear coefficient of thermal expansion of high-performance lightweight concretes with natural perlite aggregate was found to be slightly higher than that of high-strength normal weight concrete with limestone aggregate. This was attributed to the fact that thermal expansion of concretes with siliceous aggregates have tendency to show greater expansion than those with limestone

aggregates. Although the perlite aggregate is porous and thus expected to have lower expansion, it was concluded that its chemical structure was more dominant in determining its expansion characteristics.

7. The chloride permeability of self-compacting high-strength lightweight concrete with natural perlite aggregate and perlite powder was found to be only about 20 and 10% of high-strength normal weight concrete of similar specific strength at 28 and 90 days, respectively. Similarly, high-strength lightweight concrete with natural perlite aggregate has a permeability corresponds to about 50 and 25% of high-strength normal weight concrete at 28 days and 90 days, respectively. This superior performance of these lightweight concretes was mainly attributed to improved contact zone due to several factors including lower w/b ratio, internal curing and pozzolanic activity.
8. High-performance lightweight concretes with natural perlite aggregate and high-strength normal weight concrete were found to be similarly durable against magnesium sulfate attack when the change in compressive strength compared to control specimens is considered. However, the surface deterioration of these lightweight concretes in magnesium sulfate solution were found to be higher than that of limestone containing high-strength normal weight concrete.
9. Perlite containing high-performance lightweight concretes have been found to suffer considerably higher surface deterioration than that of limestone containing high strength normal weight concrete when exposed to sulphuric acid attack and a formation of a weak cover were observed on the surface of the specimens. This deteriorated and easily scrapable cover corresponds about 15% reduction in cross-sectional area of perlite containing concrete specimens, which is parallel to the percent loss in compressive strength of these specimens (12-14%).
10. At similar specific strength, the resistance to freezing-thawing cycles of high-performance lightweight concretes was found to be slightly lower than that of high-strength normal weight concrete. Since no air-entraining agent was used in the designed concretes, this result was attributed to the higher tensile strength of the normal weight concrete which resists the hydraulic pressure due to freezing of water in concrete.

CHAPTER 6

RECOMMENDATIONS

As evidenced by the findings in this study, there is a great potential especially for self-compacting lightweight concrete with natural perlite aggregate and perlite powder. Considering this potential, it can be beneficial in future studies on natural perlite aggregate to concentrate on self-compacting lightweight concrete rather than conventional lightweight concrete. Further research topics may include the following:

1. The mechanical properties and durability of perlite containing lightweight concretes may be studied for different binder contents and w/c ratios.
2. Ternary lightweight mixtures containing silica fume, fly-ash, metakaolin, etc. may be designed to improve mechanical properties and durability characteristics.
3. Concrete members in structures such as columns and beams are normally confined by reinforcements. The perlite containing lightweight concrete specimens confined by spiral reinforcements can be studied to realistically predict its structural performance.
4. The thermal conductivity and fire resistance of perlite containing lightweight concretes may be investigated.

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