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ODAY ALI AZEZ

**UNIVERSITY OF GAZİANTEP
GRADUATE SCHOOL OF
NATURAL AND APPLIED SCIENCES**

**FACTORS INFLUENCING THE PROPERTIES OF SCLC AND
STRUCTURAL STEEL TUBE COLUMNS INFILLED WITH SCLC**

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IN
CIVIL ENGINEERING**

**BY
ODAY ALI AZEZ
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**Factors Influencing the Properties of SCLC and Structural Steel Tube Columns
Infilled with SCLC**

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in
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University of Gaziantep**

**Supervisor
Assoc. Prof. Dr. Esra METE GÜNEYİSİ**

**by
Oday Ali AZEZ
April 2017**



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Name of the student: Oday Ali AZEZ

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Approval of the Graduate School of Natural and Applied Sciences


Prof. Dr. A. Necmeddin YAZICI

Director

I certify that this thesis satisfies all the requirements as a thesis for the degree of
Doctor of Philosophy.


Prof. Dr. Abdulkadir ÇEVİK
Head of Department

This is to certify that we have read this thesis and that in our opinion it is fully
adequate, in scope and quality, as a thesis for the degree of Doctor of Philosophy.


Assoc. Prof. Dr. Esra METE GÜNEYİSİ
Supervisor

Examining Committee Members:

Prof. Dr. Mustafa GÜNAL

Prof. Dr. Hanifi ÇANAKCI

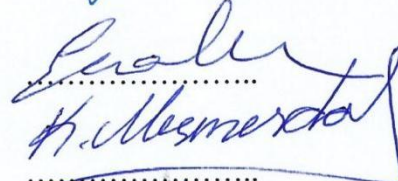
Assoc. Prof. Dr. Esra METE GÜNEYİSİ

Assoc. Prof. Dr. Kasım MERMERDAŞ

Asst. Prof. Dr. Volkan KALPAKCI

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Oday Ali AZEZ

ABSTRACT

FACTORS INFLUENCING THE PROPERTIES OF SCLC AND STRUCTURAL STEEL TUBE COLUMNS INFILLED WITH SCLC

Oday Ali AZEZ

Ph. D. in Civil Engineering

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The main utilization of structural lightweight concrete is to decrease the dead load of the structure, which then permits the structural designer to diminish the size of columns, footings, and other load bearing members. The aim of the present thesis is twofold, firstly, it attempts to identify and describe experimentally the behavior of self-compacting lightweight concrete (SCLC). Secondly, it investigates numerically the ultimate strength of concrete filled steel tube (CFST) stub columns with SCLC concentrically loaded in compression to failure. To this aim, 18 SCLC mixtures were designed by considering different water to binder ratios (w/b) of 0.25 to 0.5, nano-silica (nS) contents of 0 to 5.0%, and untreated/treated artificial lightweight aggregates. They were tested for the fresh, mechanical, and durability performance at different ages. Thereafter, to assess the compression resistance of the circular composite columns, various diameter to wall thickness ratios (D/t) of 25 to 100, steel tube yield strength (f_y) of 200 to 650 MPa and compressive strength of SCLC of 30 to 85 MPa were used in the investigation. Comparison were made with predicted column strengths using the existing design codes such as ACI, Eurocode 4, and DL/T. For each code, a total of 576 strength results for the circular CFST columns with SCLC were obtained and discussed accordingly. It was observed that the designed SCLC mixtures had comparable and satisfactory engineering performance. In addition, the results in this study showed that the codes estimated a reasonable capacity for the composite columns incorporating SCLC.

Keywords: Axial bearing capacity; Composite column; Lightweight aggregate; Performance assessment; Self compacting lightweight concrete.

ÖZET

KENDİLİĞİNDEN YERLEŞEN HAFİF BETONUN VE BU BETON DOLGULU YAPISAL ÇELİK TÜP KOLONLARIN ÖZELLİKLERİNE ETKİ EDEN FAKTÖRLER

Oday Ali AZEZ

Doktora Tezi, İnşaat Mühendisliği Bölümü

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Yapısal hafif betonun genel kullanım amacı yapı üzerindeki ölü yükü azaltmak ve bu sayede yapı tasarımcısının kolon, temel ve diğer taşıyıcı elemanların boyutlarını azaltmasına müsaade eder. Bu tez çift yönlü bir çalışma olup, ilk olarak, hafif agregalı kendiliğinden yerleşen betonun davranışları deneysel olarak tanımlanmaya çalışılmıştır. İkinci olarak, kendiliğinden yerleşen hafif agregalı beton'a sahip beton dolgulu çelik tüp kolonlar, eşmerkezli olarak basma yükü ile kırılmaya kadar yüklenmiş ve nihai dayanımı sayısal olarak incelenmiştir. Bu amaçla, su/bağlayıcı oranı 0.25 ile 0.5, nano-silika katkı oranı 0 ile 5.0% arasında değişen ve işlem görmüş/görmemiş yapay hafif agregalı 18 adet kendiliğinden yerleşen hafif agregalı beton karışımı tasarlanmıştır. Bu karışımların farklı yaşlarda taze, mekanik ve durabilite performansları incelenmiştir. Daha sonra, dairesel kompozit kolonlarının basınç dirençlerini incelemek için, çap et kalınlık oranları (D/t) 25 ile 100 arasında, çelik tüp akma dayanımı (f_y) 200 ile 650 MPa arasında ve basınç dayanımları 30 ile 85 MPa arasında değişen kendiliğinden yerleşen hafif agregalı betonlar araştırmada kullanılmıştır. ACI, Eurocode 4 ve DL/T gibi mevcut tasarım kodları kullanılarak öngörülen kolon dayanımları ile karşılaştırma yapılmıştır. Her bir kod için, dairesel beton dolgulu çelik tüp kolonlar için toplamda 576 dayanım sonucu elde edilmiş ve buna göre sonuçlar tartışılmıştır. Tasarlanan kendiliğinden yerleşen hafif agregalı beton karışımlarının tatmin edici mühendislik performansına sahip olduğu gözlenmiştir. Ayrıca, bu çalışmadaki sonuçlar, kodların hafif agregalı beton'u içeren kompozit kolonlar için makul bir kapasite tahmini yapabildiğini göstermiştir.

Anahtar Kelimeler: Eksenel yük taşıma kapasitesi; Kompozit kolon; Hafif agregalı; Performans değerlendirme; Kendiliğinden yerleşen hafif beton.



*To my wife, daughter, and son,
Rayan, Arwa, and Zaid*

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LIST OF SYMBOLS/ABBREVIATIONS

$(EI)_{eff^2}$	Effective flexural stiffness of CFST
A_c	Cross-sectional area of the concrete specimen core
A_s	Cross sectional area of steel
N_{cr}	Euler buckling load of the pin-ended CFST column
N_{pIR}	Cross-section plastic resistance of the CFST column
f'_c	Compressive strength of concrete cylinders (150×300 mm),
f_{cu}	Compressive strength of 150 mm cube,
f_y	Yield stress of steel
$\bar{\lambda}$	Relative slenderness of the CFST column
A	Cross-sectional area of the sample
A_{sc}	area of composite section of the composite section
CFST	Concrete filled steel tube
C-S-H	Calicium silicates hydrates gel
D	Outer diameter of concrete filled steel tube
E_{c2}	Elastic modulus of concrete
E_s	Elastic modulus of steel
f_{ck}	Characteristic concrete strength
f_{scy}	Nominal yielding strength of the composite section
g	Gravitational acceleration.
HRWRA	High range water reducing admixture
K	Gas permeability coefficient
L	Height of sample
LWA	Lightweight aggregate
LWC	Lightweight concrete
m	Mass of individual pellet.
nS	Nano silica
P_1	Inlet gas pressure
P_2	Outlet gas pressure

r	Radius of the pelletization disc
SCC	Self compacting concrete
SCLC	Self compacting lightweight aggregate concrete
SCM	Supplementary cementing materials
SSD	Saturated surface dry condition
t	Thickness of steel tube
T_{500}	Time required to reach 500 mm slump-flow
w	Centrifugal acceleration
WG	Sodium silicate solution
β	Angle of the pelletization disc's plane with respect to normal
μ	Viscosity of oxygen
ν	Coefficient of friction between the pellet and the pelletization disc
ξ	Confinement factor

CHAPTER 1

INTRODUCTION

1.1 General

Over the last decades, self-compacting concrete (SCC) has been known as an important progress in building construction industry and accordingly considered as the main topic of numerous scientific researches. The technology of self-compacting concrete has superior performance comparing to conventional concrete. Because of SCC consolidates under its own weight, it offers a noticeable enhancement in capability to cast fresh concrete without the need to internal or external compaction. By using SCC, absolutely any consolidating and vibration efforts will be eliminated. It can pass and/or fill all corners of the mold and complex section even with the presence of heavy steel reinforcement grids by means of its self-weight. The improvement of such type of high performance concrete is related to the scientific investigations adopted by Japanese research centers to develop conventional vibrated concrete (Okamura and Ouchi, 2003).

Occasionally, the application of SCC for building structures is limited due to its high self-weight compared to other building materials. Indeed, the significant studies that achieved to investigate the fresh and mechanical properties of SCC improved the performance of this concrete. Recently, one of the newest innovations generated from the scientific researches adopted in concrete technology is lightweight self-compacting concrete (SCLC). Thus, in the last few years there are tremendous efforts to enhance SCLC to meet the builder's necessities (Lo et al., 2007; Her-Yung, 2009; Lotfy et al., 2014).

Lightweight concrete (LWC) has been wildly used as a building material in construction sectors since the early 20th century. Typically the densities of LWC are ranging from 1400 to 2000 kg/m³. Generally, LWC characterized by higher strength/weight ratio, superior strain intensity, proper coefficient of thermal

expansion, and excellent insulation characteristics due the presence of voids in lightweight aggregate (LWA). Moreover, using of LWC provides a significant cost saving comes from the reduction in the building dead load which can lead to minimize the area of cross-section of steel reinforced columns, beams, slabs and foundations. Normally, two kinds of LWA can be used in the production of concrete; natural lightweight aggregate such as pumice, scoria, and tuff or artificial lightweight aggregate which can be produced from special process like heating clay, slate, and specific municipals solid waste (Harding, 1995; Baykal and Döven, 2000; Chandra and Berntsson, 2002; Torres and García-Ruiz, 2009; Zhang et al., 2010; Ahmaruzzaman, 2010; Hwang et al., 2012).

The increase of using lightweight aggregate concrete brought the need to produce and improve artificial lightweight aggregate. Several types of by-products materials - materials that are not the primary products of the industry producing them - are used in artificial lightweight aggregate production all around the world (Baykal and Döven, 2000). Although about 15 million tons of fly ash is generated from thermal coal-fired power stations every year in Turkey, small quantity of it is used in the construction engineering (Turkish Statistical Institute, 2010). Due to the presence of dioxins and heavy metals, fly ash considered as a hazardous waste material, and the recycle of small amount of fly ash means that the most of generated quantities are usually disposed in hazardous waste landfills and storage ponds (Kayali, 2008; Geetha and Ramamurthy, 2010; Hwang et al., 2012).

With the proper awareness and technology development, the utilization of industrial by-products and wastes powder materials in production of artificial lightweight aggregate has attracted the attentions of investigators and practitioners in the recent years as an alternative way for larger consumption (Döven, 1996; Al-Khaiyat and Haque, 1998; Videla et al., 2002; Huang et al., 2007; Geetha and Ramamurthy, 2010; Kockal and Ozturan, 2011; Gesoğlu et al., 2012; Hwang et al., 2012; Vasugi and Ramamurthy, 2014; Lo et al., 2016). The production of LWA from waste materials is an important issue since it provides an alternative mean to the dependence of construction industry on natural aggregates and the critical shortage problem of natural aggregate sources. There are other advantages in using waste materials can be explained as follows: typically, the aggregate produced from waste materials is considered as a LWA. Some of waste materials can be used as supplementary

cementitious material, thus the required amounts of cement decrease which in turn leads to lessen the emissions of greenhouse gases (Kayali, 2008).

The most commonly techniques applied to manufacture artificial lightweight aggregate are cold bonding, autoclaving, or sintering. Cold-bonding process defined as “the type of bonding method which accounts for the ability of pozzolanic powder material to react with calcium hydroxide at ordinary temperatures to form a water resistant bonding material”. This process consumes minimum energy during the production of pellets, a ball like shape of material made by agglomeration of moisturized fines with water acting as a coagulant in a rotating disc (Döven, 1996; Geetha and Ramamurthy, 2010; Güneyisi et al., 2013).

In spite of all the favorable properties of lightweight aggregate concrete mentioned earlier, the fact reflects some of difficulties associated with the use of such concrete in sites. The coarse lightweight aggregate particles segregation is the major challenge to the use of LWC. Due to the lower densities of LWA compared to the matrix, the aggregate particles tend to move upward and form aggregate lumps on the surface. Consequently, fresh mixes of LWC with high workability easily segregate than conventional concrete mixes of the similar workability. However, the employment of large volume of fine powder in the SCC components due to its design requirement enhances both of stability and segregation resistance of freshly mix (Choi et al., 2006; Abdelaziz, 2007; Rahai et al., 2015). Therefore, it can be considered that the incorporation of such aggregate in SCC with special attention may improve the quality of concrete while avoiding the segregation of aggregate particles.

It is well known that the aggregate volume in concrete mixture takes about 60-70% from the total component volume. As a result, the performance of specific aggregate significantly influences concrete properties. Additionally, the availability of different types of LWA characterized by their different properties caused distinctive behavior among SCLC. According to this, for each type of LWA the properties of SCLC must be investigated individually. The use of artificial lightweight aggregate in the production SCLC was not well reported hitherto in literature compared to conventional lightweight concrete and SCC (Müller and Haist, 2002).

It is also worthy to highlight that during the current century different supplementary cementing materials (SCMs) such as fly ash, silica fume, and slag were successfully

used to improve the mechanical and durability properties of traditional concrete due to the filling and pozzolanic reactions of such materials. However, recent investigators believe that the filling property and pozzolanic activity can remarkably be increased with small replacement level of nano-materials (i.e. nano-silica (nS)) in compared to the use of conventional SCMs (Li, 2004; Qing et al., 2007). For this reason, nowadays, considerable research activities on the use of different nano-materials in different types of concrete (except for SCLC) have been observed in the technical literature.

1.2 Composite construction

Composite sections are utilized to take the advantages of both concrete and steel properties and nowadays widespread in the worldwide. Concrete has provided a cost saving for columns and shear walls. The use of high strength concrete (over 48 MPa) in a moderate column allows carrying an extremely large axial load. Concrete supplies its potential mass, rigidity, damping, and cost efficiency, while steel contributes its speed of construction, strength, long-span competences, and light weight (Tao and Han, 2006; Lu et al., 2007).

There are two types of composite columns, namely, concrete filled steel tube (CFST) columns and wide flange units encased in reinforced concrete (Figure 1.1). Practically, the concrete filled steel tube column is favorable over the second type because the steel tube works as a formwork during cast of concrete. The steel tube of such column enhances the compressive ductility of concrete by giving confinement and preventing concrete spalling. Moreover, the concrete inside the CFST column restricts the steel tube from locally buckling. On the other side, the availability of high performance materials has made the use of CFST columns more appealing, since the member can be designed to be stronger and stiffer without increasing the column cross-section (Elremaily and Aziznamini, 2002; Giakoumelis and Lam, 2004).

Different shapes of CFST columns such as circular, rectangular, and square can be used in the construction of the composite structures. Sometimes, they can be used in different forms such as inclined, altered in cross section or even curved as seen in Figure 1.2. For example, i) the inclined columns provides the easy load transfer mechanisms for the irregular architectural systems, ii) the column with altering the

cross-section can be utilized for aesthetic or economic aims, and iii) the curved column can be employed in bridge type structures. An example about the use of the CFST column system in the tower structure is also illustrated in Figure 1.3 (Han et al., 2014).

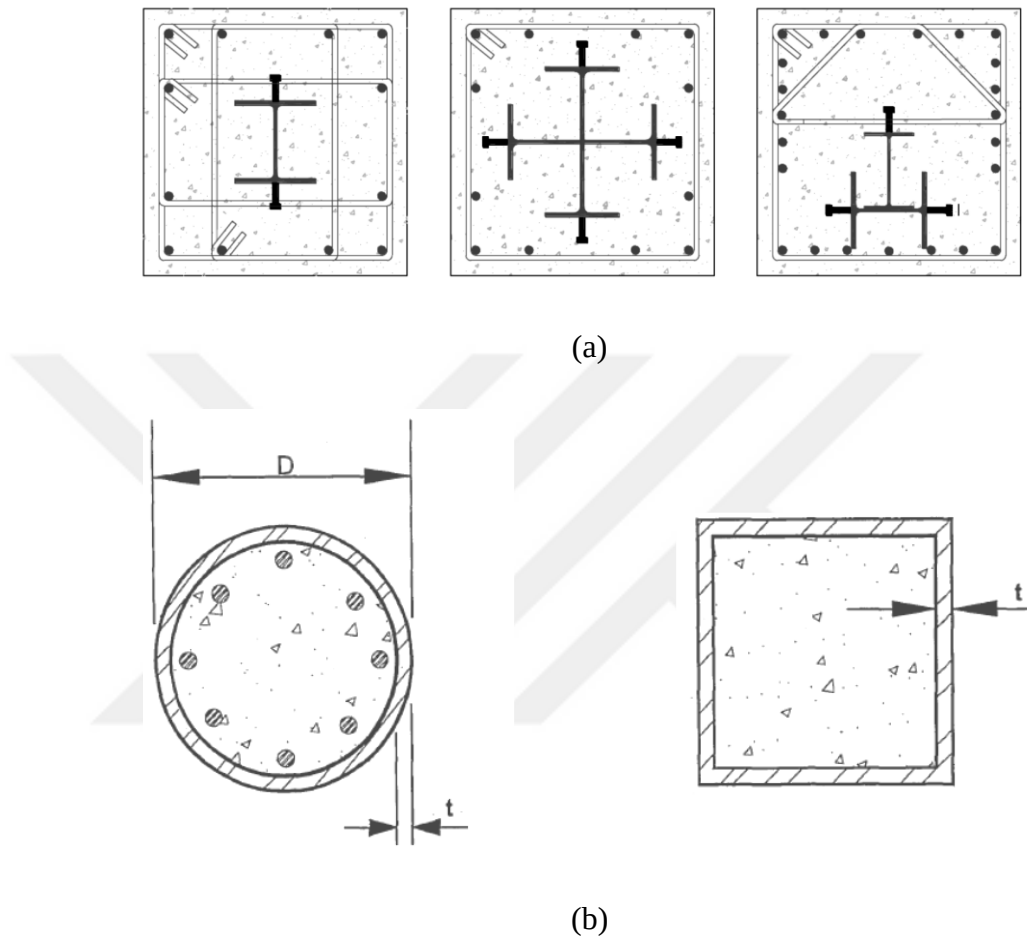


Figure 1.1 Various types of composite columns; a) Structural steel shapes encased in reinforced concrete and b) Concrete filled steel tube (Giakoumelis and Lam, 2004)

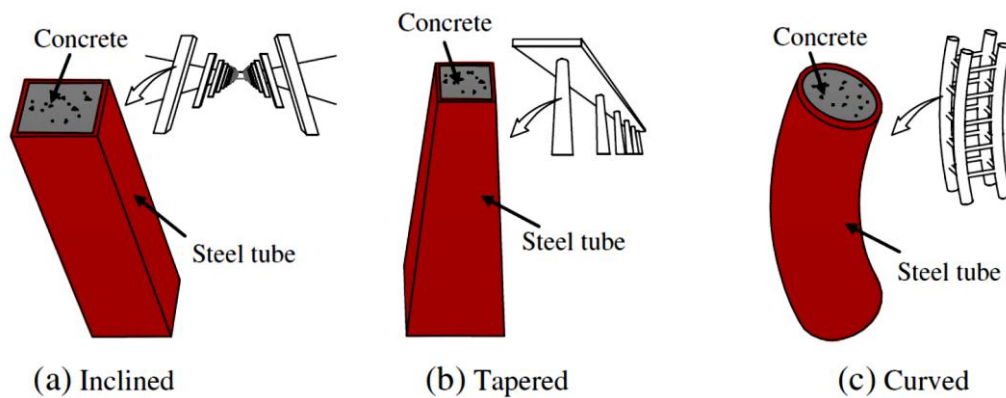


Figure 1.2 Inclined, tapered and curved CFST columns (Han et al., 2014)

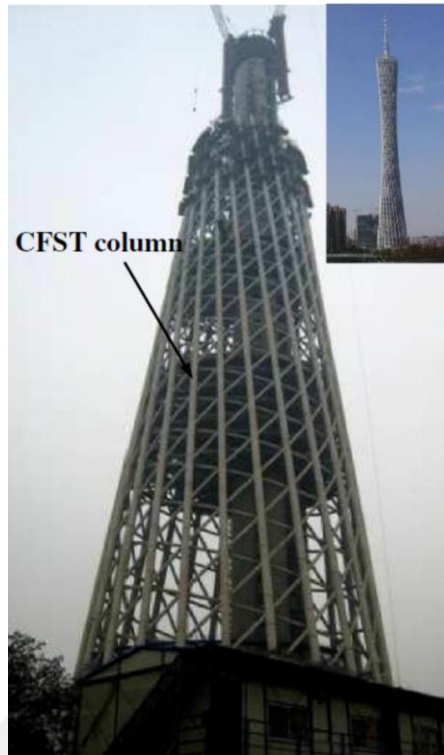


Figure 1.3 Canton tower (Han et al., 2014)

As mentioned earlier, in the recent years, the opportunity of using self-compacting concrete has been received very favorably by structural engineers. Due to its rheological properties, many benefits can be stated and new structural applications can be realized. Among them CFST columns with relatively small dimensions of cross-section and intensive reinforcement can be constructed. However, the use of self-compacting concrete will be applicable if the distance between two profiles is such that casting is almost unpractical with normal vibrated concrete (Yang and Han, 2006).

To further encourage the application of concrete-filled tubular columns, one will intend to reduce the self-weight of CFST columns. There have been several investigations about the use of lightweight concrete in concrete-filled tubular members. For example, Ghannam et al. (2004) investigated the structural behavior of concrete-filled steel tubular columns with circular, square and rectangular cross sections made with normal weight concrete or lightweight aggregate concrete. Mouli and Khelafi (2007) conducted experimental investigation on short lightweight concrete CFST columns. The high performance of lightweight aggregate concrete filled steel tube is a motivation for additional investigations in this topic.

1.3 Research significance and general objective

SCC is being more and more extensively used due to its unique structural and durability performance. Though the performance of SCC has been investigated by researchers in recent years, there is a great deal of effort that remains to be accomplished in the field. Most of the previous efforts in the field of SCC were concerned with normal weight aggregate or natural LWA. Indeed, there is a lack of knowledge concerning the incorporation of artificial lightweight aggregate in the field of SCLC. The incorporation of cold bonded LWA in SCC due to its special design requirements can be expected to enhance fresh, durability, and mechanical performance compared to LWC. On the other hand, the use of industrial by-products and wastes powder materials through cold bonded process can be satisfied the requirements of zero waste concepts. Moreover, the incorporation of nS in such concrete will be the first attempt in the technical literature.

Because of its several advantages, the utilization of composite columns has become, progressively popular. Their attractive features include greater load-bearing capacity compared to conventional concrete or steel columns with the similar size, improvement of building material performance in composite action when comparing to concrete or steel acting alone. Due to the lack of knowledge with respect to the behavior of self-consolidating lightweight concrete filled steel tube columns exposed to compressive load, a theoretical investigation was undertaken to study this issue.

In the lights of the above facts, in this thesis, two stages of the study were commonly organized. In the first stage, an inclusive experimental investigation was performed to study the effects of ameliorated cold bonded fly ash LWA on the performance of SCLC. For this, a total of 18 SCLC mixes were designed considering three water to binder ratios, two types of LWA, and three nS replacement levels. The material characterizations of SCLC were determined in fresh and hardened states. In the second stage of the study, the structural responses of the composite CFST columns made with SCLCs under the action of compressive loading were numerically investigated. To this aim, various diameter to thickness ratios and yield strengths of the steel tube were taken into account. A special procedure was adopted to take the benefit of compressive strength results of SCLCs on the behavior of CFST column. The axial capacity of the composite columns were evaluated and compared by using

three design codes, namely, American Concrete Institute (ACI), Eurocode 4 (EC4), and Chinese Design Code for Steel Concrete Composite Structures (DL/T).

1.4 Outline of the thesis

The thesis comprises of 6 chapters. **Chapter 1** offers general introduction, composite construction, and research objectives. **Chapter 2** provides a literature review which summarizes the information about nS and fly ash, artificial cold bonded fly ash lightweight aggregate, lightweight concrete, self-compacting lightweight concrete. The description, classification and structural behavior of the composite column as well as its comparison with conventional column and its practical applications are given in this chapter.

Chapter 3 comprises firstly the details of experimental program conducted throughout this thesis. Properties of materials utilized in the concrete production as well as the test procedures of fresh and physico-mechanical properties of SCLC are presented. Secondly, the details of the numerical study, design code procedures and formulas for calculating the capacity of the CFST columns having SCLC are given in this chapter.

Chapter 4 illustrates the results obtained from testing program. Moreover, the effects of experimental parameters on workability, permeability, and mechanical properties as well as drying shrinkage of SCLCs are discussed. Tables of test results and figures are included in this chapter.

Chapter 5 presents the results of theoretical study to evaluate the behavior of self-consolidating artificial lightweight concrete filled steel tube short circular columns subjected to compressive load. The design code comparisons on the calculated axial capacity of such structural members were made by means of three commonly utilized international provisions of ACI, EC4, and DL/T

Chapter 6 summarizes the conclusions of the research.

CHAPTER 2

LITERATURE REVIEW AND BACKGROUND

2.1 General

Concrete is most widely used materials of construction all over the world because of its versatility and availability. With its proper advantages such as deformability and wanted mechanical strength, concrete has been the most important element of civil engineering infrastructure for the last century and is usually preferred of the other conventional building materials (Güneyisi et al., 2014a). Concrete is a composite material that consists of three different phases; matrix, aggregates and the interfacial transition zone located between them. Therefore, the granular materials that used in concrete involve diverse sizes so as to the size range covers a wide interval. In standard concrete, cement particles can be considered the smallest as the size range of these particles normally occur between 300 nm to 90 μ m, whilst, the coarse aggregate particle can be reached up to 32 mm (Figure 2.1). As a result, the overall particle size distribution of the concrete resulted from the combination of particle size distributions of the raw materials (Neville, 1996; Quercia et al., 2012). This whole grading of the mix from 300 nm to 32 mm defines the concrete properties. Not only fresh properties are determined by the overall grading of the mixture, but also in hardened state the characteristics of concrete are significantly affected by the mix grading and resulting particle packing. One method to enhance the particle packing is to rise the size range of solid, for instance incorporating particles into concrete having sizes lower than 300 mm. Several materials can be used to attain this target, such as fly ash, micro silica, nano silica, etc. (Quercia and Brouwers, 2010).

It is well known that the use of pozzolanic materials is desirable to improve concrete performance because the Portland cement hydration products will be gained extra calcium silicate hydrates gel (C-S-H). C-S-H gel plays a key role on improving the performance of concrete. The application of silica fines in cementitious systems includes two different functions. One of these twofold is chemical effect: the reaction

between silica and calcium hydroxide (pozzolanic reaction). The second function is a physical one, because these fines can fill voids between cement particles as well as hydration products in the new hydrated cement paste, so called filler effect (Güneyisi et al., 2015a). The beneficial inspiration of silica additive on concrete has been reported from 1950s but the topic again is currently regaining considerable interest with a new focus on silica components with particles smaller by about 100 times than cement called nano silica (Oertel et al., 2014).

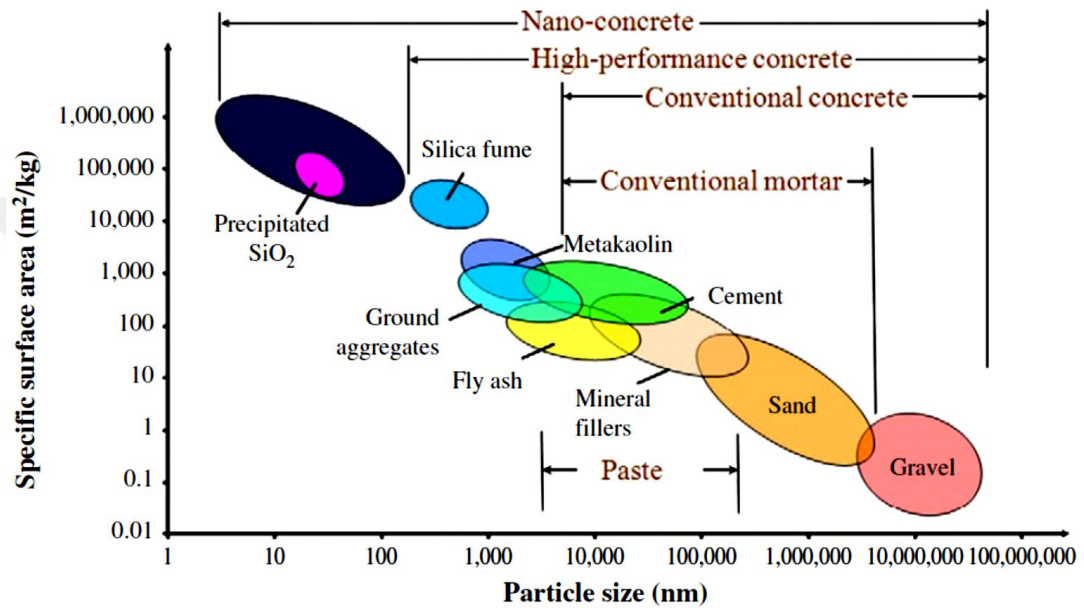


Figure 2.1 Specific surface area and size range of concrete ingredients (Quercia et al., 2012)

Management of industrial waste materials is one of the most important environmental issues. Increasing population, booming economy, huge urbanization, and the high in community living standards have significantly rise the solid waste creation in the world (Zhang et al., 2010; Song et al., 2015).

Concrete technology can be employed in the management of municipal solid waste materials like fly ash. It offers useful solutions to recycle solid waste like fly ash. Using waste powder materials such as fly ash to manufacture aggregate has attracted the considerations of practitioners and investigators. Thus, utilization of some waste materials in aggregate production can be considered as an important technique in waste management since aggregate is one of the main concrete constituents and taking about 65-75% of total concrete volume. This is because that aggregate is the least expensive of the material used in concrete leading to reduce the final cost of

concrete. In addition the shrinkage potential of a cement paste is quite high when compared to the aggregate, therefore the utilization of aggregate offers volume stability to the hardened concrete. In brief, the aggregates are incorporated to enhance economy, but more significantly do contribute considerably to the final concrete properties.

Considering fly ash for example, although there is a large amount of fly ash production up to 15 million tons per year from a wide variety of industries, only a small portion is benefited in the construction industry in Turkey. In this regard, artificial aggregates can be manufactured through processing of different materials and production methods like cold bonding pelletization and sintering. The manufacturing of aggregates by applying a cold bonding process has attracted the attention of investigators and practitioners in recent year as it is considered environmentally friendly and economical with minimum energy consumption. On the other hand, based on atomic diffusion by sintering process, the aggregate for mass production can be manufactured. In this method there is no curing period after pelletization process, because the fresh agglomerated particles are directly treated for short term with high temperatures up to 1200 °C (Baykal and Döven 2000; Gesoğlu et al., 2012; Güneyisi et al., 2013).

2.2 Nanotechnology and nano silica in civil engineering

The term of nanotechnology can be referred to the science and engineering of examining and modifying the performance of matters at scales ranging from few nanometers to submicron dimensions to generate materials with fundamentally novel specification and functions (Sanchez and Sobolev, 2010). Recently, special attentions through the visualization and measurement systems for characterizing materials at the nano-scale caused to gain nanotechnology based materials popularity in areas such as plastics, polymers, medicine, etc. For a conventional material until now, one would start with it and also can be considered for probable uses. But with the presence of nanotechnology, it is possible to develop the material specifically for an intentional use. So, the engineers can describe the end product that is required and thereafter design the material is best suitable to offer such a product (Birgisson and Roque, 2006).

Different scientific fields have been applying nanotechnology by innovating or developing materials with new functions. The applications of nanotechnology in civil engineering are new areas of interest and were increasing day by day. Applying nanotechnology in civil engineering industry significantly affects the quality of projects due to the improvements in (strength, durability, ductility, etc.) of building materials. As another advantages, combining the management of wastes and nanotechnology reduces the harmfulness of hazardous by products and provides superior performances of structural components (Nazari and Riahi, 2011a).

The most important products of nanotechnology have been commonly identified as nanoparticles (Hosseini et al., 2011). According to their unique physical and chemical characteristics as well as fine dimensions; the nano scale-size of particles can yield in dramatically enhanced properties from conventional grain-size materials of the same chemical composition. Concrete can be nano engineered by the utilization of nano-sized scale to optimize the behaviour of material and performance needed to significantly improve its properties (Jo et al., 2007). Silica nano-particles are more useful from among different available nano particles in the market for the production of cement-based composites. Their particles are spherical with sizes varying from 1nm-500nm. It is a pozzolan which has high rate of pozzolanic reaction because of its high surface area to volume ratio, providing the potential for major chemical activity (Salemi and Behfarnia, 2013). The use of conventional pozzolanic material in place was diminished as the use of nano silica was increased because the pozzolanic activity at early ages of the former lower than of the latter due to the higher of rate of consumption of calcium hydroxide Ca(OH)_2 crystals (Bahadori and Hosseini, 2012). It is thought that nano silica reveals higher pozzolanic reactivity and much finer particle sizes than silica fume, therefore can act as fillers, pozzolan and seeds more efficiently (Zhang et al., 2012).

Generally, nano silica is not industrial by-product, like fly ash and silica fume, but they are created product. It can be produced by several methods such as precipitation, vaporization of silica, biological method and sol-gel process. The production method significantly affects the main characteristics of nano silica; like particle size distribution, specific surface area, specific density, purity, and pozzolanic activity (Sobolev et al., 2006; Quercia and Brouwers, 2010). Among different biological technologies, sol-gel process is one of the widely used production methods for nano

silica. This process can be applied to produce spherical nanoparticles of SiO_2 within the size range of 1 - 100 nm. In this method, a network in a continuous liquid phase (gel) creates from the gelation of a colloidal suspension (sol). The principle steps of sol-gel process (Figure 2.2) can be summarized as follows:

- Raw materials are added in a solvent (hydrolysis of the precursor)
- Forming particles by condensing and polymerizing of monomers.
- Growth of particles.
- Formation of gel structure.
- Drying to remove the solvents (optional).
- Thermal treatment to obtain the desired a nano sized crystal materials suitable for use in concrete industry.

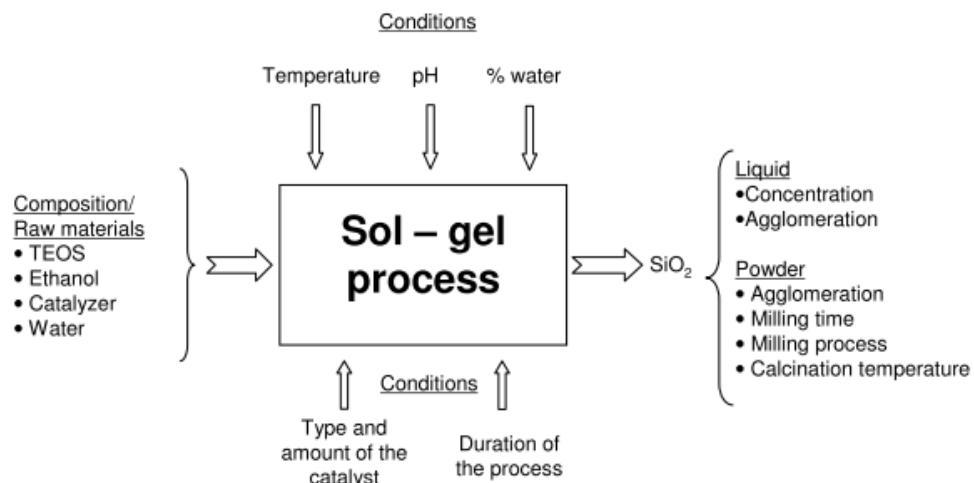


Figure 2.2 The experimental setup for nano silica synthesis using sol-gel method (Sobolev et al., 2006)

Producing nano silica by vaporization method involves vaporization of silica in an electric arc furnace between 1500 to 2000 °C and reducing SiO_2 (quartz) . Further, nano silica is manufactured as a by-product of the production of ferro-silicon and silicon metals alloys, where it is collected by subsequent condensation to fine particles in a cyclone. The nano silica powder produced by this method consists of spherical or microspheres particles with a main diameter and specific surface area of 150 nm and (15 to 25 m^2/g), respectively (Jonckbloedt, 1997).

2.3 Use of pulverized fuel ash in engineering

Material has a pozzolanic activity due to the presence of SiO_2 and/ or SiO_2 and Al_2O_3 is commonly known as a pozzolan. It possesses little or no cementitious property, for this reason such materials can be classified as a pozzolanic or as a self-cementitious and pozzolanic material. However, at ordinary temperatures, pozzolan in a finely divided form react with calcium hydroxide Ca(OH)_2 in the presence of mixture to form products having cementitious properties. Fly ash (pulverized fuel ash) is the best known, and one of the most commonly used pozzolans in the world. It is created in coal-fired electric power plants by burning process of pulverized coal. The amount of fly ash was increasing day by day worldwide due to the growth of thermal power stations and factories. The quality of fly ash strongly depends on carbon content and fly ash with low levels of carbon is utilized as a mineral admixture in concrete production. Generally, fly ash is grey in colour, abrasive, alkaline, and refractory. The properties of the fly ash, such as specific gravity, permeability, internal angular friction, etc., are affected by type of power plant, the composition of feed coal, chemical composition of the raw material, combustion operating system, and conditions of deposition of fly ash (Mehta and Monteiro, 2006; Ahmaruzzaman, 2010; Lo et al., 2016).

There are several advantages of utilization of fly ash as a mineral admixture (or supplementary cementing material) in concrete. These are: Environmental advantages: by using fly ash in the construction industry a reduction of cost of fly ash disposal, efficient land use, and an equivalent decrease in CO_2 emissions associated with a reduction in production of Portland cement. The use of raw materials such as limestone and fuel in Portland cement production obtained a large amount of CO_2 . Thus, the equivalent CO_2 emissions will be decreased due to the use of fly ash as a supplementary as a supplementary cementitious material in concrete. The second environmental advantage refers to the decrease of fly ash disposal in landfills. Landfill area is a big matter particularly in countries with limited dry-land space (Naik, 2005; Xu, 2008).

Fly ash can improve the concrete workability. The smooth spherical shape of fly ash particles reduce the quantity of water needed to yield a specific slump. On the other hand, the required volume of sand to reach target workability can be reduced when

fly ash is one of the constituents in the mixture of concrete (Ahmaruzzaman, 2010). Incorporating fly ash in concrete enhance the transport and durability properties. The pozzolanic reaction resulted from fly ash leads to fill the capillary pores in the microstructure of concrete and disconnects them due to extra formation of solid C-S-H. However, by reducing the permeability, the penetration rate of aggressive agents reduce significantly, thus, the risks of corrosion of steel embedded in the concrete will lessen (Lee et al., 2003). The addition of fly ash to concrete can be used to reduce the alkali-silica reaction and improves the concrete ability to resist sulphate attack. By using fly ash, the total quantity of aluminates decreases in a concrete mix. Therefore, the probability for expansive compounds and forming cracks drops (Ahmaruzzaman, 2010).

2.4 Lightweight aggregate

Generally, the aggregates that weigh less than 1120 kg/m^3 are considered as a lightweight irrespective of its kind (natural or artificial). The light weight of the aggregate is owing to the cellular or porous microstructure. The common source of natural lightweight aggregates is igneous volcanic rocks which are available in specific areas of the world. The crushing of some rocks such as pumice, scoria, or tuff are used to prepare natural lightweight aggregates. Synthetic lightweight aggregates involve expanded shale, clay, slate, fly ash, waste glass, and sewage slag. These materials are more consistent in their characteristics and offer more predictable properties of the hardened concrete (Baykal and Döven, 2000; Hwang et al., 2012; Öz, 2014). However, regardless the type of LWA even it natural or artificial, ASTM C330 (2009) reported the requirements for LWA.

2.4.1 Synthetic fly ash lightweight aggregate

Artificial lightweight aggregate are utilized in several application like concrete, bricks and water filtration process. In spite of their origin, different raw materials can be used to produce artificial lightweight aggregate, i.e. industrial by-product materials such as sewage sludge, waste glass, fly ash, etc. or natural raw materials like clay (Huang et al., 2007). Artificial lightweight aggregate prepared from waste materials have a positive impact on the surroundings, the community, and the construction industry. The use of waste materials in aggregate production instead of

using natural resources leads to decrease consuming of natural resources and detrimental activities of aggregate mining are avoided. However, the aggregates manufactured with waste materials can be lighter compared to the natural aggregate, which in turn leads to lightweight concrete (LWC) produced using such aggregates. In addition, the manufactures of greenhouse gases diminish as the necessity for large amounts of cement is reduced (Kayali, 2008).

In Turkey, there are several active coal-burning power plants with annual fly ash fabrication of about 15 million tons. The aim of using fly ash for LWA manufacture is an achievable way for recycling of wastes. Developing fly ash LWA can be attained by sintering, autoclaving and cold-bonding methods (Döven, 1996; Kockal and Ozturan, 2010). The cold bonding process is cheaper than other processes because of lower energy consumption needed in the production of aggregate.

2.4.2 Production of fly ash lightweight aggregate

In general, it can be said that the same basic procedures can be followed to produce LWA from clay, shale, sludge, or fly ash. Most of processes can be summarized in these three main steps.

Step1. Preparing the mixture from the mix of dry powder of raw material with water and/or additive so as to modify the engineering properties of the green pellet for hardening, transport, processing etc.

Step2. Producing of granular green pellets through granulation process.

Step3. Applying of sintering, hydrothermal or cold bonding processes to harden the fresh pellets.

Step4. Finally, the hardened pellets are crushing and/or sieving to produce the required fraction.

Hardening will be achieved at different temperatures dependent on the kind of binding of the LWA to be obtained. Several techniques of hardening are commonly possible utilizing fly ash, as can be seen below in Table 2.1.

Thermal hardening procedures such as autoclaving and sintering are applied to further develop the fly ash aggregate characteristics to improve the strength of pellets

and decrease water absorption. Sintering is an energy severe method, and yet it yields in higher strength aggregates (Kockal and Ozturan, 2011). Cold bonding provides an energy effectual environmental friendly alternative to sintering. It is a type of matrix bonding, which depends on the ability of fly ash to react with calcium hydroxide at ambient temperature to create a water-resistant bonding material and pozzolanic reactivity of fly ash recycled. Pelletized aggregates are left to cure for several days to produce an aggregate with proper strength to be used in concrete production (Döven, 1996; Kockal and Ozturan 2011). Autoclaving method is based on the use of pressurized saturated steam curing. Aggregate pellets are manufactured from 45% quartz sand, 47% fly ash, 4.5% lime, 2.0% additives and 1.5% water by weight. After pelletization process the pellets inflated in the presence of high moisture and then fired up for 6.5 hours at 200 °C so as to generate LWA (Geetha and Ramamurthy, 2010; Kockal and Ozturan, 2010). Sintering method is depending on the formation of a ceramic matrix. The ceramic matrix contains of alumina silicates as the pelletized pellets prepare from fly ash and clay. The sintering temperature is typically in the range 1100-1200 °C. In sintering method, because the aggregate particles, immediately after pelletization process are treated with high temperatures, no long term curing periods are required. The particles retaining on 2.5 mm sieve are utilized for the fabrication of LWC (Kockal and Ozturan, 2010).

Table 2.1 Hardening processes of artificial lightweight aggregate (Puri, 2015)

Method	Temperature °C	Type of bonding material
Cold bonding	Room temperature	Matrix with calcium alumina silicate hydrates
Autoclaving	100-250	Matrix with calcium alumina silicate hydrates
Sintering	1100-1200	Material bridge bonding with alumina silicates
Ultra sintering	≥1200	Alumina silicates

2.4.3 Pelletization process

Pelletization, one of the widely used processes, in powder metallurgy for size enlargement (with or without addition of binders and/or additives), has been adapted

for alleviating problems associated with fine particles for mineral, food, chemical ceramic, fertilizer, pharmaceutical, nuclear and waste treating. The fundamental of pelletization process was innovated by A. G. Anderson in the turn of the twentieth century and then devised by C. A. Brackelsberg. The revised balling process was performing though the addition of binder to the fines agglomerated and at high temperature the pellets were treating to prepare the final product. However, no published research on pelletization process founded until 1944 (Firth, 1944) and 1950 (Tigerschiold and Ilmoni, 1950). In 1960 Goldstick (1962) presented a systematic literature survey of the agglomeration investigation researches in which the investigator illustrates the major properties of fine particle aggregates through pelletizing and sintering, commercial practice of agglomeration and associated measuring performances. Besides the above mentioned reviews it can be concluded that for agglomerating fine-grained concentrate the pelletization process became economically practicable technique in 1950s and the first pelletization plant was founded in Sweden with 10-60 tones daily capacity. In 1955, the world's first factory started to yield 6 million tons of annual pellet process and later followed by the other countries (Döven, 1996; Baykal and Döven, 2000; Gesoğlu, 2004; Öz, 2014).

In construction industry, pelletization method is used to provide the desired grain size distribution of aggregate through the agglomeration of moisturized fines that has cementitious property either by themselves and/or by mixing them with special additives in special devices such as discs or drums (Vasugi and Ramamurthy, 2014). At the end of pelletization process, fresh pellets must be treated to gain particles proper strength. In general, the efficiency of pelletization process significantly depends on several factors such as; engineering properties of fines pelletized, extent of moisture of the medium, the decline degree, and rotation speed of the disc being used. In the pelletization technique, the fine particles are adhered together without applying direct forces. This is because the bonds in agglomerated grains resulted from tumbling and capillary forces. As soon as moisturizing of fine-grained started, a thin watery shell forms on the external surface of fine particles which cause to create a rudimentary bonding system as presented in Figure 2.3 a. The revolution of disc during pelletizing process enhanced bonding forces between particles with accompanying change in structure (Figure 2.3 b and c) (Jaroslav and Ruzickova, 1987).



Figure 2.3 Mechanism of pellet formation during pelletization process (Jaroslav and Ruzickova, 1987)

In pelletization process there are three states of cohesion depending amounts of liquid as presented in Figure 2.4:

- Pendular state, in this state the water exists at the contact points of grains.
- Funicular state, in addition to the previous state condition, this state characterized by some of pores completely fills with water.
- Capillary state, this state is considered the most desired one, as the intergranular voids filled with water without formation of water film on the pellet surface, hence, the utmost tension force be applied by binder between the grains.

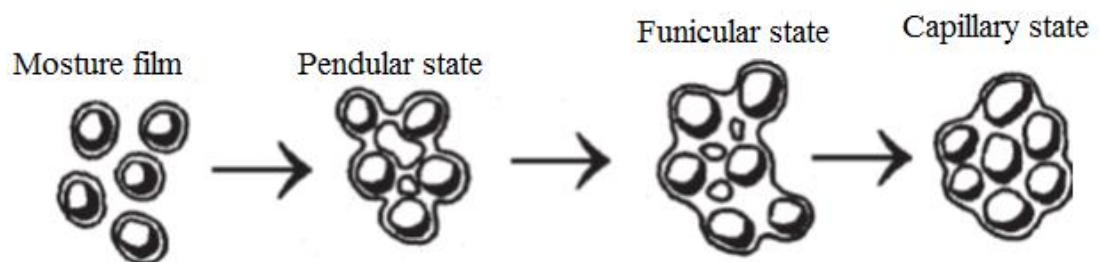


Figure 2.4 Diagrammatic representation of pellet formation (Chandra and Berntsson, 2002)

The most important mechanical parameter influence pelletization process is rotation speed of disc. This parameter managed by gravitational force and centrifugal force and significantly affects by operation tilt angle and disc diameter. In order to refrain from prepotency of gravitational or centrifugal forces, the forces following up on a pellet while pelletizing process continues in a pelletization disc with radius "R" must

be in equilibrium, Figure 2.5 (Pietsch, 1991). In the figure, m is the mass of individual pellet, g is the gravitational acceleration, β is the angle of the disc's plane with respect to normal in degrees, R is the expressions the radius of the disc, v is the coefficient of friction between the pellet and the disc, w is the centrifugal acceleration in radians/sec².

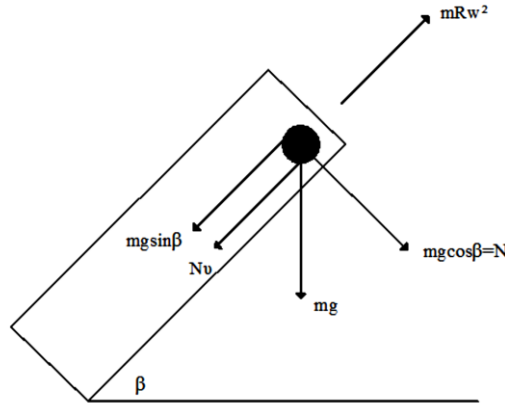


Figure 2.5 Types of forces acting on an individual pellet while pelletization process continues (Pietsch, 1991)

2.5 Lightweight aggregate concrete

Nowadays, in many applications of modern construction industry the demand for LWC is increasing due to the superior property that lower density results in a significant advantage with respect to the load-bearing elements of smaller cross sections and an identical reduction in the dimension of the foundation. For structural purposes lightweight aggregate concrete typically has density range from 1400 to 2000 kg/m³ compared with that of 2400 kg/m³ for normal weight concrete (Zhang and Gjorv, 1991; Neville, 1996).

The density of LWC can be designed by modifying the volume fraction of LWA. With respect to coarse particles, instead of normal aggregate, the use of artificial lightweight aggregate, mostly yields concrete having 2000 kg/m³ mass. At the same line, when fine lightweight aggregate is also implemented, the density is reduced more to reach 1600 kg/m³ or less. Thus, two types of concretes can be produced according to size of artificial lightweight aggregate particles; these are sand-lightweight concrete and all-lightweight concrete (JSCE, 2002; ACI 213R, 2003).

Bijen (1986) presented an overview regarding the manufacturing of fly ash lightweight aggregate. He described agglomeration and hardening techniques that adopting to produce such aggregate. His study concluded that the sintering processes accept fly ash with relatively high carbon contents, whilst autoclaving or cold bonding processes prefer low carbon fly ashes. On the other side, he believed that the strengthening of fresh pellets is the result of development of C-S-H gel throughout the reaction.

Zhang and Gjorv (1991) introduced an investigation of some artificial high-strength lightweight aggregates available on the European market. Water absorption, aggregate strength, and particle density were depended on types of pores, presence of voids and fissures, and characteristics of particles microstructure, respectively. They found that the aggregate absorbed within first 2 min through water absorption test most of the absorbed quantity after first 30 min. However, more than 50% of the 24 hr water absorption yielded within the first 30 min. The most important conclusion from this study was that the presence of dense shale for aggregate particle decreases water absorption significantly even the particles involved open pores.

Yamashita et al. (1992) tested an experiment to investigate the pelletized fly ash by means of the proper sintering conditions. In their study, fresh pellets were prepared by agglomerating process in a disc pelletizer from a mixture of fly ash, clay, and water. The experimental results revealed that the quality of LWA is greatly influenced by sintering temperature during pelletization process. Moreover, they also evaluated the optimum sintering temperature for aggregates having specific gravity of 1.40.

Baykal and Döven (2000) researched about the engineering properties of cold bonded artificial fly ash lightweight aggregate. They produced aggregate from three types of mixtures; (100% fly ash), (92% fly ash and 8.0% lime), and (92% fly ash and 8.0% cement). The utilized type of fly ash was Class-F. The study indicated that the use of additives increases crushing strength and specific gravity of aggregate. The aggregate produced within their study was adequate for nonstructural and semi structural concretes.

Geetha and Ramamurthy (2011) focused on the use of low calcium bottom ash to produce sintered aggregates through pelletization processes. They used wide range of

clay as binders in their study and involved other factors; moisture content, Ca(OH)_2 amount, sintering temperature and duration of sintering. The results induced from this study advised that for 10% bentonite containing aggregates sintering temperature and duration of about 1000 °C and 95 min. offered optimum suitable ties for the artificial aggregates formed from pulverized bottom ash. They also observed that the increase in binder quantity and sintering temperature led to decreased water absorption of manufactured aggregate.

An optimization studies were conducted by Döven (1996), Videla and Martinez (2002), and Gesoğlu et al. (2012) to define the angle of disc plane to the normal and the revolution speed of the disc during the manufacture of fly ash aggregate through pelletization process. Döven (1996) shown that for a disk with a diameter of 40 cm, the inclination angle and the optimal revolution speed were 43° and 35 rpm, respectively. Videla and Martinez (2002) said that for a 20 mm maximum size aggregate, the best size distribution was achieved with a 57° angle and 21 rpm speed in the pan with a diameter of 150 cm. Gesoğlu et al. (2012) reported that the revolution speed and the inclination angle of disc were obtained as 42 rpm and 45°, respectively, for the pelletizer.

2.5.1 Mechanical properties of lightweight aggregate concretes

With respect to ACI 213R (2003), there are many kinds of LWA available and their characteristics extend to cover wide ranges. Elastic properties, mechanical properties, durability, elevated temperature resistance, and other properties of LWC are influenced by the type of LWA incorporated in the concrete. LWC for structural applications can easily manufactured from artificial aggregate produced from fly ash, clay, shale, slate, and blast-furnace slag. In spite of high porosity and inherent weakness of LWA, 28-day compressive strength of higher than 50 MPa can be obtained for LWC. The materials utilized in LWC mix design define their engineering characteristics. LWC for structural purposes characterize by a high ratio of compressive strength to density. Generally, high performance LWC are produced utilizing rotary kiln expanded shale, slate or clay. These LWA are relatively having low weight due to the cellular structure of the aggregate particles. Harding (1995) has proposed a classification which defines the relation between compressive

strength and air dry unit weight in accordance with LWA used as presented in Figure 2.6.

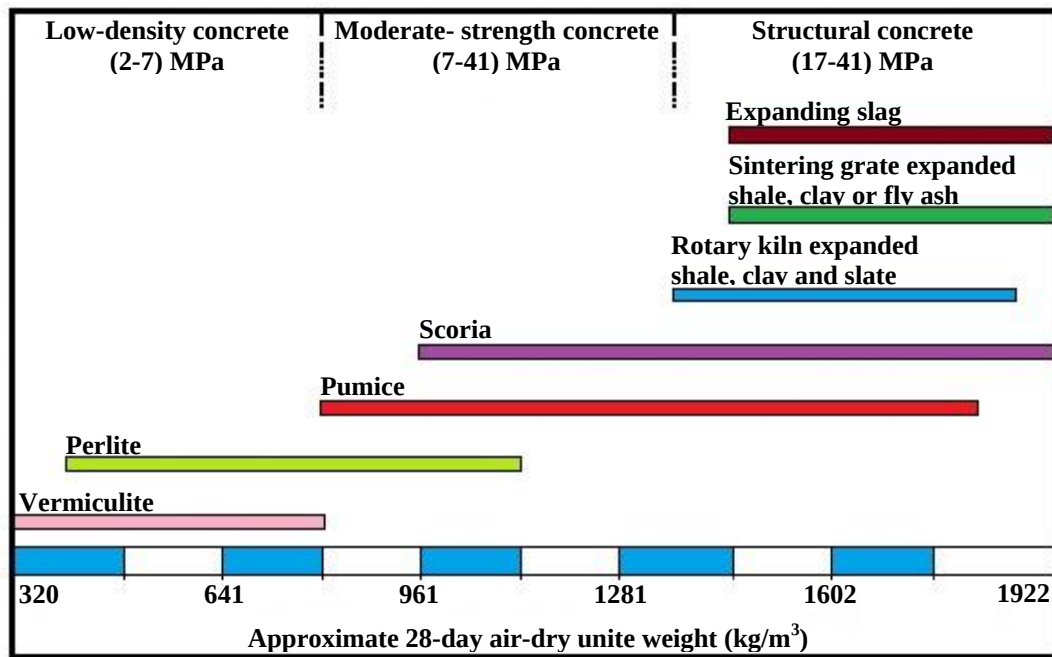


Figure 2.6 Classification of lightweight concrete and types of aggregates (Harding, 1995)

It is well known that the modulus of elasticity influences by the relative volume of paste and aggregate and their modulus of elasticity value. Due to the high modulus of elasticity of normal aggregate compared to LWA, ACI 213R (2003) reported that the modulus of elasticity of LWC is ranged between 0.5 to 0.75 that of normal weight concrete. Figure 2.7 shows the variation in modulus of elasticity values for three types of concrete. On the other hand, the lower elastic modulus of LWC would also propose less restrain to time dependent deformations like creep and drying shrinkage.

Zhang and Gjorv (1991) designed LWC has strength and density of 100 MPa and 1865 kg/m³, respectively. The concrete produced with unconventional types of LWA. The study related the strength of concrete to the strength of aggregate. Comparison of LWC with normal weight concrete in terms of modulus of elasticity and ultimate strain at peak load showed that lower modulus of elasticity and higher ultimate strain resulted for lightweight aggregate concrete.

Yang and Huang (1998) discussed the impact of the volume fraction of LWA on the mechanical properties of lightweight aggregate concrete. They prepared three types

of artificial coarse aggregates from cement and fly ash with different blends through a cold-pelletizing technique. Beside the experimental study, they also conducted a numerical approach by using a micromechanics method to assess the compressive strength of aggregate depending on the tested composite strength. They believed that the typical stress in aggregate is the aggregate strength when the concrete reaches its maximum strength. Their theoretical study concluded that the modulus of concrete and mortar are related to the volume ratio and equal about 2 to 5 of aggregate modulus.

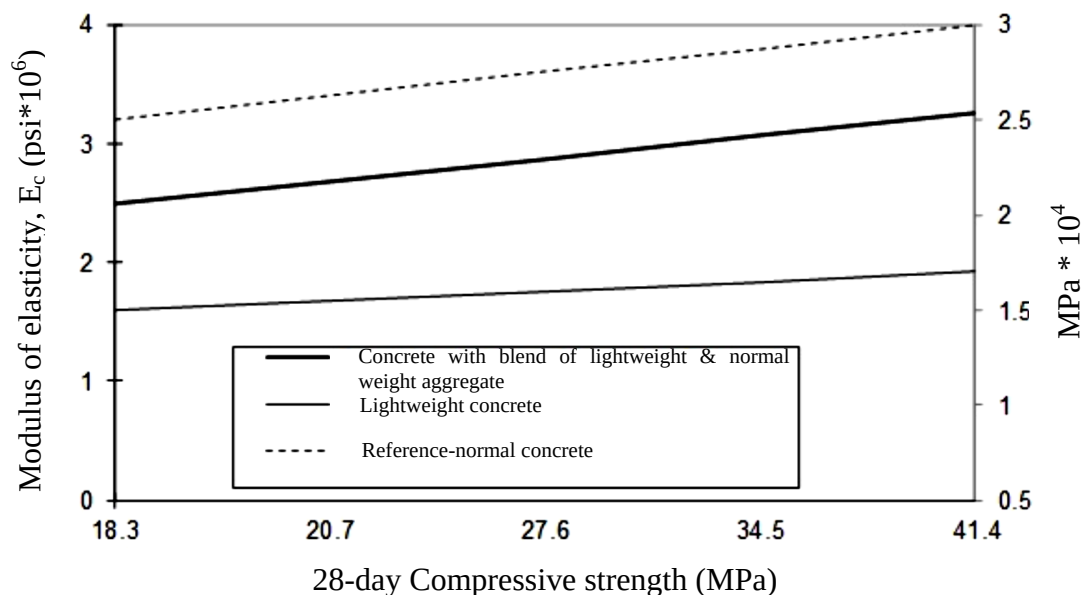


Figure 2.7 Modulus of elasticity for different types of concrete (ACI 213R, 2003)

Babu et al. (2005) presented study about the mechanical properties of concrete containing expanded polystyrene lightweight aggregate and fly ash as a mineral admixture. They reported that the compressive strength decreases systematically with the incorporation of LWA (linear behavior with the concrete density). The ultimate strains observed in the lower density samples were higher and the stress–strain curves showed lower steepness as the percentage of expanded polystyrene increased. The addition of 10% expanded polystyrene was reduced secant modulus value by about 40%.

Ke et al. (2009) selected volume fractions and aggregate qualities as a parameters to investigate the effect of lightweight expanded clay and shale aggregates on the mechanical performances of concrete. The study concluded that the elastic modulus

and the compressive strength of LWC incorporating expanded shale and clay are affected by the investigated parameters of which density less than 1000 kg/m^3 . They also reported that the impact of LWA was not only affected by aggregate density but also to the outer macro porosity, thickness of shell and presence of large pores.

Torres and Ruiz (2009) used expanded perlite, expanded glass, hollow micro-spheres and expanded polystyrene in concrete production. They aimed to investigate the effect of such types of aggregate mentioned earlier on the physical and mechanical properties of LWC. The results yielded from experimental program indicated that the density, water absorption, and compressive strength were influenced due to the use of LWA. Depending on concrete density, the compressive strength of concrete was reduced owing to the use of polystyrene and perlite as a LWC. At the same line, expanded glass and hollow micro-spheres significantly enhanced water absorption in comparison to the samples implemented with expanded perlite and expanded polystyrene.

Alengaram et al. (2011) studied the influence of oil palm kernel shell as LWA on the properties of concrete. It is seen from the experimental study in this paper, that the properties of this concrete is comparable with normal weight concrete in terms of mechanical and durability characteristics. Their study also involved the effect of water to binder and sand to cement ratios in LWC incorporating palm kernel shell. The compressive strength and elastic modulus results of the specimens reveals that increase in sand content which is fixed with decrease in palm kernel shell cause to enhance the compressive strength and modulus of elasticity. They reported that the modulus of elasticity is influenced by LWA kind, cement type, water to binder ratio, and age of curing.

As we know in lightweight aggregate concrete the aggregate is the weakest component in concrete. Thus it is normal the high amount of cold bonded fly ash lightweight aggregate reduces the compressive strength of concrete. Chi et al. (2003) produced three types of cold bonded lightweight aggregate (B1, B2, and B3) by using various cement to fly ash ratio so as to investigate compressive strengths and elastic modulus of cold-bonded lightweight aggregate concretes. They increased the replacement of coarse cold bonded fly ash lightweight aggregate by 18%, 24%, 30%, and 36%. As presented in Figure 2.8, the compressive strength decreases gradually

with the increase in LWA volume fraction regardless the water to cement ratio. Gesoğlu et al. (2004) also reported similar behavior to the previous mentioned study. They increased volume fraction of coarse cold bonded fly ash lightweight aggregate by 30%, 45%, and 60% and used different water to binder ratio of 0.35 and 0.55. They showed that the compressive strength decreases gradually by 86%, 60%, and 40% for the concretes having replacement volume fraction of 30%, 45%, and 60% of LWA, respectively.

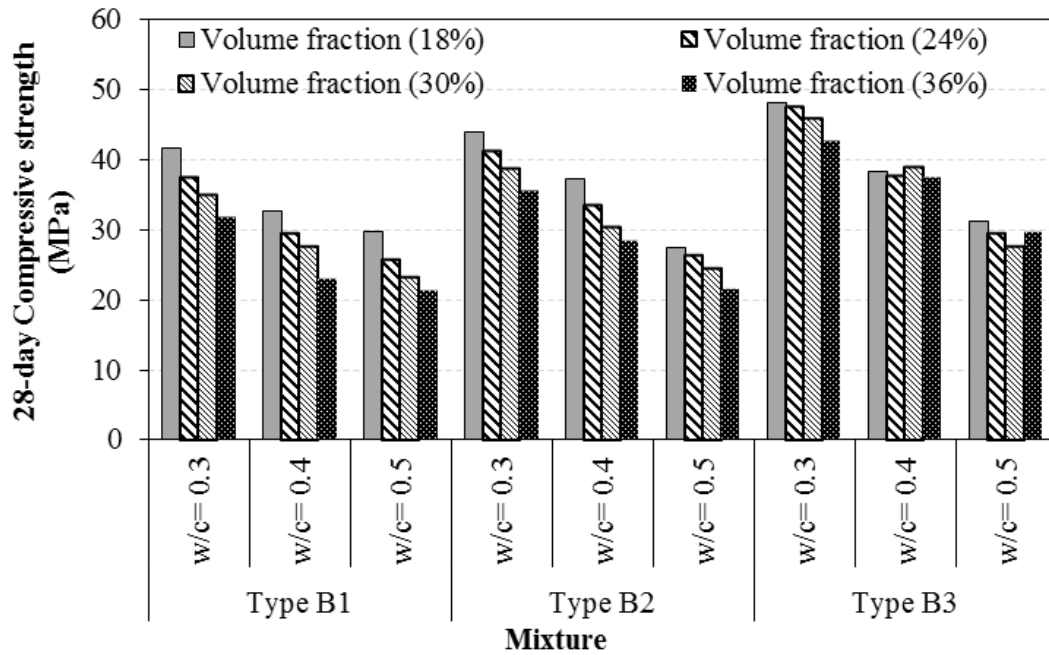


Figure 2.8 Variation in compressive strength for different lightweight aggregate volume fraction (Chi et al. 2003)

Du et al. (2015) presented special consideration to the examination of the effect of incorporation of nano-silica on the mechanical and permeability properties of LWC. LWA type expanded clay spheres were utilized in the study. The result of study concluded that the compressive strength increased with higher nano-silica level regardless the test age. An enhancement of 7.7% and 13.3% was detected for 1% and 2% nano-silica addition, respectively, at the first day. But, this improvement was dropped to be 2.6% and 5.6% at 28-day. For slag cement lightweight concrete, 1% and 2% addition of nano-silica improved 12.6% and 9.7% compressive strength on the first day. At 28 days, the improvement turned into marginal for 1% and 2% nano-silica even slightly decreased the LWC. The depth of water penetration, sorptivity, chloride migration and diffusion coefficient was lessened by 23% and 49%, 23% and

10%, 5% and 0%, 22% and 12% with respect to the reference mixes, respectively, with 1% nano-silica.

2.5.2 Permeability and drying shrinkage of lightweight aggregate concrete

The property that governs the rate of flow of a fluid into a porous solid is defined as permeability. Permeable concretes are, of course, less durable. Concrete with high permeability will offer easily to reach harmful elements by water into concrete in which causing the deterioration of concrete and steel reinforcement. The permeability of concrete influences by many factors like; mix proportions, aggregate types, compaction, and curing or even on micro-cracks caused by the ambient temperature and humidity cycles. It was reported that there is a relation between the size of voids and hardened properties of concrete. Stated that the capillary voids smaller than 50 nm considered as micropores and plays a role in shrinkage and creep, whereas the voids that larger than 50 nm classified as micropores and possesses ability to impact on strength intensity and impermeability characteristics (Mehta and Monteiro, 2006).

Chia and Zhang (2002) focused on the comparison between LWC and high strength normal weight concrete with respect to water permeability and chloride ion penetrability. The results indicated that regardless whether silica fume was incorporated, the water permeability values of the two types of concrete produced was identical. Their study also clearly concluded that the quality of the mortar matrix is the key to permeability of the high-strength concrete irrespective of the coarse aggregate utilized. The durability of normal weight concrete to the chloride penetration was similar to that of LWC in the normal and strength levels.

Babu and Babu (2003) developed structural lightweight concretes of 1500-2000 kg/m³ with the corresponding compressive strengths of about 10-21 MPa. The investigation studied compressive and split tensile strengths, and the durability related properties such as water absorption, chloride permeability, corrosion rate, and accelerated corrosion cracking. The study revealed that the expanded polystyrene concrete showed absorption rating below 3% at 30 min. Based on the results related to average daily resistance and the charge to cracking, they said corrosion progress mostly depend on the composition of matrix and are not so much effected by the properties of aggregate.

Sandvik and Hammer (1995) estimated the durability of LWC in terms of permeability and resistance to chloride ingress for Norwegian marine structures. The comparison between lightweight and normal concretes showed that the depths of fresh and sea water were equals for comparable paste compositions. It was found that the water absorption not related to LWA pores as their particles were coarse and filled with water. Consequently, the kind of aggregate incorporated does not contribute to permeability of concrete. Any deterioration of the paste such as bad curing and cracking permits a little access of water to aggregate would probably rise the permeability of concrete. The chloride intrusion depths found from the sea water capillary absorption tests also are not significantly different between lightweight and normal weight concrete with similar cementitious materials proportions.

Güneyisi et al. (2013) examined the durability of concretes composed of cold bonded and sintered fly ash LWA. Several tests conducted in the study; compressive strength was measured as mechanical property while water sorptivity, rapid chloride permeability, and gas permeability of concretes were adopted as transport properties. Moreover, the corrosion behavior of concretes was observed through an accelerated corrosion testing. Based on the results of this study the better quality of aggregate (sintered fly ash lightweight aggregate) caused to improve mechanical property. The aggregate category was also appeared to be effective on the improvement of capillary water penetration behavior of concretes. One of the most remarkable note resulted from study is the addition of silica fume for concrete made with cold bonded aggregate cased to decrease total charge value about 14% whilst for concrete contained sintered fly ash lightweight aggregate the previous value extent to be 34%. The resistances of the concrete against corrosion cracking were demonstrated to be improved through using silica fume and especially sintered aggregate.

Drying shrinkage is a common phenomenon generally defined as the volumetric change due to loss of moisture. The volume of water lost not parallel to the change in concrete volume. However, phenomenon of shrinkage can be considered as a material property that important outcomes on long-term performance of designed structures (Rashad, 2014). The initial losing in cementitious product of free water has unimportant effect upon drying of concrete. But, the continuation of concrete drying, the adsorbed water held by hydrostatic tension in the micropores (smaller than 50 nm) is drive out. In this stage the water loss is very important and causes a tensile

stress leading to shrink of concrete. Some of cracks through concrete especially in members whose surface area to volume ratio is large can be caused by drying shrinkage. These cracks lead to create less durable concrete because it will behave as conduits for salt and water or aggressive fluids (Neville, 1996; Güneyisi et al., 2010).

Kayali et al. (1999) investigated the performance of LWC containing polypropylene or steel fibers and a high proportion of fly ash. For all the LWC investigated, the binder contents, total water contents, and the total sand contents are considerably higher than those in the normal weight concrete. In this study, the results of compressive strength and indirect tensile strength for different LWC were presented. Also, the modulus of elasticity values for the samples tested through the study were reported to relate to the drying shrinkage values. In spite of the LWC produced in their study had compressive strength classified as a high strength, the concrete itself had high drying shrinkage values, and so, they are certainly not high performance concrete. The results of drying shrinkage are presented in Figure 2.9.

Neville (1997) compared the creep and shrinkage strain results of LWC with those related to normal weight concrete. He concluded that the lower modulus of elasticity, the higher shrinkage strain and vice versa. According to this behavior, he stated that the shrinkage and modulus of elasticity are inversely proportional to each other. His study also showed a relation between the proportion of fine aggregate and the intensity of shrinkage. The higher quantities of fine aggregate in concrete led to significantly increase the voids and consequently shrinkage increases.

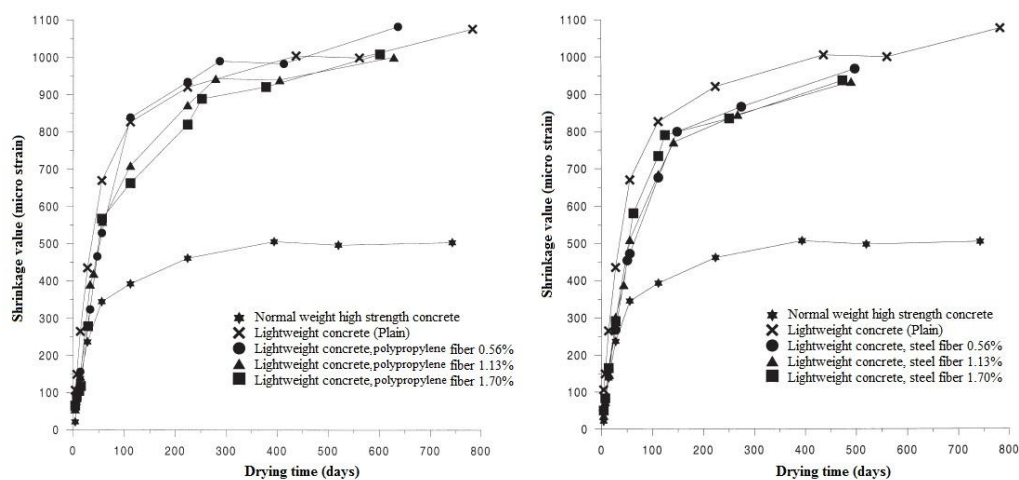


Figure 2.9 Drying shrinkage of fiber-reinforced lightweight aggregate concrete (Left: polypropylene fiber; Right: Steel fiber) (Kayali et al., 1999)

Al-Khaiyat and Haque (1998) examined the drying shrinkage of LWC as portion of an experimental research study. By using special curing conditions like severe hot and dry, hot-coastal and salt-laden exposure the long-term strength development and durability were investigated. The drying shrinkage of LWC observed on 50x50x285 mm prisms. It was reported that at the end of the first three months strain exceeded 600 micro-strains.

Kabay and Aköz (2012) said that the drying shrinkage of LWC made with pumice aggregate based on the influence of three different pre-wetting approaches. The results from the study revealed that LWC including presoaked aggregates had the maximum shrinkage value of 794 micro-strains at period of 6 months. The internal curing conducted for samples through water soaked and vacuum-soaked pumice aggregates reduced drying shrinkage

Güneyisi et al. (2014b) conducted a study to investigate effect of shrinkage reducing admixture and steel fiber incorporation on the performance properties of cold bonded fly ash lightweight aggregate concrete. To achieve the objective of study, they designed seven mixes with shrinkage reducing admixture or steel fibers. They found that drying shrinkage remarkably decreases with an increase in the dosage of shrinkage reducing admixture. They reported 42% reduction in shrinkage compared to concrete free of admixture (Plain concrete). On the other hand using of steel fiber reduced shrinkage for LWC up to 8% compared to plain.

2.6 Self-compacting concrete

The risks threatening concrete properties resulting from imperfect compaction during the vibration process and skilled labor deficiencies forced the innovation of a type of a smart concrete material. This smart concrete, known as self-compacting concrete (SCC) is capable of easy self-consolidation into place, fills voids in formwork and condenses reinforcement sections without any vibration. It also possesses enough viscosity to prevent objectionable segregation and bleeding. Additional advantages of SCC include: a reduction in required workmanship; earlier removal of formworks; ease in designing aesthetic structural members; and, high durability due to the increased packing of particles which results in a higher flowability. The most common definition of SCC as “a flowing concrete without bleeding and segregation bleeding, adequate of filling spaces and dense steel reinforcement or unreachable

members without interference or obstruction”. The use of SCC in the actual structure has progressively expanded over the world in the recent years and presents successful alternative to conventional concrete.

The design of High Performance Concrete is generally adopted through the use of low water to cement ratio and utilizing additives to provide required workability of concrete. At the same line, similar procedures are applied in the case of preparing SCC. In general, the main difference between conventional concrete and SCC is that the aggregate quantities are also adjusted to attain higher flowability while protecting the mix from segregating. Assaad et al. (2003) found that as the optimal packing density or well gradation of the combined aggregate system have been used, concrete viscosity decreases, consequently the superplasticizer dosage can be decreased to reach required concrete workability.

SCC must involve in its ingredients mineral and chemical admixtures so as to improve fresh and hardened properties. Fly ash, silica fume, and ground granulated blast furnace slag are usually used as a mineral admixture. Such admixtures enhance the SCC performance in terms of quality and reducing the amount of cement (saving cost). For example, the employment of fly ash and/or silica fume increases flowability and reduces permeability.

Water reducers classified as a chemical admixture are the solution that gives the proper consolidating for SCC. The most commonly used as water reducing agents in SCC are known as superplasticizers. The addition of superplasticizer to SCC cause dispersion of the cement particles, releasing the water molecules trapped in the cement conglomerations, and therefore, increases workability. In the present days, most of superplasticizers kinds based on polycarboxylate ethers. These types depend on electrosteric stabilization for cement particle spreading. The electrostatic stabilization consists of preventing the particle of cement from each other and providing steric stabilization to keep them apart.

The main difference between SCC and vibrated conventional concrete represent by the ability of the former to consolidate under its own weight without any vibration or compaction and transcend the existed obstacles such as narrow sections and dense reinforcement members. So, in order to identify concrete as a SCC, it must possess the following activities (Boukendakdji et al., 2012):

Sufficient filling ability: Complete filling of formwork under its own weight.

Passing ability: Flow through narrow channels or closely spaced steel reinforcement without blocking.

Dynamic and static stability: Dynamic stability refers to the segregation resistance during placement and static stability denotes to the resistance to segregation, bleeding, and surface settlement after casting.

There are different tests proposed to evaluate the fresh properties of SCC. In general, each test introduced to estimate one or two of the required fresh properties of SCC. The most widely SCC test procedures are tabulated in Table 2.2.

Table 2.2 Test for workability properties of SCC (ACI 237R, 2007; EFNARC, 2005)

Test	Property				Approaches	
	Filling ability	Passing ability	Stability	Relative viscosity	ACI	EFNARC
Slump	•				•	•
V-Funnel	•	•		•		•
U-Box	•	•				
T ₅₀₀	•		•	•	•	•
L-Box		•		•	•	•
Column Segregation			•		•	
Visual stability index			•		•	•
J-Ring		•			•	

Sonebi et al. (2007) reported that the reduction in the volume of coarse aggregate particles and the increase in the volume of paste reduce the effect of internal friction between particles. They also said that to increase paste deformability, superplasticizer quantity and water to cement ratio must be balanced within mix proportions. The reduction of internal friction and increase of paste deformability will enhance the filling ability of fresh concrete.

Khayat et al. (2004) focused on the passing ability of SCC in their study. They proposed three different factors to estimate the flowability through obstacles of

freshly mixed concrete, those are: the reinforcement density and its geometry, the stability of the fresh concrete, and the maximum aggregate size and volume fraction. Moreover, they recommend that the segregation be minimized using either viscosity modifier admixtures or low water to cement ratio. So as to evaluate passing ability of the fresh mix in their study; different tests have been done comprising the L-box and filling capacity box.

Bailey et al. (2005) advised procedure to give fresh concrete mixture higher resistance to segregation. To reach this target, the study proposed procedure to reduce bleeding resulted from free water and decreasing the segregation of solid particles by using viscosity modifying agents or using low water to cement ratio. Viscosity modifying admixtures behaves to reduce the viscosity of the paste and increase the stability of the paste.

2.7 Self-compacting lightweight concrete

The favorable properties of SCC as presented earlier in section 2.6 can be enhanced through the addition of LWA. Self-compacting lightweight concrete (SCLC) is able to promote the properties of concretes and eliminates vibration which may prevent the segregation of LWA (Güneyisi et al., 2016a). The applications of SCLC in building and construction are in increase day by a day. The use of SCLC comes to improve the quality of structure especially when it is required to lessen the dead load of buildings (Ranjbar and Mousavi, 2015). It is easy to consider that the SCLC is a type of high performance concrete derived from SCC. No doubts about the relation between the effect of fresh concrete workability and hardened properties of SCLC. A proper workability is important for fresh SCLC to encapsulate the steel reinforcement bars and completely filling the formwork. Moreover, segregation and bleeding threats should be avoided during the transportation and placement process. In addition, it is important that aggregate in SCLC uniformly distributed through the mixture (Choi et al., 2006; Wu et al., 2009).

SCLC can be produced by replacing the normal weight coarse aggregate with coarse lightweight aggregate. Although this is mostly used in SCLC production, sometimes all lightweight aggregate maybe replaced. Due to the higher porosity of LWA, extra attention is applied when designing the SCLC mix and when specifying the water of mix. In fact, some of LWAs take hours, days and weeks to absorb water (during pre-

soaking process) after first being wetted. Shape and surface texture of LWA influence the workability of SCC, rounder and smother aggregate particles increase consolidation properties of fresh concrete when comparing with angular and rougher particles. Approximately, 1/2 to 1/1.5 is the ration of LWA weigh and normal weight aggregate. In the replacement of LWA with normal aggregate the weight is reduced while the volume keeps constant (Lotfy, 2012; Lotfy et al., 2014).

2.7.1 Partial replacement of lightweight aggregate for production of SCLC

Abdelaziz (2007) studied the effect of partial replacement of coarse lightweight aggregate produced from expanded clay with normal weight aggregate on the hardened properties of SCLC. His study revealed an increase in 28 days compressive strength by about 15, 30, 50 and 90%, when 35, 50, 60 and 100% normal aggregate were used as a partial replacement of LWA. With the increase in the volume of normal aggregate the density increases systemically and reached to 2230 kg/m^3 at concrete free of LWA. The study also showed a significant impact on the porosity of concrete resulted from the replacement of LWA with normal weight aggregate. The porosity of concrete made with 100% LWA is approximately double that made with 100% normal aggregate.

Gesoğlu et al. (2014) presented study to investigate the properties of the SCC made with the cold bonded fly ash lightweight coarse and fine aggregates. They achieved the purpose of their study by an experimental program performed in two parts. In the first stage they used pelletization theory to manufacture cold-bonded fly ash. In the second part of the study, they partially replaced coarse and fine normal aggregate with cold bonded fly ash lightweight aggregate for SCC at six volume fractions 0%, 20%, 40%, 60%, 80%, and 100%. Accordingly, they produced 17 mixes at the same water to cement ratio of 0.32. The results yield from the study indicated that the increment of LWA volume fraction in concrete caused to decrease compressive strength. The specimens free of LWA yielded compressive strength of 76 MPa. Whereas the compressive strength of all lightweight aggregate concrete specimens was 43 MPa at 28 days. However, they considered this deterioration to the weakness of LWA compared to natural aggregate.

2.7.2 Fresh and hardened properties of SCLC

Yanai et al. (1999) utilized artificial perlite and coal ash aggregate to investigate the effect of the kind and volumes of LWA on workability, passing ability, and mix stability of SCLC. The study reported that by adjusting the unit quantity of LWA and water to binder ratio according to the specified aggregate, SCLC with perfect flowability, passing ability, and segregation resistance can be developed. Their study revealed that the use of high density aggregate increases flowability, filling ability, and compressive strengths.

Hwang and Hung, (2005) focused on the performance of SCLC mixtures made with different cement paste content for different water to binder ratios by incorporating coated reservoir fine sediment aggregate. They used ASTM type I cement and ASTM C618 Class F fly ash. The compressive strength after 91 days of curing was 42.5 MPa at cement content and water to cement ratio of 375 kg/m³ and 0.40, respectively. The value of compressive strength was increased to 50 MPa when 468 kg/m³ and 0.32 of cement content and water to cement ration used, respectively. The study stated that SCLC could achieve high strength.

Choi et al. (2006) provided distinct attention to the examination of the effect of replacing either the coarse or the fine volume fraction of normal weight aggregates with artificial lightweight ones on the fresh and mechanical properties of the SCLC. Slump flow and T₅₀₀ values of all mixtures were resulted to be within the normally accepted limits for SCC, whereas V-funnel and U-box values were unsatisfactory (with respect to the JSCE), especially for the mixtures in which the fine normal weight aggregate were partially or fully substituted by lightweight fine aggregate.

Lo et al. (2007) compared the workability and mechanical properties of SCLC with those of SCC. SCLC with binder content of 500-600 kg/m³ and 1650 kg/m³ concrete density was produced. The results showed to produce similar compressive strength, the bulk density reduced to 75% of SCC. The elastic modulus of SCC equal about 1.15 of SCLC.

Her-Yung (2009) estimated the effect of fly ash, slag and mixing water volume on the durability of SCLC. He designed SCLC at three water to binder ratios of 0.28, 0.32 and 0.40. It was concluded that with different water to binder ratios, SCLC can

attain 70% or more of its compressive strength at the first week. Like conventional concrete, lower mixing water resulted higher compressive strength for SCLC. The study found that the lower the water to binder ratio, the better the ultrasonic pulse velocity and electrical resistivity of SCLC specimens. SCLC with water to binder ratio of 0.28 resulted lower chloride penetration.

Topçu and Uygunoğlu (2010) studied the influence of different types of natural lightweight aggregate on the properties of SCLC. The SCLC mixtures were designed through the combinations of w/b ratio with the total powder content and superplasticizer dosages. They reported that the replacement of crushed limestone with pumice, tuff and diatomite decreased the unit weight of SCLC by 35%, 31%, and 38%, respectively. On the other hand they showed that the compressive and splitting tensile strengths significantly improved when the lower of water to binder ratio used. Moreover, the reducing intensity of flexural strength values was lighter than that of compressive and splitting tensile strengths.

Bogas et al. (2012) concern the properties of fresh SCLC and examines their performance in terms of mechanical strength, adhesion and durability. The SCLC made with two types of LWA commercially existing in the Iberian Peninsula. The compressive strength values of SCLC were ranged between 37.4 to 60.8 MPa. The slump test of fresh SCLC gave a diameter and T_{500} varied between 66-75 and 10-15 s, respectively. They also discussed the effect of coarse aggregate volume, volume of fine aggregate to volume of mortar, and powder to fine aggregate, and water to powder ingredients. The study concluded that the powder and fine aggregate ratio significantly influenced the deformation capacity of SCLC. The filling and passing ability depends on the volume of coarse aggregate. The elastic modulus of SCLC was found to be somewhat higher than SCC including normal aggregate.

Ranjbar and Mousavi (2015) investigated the effect of water to binder ratio and two curing regimes (water curing and salt/wetting) for SCLC mixtures on the strength and durability of SCLC. Expanded polystyrene lightweight aggregate contents were 10, 15, 22.5, and 30% by the total aggregate volume. Silica fume was incorporated by 10% of total weight of binder to enhance bonding between the paste of cement and expanded polystyrene aggregate as well as increasing segregation resistance. The experimental program of study involved several test; water absorption, electrical

resistivity, air permeability, chloride penetration profiles, compressive strength, and tensile strength. In spite of the use of expanded polystyrene lightweight aggregate decreases the strength of SCLC, the specimens of SCLC cured in salt/wetting system exhibited higher compressive strength than those cured in water. The volume ratio of expanded polystyrene aggregate up to 22.5% resulted absorption below 3% at 30 min indicating good quality of concrete. The study also revealed a negative effect on the air permeability and electrical resistivity due to the incorporation of such aggregate. From the result of the study, with 15% expanded polystyrene aggregate corrosion risk did not exceed the threshold of activity in SCLC at 2.5 cm covering depth.

2.8 Definition and type of the composite column

Nowadays columns made of steel (structural or reinforcing) and concrete are defined as composite columns. The use of these two materials had a number of motivations, such as resistance to fire. Generally, columns can be classified depending on the type of cross section of steel and concrete. Typically, three different kinds of composite columns widely used in construction industry - fully encased composite columns, partially encased composite columns, and concrete filled steel tube columns (Figure 2.10). Fully encased composite columns normally attend the practical requirements for high classes of fire protection. As can be seen in Figure 2.10a, fully encased composite columns require full form-work to support the fresh concrete.

Figure 2.10b and c shows the cross section of partially encased composite column. Partially encased composite column can be defined as a composite column that consists of an I or H shaped steel section with concrete between the flanges. This type of composite column requires formwork only on two sides of the column to support concrete during casting process. In the third type, the steel tube filled with fresh concrete in such a manner that the steel section works as a formwork during casing process (Figure 2.10d and e). In addition to above, special aspects of columns with inner core profiles such as concrete filled profile with additional inner profile and composite columns with hollow section and additional inner core profile can be categorized as a composite column as presented in Figures 2.10f and 2.10g.

In the concrete filled profiles (Figure 2.10d-g), the steel section serves as formwork during casting. Concrete filled sections, therefore, offer the advantage to erect the

steel frame of a building and subsequently fill the hollow column by pumping in the concrete. By so doing the erection time can be lessened.

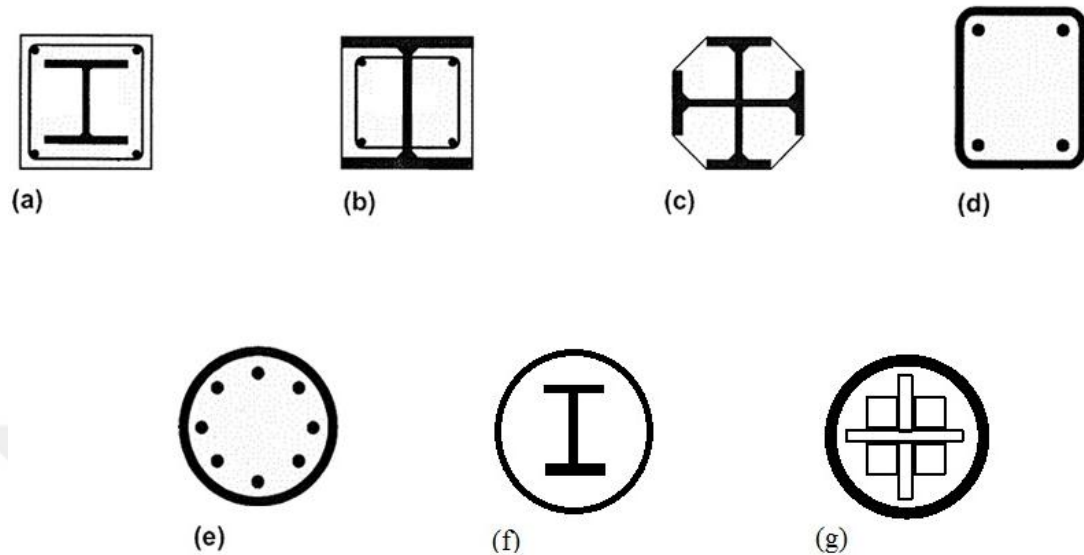


Figure 2.10 Typical cross sections of composite columns (Eurocode 4, 2004)

2.8.1 Behavior of structural CFST columns

Owing to their advantages such as high strength, proper stiffness, attractive appearance, extra capacity of energy absorption, concrete filled steel tube (CFST) columns are being used widely in construction engineering. Furthermore, the steel tube provides confinement effect that is result in an improvement in the strength of the concrete core. CFST column can restrict concrete from spalling under fire exposure. However, to resist the applied load, steel and concrete of such column need to work compositely. Transferring the load from steel to concrete in composite column significantly affects by the natural bond between the tube and the core of concrete. Furthermore, unlike the unconfined concrete, the concrete core due to the confinement effect within the composite column shows different behavior (Ding and Wang, 2008).

Several researchers studied the mechanical behavior of CFST short columns under axial compression with different steel grades (Johansson and Gylltoft, 2002; Teng et al., 2013). One of the earlier studies in the same regard was conducted by Gardner and Jacobson (1967). Their study involves several parameters; steel yield strength varied from 363 to 650 MPa, strength of concrete between 25.0 and 43 MPa, and

different ratio of column diameter to steel thickness. They found that the dilation of concrete in the initial loading stage lower than that of steel, thus steel does not provide any confinement stress to the concrete. The micro cracks begins to appear as the axial strain increases, consequently, the expansion of concrete exceeds that of steel tube, in this stage steel tube has to stretch more to monitor the rapid expansion of concrete and therefore the confining effect is motivated. Thereafter, the strength and ductility of concrete significantly improved.

Schneider (1998) investigated experimentally and analytically the behavior of concentrically loaded CFST columns having different sections such that five square, three circular, and six rectangular sections were experimented so as to study the effect of section type and thickness of steel on the behavior of CFST short columns. Their study revealed that the CFST column with rectangular or square section had the lower strength, stiffness and ductility comparing to the circular column. The reason for the better performance of circular CFST column is related to the confinement stress generated by the circular cross section is higher and more uniform than other sections.

The strength of concrete is one of the most significant factors that influence the behavior of CFST stub columns. Giakoumelis and Lam (2004) and Gupta et al. (2007) studied the behavior of CFST short column by utilizing normal strength concrete and high strength concrete. They found that the ductility and improvement strength ratio of CFST column specimens decreased as the concrete strength increased, meaning that higher strength concrete required larger confining stress by using higher steel yield stress or thicker steel tube to provide the same level of confinement. On the other hand, Giakoumelis and Lam (2004) reported that the Eurocode 4 gives values lower than that from the experiments as presented in Figure 2.11. Based on the figure, they suggested that Eurocode 4 can dependably predict the axial capacity of CFST columns utilizing strengths of concrete in the range 30–85 MPa.

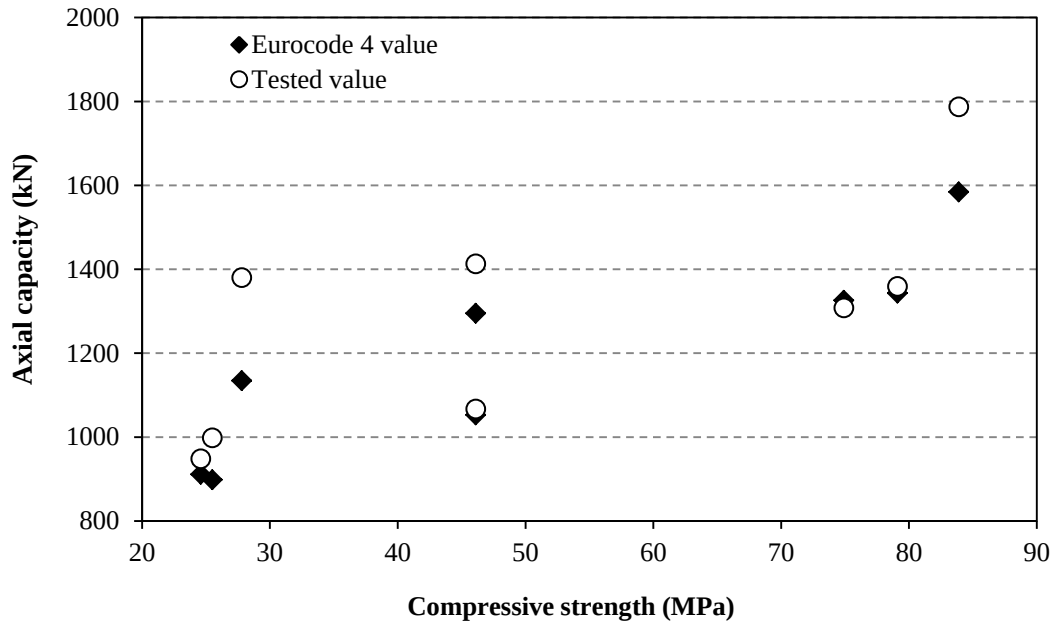


Figure 2.11 Comparison of Eurocode 4 with experimental results (Giakoumelis and Lam, 2004)

Another important parameter affecting the behavior of CFST columns is the property of steel tube. Schneider (1998) and Han et al. (2005) found for concrete-filled carbon steel tubular columns, the highest confining stress accompanying the largest yield stress. Uy et al. (2011) selected stainless steel and carbon steel to investigate the behavior of CFST column. The results of their study revealed that the stainless steel has better performance than carbon steel in terms of corrosion, maintenance, and durability. Sakino et al. (2004) conducted a comprehensive experimental investigation on the behavior of CFST columns having lower to high steel yield strength (262 MPa-835 MPa). Contrary to the results of Uy et al. (2011) study, they found that as the steel yield stress improved, the confining effect offered to the concrete core at ultimate condition increased. For this they believed that the CFST columns preparing with high strength steel can improve the behavior of composite columns remarkably.

Diameter-to-thickness ratio of tube is a key factor for estimating the behavior of CFST columns. It may be the most significant one influencing the provision of confining effect and meanwhile the ductility and strength of CFST columns. In this regard, the steel tube would buckle first before yielding when diameter to thickness ratio exceeded a specified limit. The relation between the diameter ratio to thickness

and confinement stress is considered as an inverse relationship. This is why ductility and strength of CFST column reduces when diameter to thickness ratio increases. Furthermore, the strain corresponding to peak load also affects by the diameter-to-thickness ratio. With smaller diameter-to-thickness ratio a larger peak strain would be attained (Sakino et al., 2004).

Besides the factors mentioned above that affecting the behavior of CFST columns, the length to diameter ratio of such column also influences the behavior of composite column significantly. That is to say, column with length to diameter ratio of ≤ 4 can be considered as short column since the confinement stress created from the steel can be completely derived without global buckling. Thus, in order to avoid global buckling to take place before maximum column strength, it is important to specify a practical factor when computing the strength of slender CFST column (Han et al., 2005; Uy et al., 2011).

Xue and Gong (2016) proposed a new type of composite columns to overcome the conventional disadvantages of concrete filled fiber reinforced polymer tubes and it consists of a steel reinforced concrete inside, a fiber reinforced polymer tube outside, as presented in Figure 2.12. The results indicated steel reinforced concrete-filled fiber reinforced polymer tubes tubular columns have higher specimen stiffness, strength and deformability than concrete filled fiber reinforced polymer tubes tubular columns. They related this behavior to the presentence of the section steel. Moreover, they concluded no influence of the section steel on the confining effects of the tube. Regarded the thickness of tube, they revealed that this parameter can contribute more to the strength of column and at the same time it has slight effect on the capacity of column deformation.



Figure 2.12 Concrete filled fiber reinforced polymer tubes with placed I steel section
(Xue and Gong, 2016)

Roeder et al. (1999) stated that there are strong demands in sections of geometric discontinuity like connections and foundation supports, particularly in braced frames. In these sections, external forces are transferred primarily to the concrete core or the steel tube only, and the load transfer requests to be ensured by adequate bond strength between the concrete core and the steel tube. Tao et al. (2016) studied the effect of cross-section type and size, steel type, concrete type, concrete age, interface type between the steel tube and concrete on the bond behavior between the steel tube and the concrete in CFST columns. The results indicated that when the compressive strengths of the normal concrete and recycle aggregate concrete are similar, the bond strength between smooth bars and normal concrete is lower than that developed by recycle aggregate concrete regardless the type of column cross section (circular or square), Figure 2.13. They also reported that the shorter the concrete age, the higher the bond strength. On the other hand, within their results the bond strength also affected by the shape of cross section of CFST column such that the circular column is apt to develop higher bond strength than its square for the cross section dimensions.

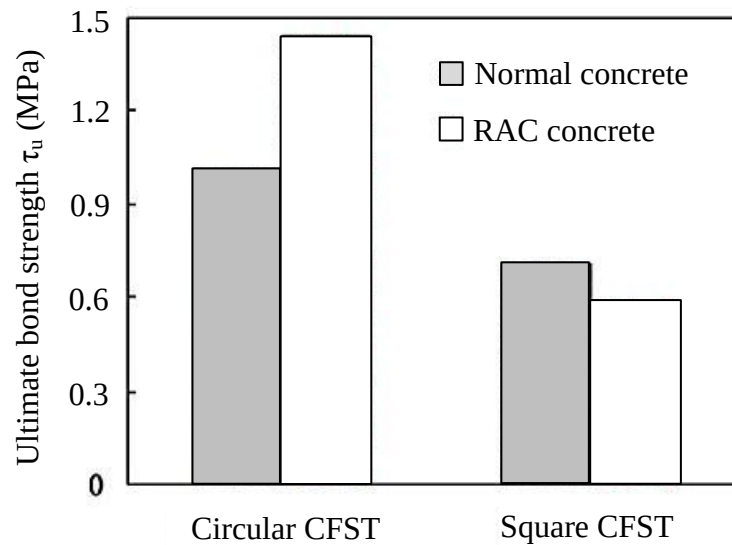


Figure 2.13 Influence of concrete type on the bond strength (Tao et al., 2016)

Mouli and Khelafi (2007) conducted an investigation to study the bond strength and axial capacity of short normal and lightweight concrete filled steel columns. Results showed that in the composite action the lightweight aggregate concrete provided greater bond strength than the normal concrete. They reasoned such behavior to the

different strain of shrinkage presented by the normal and the lightweight concretes at the same age. Also they reported that the type of concrete - normal and lightweight - did not influence the behavior of load-slip as can be seen in Figure 2.14 that all the tested specimens showed similar load-slip curves.

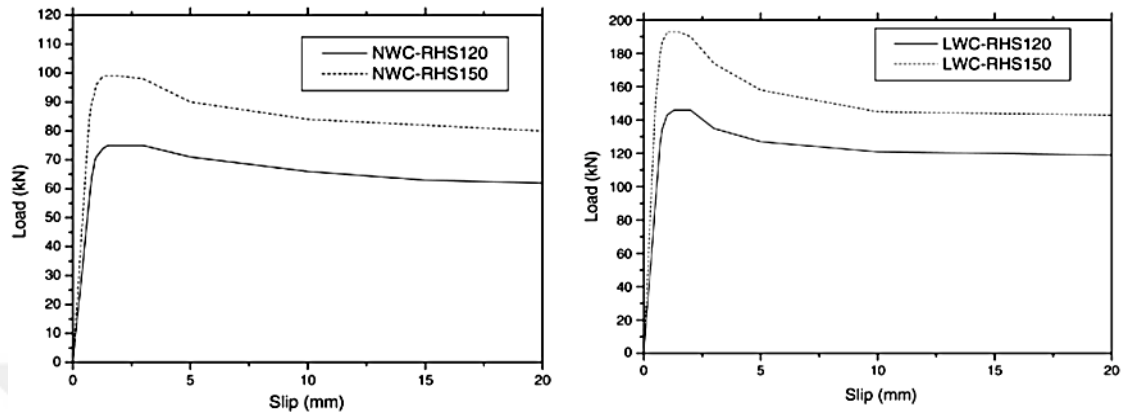


Figure 2.14 Bond slip specimen relationship, left; rectangular hollow steel sections (RHS) filled with normal weight concrete, right; RHS filled with lightweight concrete (Mouli and Khelafi, 2007)

It can be seen from the aforementioned literature, experimental studies have been accomplished in the recent decades in order to further understand the behavior of CFST columns, particularly the type of short columns subjected to uniaxial compression. At the same line, this thesis aimed to investigate analytically the behavior of CFST columns for special type of smart concrete by considering several parameters; w/b ratio, nano technology, artificial lightweight aggregate, age of curing concrete, diameter to thickness ratio, and yield strength of steel. It is hoped that the work and its findings would give engineers and practitioner better understanding of the use of SCLC made with fly ash lightweight aggregate in composite structures.

2.8.2 Comparison of reinforced concrete columns and composite concrete columns

One of the most significant decisions that must be recognized in the construction industry is the selection of materials used in buildings. In fact, the decision in many conditions depends on structure type, skill and experience of the designer, and financial issues. Concrete and steel are the materials that are utilized commonly in buildings and benefits of both are now well identified. The advantages of concretes

are its high stiffness, equitable fire resistance, and high compressive strength. On the other side as its well-known steel has high ductility and high tensile strength. Composite members combine steel and concrete, resulting in a member that has the beneficial qualities of both materials (Tao and Han, 2006).

A reinforced concrete column is a structural member consists of a combination of concrete and an embedded steel frame (reinforcement bars). The ability of plain concrete column to withstand lengthwise shortening and laterally expansion is quantified by using of longitudinal bars and lateral restraint - ties or helical spirals - for these longitudinal bars. In this regard, the reinforced concrete columns are classified to tied and spiral column, (Figure 2.15). Spirals can be considered more effective to increase column strength than ties. The use of closely spaced spirals significantly increases the ability of holding the longitudinal bars in place, and they also confine the concrete inside and enhance its resistance to axial compression (McCormac and Brown, 2014).

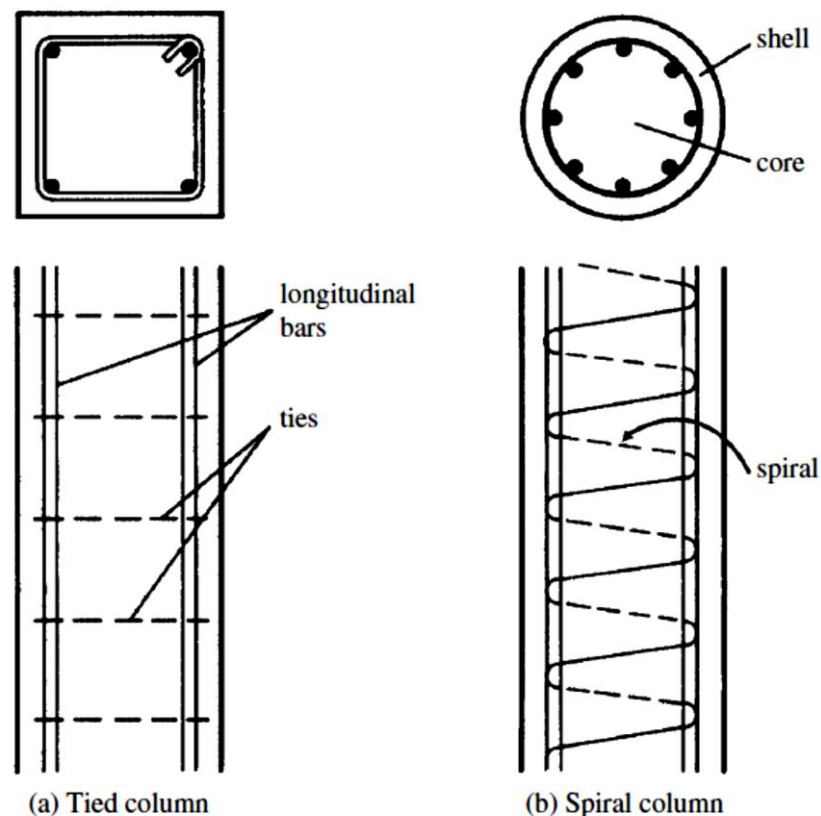


Figure 2.15 Types of conventional reinforced concrete columns (McCormac and Brown, 2014)

Although no one can deny the favorable performance of conventional reinforced concrete columns, but recently with a sharp increase in labor costs, the on-site temporary works involving formworks of concrete require high costs in construction market. Particularly for the projects like building with high story heights, the time of construction and costs for the temporary works have considerably increased, and thus the competitiveness of the reinforced concrete construction method has been adversely affected. Obviously, construction process of columns in typical reinforced concrete frame constructions is one of the challenges, therefore, absolutely advantageous to minimize the on-site temporary works for reduction of the construction time (Oh et al., 2016).

The concrete filled steel tube column as discussed earlier is a composite member made by filling steel tube with concrete. Concrete inside the steel tube enhances the performance of the steel tube column, and the stainless steel tube in turn offers lateral confinement or lateral reinforcement to the concrete. Unlike conventional reinforced concrete column and thin walled steel tube column, the CFST column has higher load-carrying capacity, toughness, and ductility. For a specified load, the required cross sectional area for CFST column smaller and the ratio of strength to weight higher than that of a conventional reinforced concrete column. However, the external material of CFST column is steel which can be represented the main disadvantage of such column because causing to a lower fire resistance compared with conventional reinforced concrete column (Yin et al., 2006; Jamaluddin et al., 2013).

In general there are several advantages of composite columns; some of them can be summarized as follows:

- Using of concrete encased column increases corrosion and fire resistance.
- Relatively small cross section of column can be designed.
- Enhances buckling resistance and stiffness with little effect of slenderness of column.
- Economically advantageous over either structural steel or pure reinforced concrete.
- Eliminating formwork for some types of composite columns which lead to savings of manpower and constructional cost and time.

- Drying shrinkage and creep of concrete are much smaller than ordinary reinforced concrete.
- The force resulting from earthquake is smaller due to the reduction in the weight of some types of composite column.
- For some types of composite column such as CFST columns the use of high strength concrete will not lead to brittle failure.

2.8.3 Use of the composite members in construction practice

A composite column defined as a member subjected mainly to compression or to compression and bending (EN 1994-1-2). The combination of steel and concrete in the same column are very important application of composite construction, and they find widespread use in different type of buildings. For example they commonly used in multi-storey commercial buildings and factories, as well as in bridges. The historical background of composite column indicated that the significant research and applications of such columns have started from 1950 till today. However, composite columns of steel & concrete were infrequently applied in buildings from the end of World War II till the early 1970s, research had begun a long time afore, at the beginning of the 20th century. Combining of these materials had revealed interest; concrete columns were coalesced with structural steel as reinforcement, while steel columns were typically encased in concrete to avoid them from fire.

Nowadays, the composite structural columns are widely applied in the construction of multi-storey buildings, and other types of structures all around the world. In fact, there are several other applications of composite construction including multi-storey car parks, industrial and residential buildings, metro station buildings, apartments, etc. In China CFST columns have been extensively utilized in arch bridges as they reduce cost of construction of such projects, Figure 2.16. The weight of a steel tube is relatively low; steel tube erection can often be achieved with comparatively simple construction technology (Wang, 2014).

Commerzbank Tower is a 56-story, 259 m skyscraper in the Innenstadt district of Frankfurt, Germany. It is the highest building in Frankfurt and the highest building in Germany. It had been the highest building in Europe from its completion in 1997 till

2003 (Wikipedia, 2017). Figure 2.17 shows the type of composite columns used in Commerzbank Tower.



Figure 2.16 Using of CFST columns in some of arch bridges in China (Wang, 2014)

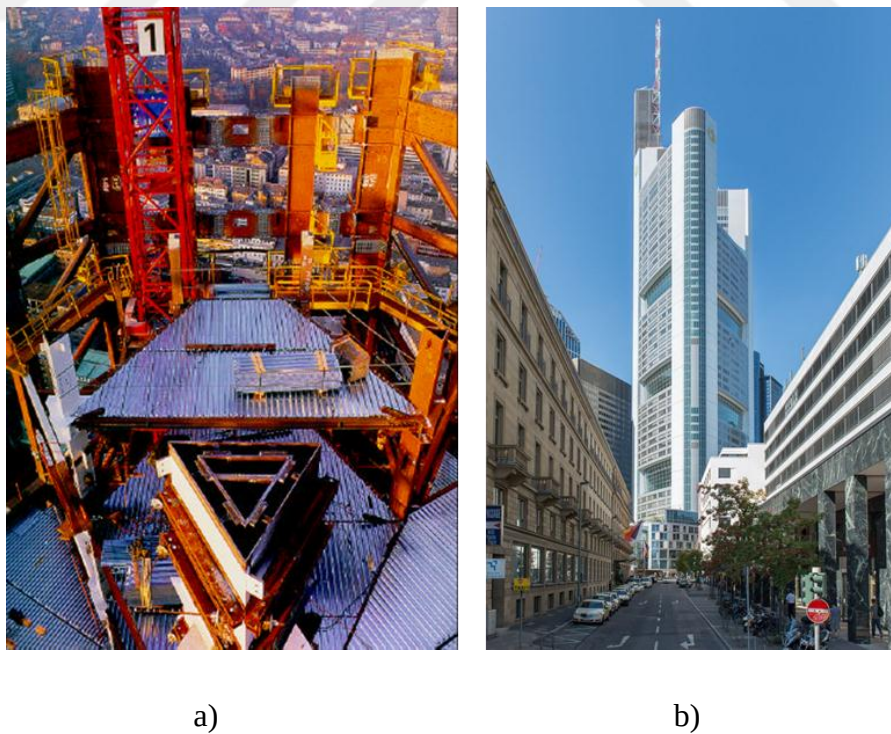


Figure 2.17 Using of composite columns in Commerzbank Tower: a) composite columns with hollow section and additional inner core profile, b) Commerzbank Tower after construction (Eurocode 4, 2004; Wikipedia, 2017)

As presented earlier, CFST columns are widely used in prestigious high rise building. An example is the SEG Plaza in Shenzhen, which is the 21st tallest building in China and the 72nd tallest in the world. The roof height of the building is 292 m. There are a total of 71 stories above ground (72 when counting the helipad) and 4 stories underground (Wikipedia, 2016). Figure 2.18 shows SEG Plaza during and after construction.

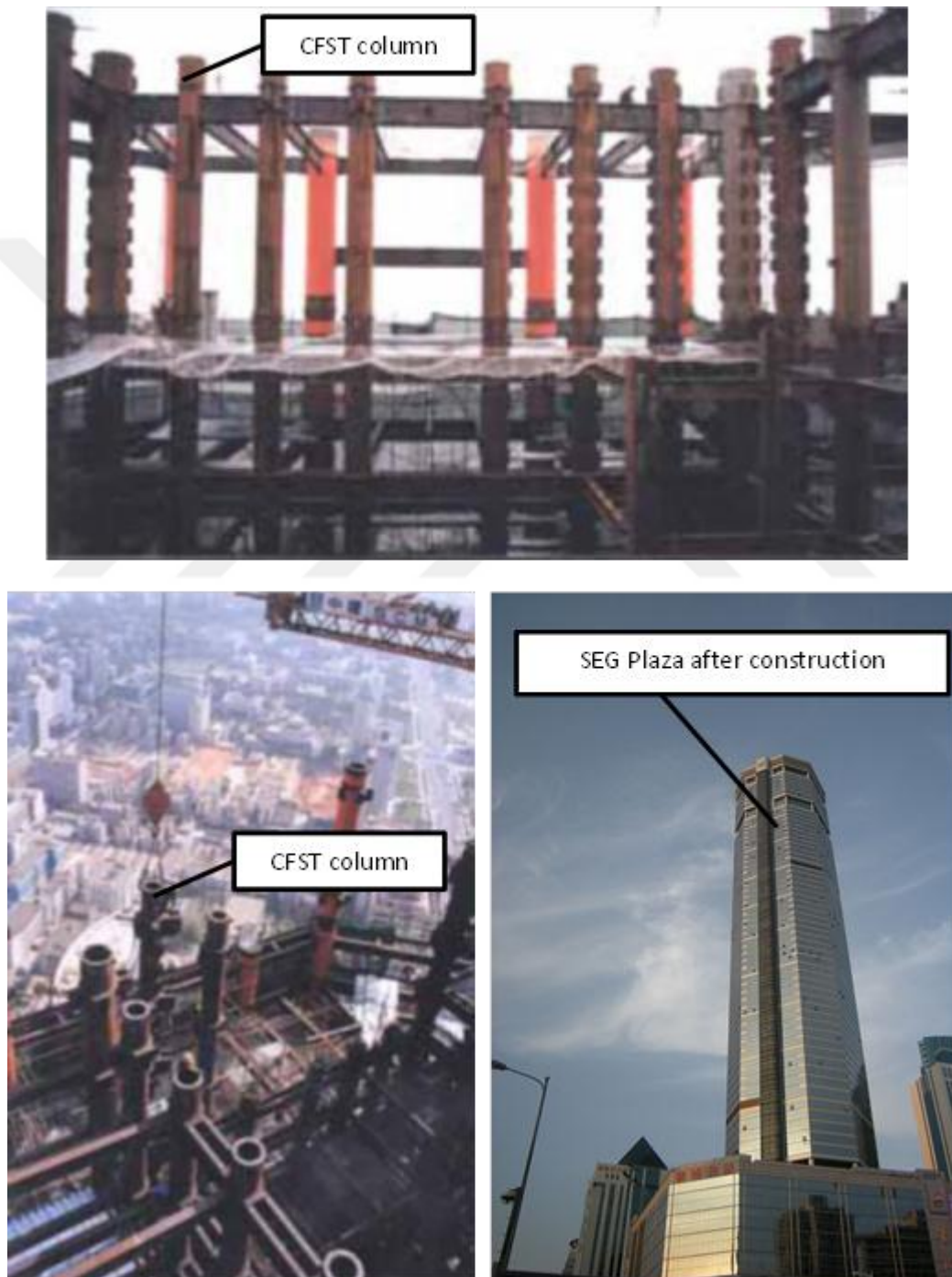


Figure 2.18 SEG Plaza constructed with CFST column (Wang, 2014)

CHAPTER 3

METHODOLOGY

In this study, a comprehensive experimental was performed to study the material properties of self-compacting lightweight concrete (SCLC). For this, synthetic fly ash lightweight aggregate was manufactured in laboratory and used in the production of 18 SCLC mixtures. Fresh and hardened properties of SCLCs were evaluated and compared with respect to the water to binder ratio (w/b), types of synthetic lightweight aggregate, and presence of nano-silica (nS). After that, a detailed numerical analysis was conducted to understand the structural response of the steel tubular columns filled with SCLCs under the action of uniaxial compression. For this, the capacity of such composite columns were evaluated by using three international design codes, namely, ACI, Eurocode 4, and DL/T. The main experimental and numerical parameters of the study were w/b ratio, type of LWA, nS content, strength level of core concrete, diameter to thickness (D/t) ratio, yield strength of the steel tube used. The details of the study are given below.

3.1 Experimental study

3.1.1 Materials

The SCLC mixtures produced in this investigation were prepared with CEM-I 42.5 Portland cement, fly ash, and silicon dioxide nanoparticles. The fly ash was provided by the Çatalağzı Thermal Power Plant in Turkey. In all mixes 25% of cement was replaced with FA by the weight. The hydrophilic fumed silica with a specific surface area of 150 m²/g was imported from Evonik industries AG Company, the commercial name of this product is AEROSIL 150 and manufactured in Belgium. In order to assess the effect of nano-particles in this study, ordinary Portland cement was replaced with 0%, 2.5% and 5.0% of nS. Figure 3.1 shows silicon dioxide nano particles sample. Table 3.1 presents the physical and chemical properties of the cement and mineral admixtures.

High range water reducing admixture (HRWRA), based on polycarboxylic-ether formulation with a specific gravity of 1.07, was used in all mixes to attain the required slump flow for SCLC. The properties of High range water reducing admixture are tabulated in Table 3.2.



Figure 3.1 Silicon dioxide nano particles sample. a) Nano silica packages, b) Nano silica powder

Table 3.1 Chemical composition and physical properties of cement, fly ash, and nS

Analysis Report	Portland Cement	Fly Ash	Nano Silica
CaO (%)	62.12	2.24	-
SiO ₂ (%)	19.69	57.2	99.8
Al ₂ O ₃ (%)	5.16	24.4	-
Fe ₂ O ₃ (%)	2.88	7.1	-
MgO (%)	1.17	2.4	-
SO ₃ (%)	2.63	0.29	-
K ₂ O (%)	0.88	3.37	-
Na ₂ O (%)	0.17	0.38	-
Loss of ignition (%)	2.99	1.52	≤ 1.0
Specific gravity	3.15	2.04	2.2
Blaine fineness (m ² /kg)	394	379	-
Surface–volume ratio (m ² /g)	-	-	150±15
Average primary particle size (nm)	-	-	14

Table 3.2 Properties of HRWRA

Property	HRWRA
Name	Glenium 51
State	liquid
Specific gravity	1.07
Appearance	Dark brown
Chemical description	Modified polycarboxylic polymer

Figure 3.2 shows the pelletization disc which was utilized to produce cold bonded fly ash lightweight aggregate. The device consists of speed controller unit to control the revolution speed during agglomeration process, pressurized water injection nozzles to spray required water on the mixture, and a hydraulic system to arrange the angle of disc. The sticking of wet mixture on the disc surface is avoided by scrapping blades which produces an energy barrier being ensured more compacted pellet (Gesoğlu et al., 2012). On the other hand, the most advantage of scrapping blades is to segregate coarse from fine particles where the coarse size forms in the region near the center whilst fine size groups at outer paths (Baykal and Döven, 2000).

A combination dry mixture of 90% fly ash and 10% cement by weight was agglomerated through moistening at a rotation of 42 rpm and in a 45 degree angle tilted pan at room temperature. The aggregate particles were formed during the first 10 min as sprayed water acted as a coagulant in the pelletization process. An additional 10 min was important to gain fresh pellets, more uniformity, and further stiffness. As the pelletization process completed, the self-curing of fresh pellets directly started, storing the pellets in plastic sealed bags in a curing room at a temperature of 20°C and 70% relative humidity for 28 days. At the end of the curing period, the cold bonded lightweight aggregates were sieved into two fractions of 4-16 mm as lightweight coarse aggregates and 2-4 mm as lightweight fine aggregates to be utilized in the production of SCLCs. Figure 3.3 demonstrates the main steps of production of LWAs.

The coarse cold bonded FA aggregate was grouped into two parts; the first one was kept in its natural condition whilst the second was treated by sodium silicate solution

(water glass). The treatment method, i.e. the use of water glass treatment, aims to create shells with the ability to improve the quality of the aggregate particle by reducing absorption and increasing strength. The treatment procedure involved soaking aggregate particles into a soluble solution of sodium silicate (water glass) for about 60 min and afterwards allowing the excess solution to drain for approximately 5-10 min. The hardening stage was accomplished by using hot air to avoid sticking of aggregate particles followed by oven drying at about 110 °C to improve the strength of the water glass shells. Noteworthy, during the oven drying phase the temperature was increased gradually in order to prevent any possibility of cracking, and then remaining in the oven until the alteration in mass during which occurs at approximately 24 h is less than 0.1% (Kockal and Ozturan, 2011). Eventually, the treated aggregate was cooled and was ready for use in concrete casting.

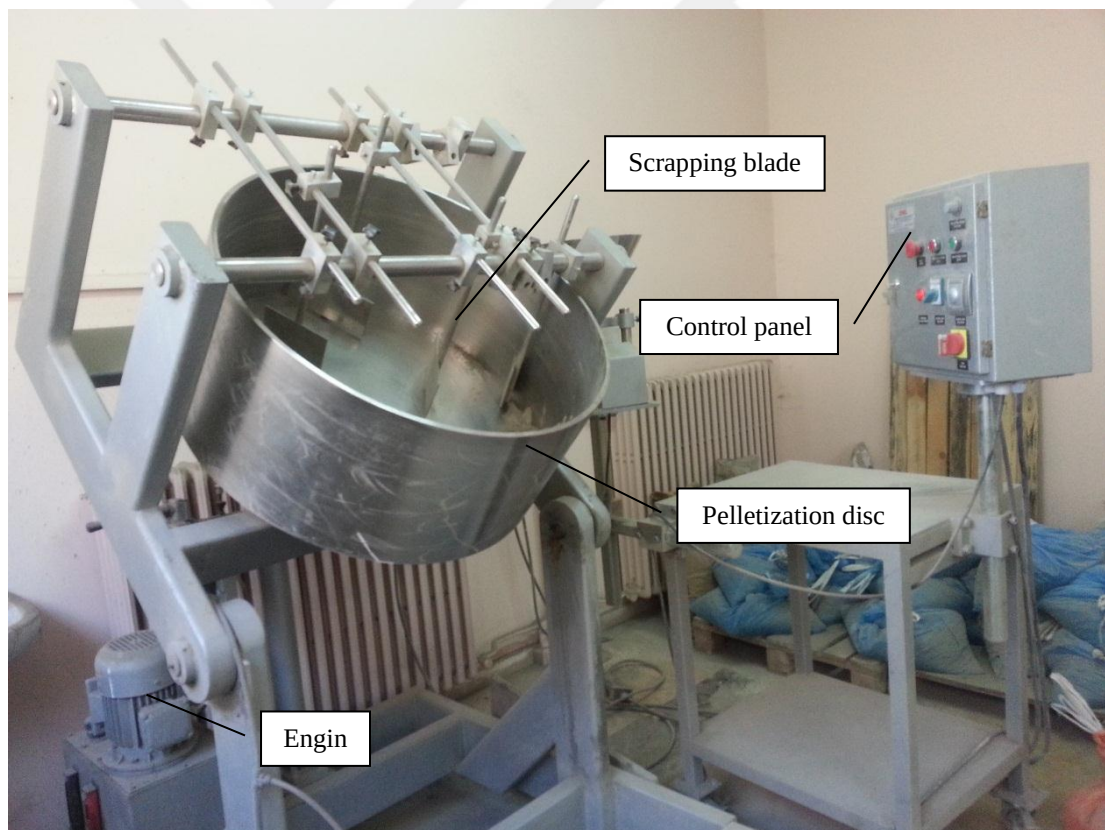


Figure 3.2 Photographic view of the pelletization disc



Figure 3.3 Production of artificial lightweight aggregate through cold bonded process

The specific gravity and water absorption of lightweight aggregates were calculated as per ASTM C127 (2015). The specific gravity of fine and untreated coarse lightweight aggregate for bulk, apparent, and saturated surface dry conditions were 1.45, 2.0, and 1.74 g/cm³ and 1.5, 2.0, and 1.75 g/cm³, respectively. On the other hand, the specific gravity of treated surface lightweight aggregate for bulk, apparent, and saturated surface dry conditions were 1.55, 2.15, and 1.76 g/cm³, respectively. The percentages of water absorption of lightweight aggregates were calculated from oven dry masses and SSD samples. It was detected there was a prominent reduction in water absorption owing to treatment technique, where the measured value for natural surface lightweight coarse aggregates was 21.6% whilst it was about 4.2% for treated. Furthermore, the treatment process changed the surface appearance; the treated aggregate particles appeared lustrous, as can be clearly seen from Figure 3.4.

In addition to the tests presented above, crushing strength of coarse aggregate particles was performed according to the standard BS 812, part 110 (1990). The influence of aggregate on the mechanical performance of aggregate can be indicated by crushing strength of an aggregate (Lo et al., 2016). In this test, crushing value of lightweight aggregate was determined on aggregate particles having the sizes of 4, 6, 8, 10, 12, and 14 mm. A known size of particle is loaded diametrically one by one between two parallel plates until failure occurred. The load that is needed to crush the pellet was used as a measure of the aggregate strength. A number of representative agglomerates were statistically tested and the mean of the results was

defined as crushing value of artificial lightweight aggregate. The crushing strength test configuration and the mean crushing values of the treated and untreated lightweight aggregate are shown in Figures 3.5 and 3.6, respectively. As seen in Figure 3.6, the crushing values of treated aggregates were remarkably higher than untreated lightweight aggregates. Crushed sand fractions of 0.25-2.0 mm and lightweight fine aggregates were used together as fine aggregates in all SCLC mixes. The specific gravity of crushed sand was 2.4 g/cm^3 .

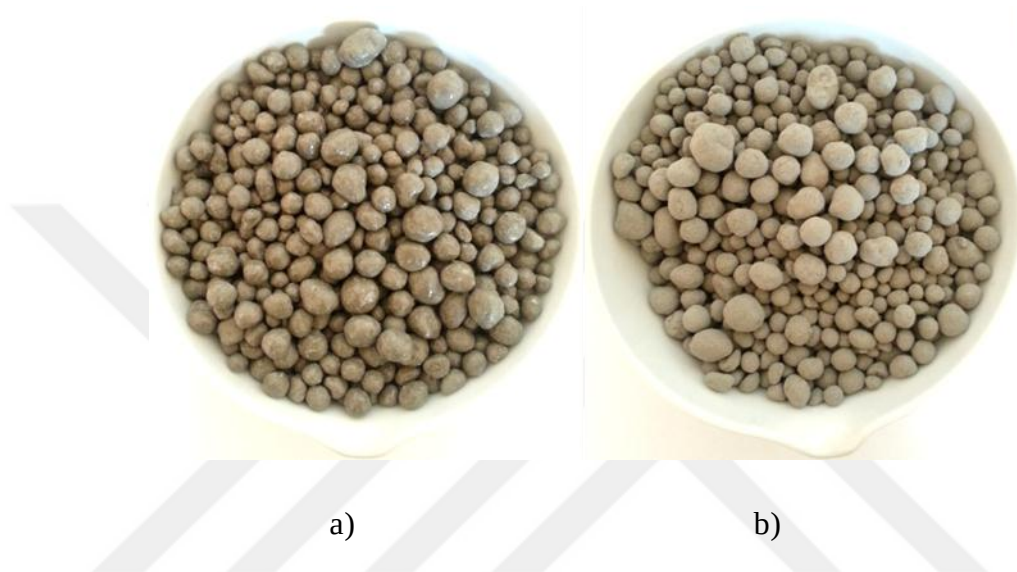


Figure 3.4 Samples of artificial lightweight aggregate a) Treated lightweight aggregate, b) Untreated lightweight aggregate

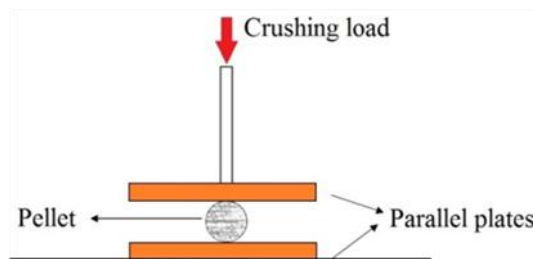
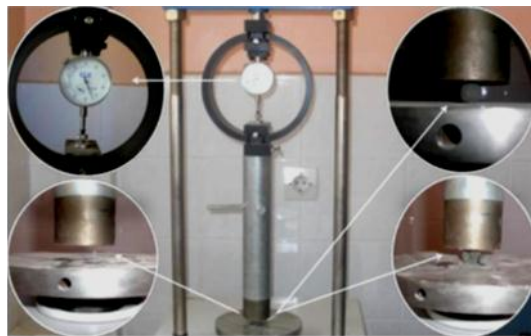


Figure 3.5 Aggregate crushing strength test configuration

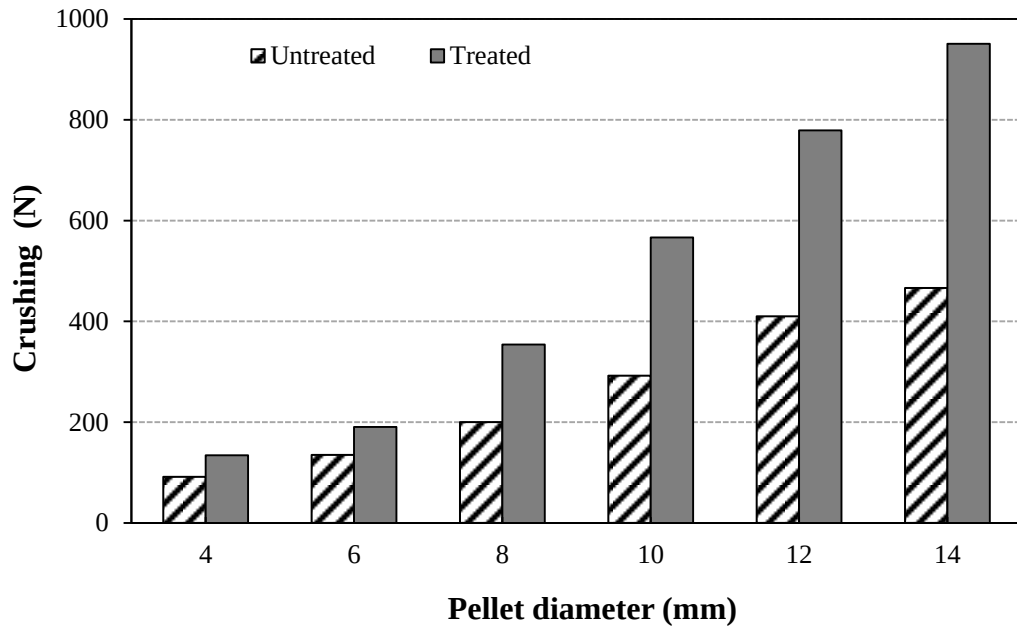


Figure 3.6 Crushing strength of treated and untreated artificial lightweight aggregate

3.1.2 Mixture proportioning and casting

A total of 18 SCLC mixtures were designed at three different w/b ratios of 0.25, 0.37 and 0.50, with total binder contents of 600 kg/m^3 , 550 kg/m^3 and 450 kg/m^3 , respectively. The total volume of aggregate in all SCLC mixes was covered by 50% cold-bonded lightweight coarse aggregates, 25% cold-bonded lightweight fine aggregates, and 25% crushed sand. Figure 3.7 shows the gradation curve of coarse and fine aggregate mixture. The mixes at each w/b ratio were varied upon binder contents and coarse aggregate type. The cementitious blends incorporated in SCLC mixtures were binary (cement+FA); and ternary (cement+FA+nS) in which a proportion of cement was substituted with mineral admixtures. The binary blend was obtained by joining 25% fly ash and 75% cement by weight, while the replacements of nS in the ternary blends were 2.5% and 5.0% by weight of cement.

The actual mixture proportions are tabulated in Table 3.3, in which the mixes were designated according to type of cold-bonded lightweight coarse aggregate, followed by the replacement level of incorporated nS. For example, for a given w/b, the mix “LWA-nS0” indicates concrete made with untreated lightweight coarse aggregates and 0% nS, whereas “LWA-WG-nS5.0” represents the concretes produced with treated lightweight coarse aggregates incorporating 5.0% nS.

The high absorption capacity of LWAs can cause the early loss of slump. Thus, to eliminate such behavior, a special procedure was adopted. Lightweight fly ash aggregates were steeped in water up to saturation. Afterwards, the aggregates were manually dried to SSD condition with a dry towel. The SCLC casting procedure began by mixing aggregates for 1 min. Then, the binder was added and was mixed for an additional 1 min in order to attain a better possibility of mixture homogeneity. It is worth mentioning that the 2.5% of nS was added to binders directly, whilst the 5.0% of nS was first mixed with cement powder by shaking in a dry closed container in order to promote uniform distribution of nano particles throughout the mix and to prevent any agglomeration which may occur during the mixing process. HRWRA was mixed with water and dispensed into the concrete. The concrete was mixed for 3 min and then allowed to rest for 2 min. Finally, the concrete was mixed for an additional 2 min to complete the mixing sequence. The slump flow diameter of SCLC was designed to be in the range of 700 ± 50 mm to satisfy the EFNARC (2005) (European Federation for Specialist Construction Chemicals and Concrete Systems). For this, trial batches were prepared for each mixture by modifying the HRWRA used until the desired slump flow was achieved. The schematic illustration of the logical sequence of mixing production is presented in Figure 3.8.

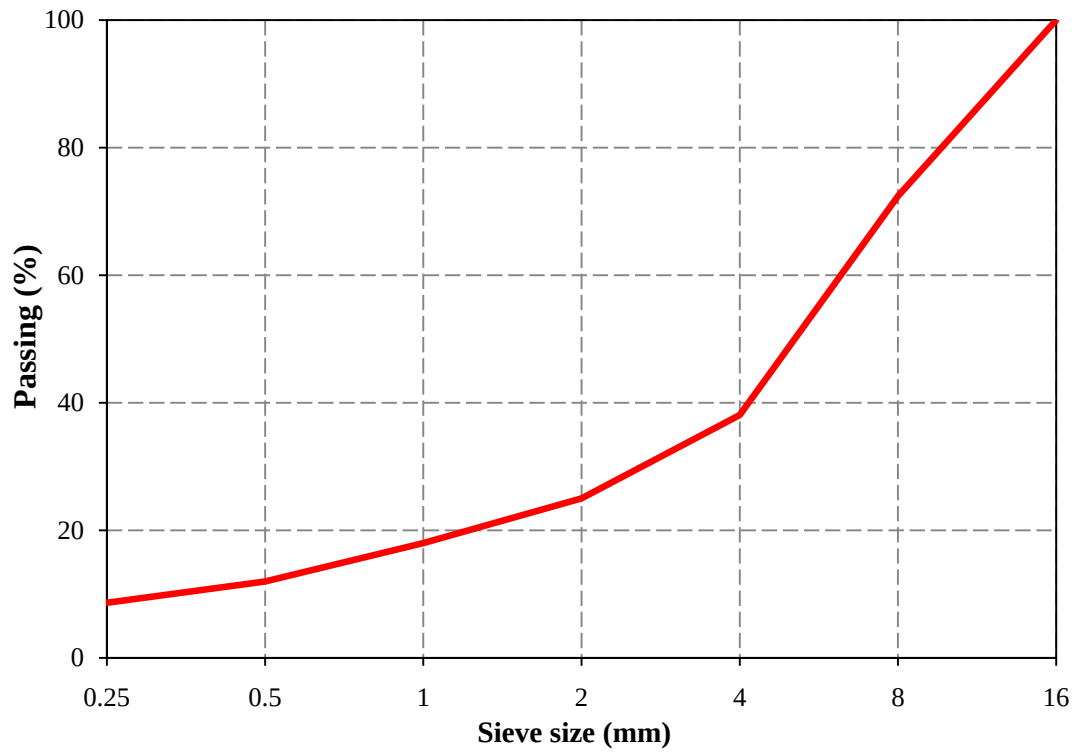


Figure 3.7 Aggregate mix sieve analysis



Figure 3.8 Logical sequence of SCLC production

Table 3.3 Details of mix proportions in kg/m³

Mix type	Code	w/b	Cement	Fly ash	Nano silica	Water	LWA (4-16) mm	LWA (2-4) mm	Natural (0-2) mm	Fresh density
Mix 0.25	LWA-nS0	0.25	450	150	0	150	551.7	273.9	375.9	1959.4
	LWA-nS2.5	0.25	438	150	12	150	550	273.1	374.8	1959.2
	LWA-nS5.0	0.25	420	150	30	150	548.1	272.2	373.5	1963.1
	LWA-WG-nS0	0.25	450	150	0	150	554.8	273.9	375.9	1962.5
	LWA-WG-nS2.5	0.25	438	150	12	150	553.1	273.1	374.8	1961.6
	LWA-WG-nS5.0	0.25	420	150	30	150	551.2	272.2	373.5	1961.7
Mix 0.37	LWA-nS0	0.37	412.5	137.5	0	203.5	520.1	258.3	354.4	1889.7
	LWA-nS2.5	0.37	401.5	137.5	11	203.5	518.6	257.5	353.4	1889.2
	LWA-nS5.0	0.37	385	137.5	27.5	203.5	516.8	256.6	352.2	1892.0
	LWA-WG-nS0	0.37	412.5	137.5	0	203.5	523.1	258.3	354.4	1892.7
	LWA-WG-nS2.5	0.37	401.5	137.5	11	203.5	521.6	257.5	353.4	1892.2
	LWA-WG-nS5.0	0.37	385	137.5	27.5	203.5	519.8	256.6	352.2	1892.1
Mix 0.50	LWA-nS0	0.5	337.5	112.5	0	225	531.9	264.1	362.4	1835.4
	LWA-nS2.5	0.5	328.5	112.5	9	225	530.6	263.5	361.6	1834.7
	LWA-nS5.0	0.5	315	112.5	22.5	225	529.2	262.8	360.6	1836.4
	LWA-WG-nS0	0.5	337.5	112.5	0	225	534.9	264.1	362.4	1838.4
	LWA-WG-nS2.5	0.5	328.5	112.5	9	225	533.7	263.5	361.6	1837.7
	LWA-WG-nS5.0	0.5	315	112.5	22.5	225	532.2	262.8	360.6	1838.4

3.1.3 Evaluation of fresh properties

For traditional vibrated concrete the slump test are used to measure the vertical settlement of concrete, while for self-compacting concrete the slump test allows a sample of fresh concrete to flow out in all directions. Thereby, the test measures an unconfined horizontal spread of the sample which can be considered as a primary check to the specification in terms of flow. Furthermore, visual observations during the test can give important information on the segregation resistance and uniformity of each mixture. However, the visual stability index observation was carried out in this study to detect the SCLC stability. In this index, the concrete classified into four groups according to their stability (high, slight, moderate and weak). The visual observation start after removing slump cone and the segregation resistance of fresh concrete can be estimated visually by measuring a coarse aggregate lump or the thickness of cement paste spread away from the coarse aggregate (Madandoust and Mousavi, 2012). It was reported in EFNARC (2005) that the self-compacting concrete can be classified into three classes with respect to slump flow diameter. The fresh concrete speed had been of interest from the very first tries to evaluate slump-flow. The time taken from the moment the slump cone is being removal to circle of 500 mm in diameter is a flow-rate namely as $T_{500\text{ mm}}$ slump flow time.

The V-funnel flow time test value describes the filling ability (fluidity) as well as flow-rate of fresh concrete. The V-shaped funnel used in this research had a volume of approximately 10.5 l, the time recorded by a stopwatch between the moments of opening the gate and when it is possible to see vertically through the funnel into the container below for the first time is the V-funnel flow time. Figure 3.9 illustrates a general view of the V-funnel test procedure.

The primary application of L-box test is to measure the passing ability of fresh concretes. The set of vertical reinforcing bars fixing in the horizontal tank directly behind the separating gate making the test simulates the fresh concrete passing through restricted and narrow opening as these two parameters may be cause a blocking or segregation during the flow which is generated by the static weight of the fresh concrete in the vertical column at the beginning of the test. The L-box used in this work has a vertical column with a capacity of 12 l. After pouring the fresh concrete into a chimney section, the concrete lets to flow to the horizontal channel by

opening the gate that is used to prevent concrete to flow from vertical level to horizontal level. At the end of concrete flowing, the test procedure used to determine the percentage of H_2/H_1 where H_1 and H_2 represent the depths at the start and end of horizontal channel of settled fresh concrete



Figure 3.9 Photographic views of V-funnel test

3.1.4 Evaluation of physical and transport properties

To monitor the drying shrinkage of the SCLC, as shown in Figure 3.10, seventy-two 70x70x280 mm prisms were used in order to provide averages of four samples for each mixture as per ASTM C157 (2014). Immediately after demolding the prisms, two pins were glued on the cast face to ensure better bonds. A dial gage extensometer with a 200 mm gage length was used to measure the length change, Figure 3.11. The length variation was measured every 24 h for the first 21 days, and then three times per week. At each length reading, the identical samples were weighed so as to investigate weight loss. The free shrinkage strain and the weight loss measurements were followed through the drying period for 61 days at about 21 °C and 50% relative humidity.

Water sorptivity of three specimens of 100x100x100 mm was measured in accordance with ASTM C1585 (2004). The specimens must be fully dried before the test; therefore, to prepare samples, specimens were first dried in an oven at $105\pm5^{\circ}\text{C}$ until they reached constant mass and then stored in a sealed container to cool to the ambient temperature. Afterwards, the four sides of each sample were coated with a waterproof material, paraffin wax. The sorptivity test was accomplished by setting the specimens on glass rods in a tray such that their bottom surfaces were in contact with a thin water layer of about 5 mm high, as can be seen in Figure 3.12. This procedure allowed free water movement along the bottom surfaces of the specimens. Data on mass variation was collected during the tests at various time intervals up to 64 min. In order to find the sorptivity index, the volume of water soaked up was graphed with respect to the square root of time. The slope of the drawn line represented sorptivity index. In this experimental work to evaluate the water sorptivity index, two ages of 28 and 90 days were elected.

Following TS EN 12390-8 standard (2002), water permeability tests at 28 and 90 days were conducted on three SCLC cubes of 150x150x150 mm. During a 72 h period, a 500 ± 50 kPa pressurized tap water was applied through the bottom side so as to penetrate the test specimen. Immediately upon the passing of 72 h, the test specimens were split into two symmetric parts and the greatest penetration depth that occurred due to the pressurizing water process was measured in mm. As supposed by TS EN 12390-8 (2002) regarding concrete resistance against chemical attack, the water penetrated depth must be no more than 50 mm for moderately aggressive media and 30 mm for concretes with extremely aggressive media. A photograph of the water permeability test is presented in Figure 3.13.

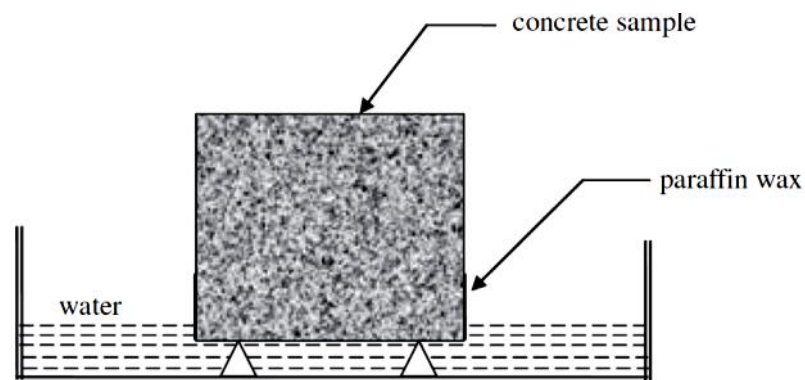


Figure 3.12 Sorptivity test set up

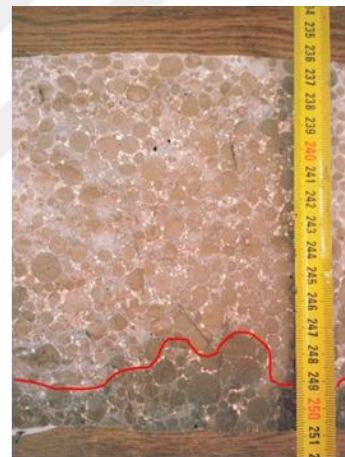


Figure 3.13 Photographic views of water permeability test

The CEMBUREAU method given by RILEM TC 116 (1999) was employed to evaluate gas permeability of all SCLC mixtures at 28 and 90 days. The gas permeability test sample was prepared as a concrete disk specimen of 50 mm thick and 150 mm diameter, cut from the mid-section of $\text{Ø}150 \times 300$ mm cylinder. At the end of the curing period, the samples were dried in an oven until the variation in sample weight did not exceed 1%, then allowed to cool to ambient temperature in a sealed container. Oxygen gas was implemented as the permeating medium. Inlet gas pressures of 150, 200, and 300 kPa were applied and gas permeability coefficients were determined for each level. The Hagen–Poiseuille relationship for laminar flow of a compressible fluid through a porous media with small capillaries under steady-

state condition was utilized to determine the coefficients of gas permeability. As proposed by RILEM TC 116 (1999), the average of calculated coefficients was apparent gas permeability coefficient. Three specimens for each SCLC mix were tested and the mean of them was represented as K. The apparent gas permeability coefficient K can be calculated from the modified Darcy's equation as follows:

$$K = \frac{2P_2QL\mu}{A(P_1^2 - P_2^2)} \quad (3.1)$$

In the equation 3.1, K is the gas permeability coefficient (m^2), P_1 is the inlet gas pressure (N/m^2), P_2 is the outlet gas pressure (N/m^2), A is the cross-sectional area of the sample (m^2), L is the height of sample (m), μ is the oxygen viscosity ($2.02 \times 10^{-5} \text{Ns/m}^2$), and Q is the volume flow rate (m^3/s).

The photographic view of some sample disk SCLC specimens and general view of the apparatus, as well as the detail of the testing cell are presented in Figure 3.14.

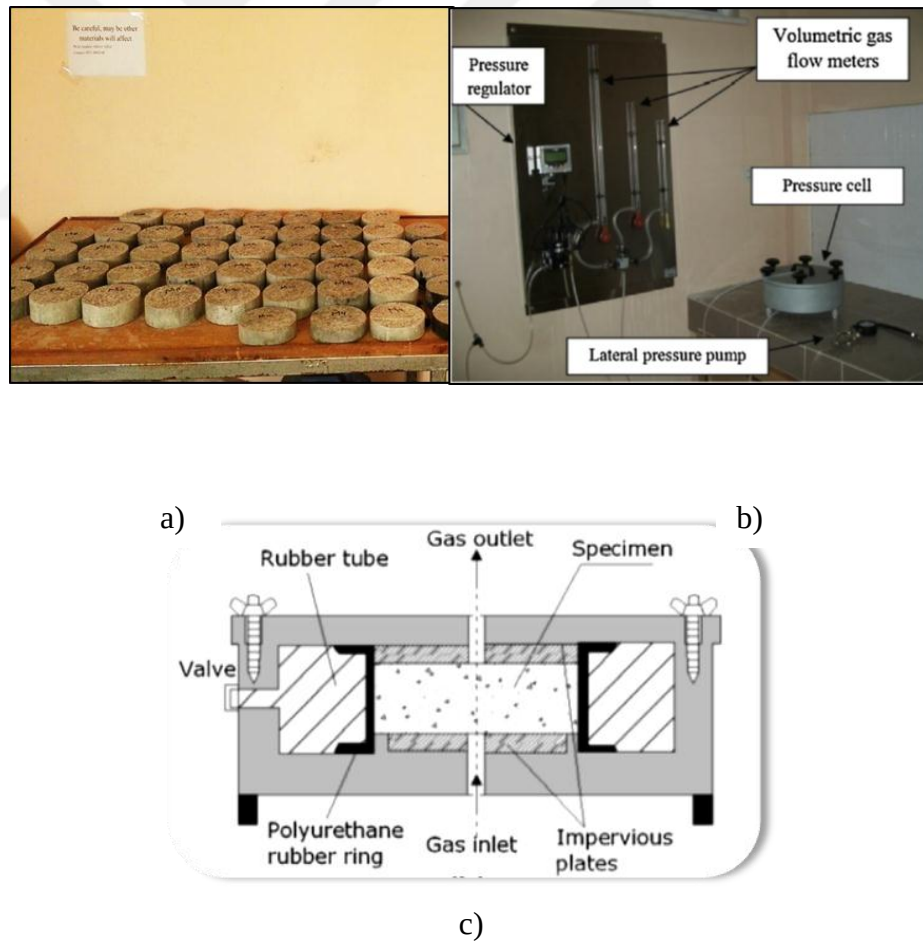


Figure 3.14 Gas permeability details: a) photographic view of disk specimens, b) photographic view of gas permeability device and (c) detail of pressure cell

3.1.5 Evaluation of strength properties

Compressive strength test was conducted on water cured samples of 150x150x150 mm by means of a 3000 kN capacity testing machine at 28 and 90 days of age as per the directions detailed in ASTM C39 (2015). Three cubes were tested and the average values were reported for each type of SCLC mixture at each age.

For splitting tensile strength testing, cylindrical specimens with $\text{Ø}100 \times 200$ mm dimensions were used as it is recommended by ASTM C496 (2011). Three cylinders were tested and the average values were reported for each type of SCLC mixture at ages of 28 and 90 days.

BSI 1881-121 (1983), “Method of determination of static modulus of elasticity in compression,” was applied to determine modulus of elasticity. The maximum stress applied was fixed at 33.33% of the ultimate load because secant modulus (modulus of elasticity) lessens with an increase in stress (Gul et al., 2014). The first set of readings from each cube was neglected and the elastic modulus was reported as the average of the other two sets of readings.

For each test mentioned above, three specimens were used and the mean of them was recorded as a test result. All of the tests were performed at the end of 28 and 90 day of water curing period. Figure 3.15 shows the test setup and specimen for static modulus of elasticity.



Figure 3.15 Modulus of elasticity test setup

3.2 Numerical study

3.2.1 Introduction to CFST column

Concrete-filled steel tubular (CFST) columns have been extensively applied in construction and building engineering. Like these members can be used successfully in industrial plants, subways, high-rise buildings and arch bridges. They characterized by their excellent structural performance such as superior strength, perfect stiffness, low brittleness and large strain energy absorption capabilities. CFST columns construction has established to be cost-effective in building resources materials as well as satisfying for rapid construction and therefore becomes popular in construction building industries worldwide (Yuan and Yang, 2013).

The strength of steel and concrete for building structures have become higher with the improvement of innovative materials, the cross section with these materials becomes lighter, and hence the load carrying capability of the column is significantly influenced. The structural benefits of CFST columns is due to the provision of confinement effect of surrounding steel as well as contribution in load carrying capacity. Moreover, the construction time is also decreased by excluding the permanent formwork. Another noteworthy property is inhibition or delay of local buckling of the steel tube because of core of concrete. Consequently, the use of concrete-filled steel tubes in various areas of constructions is becoming an attractive solution. Moreover, as a result of the increase in the load carrying capacity, stiffness, and ductility as well as economy and rapid construction, use of CFST columns has become prevalent (Chang et al., 2013; Güneyisi et al., 2016b).

In order to dependably predict the strength and ductility of circular CFST columns, correct constitutive models must be developed for confined concrete. However, it has been found that there are somewhat inconsistencies in empirical models projected by investigators for confined concrete (Liang and Fragomeni, 2009). There are also inconsistencies in the capacity prediction of axially loaded CFST column given in current design codes such as Eurocode 4 (2004) and ACI 318 (2005).

3.2.2 Details of study

The main objective of this chapter is to evaluate the behavior of cold bonded fly ash lightweight CFST short circular columns subjected to compressive load. Towards this aim, three different design models for computing axial load carrying capacity of circular CFST short columns presented by ACI 318 (2005), Eurocode 4 (2004), and Chinese (DL/T) (1999) codes were used for prediction of the ultimate axial loads. Four ratios of diameter to thickness (D/t) and three yield stress of steel values were specified for CFST columns. The results of compressive strength obtained from experimental program in this thesis are used for theoretical study. Moreover, the study involves illustration of the effect of curing age, aggregate type, water to binder ratio, and incorporation of nS on the axial load carrying capacity as well as the comparison between the studied codes.

The test configuration taken into account is uniaxial compression test which is schematically described in Figure 3.16.

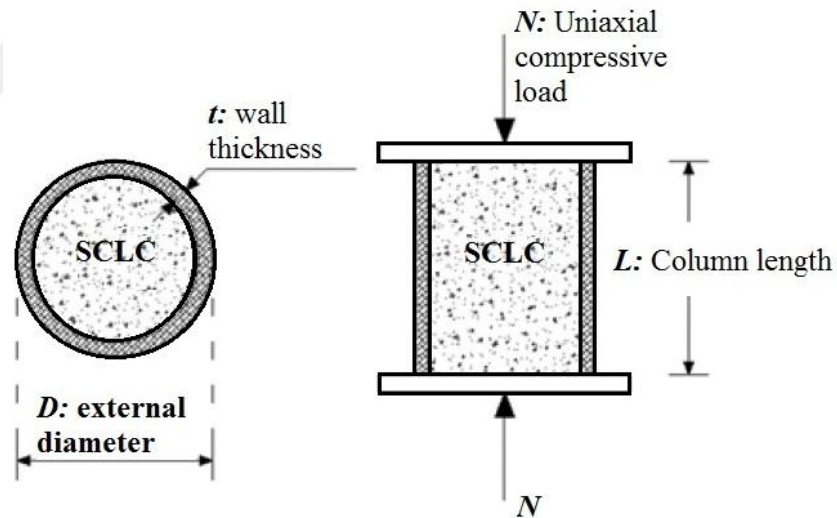


Figure 3.16 Presentation of load applied to the section of circular CFST column

3.2.3 Design code formulas

3.2.3.1 ACI 318R code

The American Concrete Institute (ACI-318R, 2005) for calculating the ultimate axial capacity of the circular CFST column was designated as $N_{u, ACI}$ in calculations. This relation does not contain effectiveness of the concrete confinement and the

interaction between the concrete core and steel tube. The limiting thickness of steel tube to avoid buckling is depend on realizing yield stress in a hollow tube under monotonic axial loading which is not a significantly requirement for CFST column. The equation for axial capacity of a circular CFST column, N_u is expressed by following formulae:

$$N_{u,ACI} = 0.85 f'_c A_c + f_y A_s \quad (3.2)$$

Where f'_c is the compressive strength of concrete cylinders (150×300 mm), A_c is the cross-sectional area of the concrete specimen core, f_y is the yield stress of steel, and A_s is the cross sectional area of steel.

3.2.3.2 Eurocode 4

In the relation recommended by Eurocode 4 (2004), the contribution of the concrete core and the steel tube as well as the confinement effect is considered. The confinement coefficients for the steel (η_a) and concrete tube (η_c) are employed. Therefore, the concrete strength is increased by η_c due to the occurrence of a triaxial state of stress condition while the steel tube strength is decreased by η_a as the hoop stresses yield a reduction in the effective yield steel stress. The Eurocode 4 (EC4) equations for axial capacity of a circular CFST column ($N_{u,EC4}$) are given as:

$$N_{u,EC4} = \left(1 + \eta_c \frac{t}{D} \frac{f_y}{f'_c}\right) f'_c A_c + \eta_a f_y A_s \quad (3.3)$$

where

$$\eta_a = 0.25(3 + 2\bar{\lambda}) \quad (\eta_a \leq 1.0) \quad (3.4)$$

$$\eta_c = 4.9 - 18.52\bar{\lambda} + 17\bar{\lambda}^2 \quad (\eta_c \geq 0) \quad (3.5)$$

in which the relative slenderness ($\bar{\lambda}$) of CFST column can be calculated from following equation:

$$\bar{\lambda} = \sqrt{\frac{N_{pIR}}{N_{cr}}} \quad (3.6a)$$

Where:

N_{pIR} is the cross-section plastic resistance of the CFST column, given as:

$$N_{pIR} = f_y A_s + f'_c A_c \quad (3.6b)$$

N_{cr} is the Euler buckling load of the pin-ended CFST column, represented by:

$$N_{cr} = \frac{\pi^2 (EI)_{eff^2}}{l^2} \quad (3.6c)$$

Where:

$(EI)_{eff^2}$ is the effective flexural stiffness of CFST and can be computed as:

$$(EI)_{eff^2} = E_s I_s + K_e E_{c2} I_c \quad (3.6d)$$

Where:

E_s and E_{c2} is the elastic modulus of steel and concrete, respectively. The elastic modulus of concrete = $22000[(f'_c + 8)]^{0.3}$

Typically, when $\bar{\lambda} = 0$, the sectional capacity (the zero length strength), $P_{0;EC4}$ can be simplified as

$$P_{0,EC4} = \left(1 + 4.9 \frac{t}{D} \frac{f_y}{f'_c}\right) f'_c A_c + 0.75 f_y A_s \quad (3.7)$$

3.2.3.3 Chinese code DL/T

The formulae proposed by Chinese code DL/T (1999) considered CFST column has one material not as a composite with a total area of A_{sc} with a nominal yielding strength of f_{scy} . Moreover, Chinese code DL/T (1999) introduces a factor of ξ for the confinement effect to describe the action between the concrete and steel tube. The equations for axial capacity of a circular CFST column ($N_{DL/T}$) are given as:

$$N_{u,DL/T} = f_{scy} A_{sc} = f_{scy} (A_s + A_c) \quad (3.8)$$

In which:

$$f_{scy} = (1.212 + B\xi + Cx^2) f_{ck} \quad (3.9a)$$

$$B = 0.1759 \frac{f_y}{235} + 0.974 \quad (3.9b)$$

$$C = -0.1038 \frac{f_{ck}}{20} + 0.0309 \quad (3.9c)$$

$$\xi = \frac{A_s f_y}{A_c f_{ck}} \quad (3.9d)$$

$$f_{ck} = 0.67 f_{cu} \quad (3.9e)$$

Where: ξ the confinement factor; A_{sc} and f_{scy} the area of composite section and the nominal yielding strength of the composite section respectively, f_{cu} the compressive strength of 150 mm cube, and f_{ck} the characteristic concrete strength.

3.2.4 Summary of main computation steps

The computation of ultimate axial capacities of the circular self-compacting lightweight concrete-filled steel tubular by using codes illustrated in section 3.2.3 conducted as follows:

- The experimental data of compressive strength have been used.
- So as to comprise different specification regarded steel material characteristics, yield strength of the steel tube f_y selected as 200, 350, 500, and 650 MPa.
- Four different diameter-to-thickness D/t ratios of the specimens were specified, those are: 25, 50, 75, and 100.
- The length to diameter ratio of short composite columns was 3 to ensure that they would be with a small slenderness effect and would not fail in overall buckling. At the same time wall thickness of the steel tube was 3 mm.

It should be noted that the conversion of compressive strength of standard concrete cylinders 150*300 mm (f'_c) to compressive strength of 150 mm cubes (f_{cu}) calculated by using Table 3.4 (Eurocode 2, 2004).

Table 3.4 Conversions between f'_c and f_{cu} (Eurocode 2, 2004)

f'_c (MPa)	12	16	20	25	30	35	40	45	50	55	60	70	80	90
f_{cu} (MPa)	15	20	25	30	37	45	50	55	60	67	75	85	95	105

CHAPTER 4

RESULTS OF EXPERIMENTAL STUDY AND DISCUSSIONS

4.1 Fresh properties of SCLC

The fresh densities of different concrete groups (untreated and treated LWAs) are presented in Figures 4.1 and 4.2. It can be seen the fresh density of concretes varies from 1959 to 1963 kg/m³, 1889 to 1892 kg/m³ and 1834 to 1838 kg/m³ for 0.25, 0.37 and 0.5 w/b ratios, respectively. According to results of fresh density analysis, all concrete mixes for second and third group can be considered as a LWA, whilst the recorded results of the first group with 0.25 w/b ratio slightly increased and may be classified as a semi-lightweight concretes. This might be attributed to three reasons. First, using of lower w/b ratio in this group reduces porosity; consequently, increases density. Second, the binder content was the higher in comparison with other groups. Finally, the combined use of crushed sand and lightweight fine aggregates which caused the fresh density of concretes to exceed the ACI 213R (2003). It was observed that there are a very slight increment in fresh density due to the replacement of untreated with treated coarse aggregates and this is related to specific gravity of treated aggregate relatively higher than untreated aggregates. Another indication must be mentioned that the replacement of cement by nS decreases slightly the fresh density state of concrete. This behavior is due to the replacement of denser particles of cement by lighter particles of nS (Senff et al., 2009).

As refereed earlier, the concretes were designed to give a slump flow diameters of 700±50 mm which were achieved by adjusting the amount of HRWRA used. The visual stability index observation indicated that there is homogeneity in fresh concrete for all mixes and there is no bleeding or segregation even for high w/b ratio mixes. The slump flow diameters of SCLC were determined in the range of 680-738 mm. According to the results, the highest diameter was achieved for LWA-WG-nS0 at 0.25 w/b ratio, whilst the lowest was obtained for LWA-nS5.0 at 0.25 and 0.50 w/b ratios. As represented in Figures 4.3 and 4.4, it can be seen that the lowest

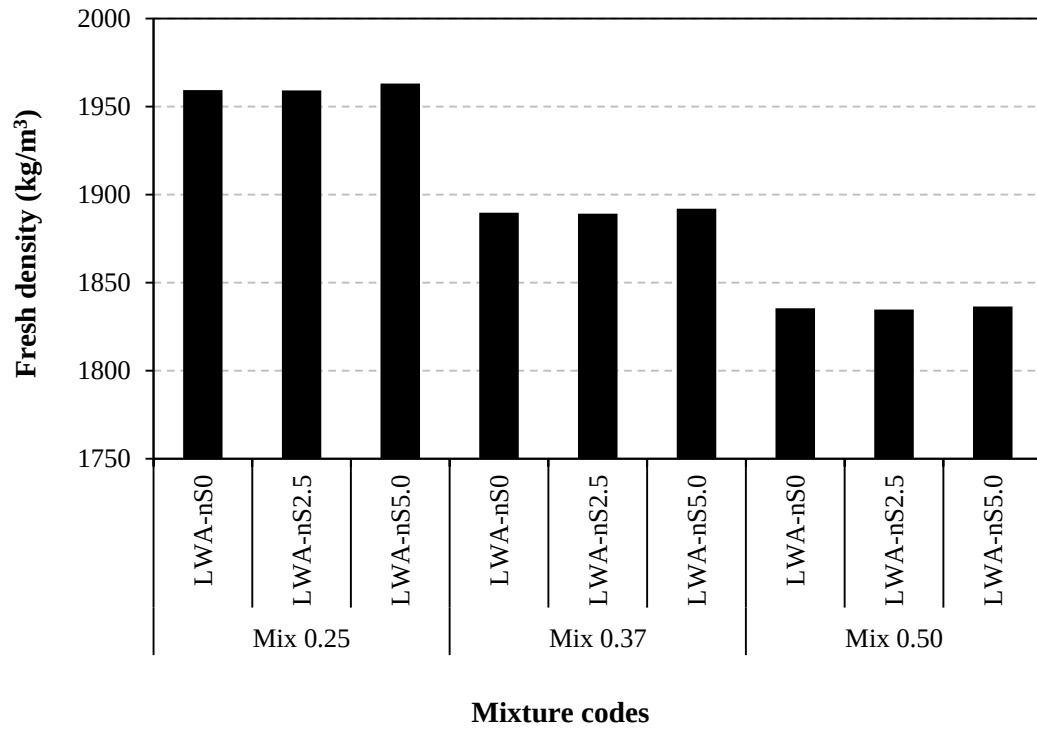


Figure 4.1 Comparison in the fresh density of SCLC with untreated LWA

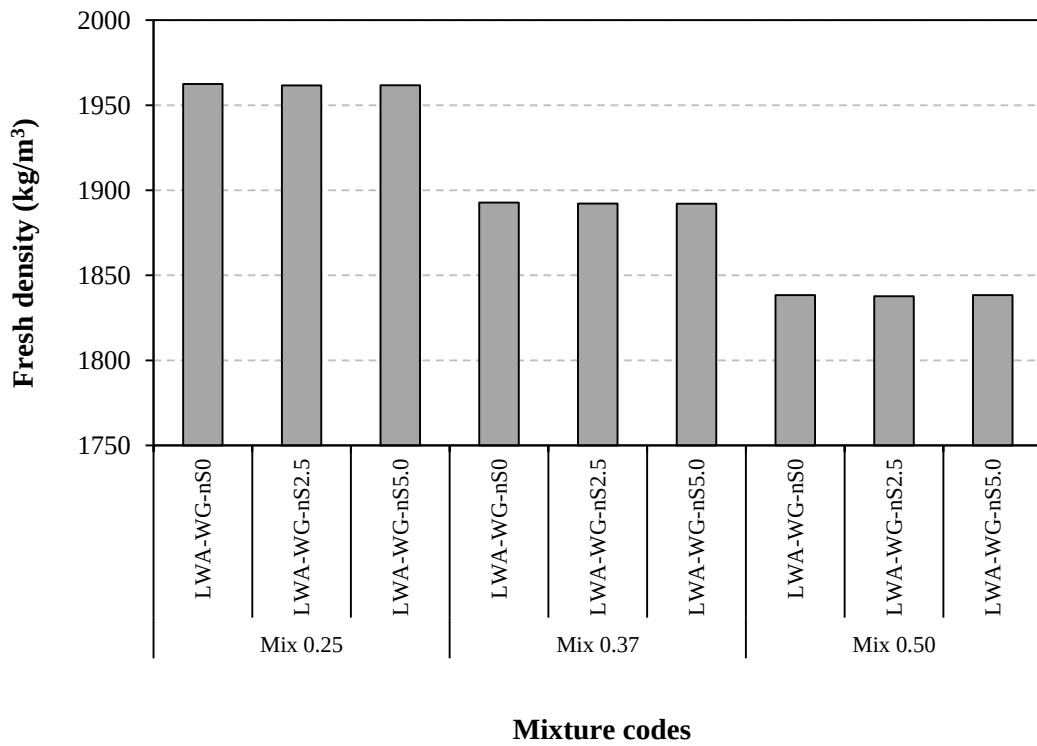
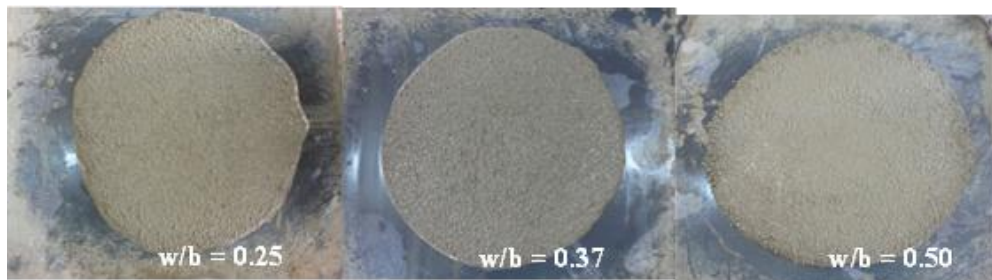


Figure 4.2 Variation of fresh density of SCLC with treated LWA

slump diameters were always recorded to SCLC employed with untreated aggregates. For instance, 700, 710 and 702 mm were deducted for LWA-nS0 mixtures at 0.25, 0.37 and 0.50 w/b ratio, respectively, whilst in the mixtures produced with surface treated aggregates the values became 738, 735 and 730 mm. This behavior is attributed to the weaker cohesion between the cement mortar and water glass treated aggregates due to the smooth and comparably impermeable surface of treated aggregates (Güneyisi et al., 2014c). It can be also indicated that the slump flow diameter influenced by incorporating of nS into the binary blended SCLC mixtures. The slump flow diameter decreases with increasing nS concentration at the same w/b ratio, i.e at 0.50 w/b ratio the diameter was 702.5 mm for the reference mixture, whereas LWA-nS2.5 and LWA-nS5.0 mix gave a slump flow diameter of 695 and 680 mm. (Senff et al., 2009) reported that the addition of nS to the mixture decreases the spread on flow table, due to the increase of cohesion in the mortar. Additionally, the results presented in Figures. 4.4 and 4.5 show that all SCLC mixtures can be diagnosed as slump flow SF2 (class 2) with respect to EFNARC (2005) limitations. Thus the concrete produced in present study can be used in most of the structural members. Moreover, as seen Figure 4.3, aggregate particles exhibited uniform distribution so that no segregation was observed for all mixtures, even for w/b ratio of 0.5.



a) LWA-nS2.5



b) LWA-WG-nS2.5

Figure 4.3 Slump flow test of a) LWA-nS2.5 and b) LWA-WG-nS2.5

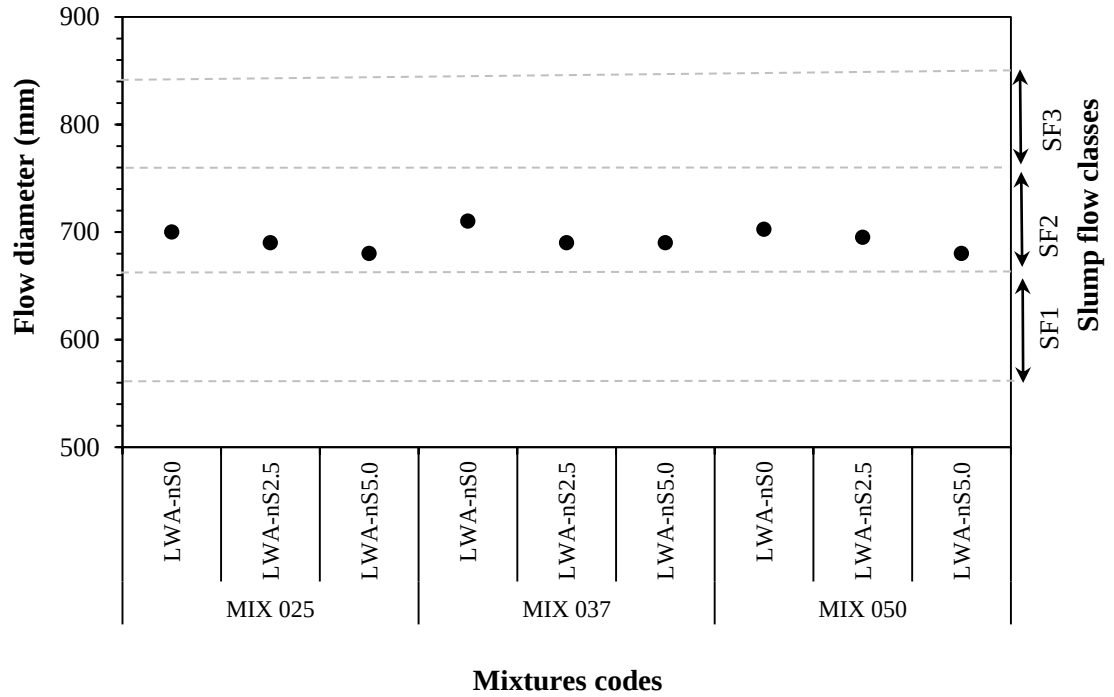


Figure 4.4 Variation of slump flow diameter and slump classes for SCLC with untreated LWA

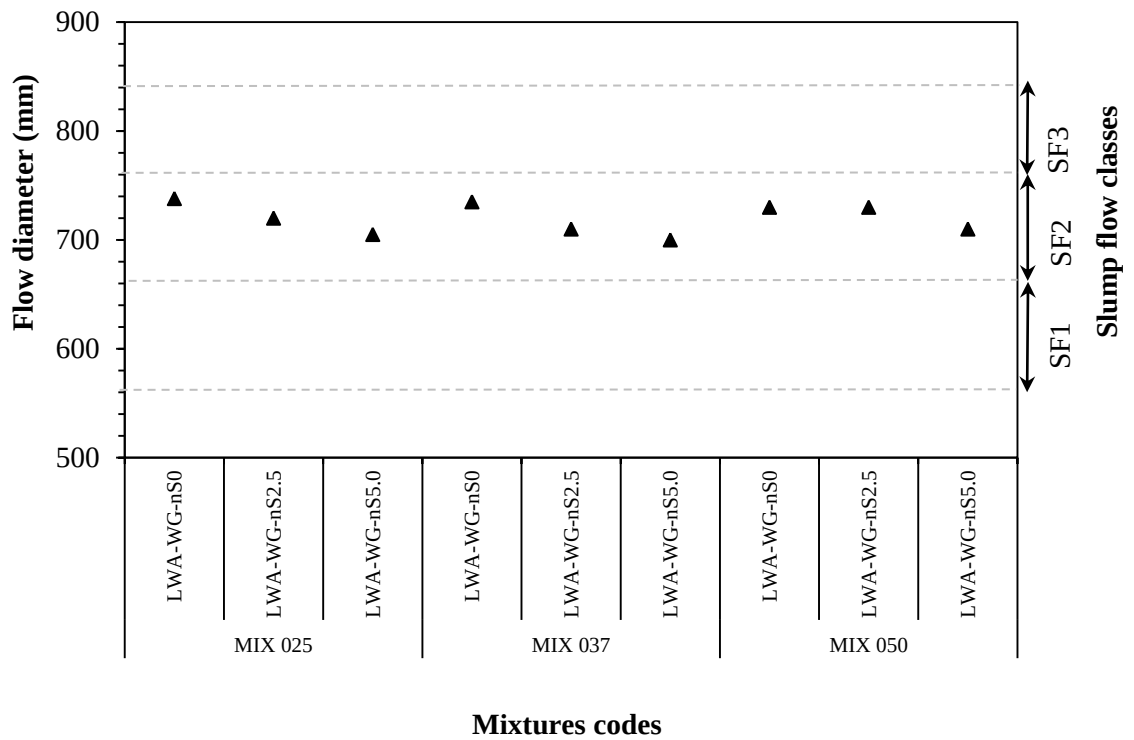


Figure 4.5 Variation of slump flow diameter and slump classes SCLC with treated LWA

The required dose of HRWRA to obtain the aimed slump flow for SCLC made with untreated and treated lightweight aggregate at different w/b ratios are presented in Figures 4.6 and 4.7, respectively. First of all, as could be expected the variation in w/b ratios causes a remarkable dissimilarity in the HRWRA content. For example, as can be clearly seen in Figure 4.6 the optimum dosage of HRWRA was dropped dramatically from 7.9, 3.4 and 2.04 kg/m³ for the reference mixtures namely LWA-nS0 at 0.25, 0.37 and 0.50 w/b ratio, respectively. This is attributed to the fact, an increases in w/b ratio increases the available free water which causes in reducing HRWRA dosage to obtain the required deformability. The same behavior was reported by previous studies related to SCC (Khayat, 1999; Felekoğlu et al., 2007). Moreover, the results indicate when the w/b ratio was kept constant in mixtures, the addition of nano particles in SCLC mixtures had an important effect to increase the required amount of HRWRA and this increase depends on the replacement level of these particles. In the case of control mixture (LWA-nS0) at 0.37 w/b ratio, only 3.4 kg/m³ of HRWRA have been used while LWA-nS2.5 and LWA-nS5.0 required 6.2 and 12.8 kg/m³ of HRWRA, respectively, to achieve the desired slump flow diameter. The mixtures incorporated nS needed higher amount of HRWRA to attain workability reduction. This phenomenon is attributed to the absorption ability of nano-particles to the mixing water (Felekoğlu et al., 2007; Hosseini et al., 2010). The high surface area of nS particles and the availability of unsaturated bonds, making them reactive and attraction of water molecules with surface these particles and caused to create a chemical bond between water and these particles. Therefore, the fluidity of concrete is affected as the free water available will be decreased, and resulting in viscosity increments of the mix (Senff et al., 2009; Hosseini et al., 2010). Regarding the effect of water glass treatment on the required amount of HRWRA, it can be seen from Figure 4.7 that the dosage of HRWRA for SCLC implemented with treated lightweight coarse aggregate was similar to those implemented by untreated surface lightweight coarse aggregate except for mixtures incorporated 5.0% nS. From the results presented in Figure 4.7, (LWA-WG-nS5.0) mix had the higher reduction in HRWRA content about 22% to get a slump flow diameter of 705 mm at 0.25 w/b ratio. The same attitude with previous study which had shown that the surface improvement of recycle aggregate contributes to the HRWRA reduction of SCC mixtures at 0.38 w/b ratio (Güneyisi et al., 2014c).

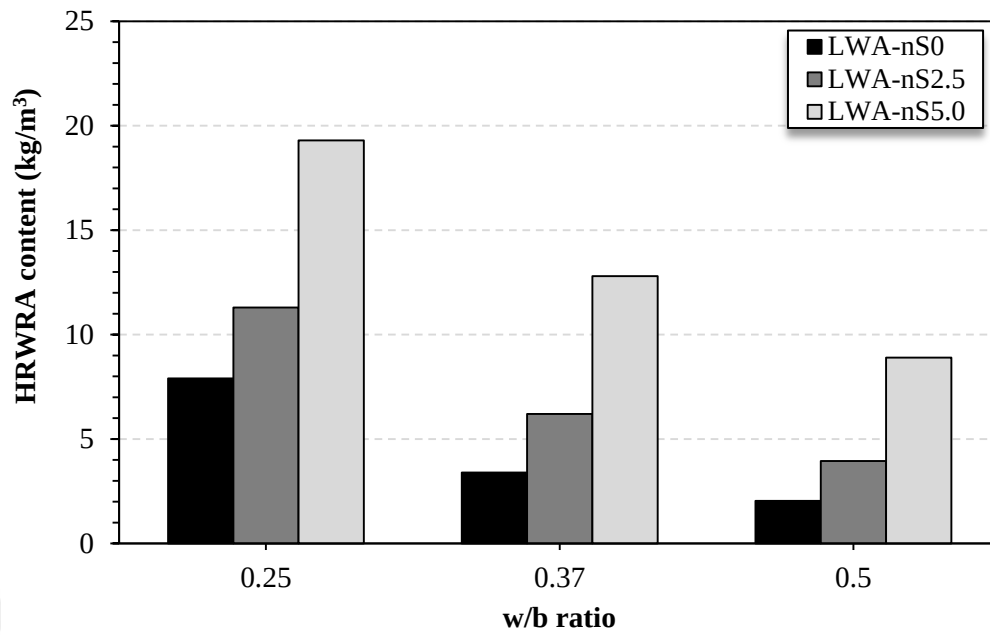


Figure 4.6 Variation of HRWRA content with respect to w/b ratio

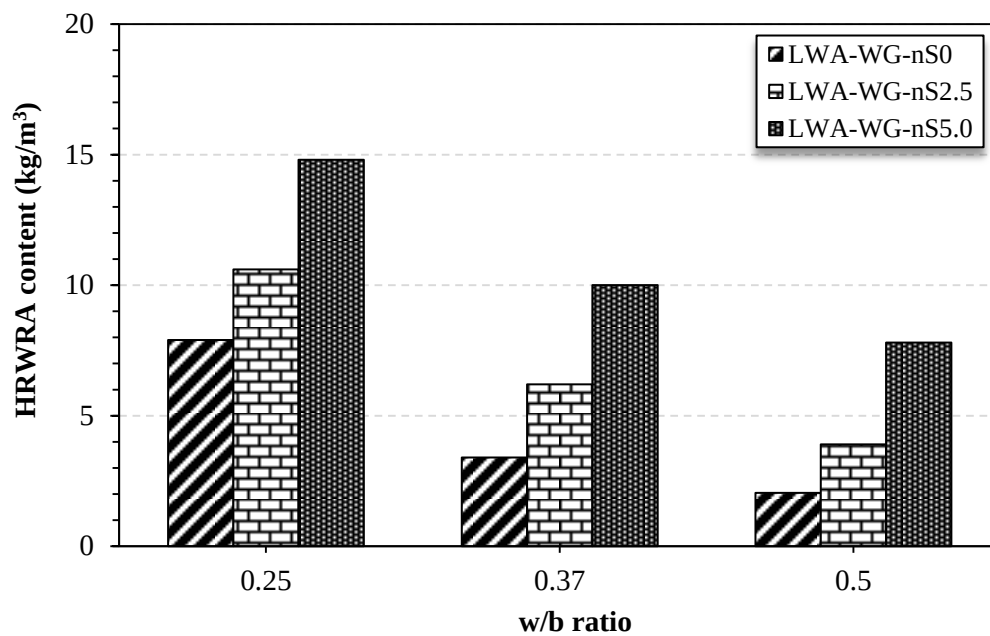


Figure 4.7 Variation of HRWRA content with respect to w/b ratio

The time required to reach 500 mm slump-flow were demonstrated Figures 4.8 and 4.9 with respect to concretes made with untreated and treated LWAs, respectively. It was observed from Figure 4.8 that $T_{500 \text{ mm}}$ slump flow time of reference mixtures LWA-nS0 were 3.66, 1.16 and 0.28 s, respectively, depending on the w/b ratio increment. These results provided an indication about the relation between free water increasing and flow resistance decreasing. The ball bearing effect produced by the spherical particles of conventional mineral admixtures has resulted in reducing slump flow time. Similar behavior was approved in previous investigations which stated a clear trend of lowering of the slump flow time when the mineral admixtures was used up to certain dose (Boukenda *et al.*, 2012; Uysal and Yilmaz 2011). However, as presented in Figures 4.8 and 4.9, it can be indicated that the slump flow time of SCLC increased slightly with the replacement level of Portland cement by nS in the mix. For example, mixture made with ordinary surface aggregate at 0.25 w/b ratio (LWA-nS0) recorded 3.66 s increased to 4.68 and 4.98 s for LWA-nS2.5 and LWA-nS5.0 mixture, respectively. The spherical shape of nS particles can reduce internal frictional forces due to ball bearing effect (Laskar and Talukdar, 2008). However, the addition of nS can improve the packing particles, causing to reduce the volume between the fine particles in mixture, consequently reduces the amount of free water. Therefore, the internal friction between solid particles contributes to create flow resistance higher than ball bearing effect (Senff *et al.*, 2009). The range of T_{500} between 2-5 s maybe preferred because the viscosity is enough to prevent segregation as reported by Barfield and Ghafoori, (2012). According to the test results of time required to reach 500 mm slump-flow related with nS incorporation, it may be concluded that nS has positive effect as it lead to increase the slump flow time with this range.

Figures 4.8 and 4.9 show that T_{500} slump flow time value for all produced SCLC mixtures made with treated aggregate are lower than mixtures produced with untreated aggregate. For example, the time needed to reach 500 mm equal to 1.31 s for LWA-nS2.5 mix at 0.37 w/b ratio, whilst LWA-WG-nS2.5 recorded 1.2 s. This is because of lower water absorption capacity for treated LWA caused to provide more free water between the aggregate particles in SCLC mixes, hence resulted in a slump flow time reduction. Nevertheless in the current investigation there is no segregation even with T_{500} lower than 2 s.

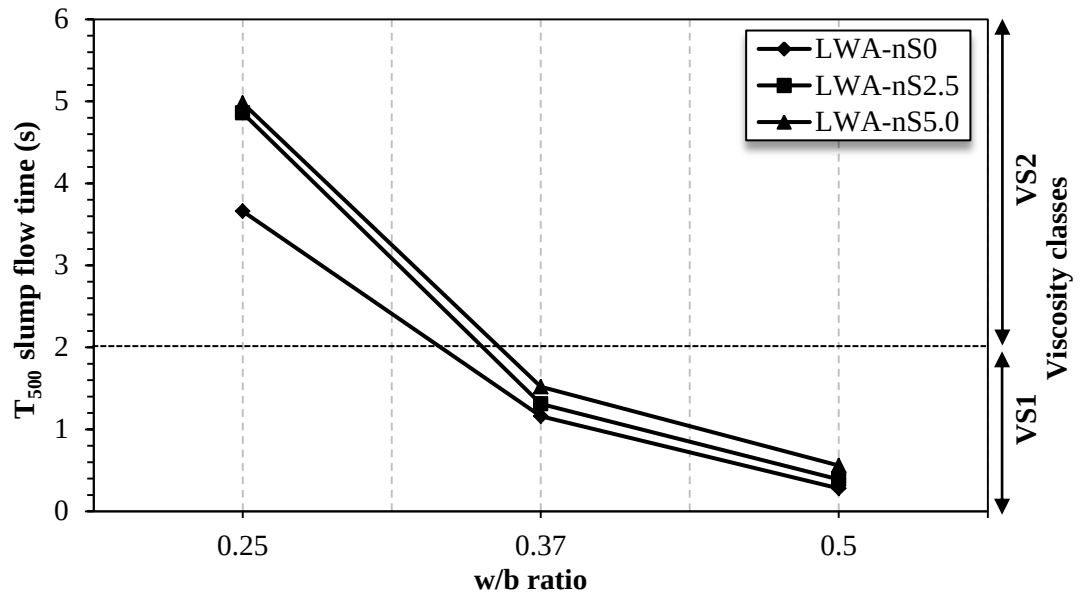


Figure 4.8 Variation of T_{500} slump flow time for SCLC with untreated LWA

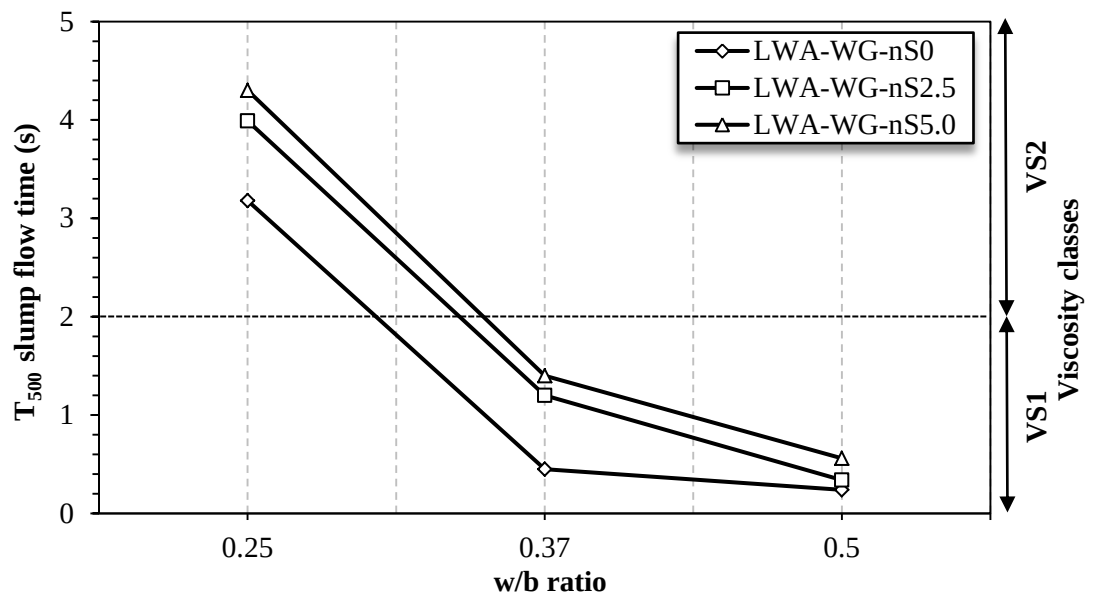


Figure 4.9 Variation of T_{500} slump flow time for SCLC with treated LWA

The V-funnel test measures the filling ability of a mixture. It is known that the V-funnel flow time is related to the cohesiveness and viscosity of the mixtures. There are two viscosity classes expressed by V-funnel time, VF1 less or equal to 8 s while VF2 limits should be between 9 and 25 s (EFNARC, 2005). The determined V-funnel flow times of eighteen SCLC mixtures at different w/b ratios are presented in Figures 4.10 and 4.11 for untreated and treated LWA, respectively. Almost results of all concrete met the requirements specified in (EFNARC, 2005) except mixture namely LWA-nS2.5 and LWA-nS5.0 at w/b 0.25 recorded about 26 s. In general, the effect of w/b on V-funnel flow times seems to be more dominant than the effect of nS and treated LWA incorporation. For reference mixture LWA-nS0 had V-funnel flow time 24.5, 7.84 and 3.36 s at 0.25, 0.37 and 0.50 w/b ratio, respectively. Moreover, the results presented in Figures 4.10 and 4.11 show that the SCLC mixtures made by 600 kg/m³ binder content at 0.25 w/b ratio resulted the higher V-funnel time, whilst the lower values were observed in mixtures produced by 450 kg/m³ binder contents at 0.5 w/b ratio. On the other side, although the incorporation of extreme dosage of HRWRA in LWA-nS2.5 and LWA-nS5.0 at 0.25 w/b ratio obtained desired slump flow diameter as discussed earlier, but it is not sufficient to modify V-funnel flow time in the range of permissible values, Figure 4.10. This in agreement with Utsi et al. (2003), they reported that the slump flow below 650 mm some relation can be occurred with rheology parameters but for more flowable mixes, slump flow diameter over 650 mm, the correlation may be not exists at all. It was evident from Figures 4.10 and 4.11 that the V-funnel time showed a tendency to increase with increasing nS content. For example, at w/b ratio 0.5 the LWA-nS0 mixture had a V-funnel flow time of 3.36 s which increased to 4 and 6.36 s as nS introduced to 2.5% and 5.0% by weight of cement. This supports that nS addition made the concretes more viscous which is in agreement with those incorporated mineral admixtures have a high specific surface in cement mortar or SCC (Madandoust and Mousavi, 2012; Ranjbar et al., 2013). Figure 4.11 shows the SCLC containing water glass treated LWA recorded shortest flow V-funnel time compared to mixes made with natural surface aggregate. With the replacement untreated by treated aggregates the viscosity classes of the LWA-nS2.5 and LWA-nS5.0 produced at 0.25 w/b ratio turn into the VF2. This indicates that the viscosity is reduced for treated LWAs due to the lower cohesion forces generating from lower water absorption capacity generally cause lower flow viscosity (Güneyisi et al., 2014c).

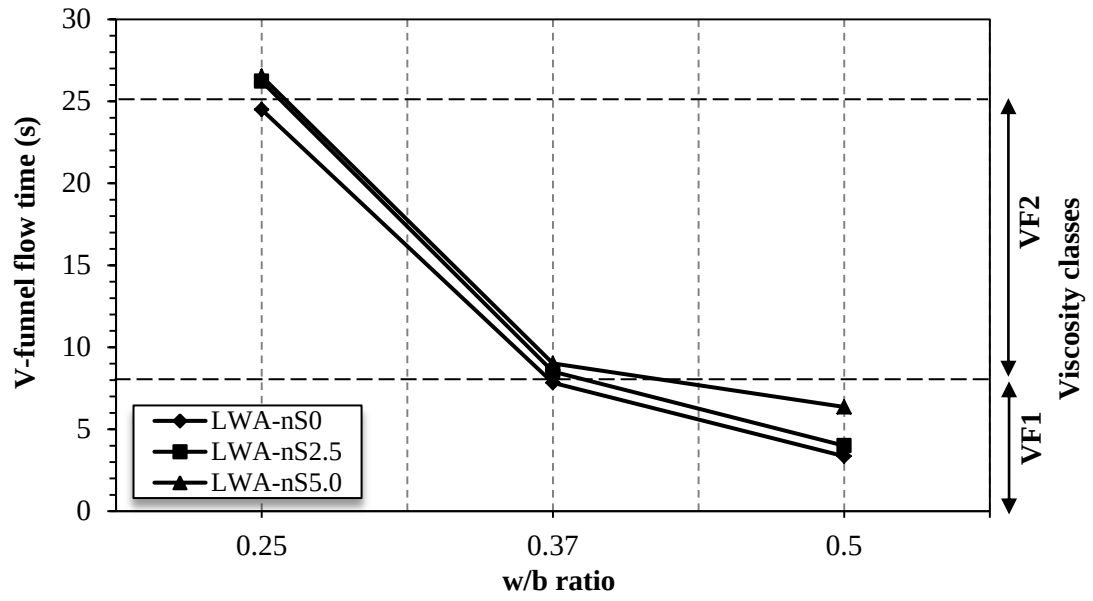


Figure 4.10 Variation of V-funnel flow time (s) for SCLC with untreated LWAs

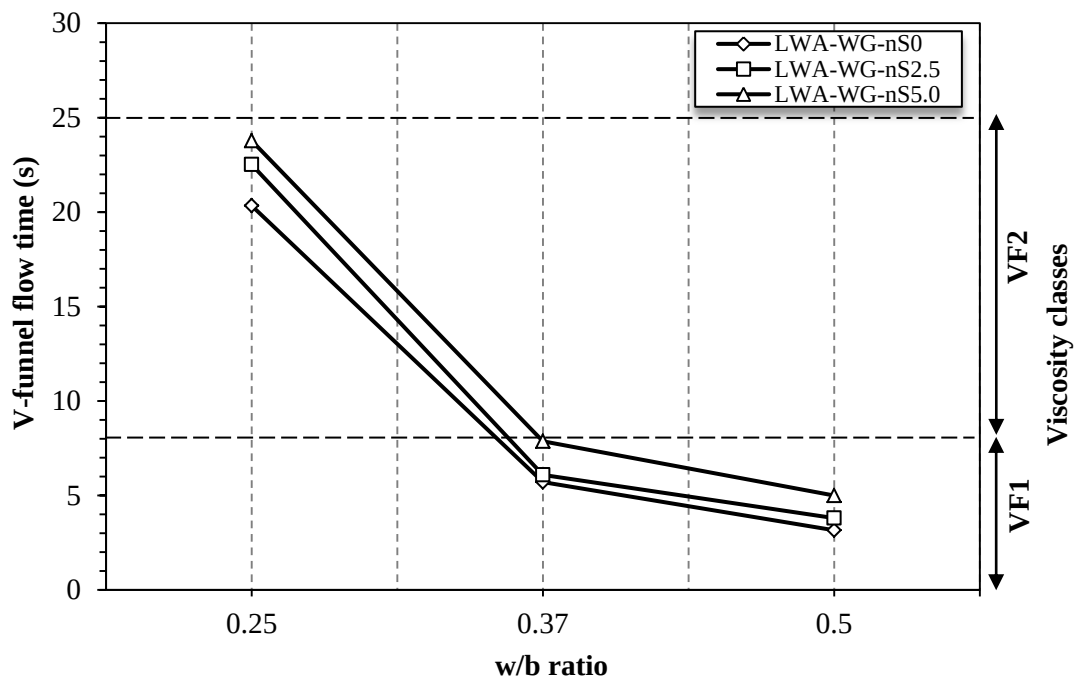


Figure 4.11 Variation of V-funnel flow time (s) for SCLC made treated LWA

Figure 4.12 gives the variation of viscosity classes with T_{500} slump flow and V-funnel flow times. Out of eighteen SCLC, twelve mixtures produced with 0.37 and 0.50 w/b ratio were classified as VS1/VF1 except two mixes (0.37 w/b) namely LWA-nS2.5 and LWA-nS5.0, while most the six mixtures (0.25 w/b) were in the classification of VS2/VF2. As clearly seen, at 0.25 w/b ratio the incorporation of nS in SCLC mixtures made with untreated LWA slightly exceeded the upper limit of VS2/VF2 class. On the other hand, the results of all SCLC with nS clearly reveal the slight increase of viscosity by increasing the nS percentage from 0 to 5.0%. This conclusion agrees with research stating that incorporating nanoparticles generally made the concretes a little more viscous (Senff et al. 2009; Hosseini et al., 2010). The properties of the cement paste of SCLC were found to be very important to prevent segregation. If the mortar viscosity is high enough, the coarse aggregates will be supported by the mortar, thus avoiding segregation. Often, viscosity modifying agents or mineral admixtures are utilized to increase the viscosity, without significantly increasing the yield stress (Ferraris and Larrard, 2001). It can be concluded from the results, the consistency of SCLC mixtures at 0.37 and 0.50 w/b ratio improved as the addition of nS increased, so it may be considered, it is not necessary to use viscosity modifying agent in the nS concretes. Furthermore, as seen from Figure 4.12, good correlation coefficient was observed for T_{500} flow time and V-funnel flow time ($R^2 = 0.979$). Considering viscosity class of VS1/VF1, these concretes are appropriate for the slabs and floors and VS2/VF2 feasible for ramps and walls, columns with respect to EFNARC recommendations (EFNARC, 2005).

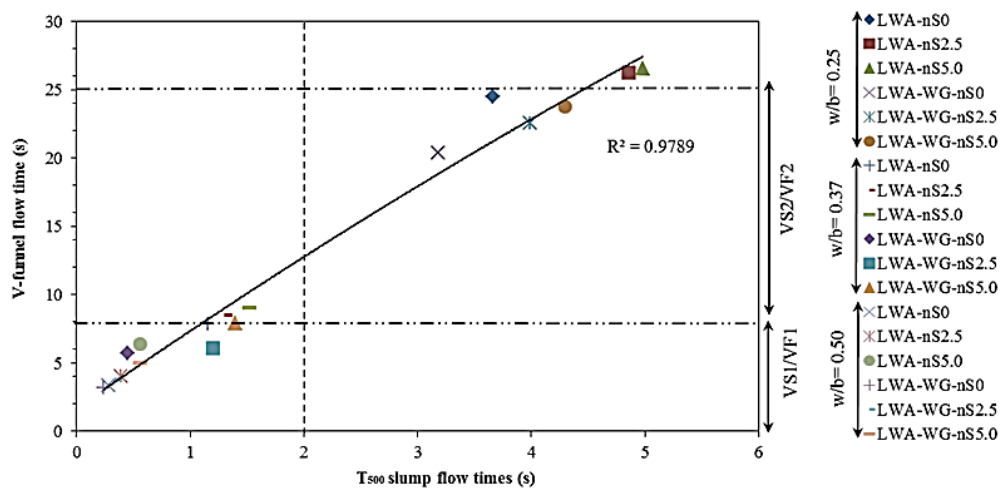


Figure 4.12 Variation of viscosity classes with T_{500} slump flow and V-funnel flow times

To identify the passing ability of the produced SCLC, L-box test have been used. The three bars test was adopted in this study instead of two bars test as it is simulate more congested reinforcements. The test provided H_2/H_1 ratio as a measure of the flowability among reinforcing bars. The variation in the three bar L-box height ratio are demonstrated in Figures 4.12 and 4.13. To affirm that a self-compacting concrete has no blocking risk, height ratio must be equal to or greater 0.8 as per by EFNARC (2005). According to the test results, all SCLC mixtures were met the expectations of EFNARC.

As clearly seen from Figures 4.12 and 4.13, increased the w/b ratio caused to increase in H_2/H_1 ratio slightly. For instance, at the 0.50 w/b ratio the H_2/H_1 ratio of LWA-nS0 was 0.91 slightly higher than that of the same mixture at 0.37 w/b ratio in which the H_2/H_1 ratio decreased to 0.86. Felekoğlu et al. (2007) reported that the risk of blocking of SCC decreases as w/b ratio increases. Figures 4.12 and 4.13 indicated that when considering the fixed w/b ratio, changing type of coarse aggregate affected the passing ability behavior of concrete. The highest values of blocking ratio always have been recorded for the mixtures made with treated LWA. For example, at 0.37 w/b ratio the H_2/H_1 ratio varied from 0.86 to 0.93 for LWA-nS0 and LWA-WG-nS0, respectively. Findings of this test emphasized that utilization of treated LWA provided a systematic increase in the L-box height ratio.

As evident from test results of L-Box, the concretes with ternary systems (FA+nS) provided slightly better performance in terms of L-box. Especially, a perfect fluid behavior was observed in LWA-WG-nS5.0 at 0.50 w/b ratio due to having H_2/H_1 ratio being 1.0. This may be related to the combined effects of high w/b ratio, high quality of treated LWA and nS. Regarding nS effects on L-box test, a similar conclusion was also observed by Jalal et al. (2012). They reported that the addition of micro and nano-silica materials improved the passing ability of the high performance self-compacting concrete and reduced the probability of bleeding and segregation.

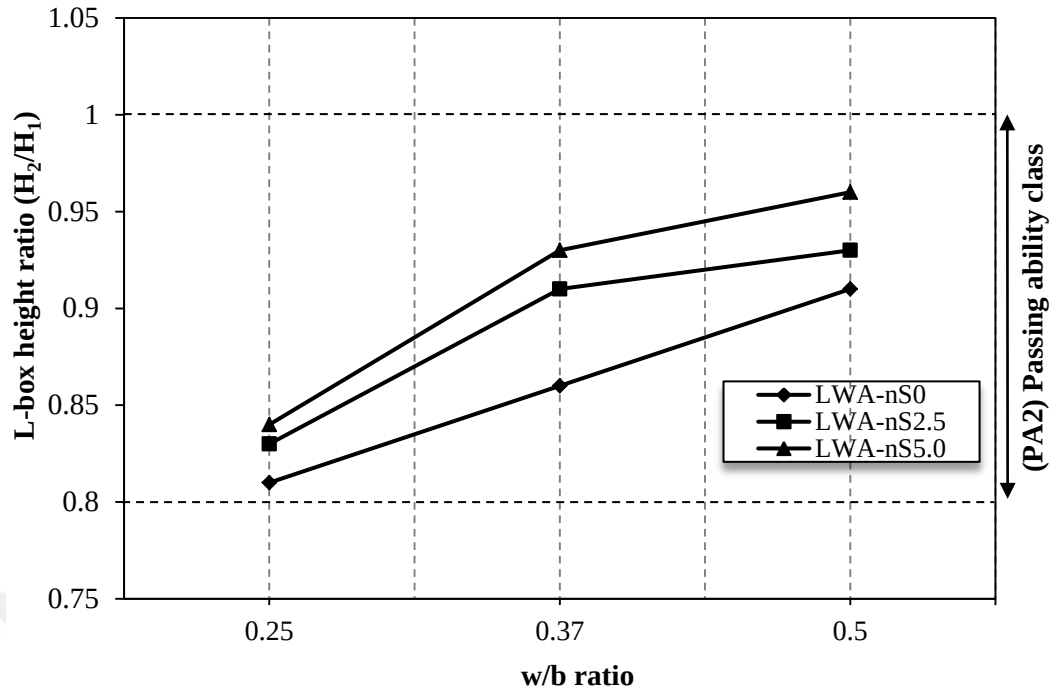


Figure 4.13 Variation of L-box height for SCLC with untreated LWA

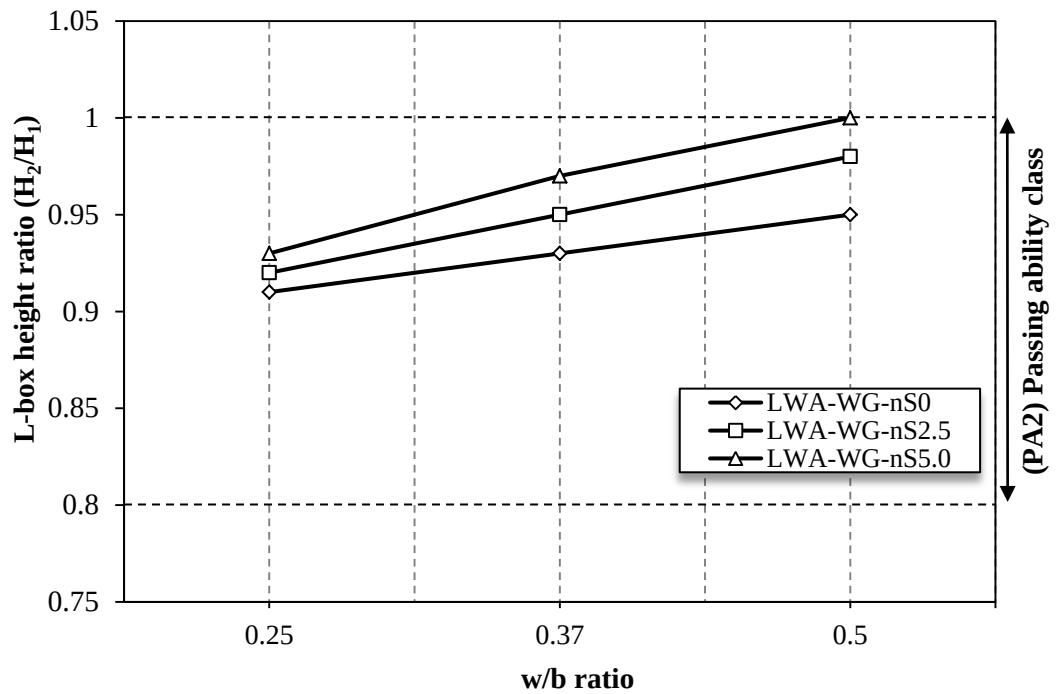


Figure 4.14 Variation of L-box height for SCLC with treated LWA

4.2 Shrinkage and permeability of SCLC

A change in volume of concrete due to shrinkage may result in the cracking of concrete members, thereby exposing the reinforcement bar to corrosion and deterioration. This occurs primarily in structural concrete applications. The influences of w/b ratio and types of aggregates on the drying shrinkage of SCLC are shown in Figures 4.15 and 4.16. These figures demonstrate that the w/b ratios had significant influence on the drying shrinkage strain of the SCLC for both LWA types (untreated and treated LWA). The drying shrinkage values decreased with decreasing w/b ratios as a result of enhancement in strength and refinement in microstructure. The effect of w/b ratios on shrinkage results in the present study are comparable to that reported by (Zhang et al., 2003; Bogas et al., 2014) found that a small reduction in w/b ratio produced a significant reduction in drying shrinkage.

For LWA-nS0 mixes, a reduction of w/b ratio from 0.50 to 0.37 generated a significant decrease in drying shrinkage from 1005 to 905 micro strains at 61 days, whereas a further reduction of w/b ratio to 0.25 produced 820 micro strains. Figure 4.16 also shows a similar trend in the concretes group with water glass treated aggregates.

It is well known that aggregates oppose the contraction of the concrete system, thereby, it is expected that the type of aggregates will create different observations in shrinkage behavior. First, LWA with high absorption correlate with high shrinkage at the end of 61 days in SCLC samples. This is assured by the reduction in drying shrinkage when untreated LWA are replaced with treated LWA in mixtures as presented in Figure 4.16. Gesoğlu et al. (2006) reported that concretes with less water absorbency and harder LWAs revealed lower shrinkage. At 61 days, the shrinkage strains decreased by approximately 15%, 11% and 7% for low, moderate and high w/b ratios with the use of water glass aggregates.

In another observation, it was found that the shrinkage of SCLCs specimens with untreated aggregates is lower than that of SCLCs with treated aggregates during the first week. This behavior is attributed to the water absorbed by untreated LWAs when liberated into the system by internal curing and therefore counterbalances the loss of the initial water from drying and self-desiccation. The same conclusion has also been documented by (Zhang et al., 2005).

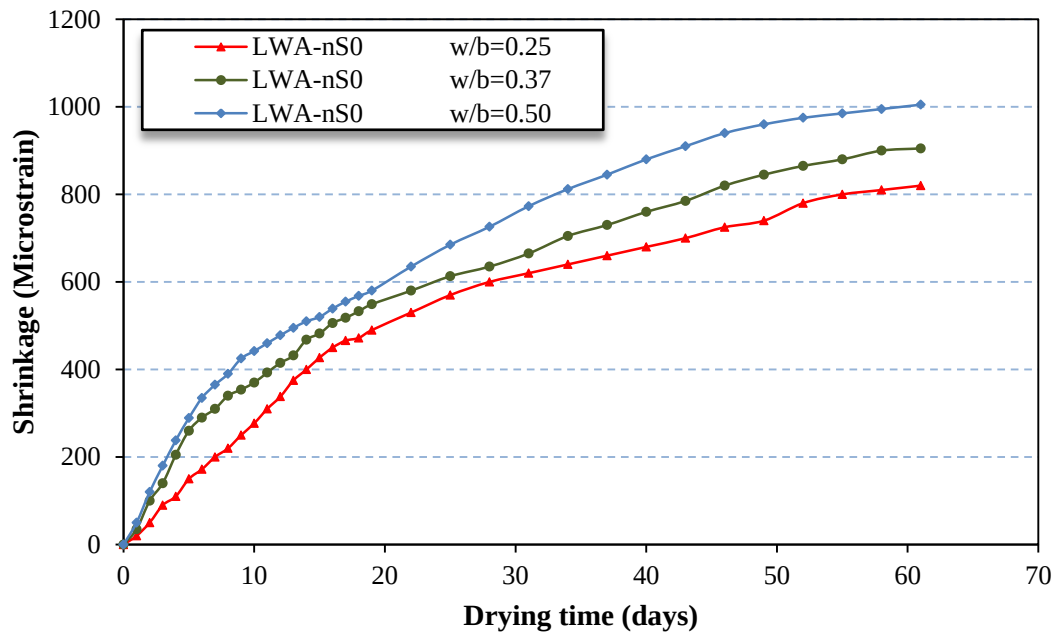


Figure 4.15 Shrinkage deformation for SCLC with untreated LWA

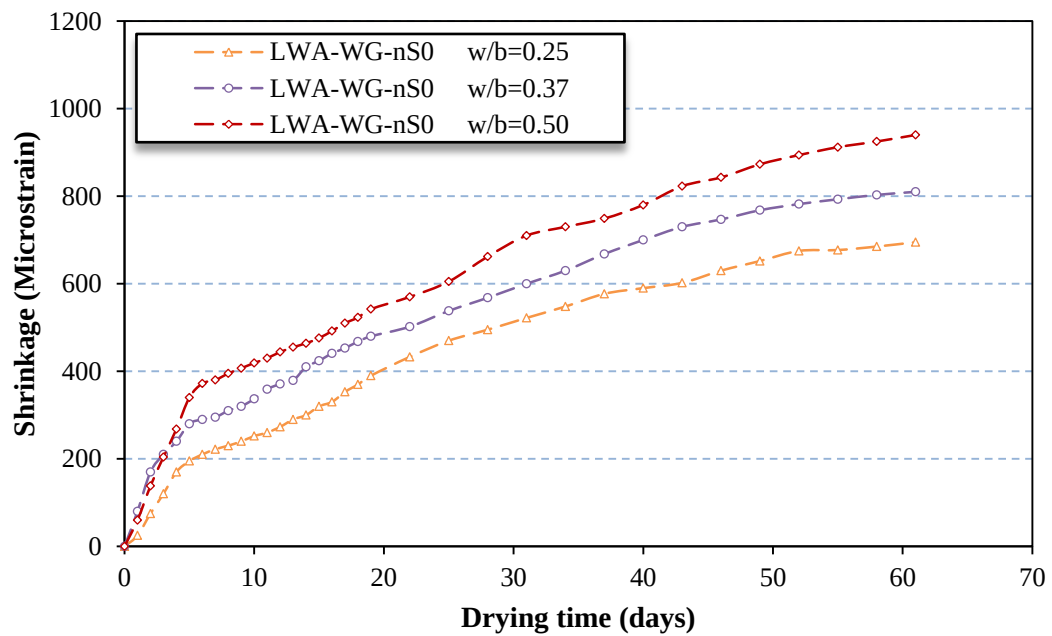


Figure 4.16 Shrinkage deformation for SCLC with treated LWA

The influence of nS on strain developments of the drying shrinkage samples made with untreated and treated LWAs are shown in Figures 4.17 and 4.18, respectively. As seen in Figures 4.17 and 4.18, at early test ages, the variances between values of drying shrinkage of SCLCs exhibit an extremely steep development. Although the free shrinkage strains were affected by incorporating nS, the w/b ratios still have the main role in controlling the results even at initial readings.

According to Figures 4.17 and 4.18, the binary use of cement+FA yielded higher shrinkage in comparison to the specimens with the ternary blends of cement+FA+nS, irrespective of w/b ratio. For example, at 0.50 w/b ratio, the drying shrinkages of LWA-nS2.5 and LWA-nS5.0 were approximately 6% and 10% lower than that of the concrete free of nS, namely LWA-nS0, respectively. On the one hand, this positive effect seemed to be higher with the increase in the replacement level of nS. On the other hand, the lower replacement level of nS appeared to be more influential than the highest level, i.e. an increase in nS did not directly correlate with the same percentage increase in positive effect. The beneficial effect of nS on drying shrinkage was also reported by Sadrmomtazi and Barzegar (2010) and Bogas et al. (2014). Moreover, the test results discussed herein agree with the findings of the investigations conducted by Al-Khaja (1994), Li and Yao (2001) and Güneyisi et al. (2007) regarding the influence of ultrafine mineral admixtures on shrinkage.

Interestingly, the efficiency of nS in improving drying shrinkage of SCLCs appeared to be more pronounced as the w/b ratio increased. For instance, at the end of the drying period, indeed, the drying shrinkages of the LWA-nS5.0 mix were approximately 6%, 7% and 10% lower than those of the LWA-nS0 mix at 0.25, 0.37 and 0.50 w/b ratio, respectively, while for the LWA-WG-nS5.0 mix, the enhancements were about 9%, 13% and 14% compared to LWA-WG-nS0 at lower, moderate and high w/b ratios, respectively. This indicates that porous aggregates reduced the effectiveness of nS on reducing shrinkage strains. (Bogas et al. 2014) reasoned that this behavior of porous aggregates may be behind the absorption of a part of this mineral admixture.

The relationship between shrinkage after 61 days and the replacement levels of nS can be seen in Figure 4.19. It was denoted that there was an inverse relationship, indicating a reduction in drying shrinkage of concretes for all w/b ratios and

aggregates types with increases in nS levels. However, maximum shrinkage strains at 61 days were achieved within LWA-nS0 at 0.50 w/b ratio of about 1005 micro-strains, whereas the lowest of about 630 micro-strains was reached in LWA-WG-nS5.0 at 0.25 w/b ratio. A part of this behavior may be due to the effect of using higher nS replacement level which worked as shrinkage reducer or compensator. Moreover, Figure 4.19 reveals that there is a positive correlation between nS replacement level and shrinkage. Mixtures containing untreated aggregates and treated aggregates at w/b ratio of 0.37 resulted in the highest correlation coefficient with values of 0.987 and 0.97, respectively.

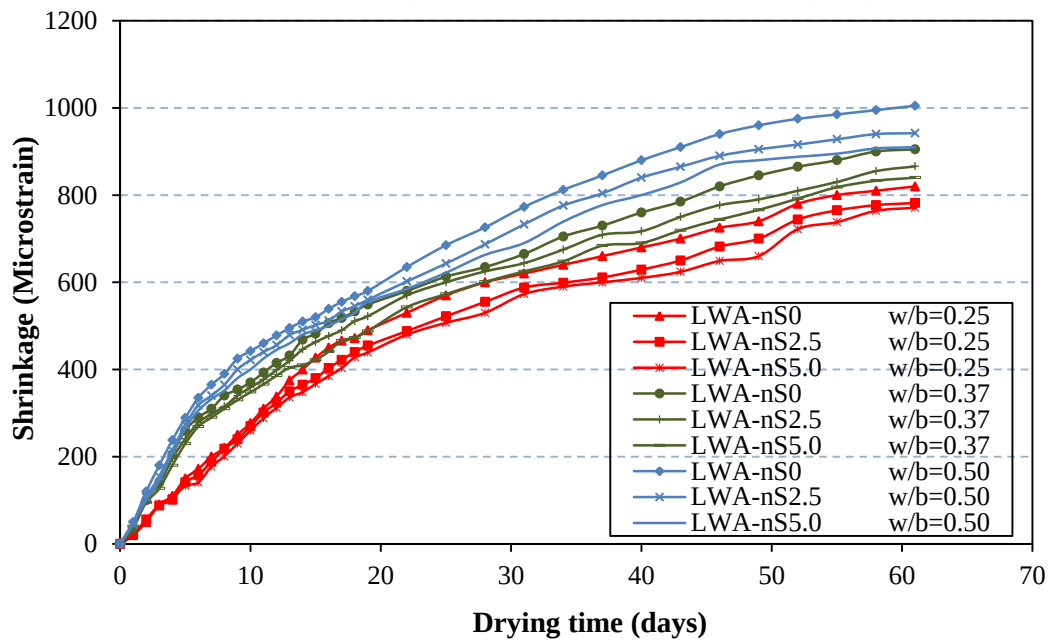


Figure 4.17 Shrinkage deformation for SCLC with untreated LWA having different nS

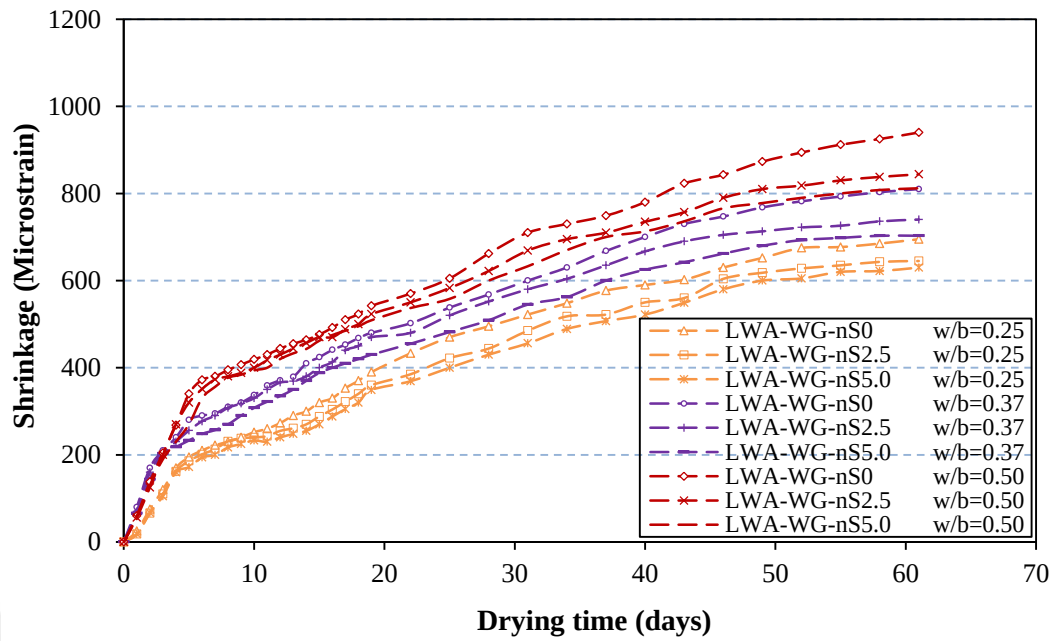


Figure 4.18 Shrinkage deformation for SCLC with treated LWA having different nS

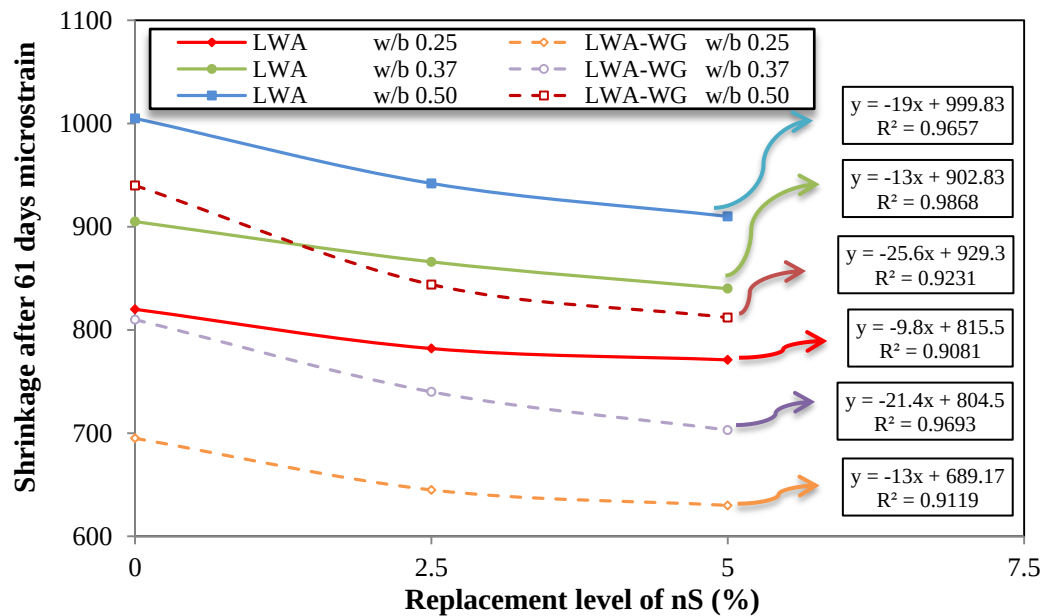


Figure 4.19 Shrinkage deformation results with varying nS values

The weight losses with respect to drying time of SCLC mixes made with cement and FA are shown in Figures 4.20 and 4.21 with respect to untreated and treated LWA, respectively. Although the differences in weight loss appeared at the early ages of the drying period, a distinction was made as weight loss increased as the drying time increased for both types of LWA. Similar to the drying shrinkage test results, for the same w/b ratio, increased water loss was occurred by natural coarse LWA more than treated coarse LWA. This matches the expected outcome that saturated aggregate particles act as water reservoirs, taking into account in this work that the water absorption of untreated lightweight aggregates were remarkably higher than treated lightweight aggregates as discussed earlier. Therefore, the almost saturated untreated particles are providing water into the drying matrix (Kohno et al., 1999; Bentur et al., 2001), thus making the unit water content higher compared to a treated situation, and consequently, increases weight loss during the drying period (Gesoglu et al. 2004). This also implies that weight loss upon drying can be considered as a function of aggregate porosity.

The weight loss variations at the end of the 61 day drying period with respect to the w/b ratio of SCLC mixes free from nS are presented in Figure 4.22. It is evident from the results there is a substantial increase in weight loss with an increase in w/b ratio. For instance, with the utilization of 0.25, 0.37 and 0.5 w/b ratios, the weight loss extended by about 8%, 9.5% and 13.7%, respectively. However, as mentioned before, the SCLC samples with lowest water absorption-treated aggregates- had lower weight losses and shrinkages than those with untreated aggregates, proving the role of the aggregate quality in controlling shrinkage and weight loss.

The influence of nS incorporation on the behavior of weight loss of SCLC samples at 23 ± 2 °C with $50\pm 5\%$ relative humidity for 61 days made with normal and treated coarse aggregates is given in Figures 4.23 and 4.24, respectively. These graphs cover the three w/b ratios employed in this study. It can be seen that the addition of nS decreased the weight loss of SCLC specimens for all w/b ratios. In general, this trend is similar to the drying shrinkage results as discussed earlier. For instance, in the case of using 2.5% nS at 0.25 w/b, the reduction in weight was approximately 9.8% for LWA-nS2.5 and 7.58% for LWA-WG-nS2.5. Zhang et al. (2015) stated that the greater pore refining effect of nS results in a smaller water loss. In the same previous study, it was concluded that the nano particles decrease the contact angles of the

cement-based material which contributes to the reduction of water loss through the drying period. However, in the present study, the results also indicate that with decreasing w/b ratio, the effect of nS on reducing weight loss increases accordingly. The current behavior did not match the drying shrinkage results. This may be due to the free shrinkage of concrete being related to some other factors in addition to weight loss during drying (Wiegrink and Marikunte, 1996).

Observing Figures 4.23 and 4.24, it is obvious that the weight loss decrement is slighter with the second level of nS. It can be concluded that the addition of nS to SCLC can enhance the elimination of water loss caused by curing (Esmaeili and Andalibi, 2013).

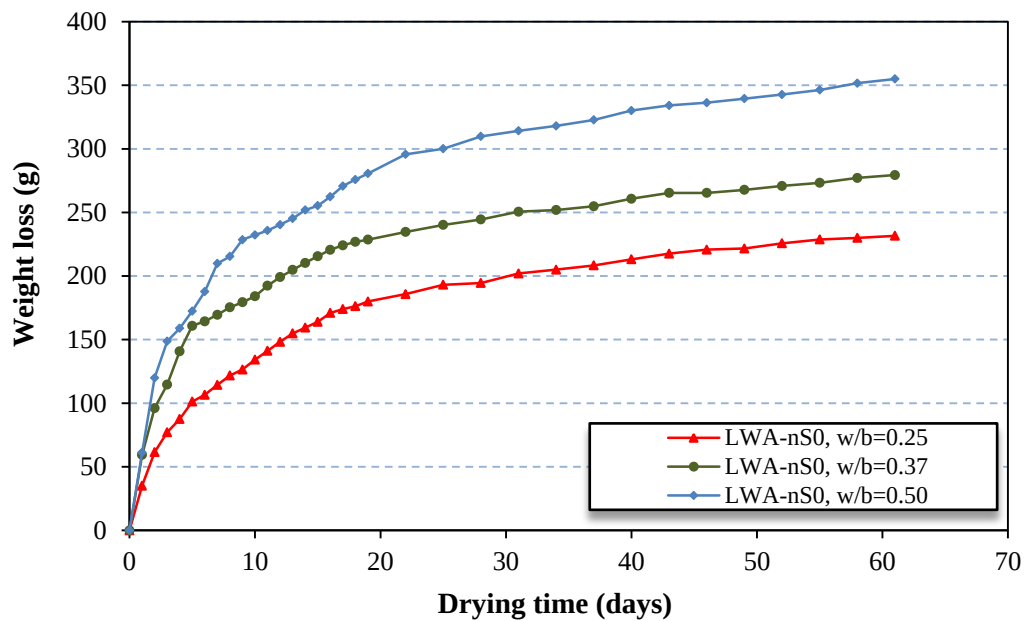


Figure 4.20 Weight loss of untreated LWA included SCLC without nS

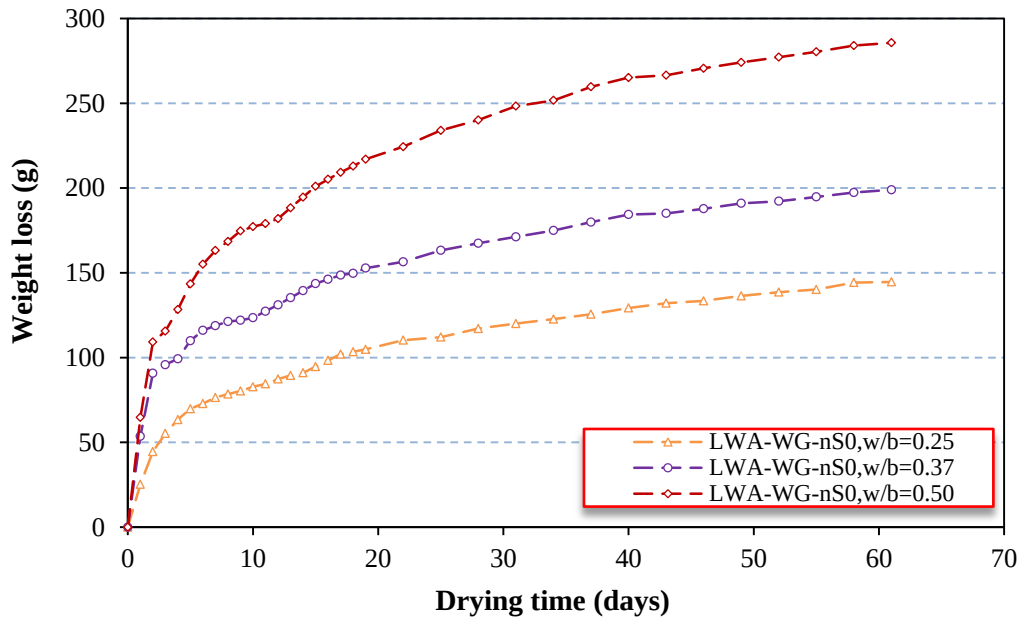


Figure 4.21 Weight loss of treated LWA included SCLC without nS

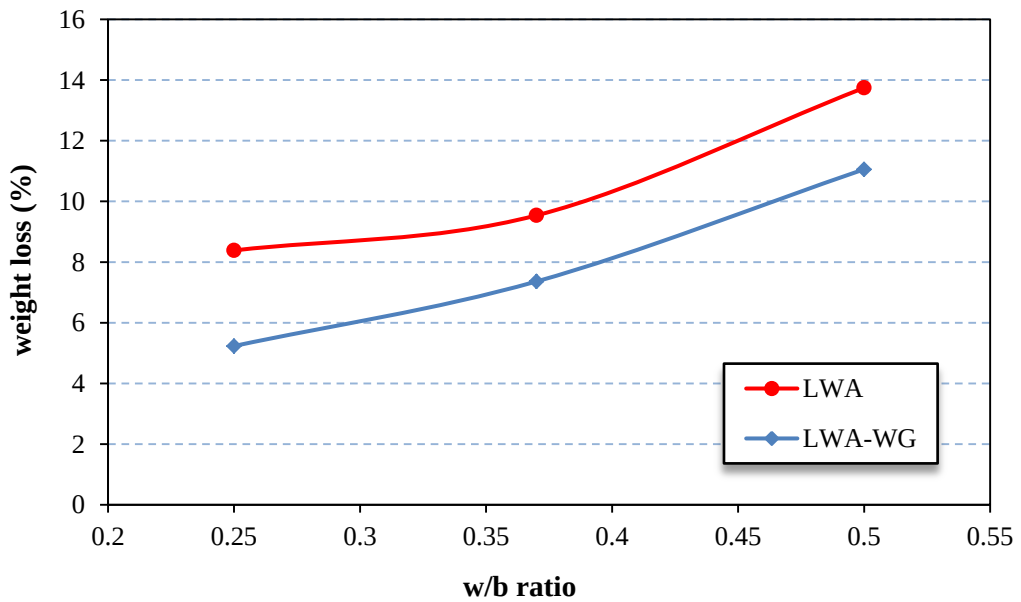


Figure 4.22 Variations in weight loss at the end of 61 days drying period

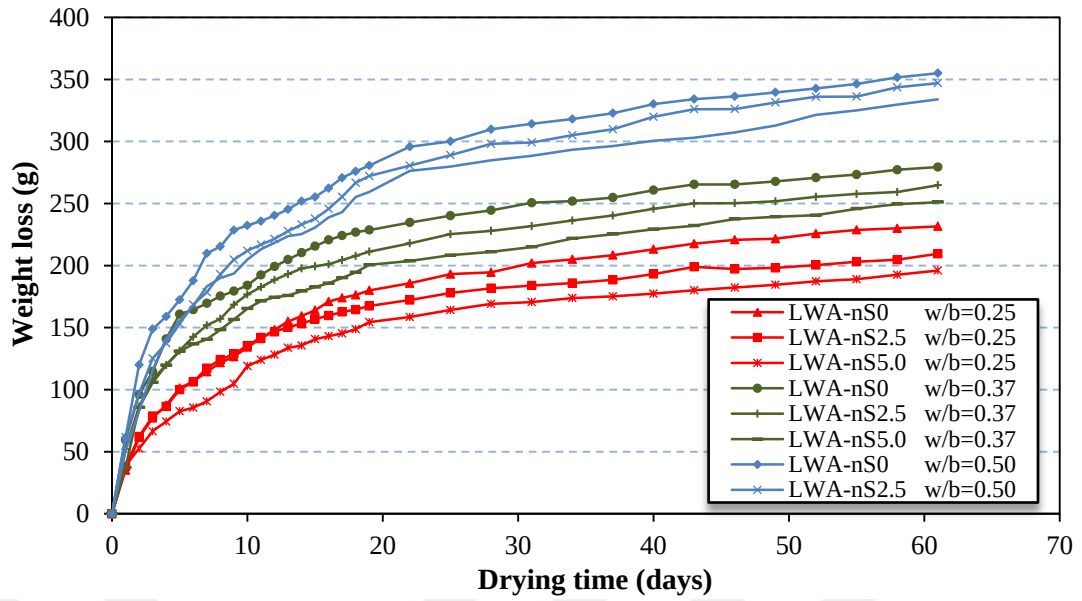


Figure 4.23 Weight loss of SCLC with natural LWA having nS

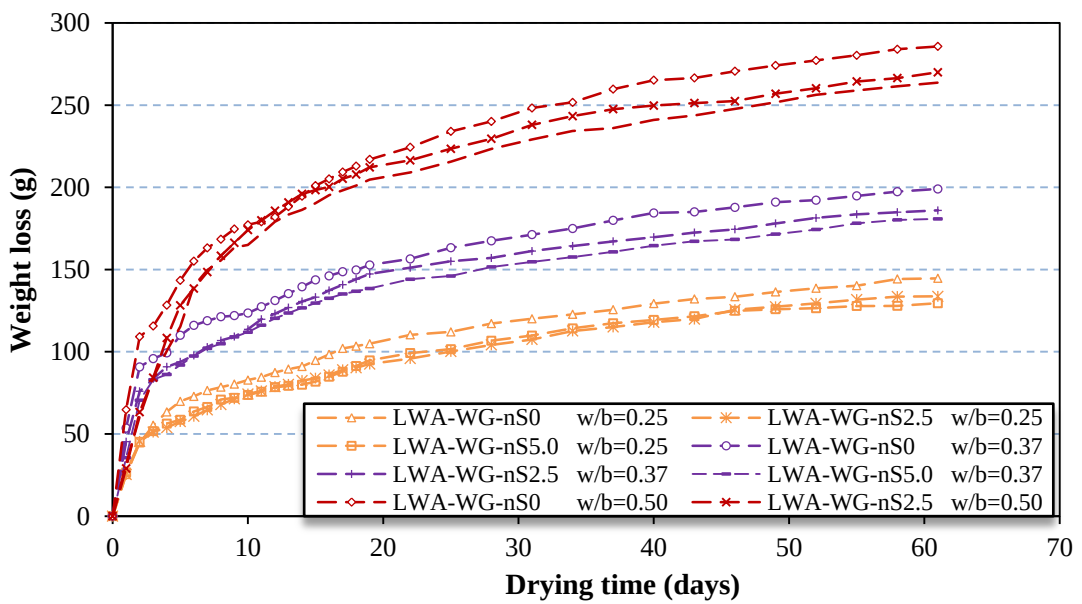


Figure 4.24 Weight loss of SCLC with treated LWA having nS

The sorptivity coefficients of various SCLC mixtures at 28 and 90 days are graphically presented in Figures 4.25 and 4.26. As expected, the w/b ratios and age of samples affected the results of sorptivity coefficients of SCLC as these two factors affect the pore structure intensification, where the reduction of w/b ratio from 0.50 to 0.25 caused a decrease in the sorptivity coefficient from 0.295 to 0.134 mm/min^{1/2} and 0.26 to 0.105 mm/min^{1/2} for LWA-nS0; and 0.244 to 0.091 mm/min^{1/2} and 0.163 to 0.066 mm/min^{1/2} for LWA-WG-nS0, at 28 and 90 days respectively. SCLC containing untreated LWA revealed an increase in sorptivity coefficients when compared to samples containing treated aggregates. The increments in the percentages of sorptivity coefficient at 28 and 90 days were about 54%-90%, 31%-54% and 3%-29% for low, moderate and high w/b ratio, respectively. These increments may be related to the higher porosity of untreated LWA which leads to lower matrix quality by affecting the capillarity of concrete. Similar findings can be found in the literature for concrete incorporated LWA (Topçu and Uygunoğlu, 2010; Güneyisi et al., 2013; Öz, 2014).

All coefficients of sorptivity measured for mixtures implemented with nS were less than those of mixtures without nS and the higher level of nS tends to be more efficient to lessen sorptivity than lower one. The sorptivity index results of the samples including nS were about 2.7% to 35% and 5% to 37% lower than that of the samples containing cement and fly ash only at 28 and 90 days, respectively, depending on w/b ratio, coarse aggregate situation and level of nS. The lower sorptivity values indicate a finer pore structure that prevents the penetration of aggressive solution into the pore structure. According to Ghafari et al. (2014), the use of nS in cement paste produced an increase in hydration products due to the pozzolanic reaction activity of nS which created a lower number of continuous capillary pores resulting in lower water absorption.

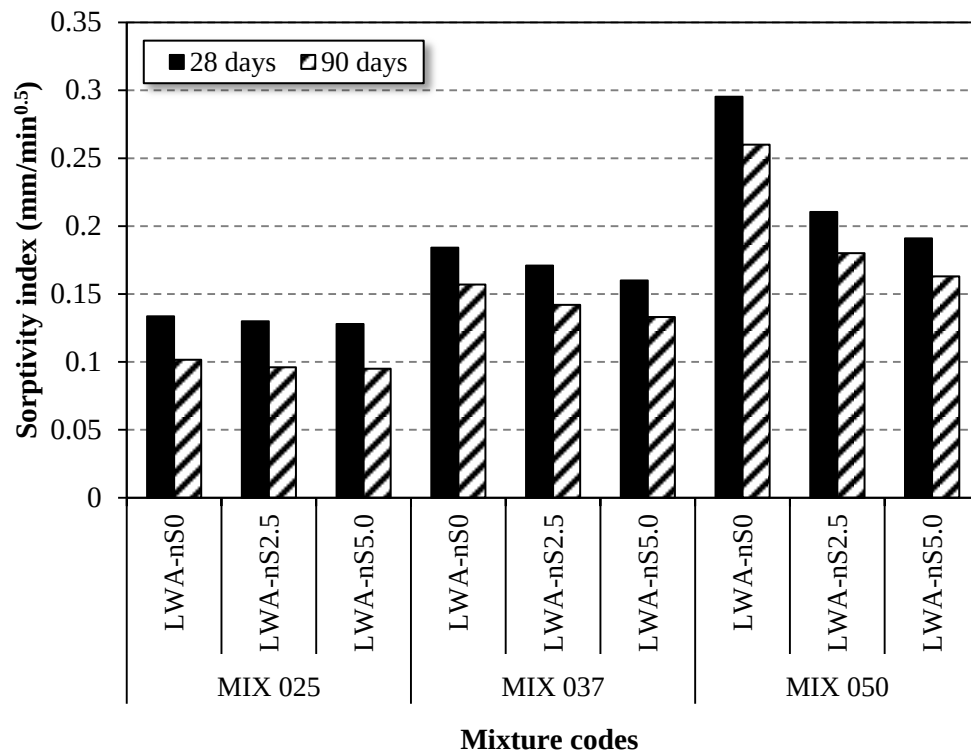


Figure 4.25 Variation of water sorptivity index of SCLC incorporated untreated LWA

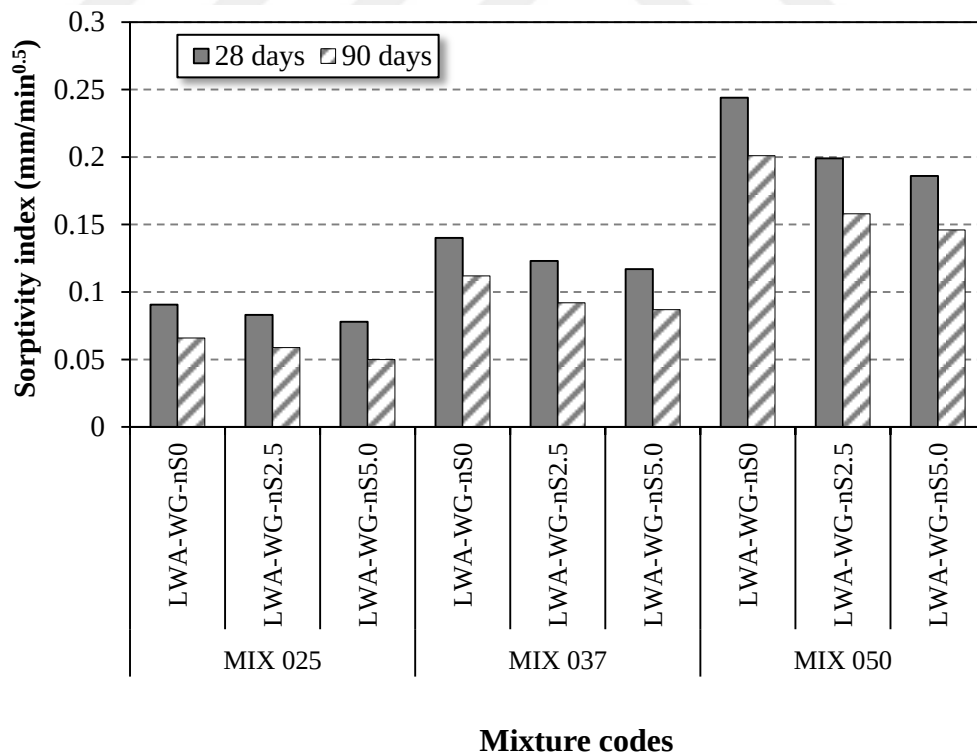


Figure 4.26 Variation of water sorptivity index of SCLC incorporated treated LWA

There are many methods typically used to measure pore structure; the gas permeability method can be considered one of the most important methods to investigate the porous material and characterize pore structure (Schutter et al., 2008). The effects of w/b ratios and the incorporation of hydrophilic fumed nS at 28 and 90 days on concretes casted with untreated aggregates and water glass surface treated aggregates are presented in Figures 4.27 and 4.28, respectively. Minimum gas permeability was detected at LWA-WG-nS5.0% at w/b ratio of 0.25 as $0.8 \times 10^{-16} \text{ m}^2$, while the maximum was observed at LWA-nS0% as $17.5 \times 10^{-16} \text{ m}^2$ with w/b ratio of 0.50. In the study of Güneyisi et al., (2013), it was reported that gas permeability of concrete depends on the quality of the matrix, as well as on porosity of the aggregates. It can be clearly seen that there is a remarkable decrease in the gas permeability coefficient due to the use of treated instead of untreated LWAs. For instance, the coefficients were observed as $2.46 \times 10^{-16} \text{ m}^2$ and $1.49 \times 10^{-16} \text{ m}^2$ for LWA-nS0% and LWA-WG-nS0%, respectively, at w/b ratio of 0.25.

In comparison to the mixture without nS, the incorporation of 2.5% and 5.0% nS caused a reduction in gas permeability of SCLCs implemented with natural LWA by 12% and 13%; 21% and 28%; and, 34% and 50% at 28 days, whereas at 90 days the percent of decrement increased slightly to 19% and 17%; 27% and 33%; and, 39% and 54% for low, moderate and high w/b ratios, respectively. The results also showed that the gas permeability coefficient of water glass treated aggregate concrete samples was once again lowered by the addition of nano particles. Critical observation of Figures 4.27 and 4.28 reveals that when considering the same aggregate types, an increase in nS content to 5% lowers the gas permeability coefficient of SCLCs with respect to the lower value used in this experimental study. Moreover, this behavior tends to increase as much with the increase of w/b ratio. This evidence indicates that the use of high nS levels are needed in the mixing water in order to increase the proofing efficiency of nano particles. The results of the gas permeability test clearly showed that the presence of nS is beneficial in improving the packing and the refined pore structure and microstructure of SCLCs. It was reported that the higher the Ca(OH)_2 content in the hydrated matrix, the higher the volume of continuous pores (Li and Ding 2003). The pozzolanic reaction caused a decrease in the volume of continuous capillary pores owing to the reduction in Ca(OH)_2 content in the hydrated matrix.

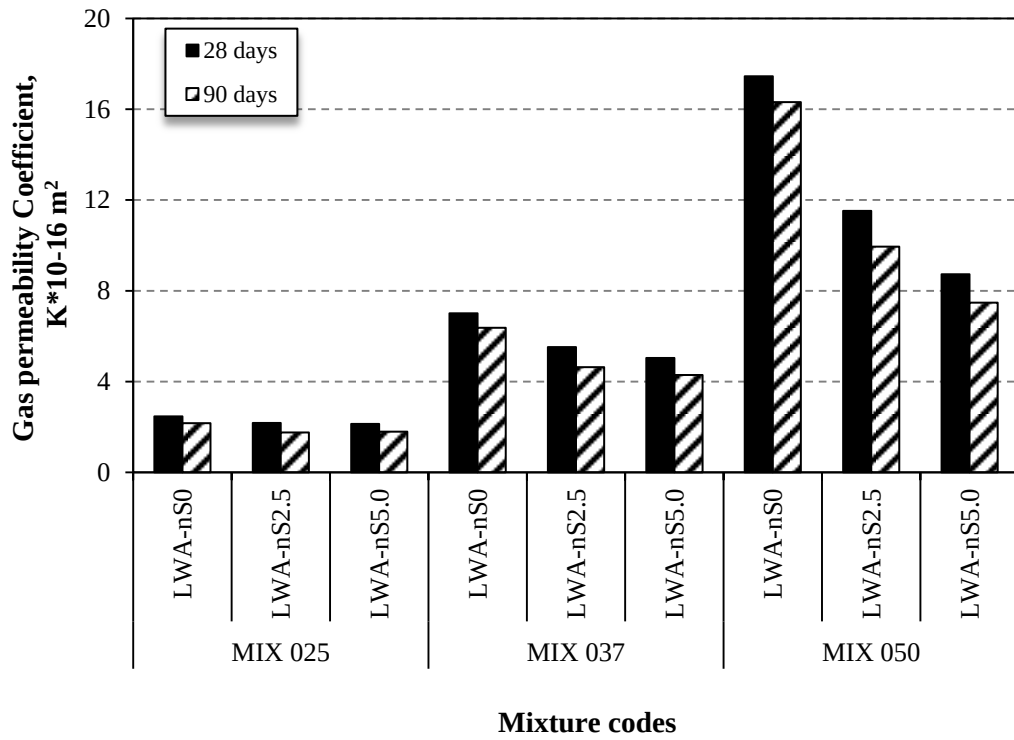


Figure 4.27 Variation of gas permeability coefficient of SCLC incorporated untreated LWA

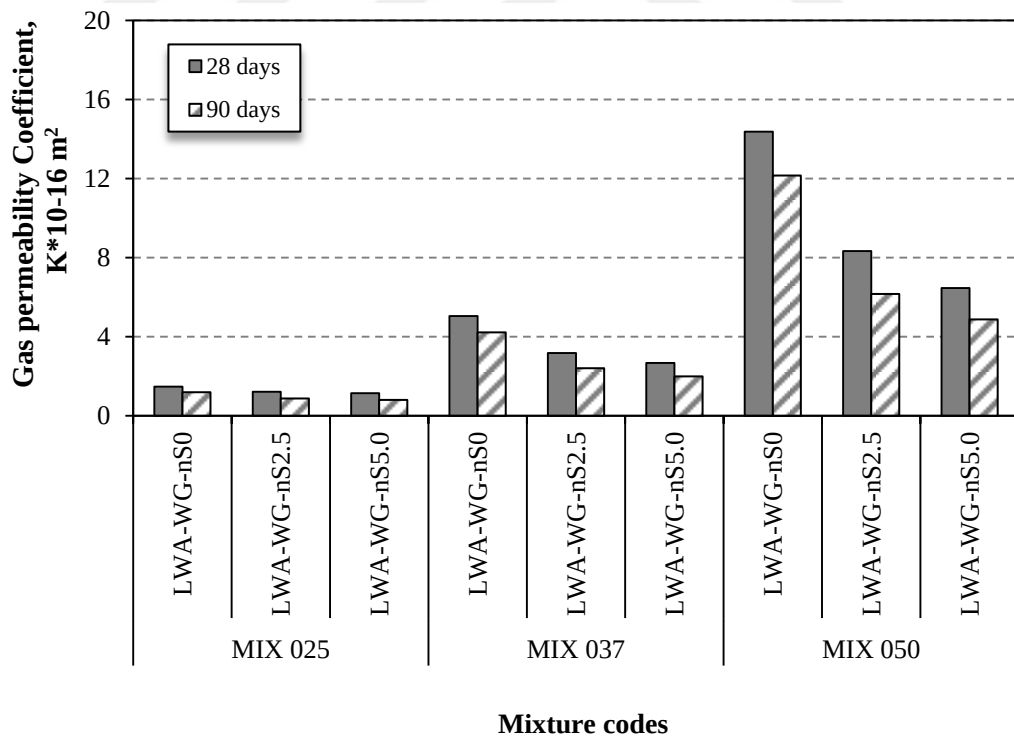


Figure 4.28 Variation of gas permeability coefficient of SCLC incorporated treated LWA

Higher water permeability depth usual denoted lower durability for concrete materials (Ahmad et al., 2014). Figures 4.29, and 4.30 illustrate the water penetration depth at 28 and 90 days of curing of the cubes prepared using the developed SCLC mixes. As expected, the rates of water penetration of SCLC at 90 days are lower than that of 28 days. Also from Figures 4.29 and 4.30, it is clear that the depth of water penetration increases systematically with the increase of w/b ratio from 0.25 to 0.50, and there is a tendency of reduction when treated aggregates are used instead of untreated. The water permeability values for the LWA-nS0 mixes were 12.3% and 9.7%; 24.3% and 19 %; and, 43.3% and 35.3% at 28 and 90 days for low, moderate and high w/b ratio, respectively. However, in the case of using treated aggregates, the water permeability results seem beneficial even with the higher w/b ratio used in this research. The SCLC mixture, namely LWA-WG-nS2.5, had a water permeability rate lower than that of LWA-nS2.5 by 3.7 times at 90 days, with a lower water mixing ratio of 0.25, while the LWA-WG-nS0 mixture at a w/b ratio of 0.37 reported 1.2 times less than LWA-nS0 at curing of 28 days. This behavior is attributed to the nature of unprotected aggregate where the porosity of its structure makes deeper water penetration easy into SCLCs that include such aggregates (Liu et al., 2011). This finding agrees with some research observed that many types of synthetic LWA in concrete cause higher water penetration (Güneyisi et al., 2014c).

Since the transport properties of concrete strongly depend on its pore structure, it can be stated that the use of mineral admixtures affect the mechanism positively. As a result, the enhancement of the cement paste filled the voids through aggregates and this affected the interfacial transition zone (Güneyisi et al., 2015b). In this study we observed that the samples made with nS had a water permeability ranging from 1.7% to 9.3%, 8.3% to 19.3%, and from 10% to 22.3% for 0.25, 0.37 and 0.50 w/b ratio, respectively. It was shown that by Esmaeili and Andalibi (2013), the application of nS particles to concrete samples can cause a systematic reduction in water permeability of the concrete. The drop in the water permeability reached up to 53% with the replacement level of 9% nS. The plotted results also illuminate the incorporation of 5% nS behind the lowest water permeability rate of 2.5% nS. For example at w/b ratio of 0.50, the water permeability of LWA-WG-nS2.5% was 18.0%, while for LWA-WG-nS5.0%, the value decreased to 13.3% at 28 days. Therefore, it can be concluded that the use of nS as mineral admixtures provides a

significant drop in the water permeability of SCLC when compared to that of the SCLC made with cement and fly ash only. Figure 4.31 shows water penetration of six samples tested at 28 days.

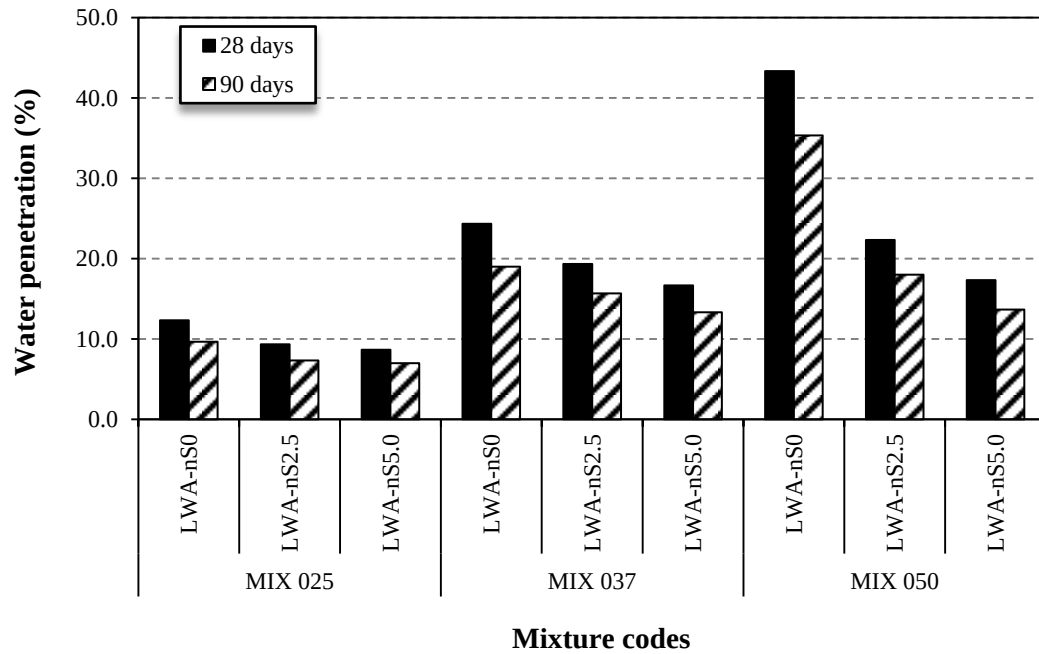


Figure 4.29 Variation in water permeability of SCLC produced by using untreated LWA

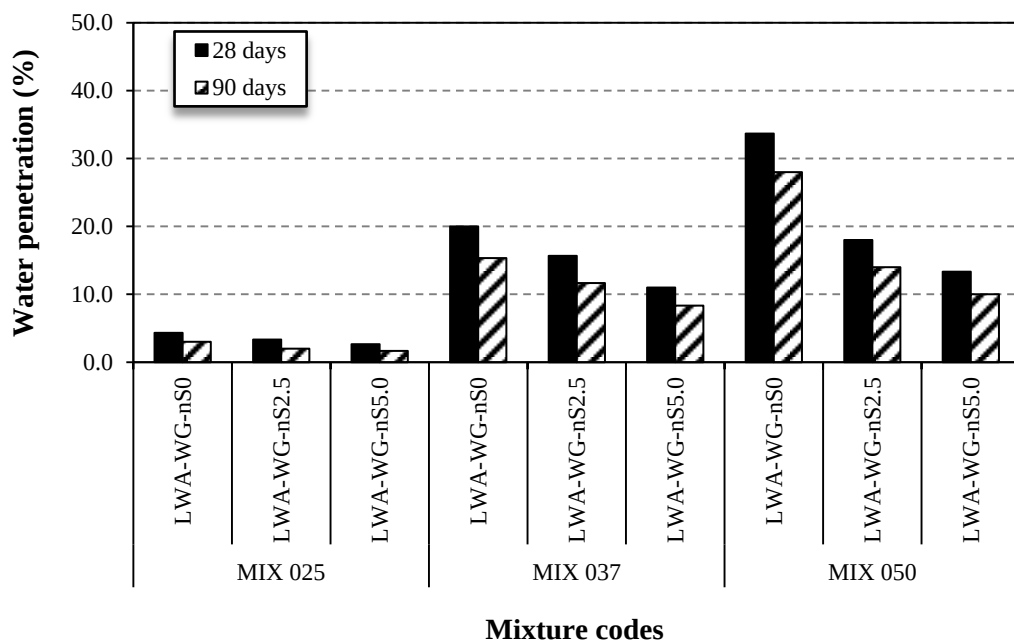
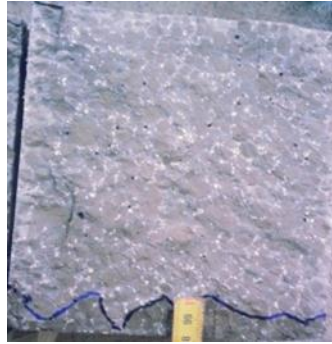
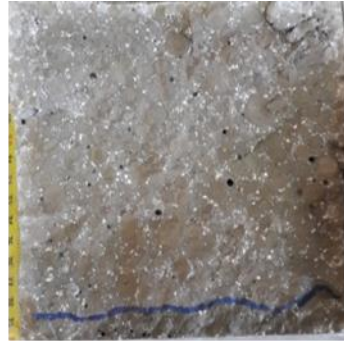


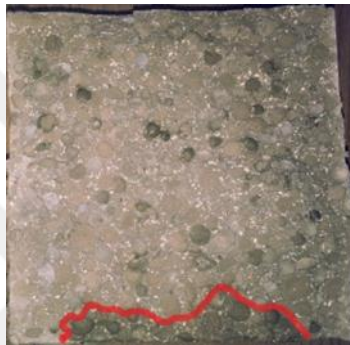
Figure 4.30 Variation in water permeability of SCLCs SCLC produced by using treated LWA



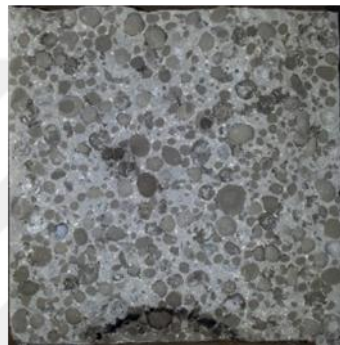
LWA-nS2.5
w/b= 0.25



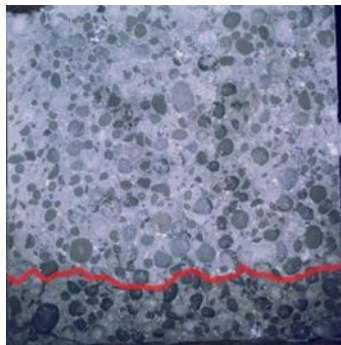
LWA-nS5.0
w/b= 0.25



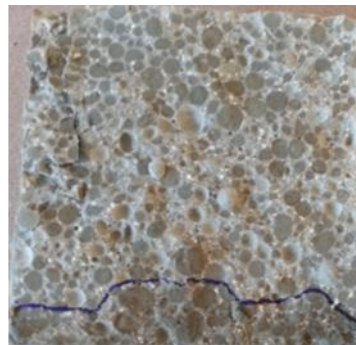
LWA-WG-nS2.5
w/b= 0.37



LWA-WG-nS5.0
w/b= 0.37



LWA-nS2.5
w/b= 0.50



LWA-nS5.0
w/b= 0.50

Figure 4.31 Water penetration through samples of SCLC at 28 days

4.3 Mechanical properties of SCLC

The results relevant to the compressive strength of SCLC at 28 and 90 days are graphically presented in Figure 4.32 and 4.33. It can be seen that the increase in the ratio of w/b was remarkably decreased the compressive strength of all SCLC types at both 28 and 90 days. For instance, the compressive strength at 28 days of LWA-nS0 varied from 55.06 to 30.57 MPa as the w/b ratio of 0.25 increased to 0.50. In the literatures, it has been shown that several factors can influence the mechanical properties of cementitious materials; such as porosity, pore size distribution, microcracks, interface, etc (Rahai et al., 2015). On the other hand, it was stated that the compressive strength of concrete is governed by the porosity within the cement paste and interfacial transition zone. In this regard, increasing water to cement ratio develop porous microstructure, which will reduce the concrete strength (Tanaka and Kurumisawa, 2002; Elsharief et al., 2003). Fernandes et al. (2005) and Nikbin et al. (2014) also reported decreases trend similar to the trend of the present study. In addition, at high water content condition the crystallization of harmful crystals (calcium hydroxide and ettringite) will be higher than low water content as the C-S-H gel in the later higher than in the former. At the same line, increasing w/b ratio reduces gel/space ratio, consequently, the strength of concrete detracts. Finally, with high binder content a stronger and denser cement paste and interfacial transition zone can be developed by creating more C-S-H gel (Neville, 1996).

The treated LWA containing SCLC caused an increase in the compressive strength values when compared with those implemented with untreated surface LWA, regardless of w/b ratio. Irrespective of nS addition, the percent increase in the strength with the treated aggregate ranged from 17% to 32% and 31% to 33% at 28 and 90 days, respectively. The same trend also detected in the case of concretes incorporating nS. The reason behind this behavior is the enhancement of synthetic FA coarse aggregate properties through the water glass treatment which leads to the development of denser and harder aggregates. Furthermore, the lowest water absorption of water glass treated aggregates utilized in saturated surface dry conditions would not occasion a rise in the unit water cement ratio of the matrix (Gesoglu, 2004).

The effect of nS on compressive strength of SCLC is well observed in Figures 4.32 and 33. These figures indicate that there was an increase in compressive strength with the increase in nS content. According to the experimental values, the compressive strength of samples incorporating nS are 4-36% and 5-40% higher than those free of nS at 28 and 90 days, respectively. It can also be seen that the effectiveness of nS to improve compressive strength has a relation to the parameters of w/b ratio, age of curing, and the replacement level of nS. As it is observed in Figures 4.32 and 4.33, nS more effective to improve compressive strength in higher w/b ratio (lower cement contents) (Hosseini et al., 2011). However, the enhanced intensity of SCLC compressive strengths decreases with the increasing content of nS up to 5.0%. As discussed earlier, the mixes that included binary blends and made with untreated aggregates exhibited lower values of compressive strength regardless of w/b ratio. The addition of nS remedies the compressive strength loss resulted from using inferior quality of LWA. For instance, the compressive strength at 28 days for LWA-nS0 with respect to a w/b ratio of 0.25 was 55.06 MPa has increased to 61.68 and 64.83 MPa with the addition of 2.5% and 5.0% nS, respectively, as can be seen in Figures 4.32 and 4.33.

In other related studies with different nS replacement levels compressive strengths are also exhibited similar trend in contribution to the compressive strength of the concretes. Qing et al. (2007), Hou et al. (2013), Li et al. (2004), Nazari and Riahi (2011b), and Nik and Orman (2013) found that the replacement of cement with 5.0% nS can significantly improve compressive strength. At the same time, other previous works stated dissimilar behavior regarding this replacement level (Naji et al., 2010; Nazari and Riahi, 2011c; Najigivi et al., 2013). These contradictions witnesses rather than compressive strength test must be considered to verify the optimal level of nS.

To discuss the effectiveness of nS on concrete strength, deep description concerning the modification of cement based mixtures incorporating nS must be presented. nS particles are pozzolan and its pozzolanic reaction can be divided into physical and chemical aspects. Unlike conventional pozzolanic materials, nS particles have a high surface area to volume ratio. Based on the physical effect, infinitesimal particles increase the reduction extent in specific pore volumes (filling effect of nS) (Jalal et al., 2013). Filling effect can also lead to converting the most pore probable diameters in cement paste and transition zone to smaller pores. Thus, the refinement of pore

structure will lessen porosity and creates a dense system; hence increasing strength. In addition, SiO₂ nanoparticles possessing an ability to accelerate C-S-H generation (accelerate cement hydration) due to their high activity and tightly bond with its particles by acting as a nucleus, as a consequence, enhancing concrete strength (Hosseini et al., 2010; Nazari and Riahi 2011b; Najigivi et al., 2013).

The chemical effect representing by the ultra-high pozzolanic activity is the favorable reactions of nS in cementitious environments. Generally, the cement hydration products involved a harmful Ca(OH)₂ crystals which are arrayed in the interfacial transition zone among aggregates and cement paste matrix and can reduce the compressive strength. The presence of the desired amount of nS increases compressive strength of concrete due to the ultra pozzolanic activity of nS which can react with Ca⁺² liberated from Ca(OH)₂ and produce C-S-H gel (Salemi and Behfarnia, 2013; Oertel et al., 2014; Khaloo et al., 2016). The superior performance of nS does not stop at this level, but will expand to be the reason for restricting the growth of unfavorable crystals; because of the existence of large numbers of nanoparticles between hydration products and their confinement role (Li et al., 2006). As a result, the pozzolanic action and filler effect of nS particles develop a high quality matrix and interfacial transition zone of SCLC, and these events strengthen the concretes.

Moreover, Figure 4.34 (a-d) illustrates the relative compressive strength of cubical concrete specimens versus nS contents at 28 and 90 days. The relative compressive strength is calculated by dividing the compressive strength of each nS incorporated concrete by the compressive strength of the specimens free of nS at the same age. The calculated values indicated that the highest percentage strength improvement due to the nS incorporation was about 40% for LWA-WG-nS5.0 at 0.50 w/b ratio for 90 days of curing. Although the use of nS increased the compressive strength of SCLCs at different w/b ratios, a clear distinction can be observed between concrete serious made with untreated lightweight aggregate and those with treated lightweight aggregate, Figure 4.34 (a-d). That is to say for SCLC made with untreated LWA, the higher rate of compressive strength increment was observed at a w/b ratio of 0.37, whilst the lower rate occurred at 0.25 w/b ratio. This behavior appeared to be contradicted by replacing untreated with treated aggregates.

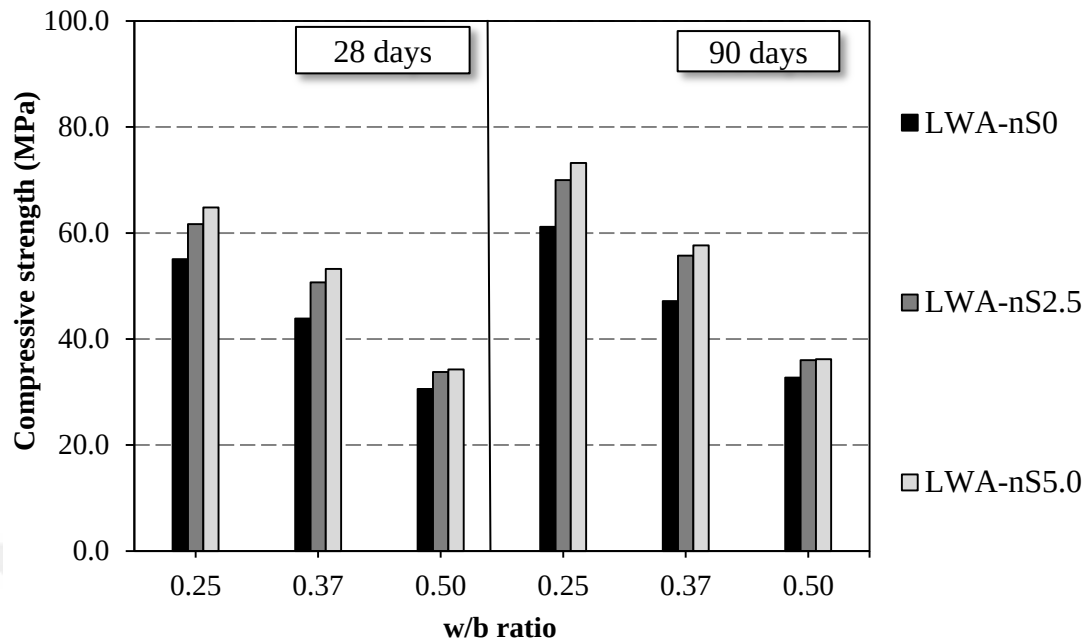


Figure 4.32 Compressive strength values for untreated LWA cases

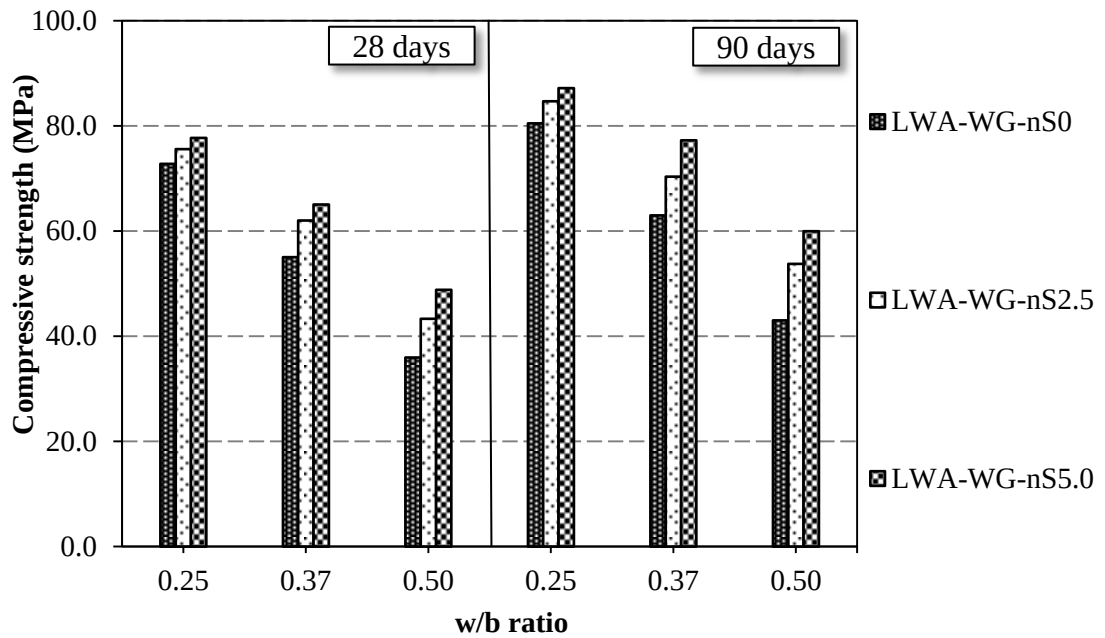
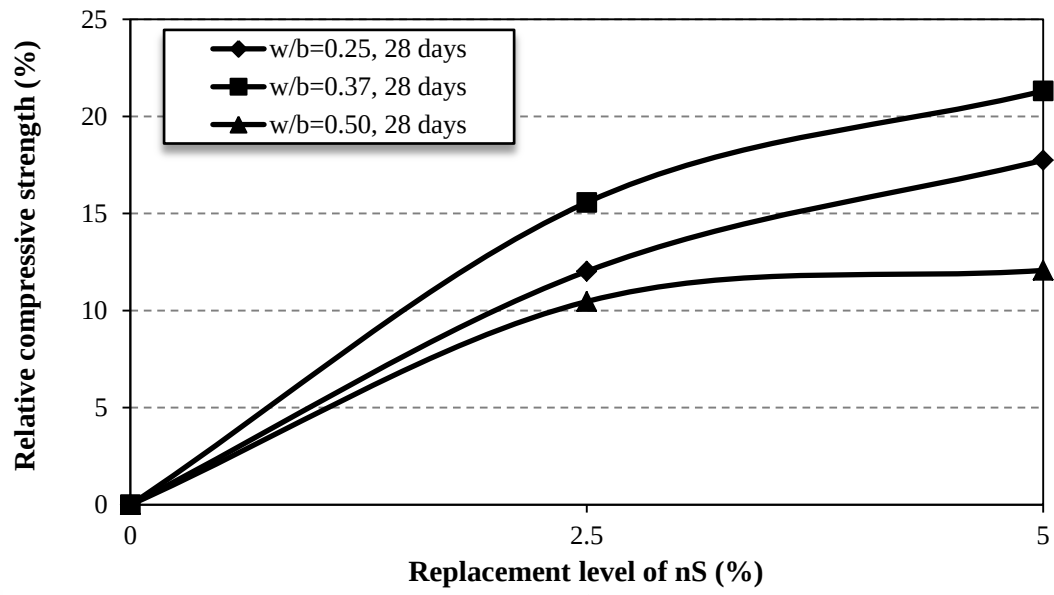
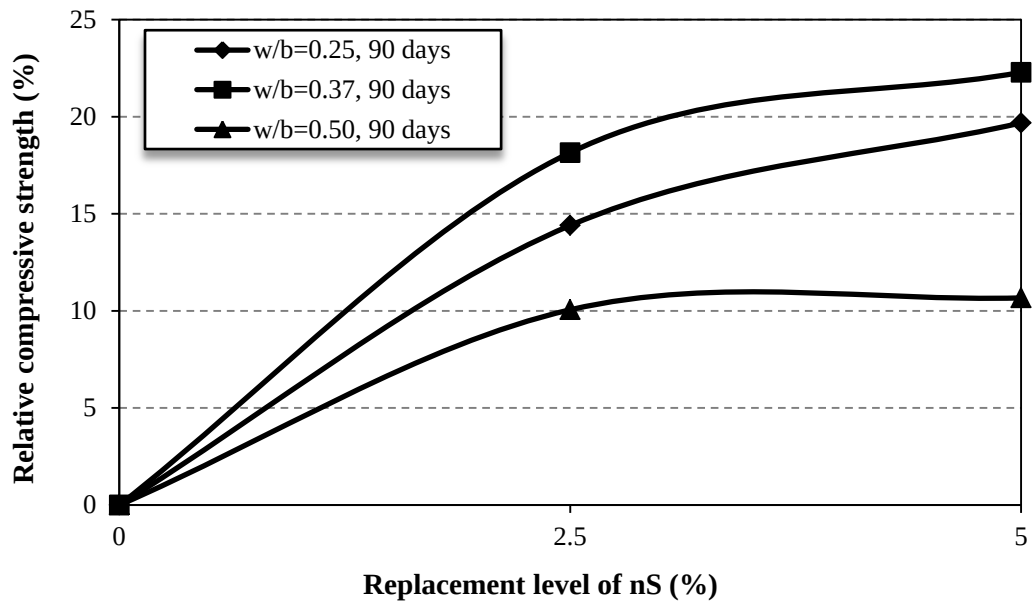


Figure 4.33 Compressive strength values for untreated LWA cases



a)



b)

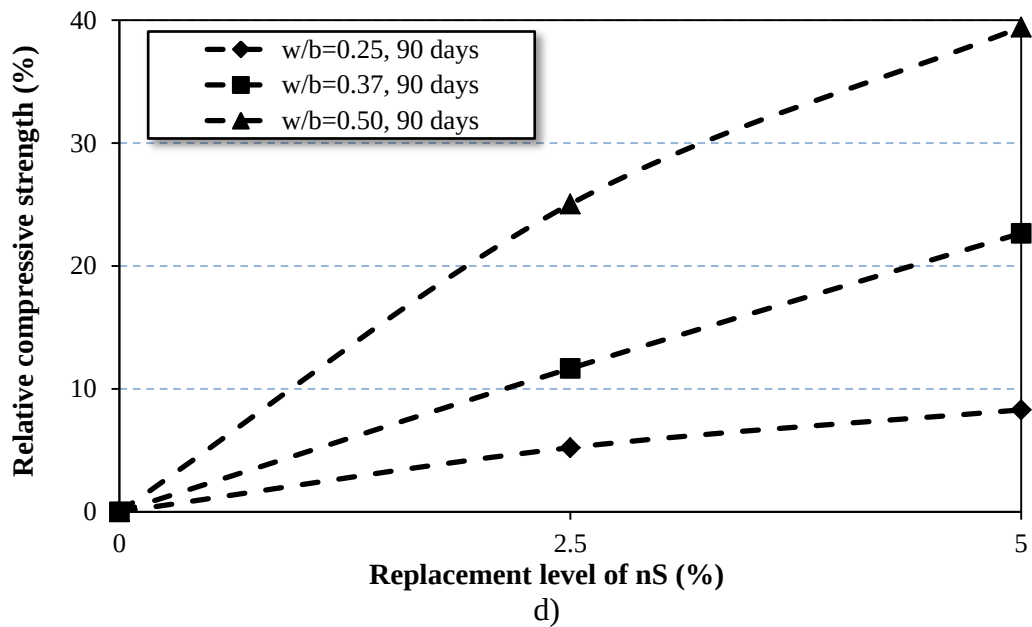
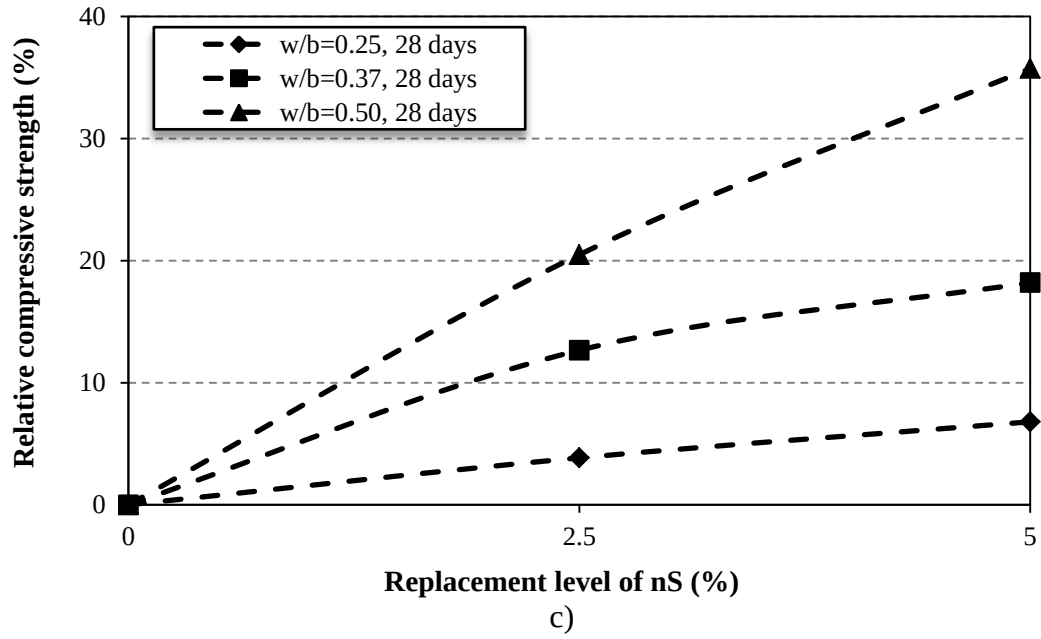


Figure 4.34 Influence of nS replacement level on compressive strength of SCLC made with: a untreated LWA at 28 days, b untreated LWA at 90 days, c treated LWA at 28 days, and d treated LWA at 90 days

The splitting tensile strength values of the SCLC cylinders tested at 28 and 90 days are presented in Figures 4.35 and 4.36.

Considering the lower, moderate, and higher w/b ratio for each series it was indicated that the splitting tensile strength decreases as the water to binder ratio increased. This trend is noticeable for all types of concretes at all ages, for example, 28 and 90 days splitting tensile strength values for LWA-nS5.0 were decreased from 2.27 and 3.19 to 2.00 and 2.63 MPa as the 0.37 w/b ratio increased to 0.50, respectively.

Tommy et al. (2007) observed that the number of pores in the hardened cement paste increases with increasing w/b ratio because as the water consumed by hydration, it leaves a coarse air void in the cement paste, consequently, strength decreases. The influence of increased water to binder ratio was observed to be higher on the splitting tensile strength of concretes free of nS. A type of this trend can clearly be observed with LWA-WG-nS0 samples tested at 90 days, where the increment of w/b ratio from 0.37 to 0.50 caused a drop in the splitting tensile values of 20.4% whereas it was 10.7% for LWA-WG-nS5.0 specimens. This observation may be due to the fact that the interfacial transition zone considered as a crucial factor in influencing concrete tensile strength (Quercia et al., 2014).

As Figure 4.36 presents, the splitting tensile strength development in SCLCs with treated LWAs revealed higher values compared to SCLCs made with natural surface lightweight aggregates. These strength improvements are approximately in the range of 21-33%, 24-32% and 21-33% at 28 days of curing, on the other hand, at 90 days had also comparable enhancement extent to the former as the percentage of splitting strength improvement by 14-22%, 17-30% and 18-27% was calculated for 0.25, 0.37 and 0.50 w/b ratio, respectively. Güneyisi et al. (2015b) investigated the effect of surface treatment methods of recycled aggregate on the properties self-compacting concretes. They performed splitting tensile strength in their experimental study, with the recycled aggregate condition as the main variable. They concluded that, all samples containing water glass treated aggregate revealed significant development in splitting strength values at both 28 and 90 days of curing.

Figures 4.35 and 4.36 also present the variation of splitting tensile strengths of SCLC depending on the contents of nS. It was detected that there was a systematic increase

in splitting tensile strength as the nS percentage increased from zero percent up to 5.0%, irrespective of w/b ratio and type of LWA. For this detection, at a given w/b ratio and coarse aggregate condition, the lowest splitting strength was observed for specimens free of nS, while the maximum value was observed for specimens of 5.0 nS. The improvement of splitting tensile strength of nS incorporated concrete was also reported in previous researches (Nazari and Riahi 2011a; Quercia et al., 2012).

In the study of Quercia et al., (2012) the mechanical and durability properties of self-compacting concrete containing precipitated silica in colloidal suspension or fumed powder nS were studied. The percentage of nS was 3.8% by weight of cement. They reported that the splitting tensile strength after 28 days of curing was enhanced by about 21.51% and 9.91% for powder and colloidal nS, respectively. The development of denser and harder C-S-H gel due to the addition of nS improves the interfacial transition zone cause to improve tensile strength of concrete (Mukharjee and Barai 2014).

The test results presented herein indicated that with the addition of nS the enhancement extent of splitting tensile strength of SCLCs produced with treated LWA was slightly less than those of SCLCs produced with untreated LWA. For instance, at w/b ratio of 0.37 the improvement of splitting tensile strength after 28 days due to the use of nS in LWA-WG-nS2.5 mixture is lower by about 6% comparing with LWA-nS2.5. This can be attributed to the untreated LWAs could promote relatively a better bond with the mortar due to its porous surface able to absorb even fine content. Thus, the test results suggested that, at a given water to binder ratio, it was the treated LWAs aggregates governing the increment in the strength of the SCLCs when used instead of untreated LWAs (Topçu and Uygunoğlu, 2010).

As presented earlier in the compressive strength results, the splitting tensile strength values exhibited similar increase behaviors for the age of 90 days compared to that of the 28 days (Figures 4.35 and 4.36). These strength increasing percentages are in the range of about 5-32% for SCLCs incorporating untreated LWAs and treated LWAs. Another evident finding from the splitting strength testing is that unlike compressive strength, the splitting tensile strength increment capacities of SCLCs containing untreated LWAs due to the increased amount of hydration products produced at the

prolonged curing age seem to be slightly higher than those tested obtained for samples containing treated LWAs.

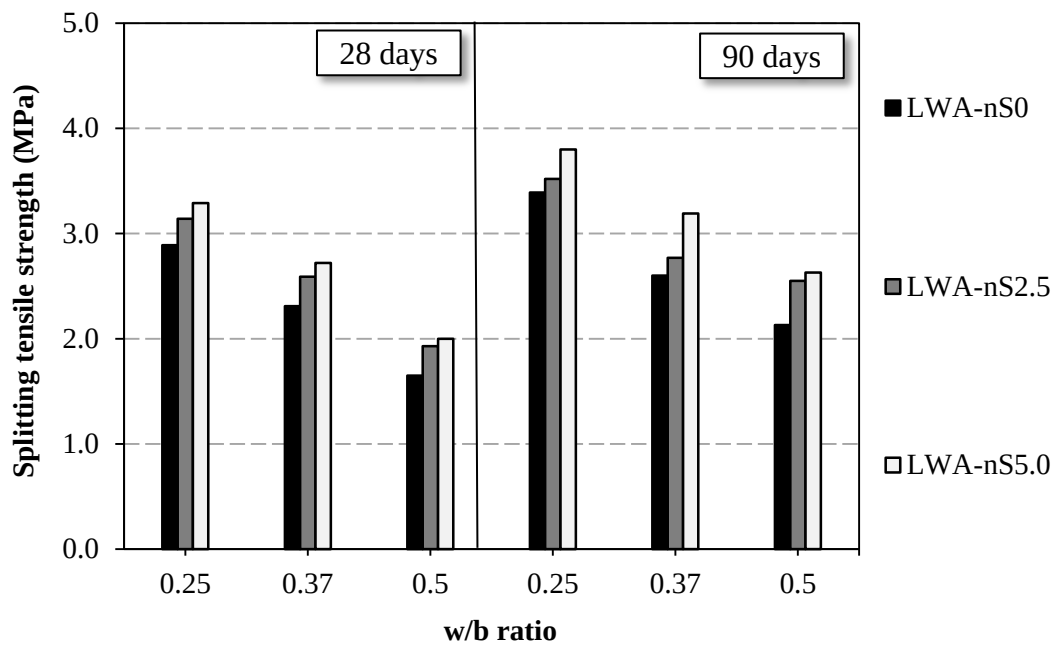


Figure 4.35 Splitting strength values for untreated LWA cases

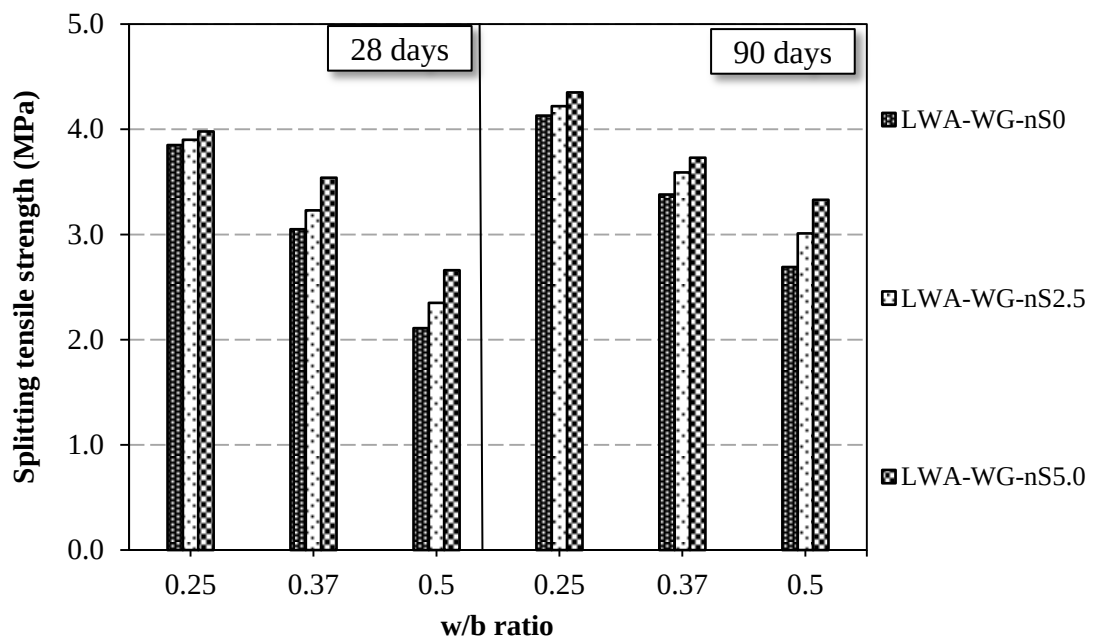


Figure 4.36 Splitting strength values for treated LWA cases

The variations in 28 and 90 days modulus of elasticity values of the SCLC are plotted within the Figures 4.37 and 4.38 taking into consideration untreated and treated LWA, respectively. The modulus of elasticity values varied between 12.64-17.82 GPa and 18.76-28.06 GPa for concrete containing untreated LWA and treated LWA, respectively. As in the case of compressive strength and splitting tensile strength, the modulus of elasticity decreased with the increase of w/b ratio. The same trend has been reported by Nikbin et al. (2014). It is clearly seen; the rigidity of LWAs significantly affected the static elastic modulus of SCLCs. The static elastic modulus of the concrete containing untreated LWA varied from 12.64-17.13 and 14.10-17.82 GPa at 28 and 90 days respectively. However, the modulus of elasticity of SCLC specimens made with treated LWA was in the range of 18.76-26.01 and 20.89-28.06 GPa at the age of 28 and 180 days, respectively. In other words, at a given age, w/b ratio, and dosage level of nS the mixture containing treated LWA has higher value of static elastic modulus than that of untreated LWA concrete. This type of behavior of the response is related to the modulus of elasticity of untreated LWAs is lower than of treated LWAs (Akçay and Tasdemir, 2009; Mukharjee and Barai 2014).

Regardless of w/b ratio and age of curing the incorporation of nS has dissimilar efficiency in enhancing modulus of elasticity depending upon type of LWA used, Figures 4.37 and 4.38. For example, at w/b ratio of 0.37, the 90 days static elastic modulus of LWA-nS2.5 and LWA-nS5.0 were approximately 3.0% and 3.4% higher than that of the concrete free of nS namely LWA-nS0, respectively. On the other hand, there is a clear trend that the positive effect of nS seemed to be higher with mixtures containing treated aggregate, for instance, at the same w/b ratio and age mentioned above the increment by about 9.0% and 14.5% were found with LWA-nS2.5 and LWA-nS5.0 when comparing with LWA-nS0, respectively. These results are in agreement with results reported by Mukharjee and Barai (2014) and Corinaldesi and Moriconi (2009). In the study of Mukharjee and Barai (2014), the effect of incorporation 0%, 0.75%, 1.50%, and 3% nS in normal and recycled coarse aggregates concrete were studied. They performed elastic modulus tests of concrete at 28 days in their experimental program. The study indicated that the addition of nS in concrete mixes containing recycled coarse aggregate and or natural aggregate slightly increases the elastic modulus, whilst, the modulus of elasticity significantly

increases with the replacement of recycled aggregates with natural aggregates. They concluded that the aggregate quality affects modulus of elasticity rather than incorporation of nS. Corinaldesi and Moriconi (2009) also observed the addition of nS in concrete mixes improves modulus of elasticity slightly due to increase in bonding between the aggregates and mortar. However, this enhancement is not so significant when compared with compressive strength, as modulus of elasticity of concrete depends upon the aggregate characteristics rather than incorporation of pozzolanic materials. To expound this issue we can refer to the inferior properties of untreated LWA and the influential zones on the mechanical properties. In general, the interfacial transition zone considered the weakest zone in concrete comparing to paste and aggregate zones because of strength of normal aggregate greater than strength of bond, while the use of such aggregate (untreated LWA) make this fact is no longer valid (Mehta and Monteiro, 2006). The particles of untreated LWA can easily crack and loads fail to transfer to matrix, thus the strengthening of interfacial transition zone and cement paste cannot contribute effectively to modulus of elasticity development of SCLCs.

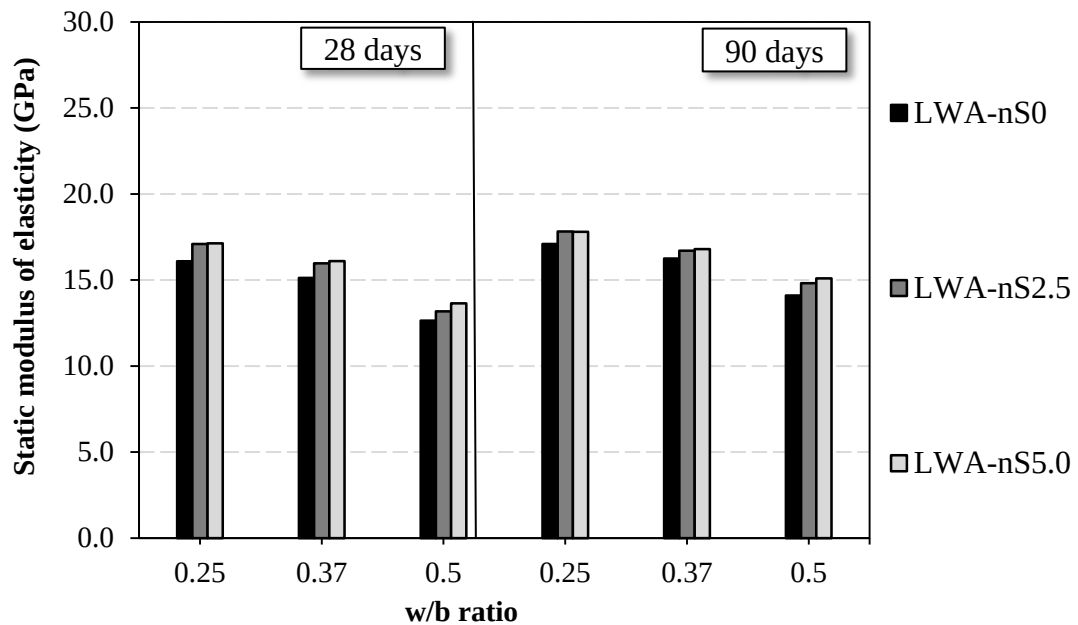


Figure 4.37 Modulus of elasticity values for untreated LWA cases

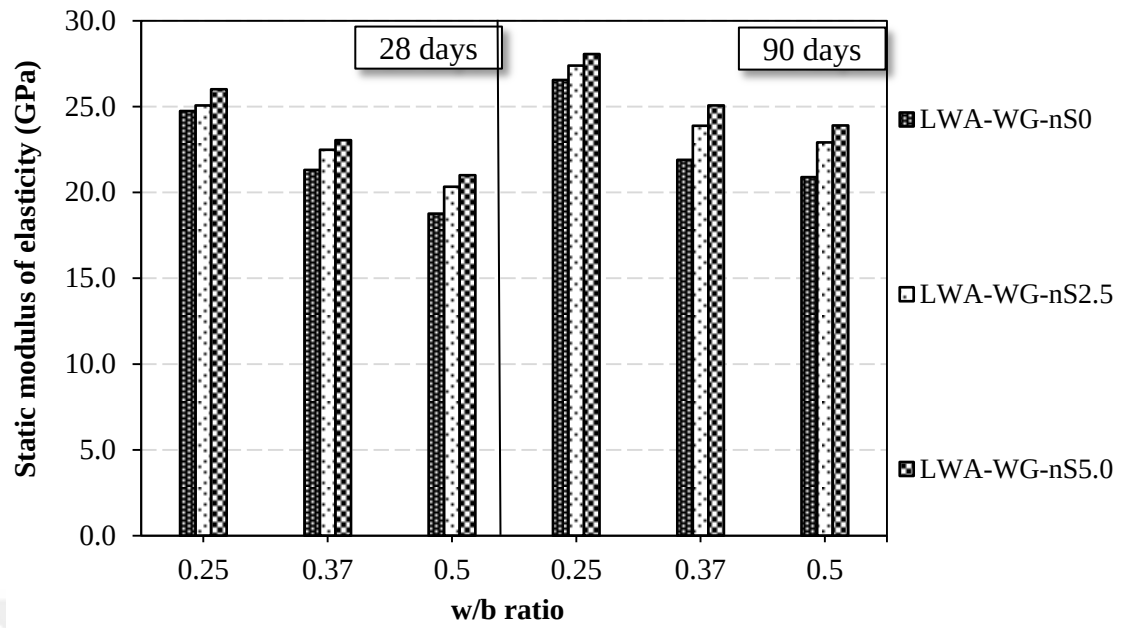


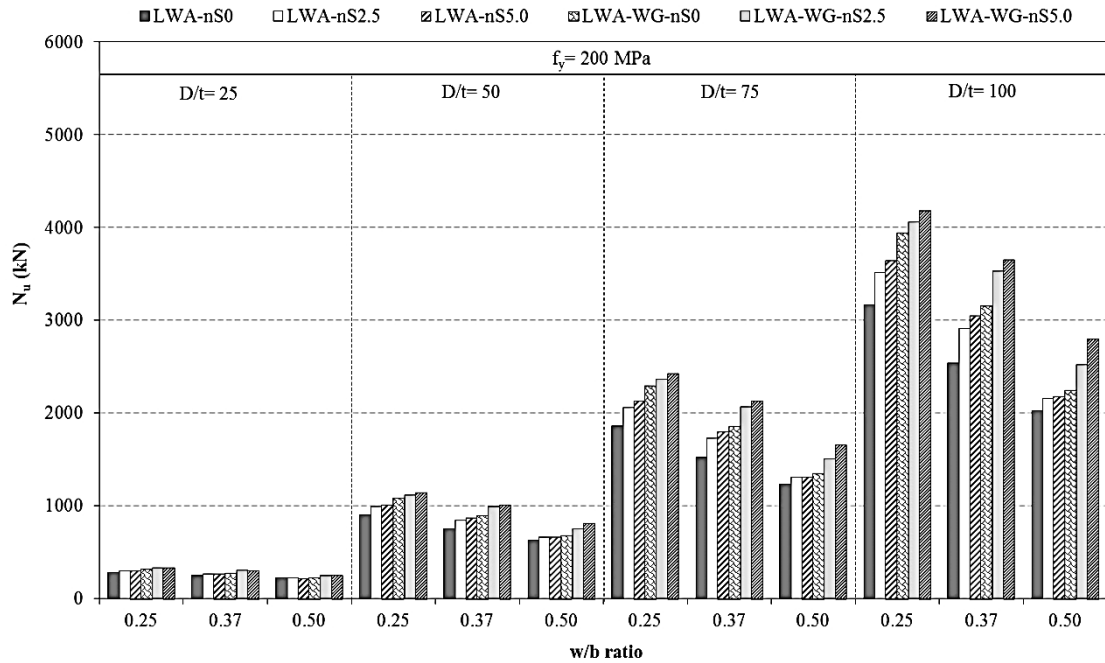
Figure 4.38 Modulus of elasticity values for untreated LWA cases

CHAPTER 5

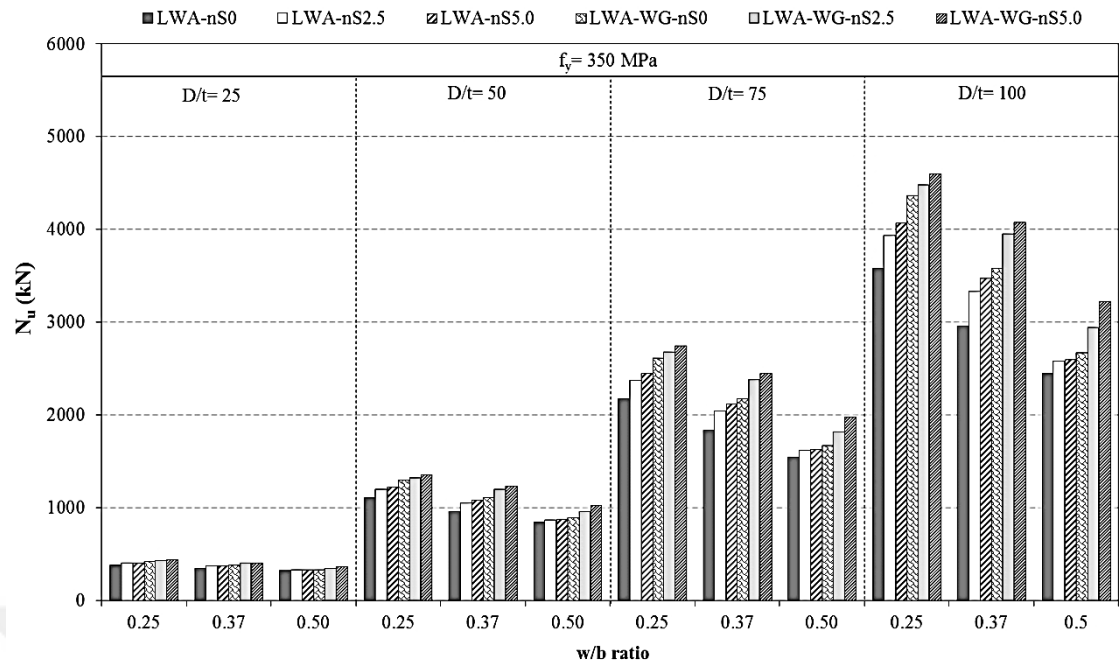
RESULTS OF NUMERICAL STUDY AND DISCUSSION

5.1 Axial capacity of lightweight concrete filled CFST columns

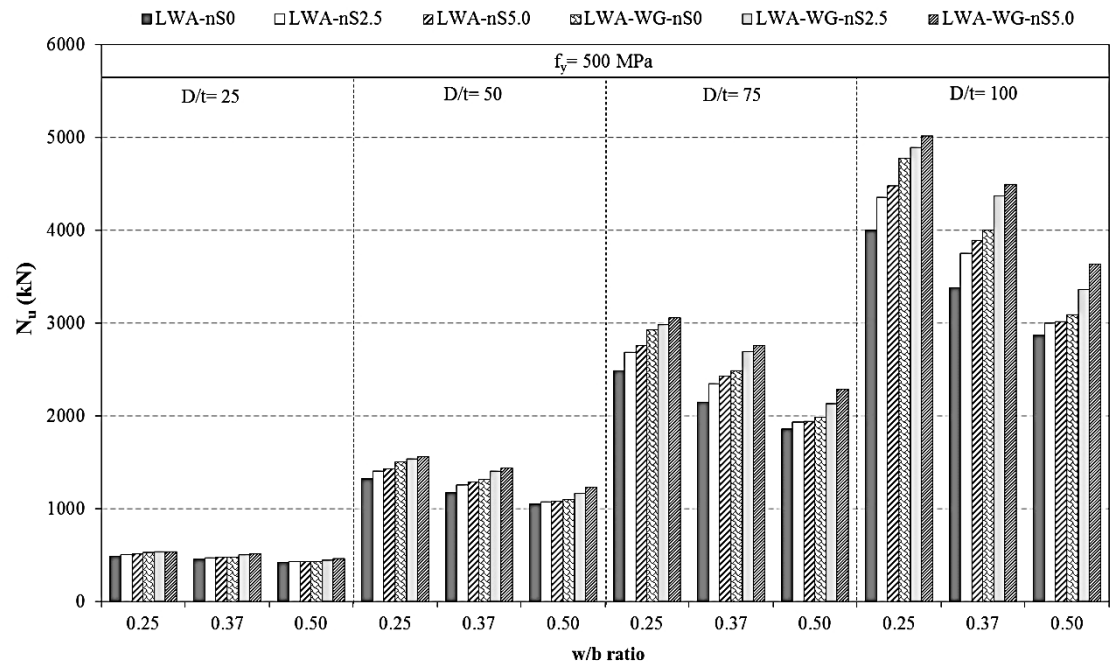
The axial capacity of lightweight concrete (N_u) of the composite columns calculated according to ACI-318R (2005), Eurocode 4 (2004), and Chinese code DL/T (1999). The details of the prediction values were given in "Appendix A" and graphically presented in Figures 5.1 to 5.3. As can be seen that the predicted N_u values were calculated for all SCLC mixtures at ages of curing of 28 and 90 days, made with untreated or treated LWA, and plain concrete or nS incorporated concrete. On the other side, various ratios of diameter to length and yield strength steel values were used. In this regard, the databases for N_u calculations were regulated by altering D/t ratio and keeping the corresponding f_y value constant.



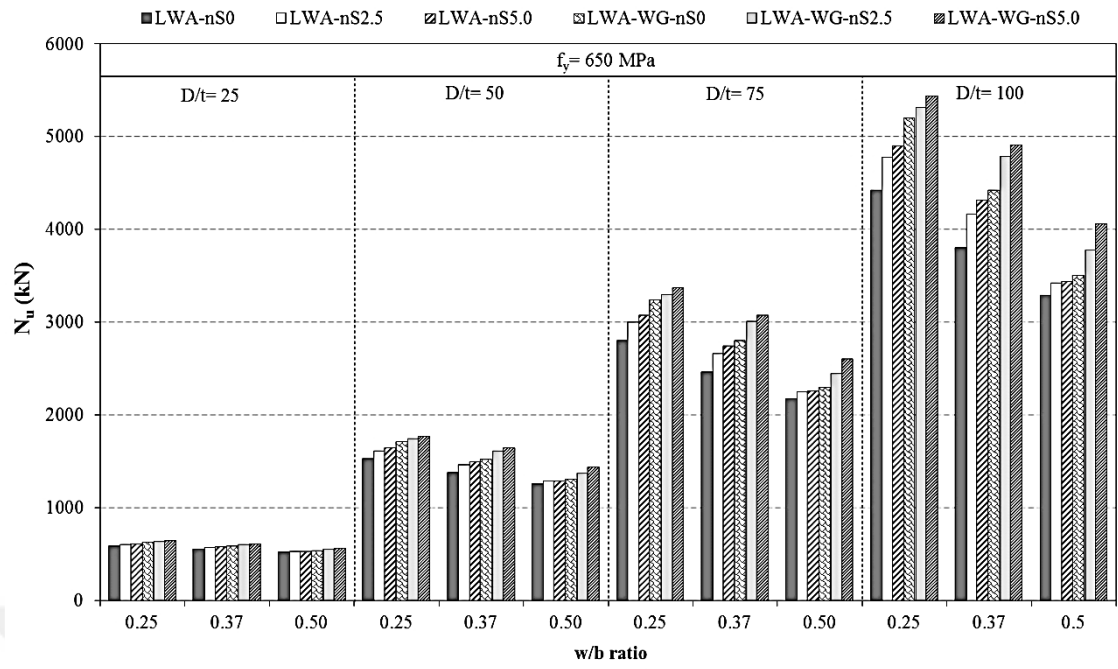
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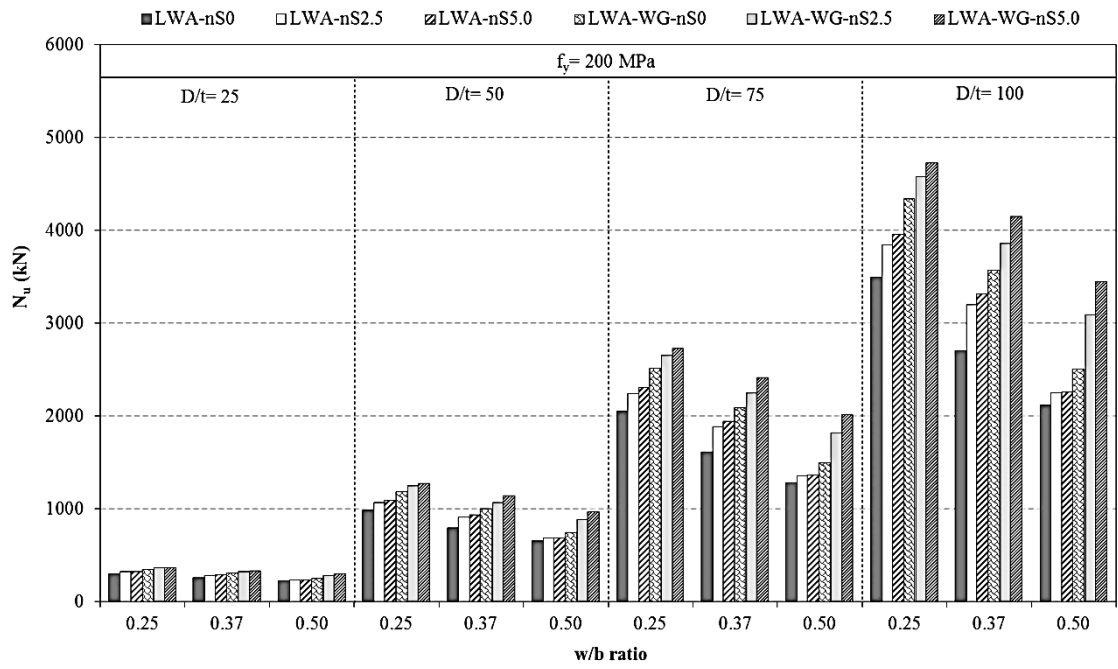
b)



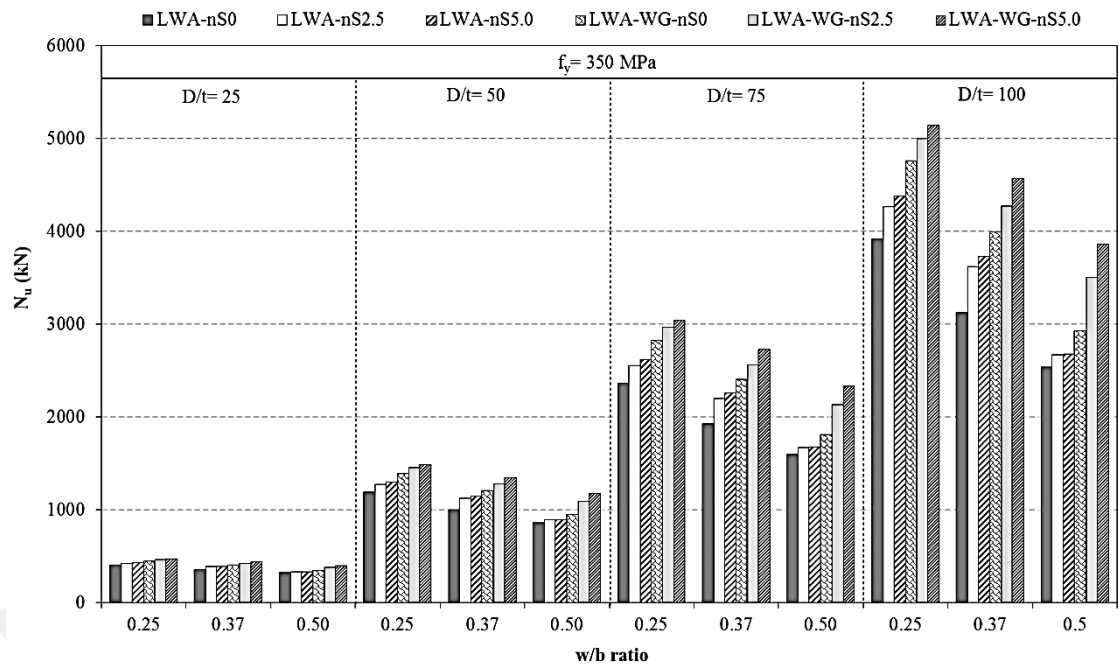
c)



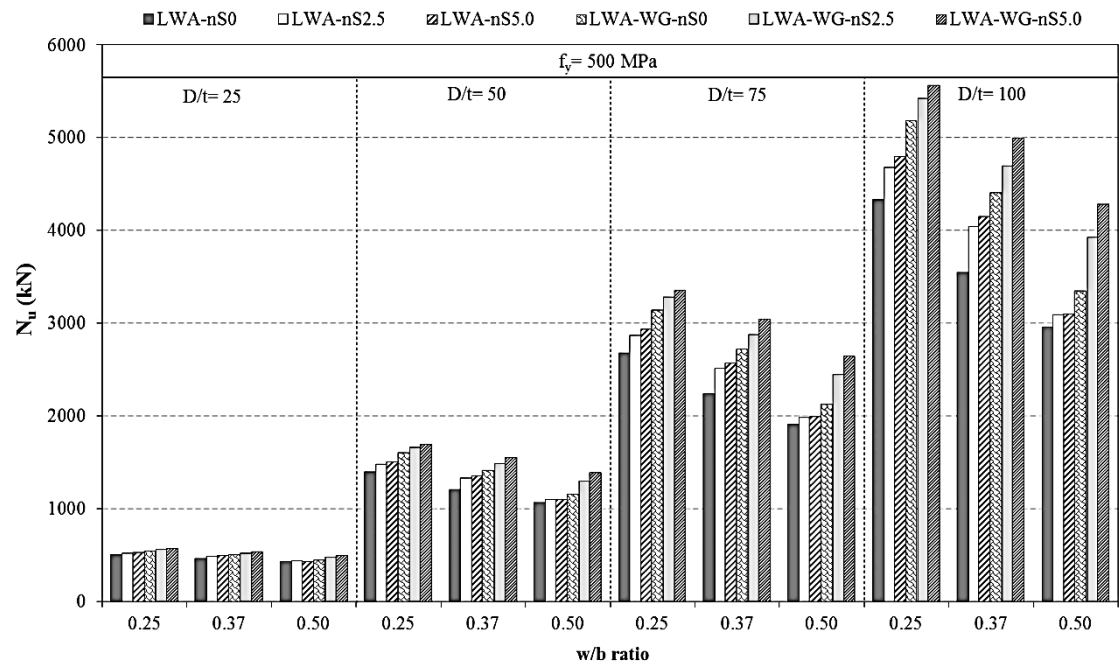
d)



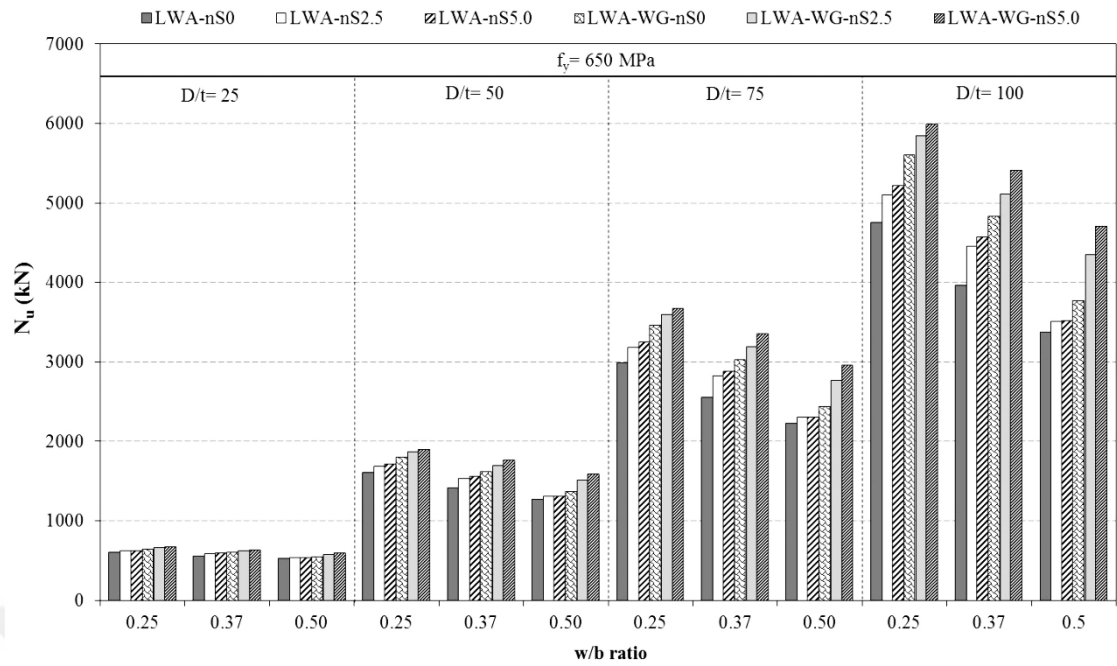
e)



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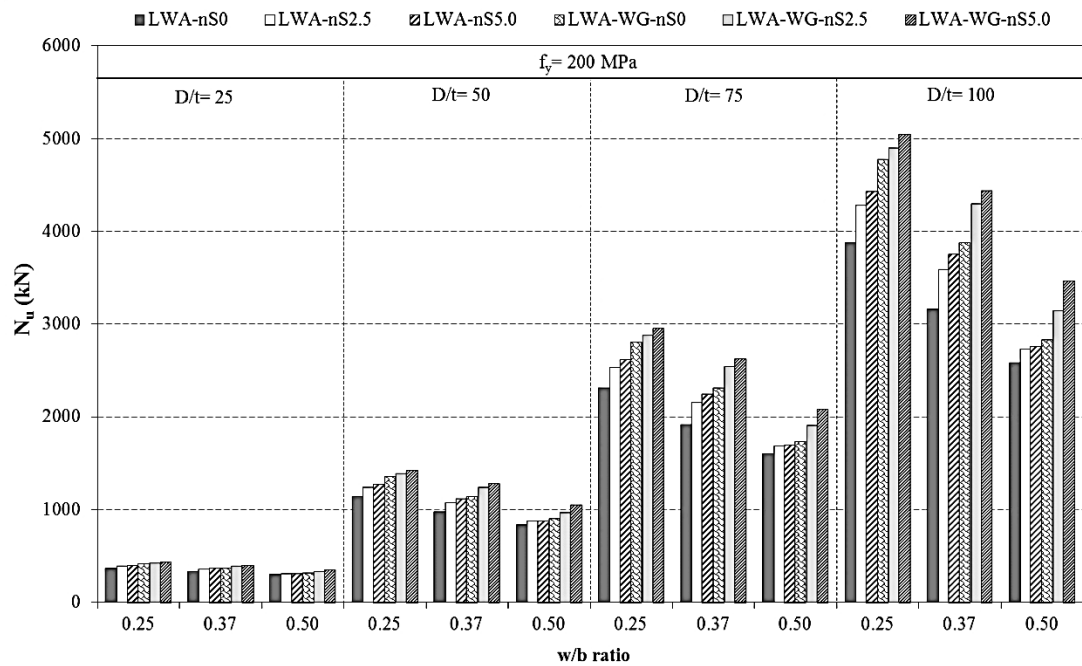


g)

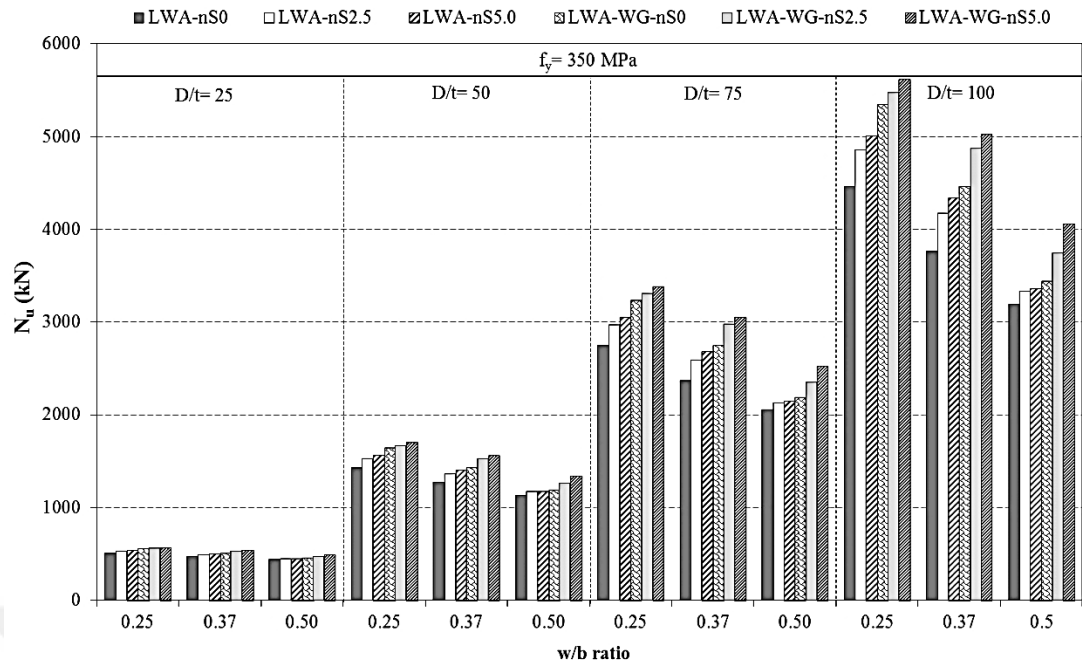


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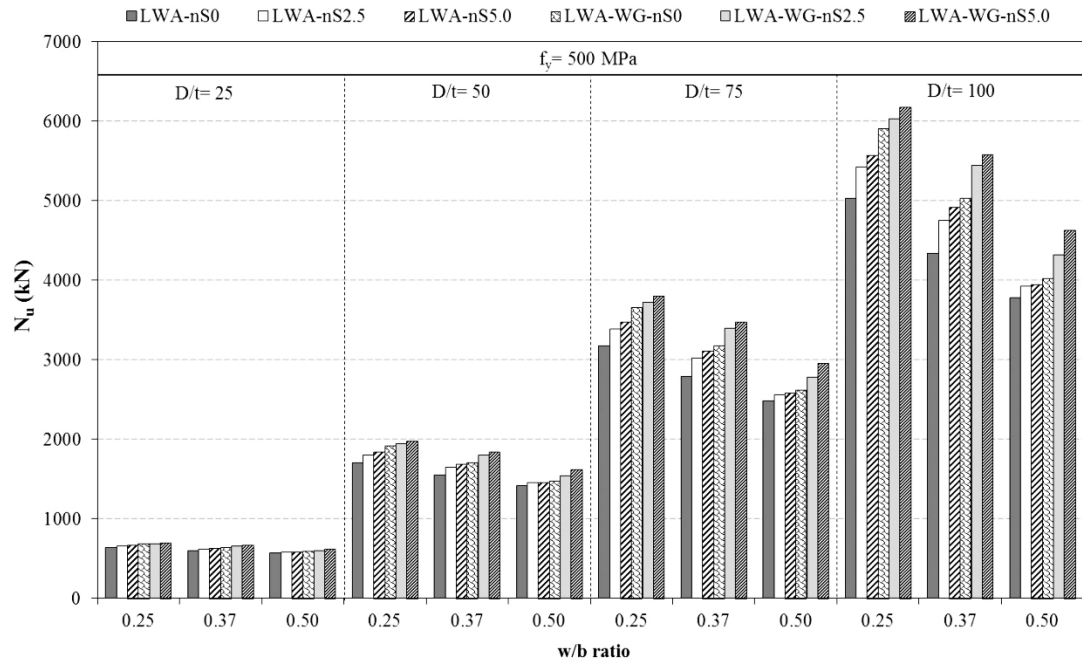
Figure 5.1 Axial capacity calculated via ACI 318 with respect to w/b ratio and steel yield strength; a, b, c, and d for 28 days and e, f, g, and h for 90 days



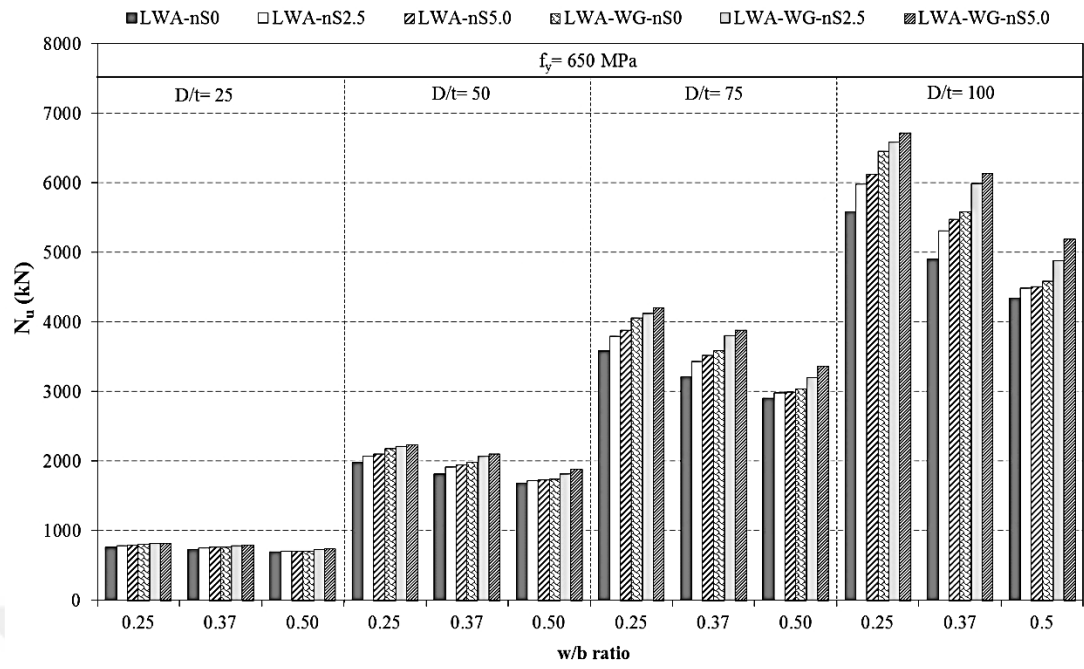
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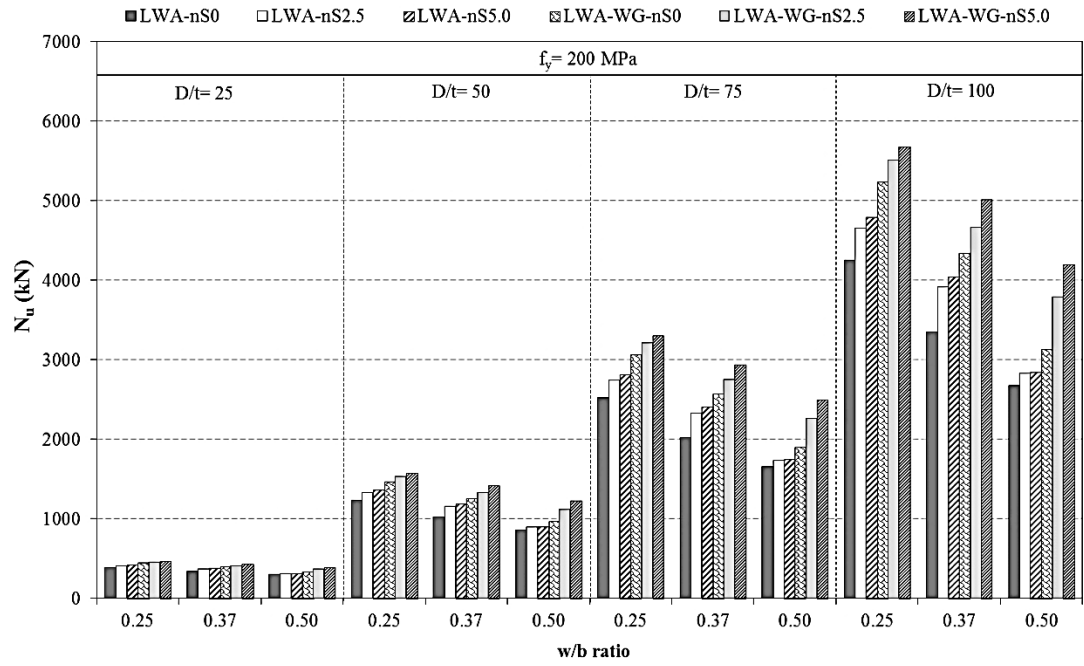
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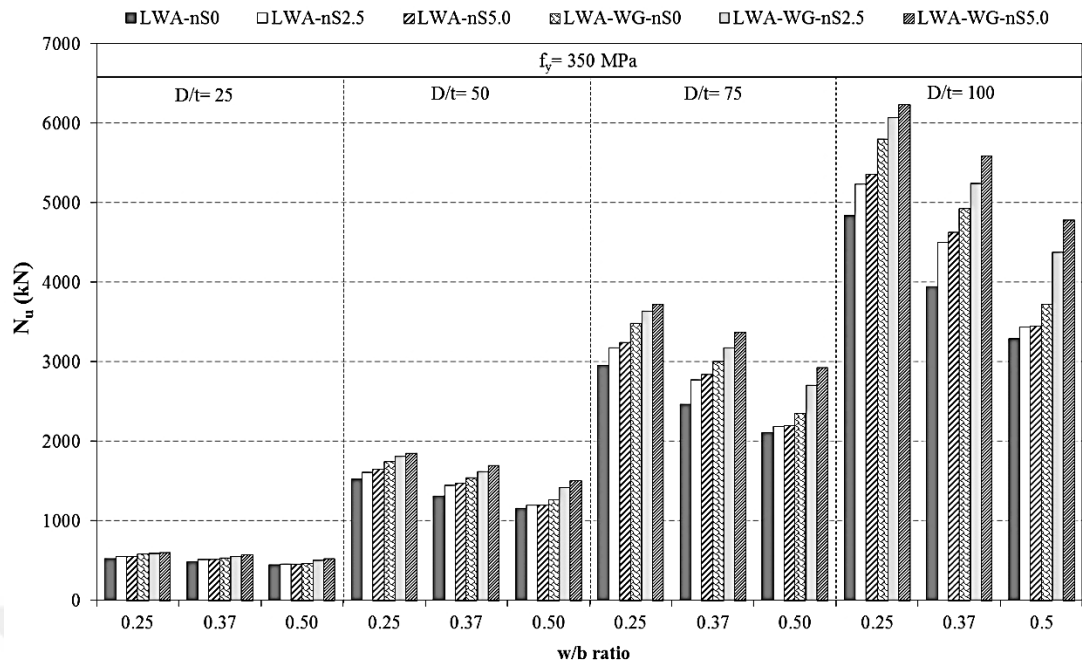
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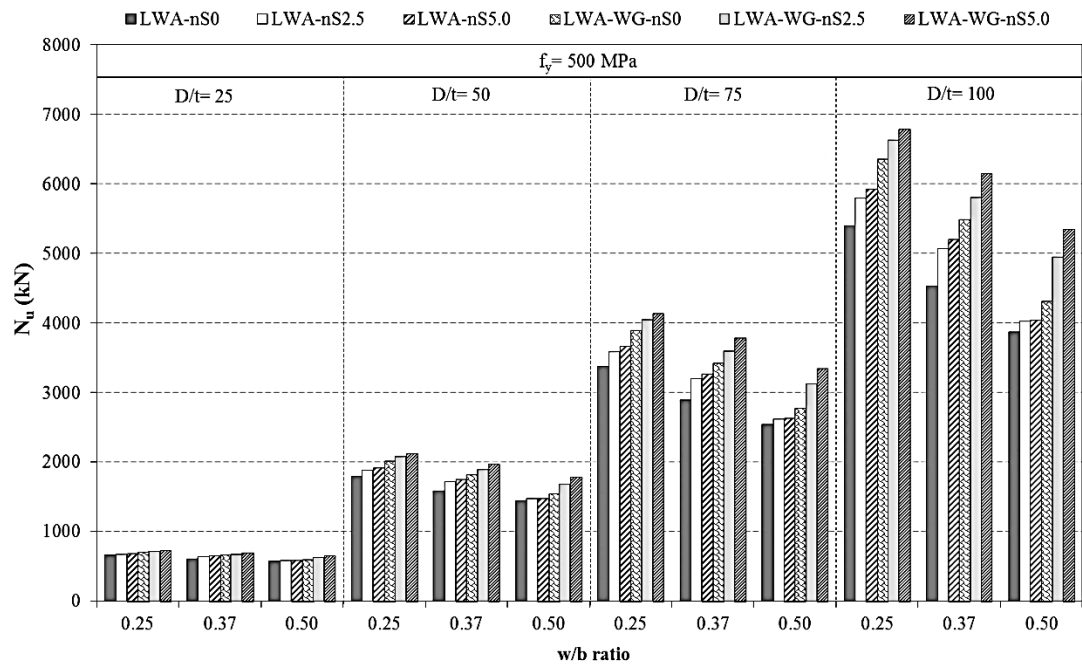
d)



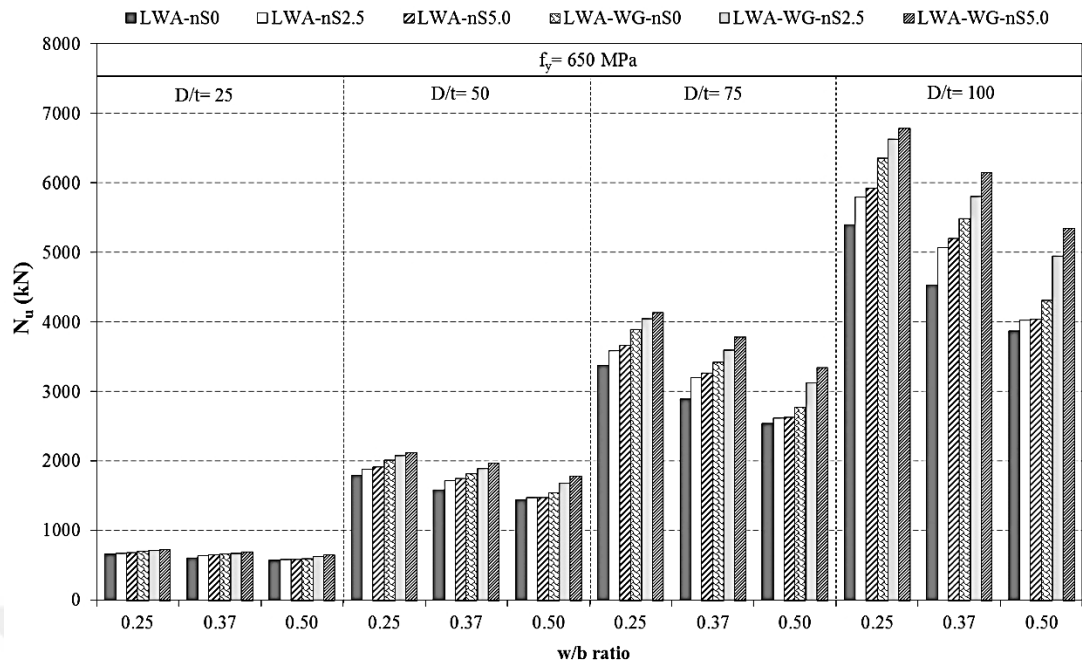
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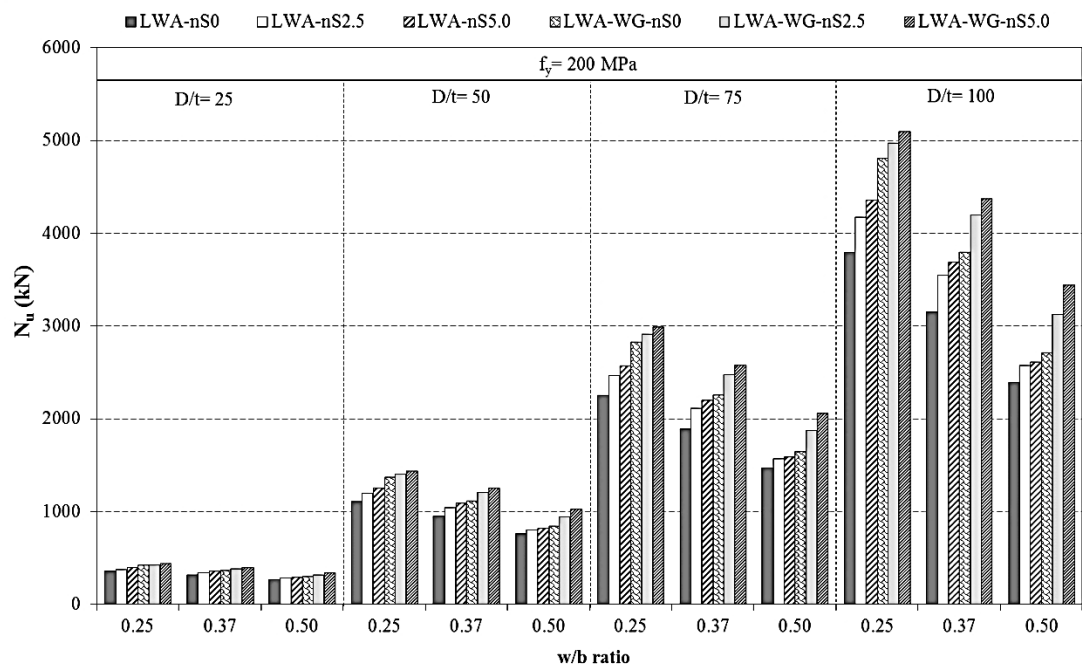


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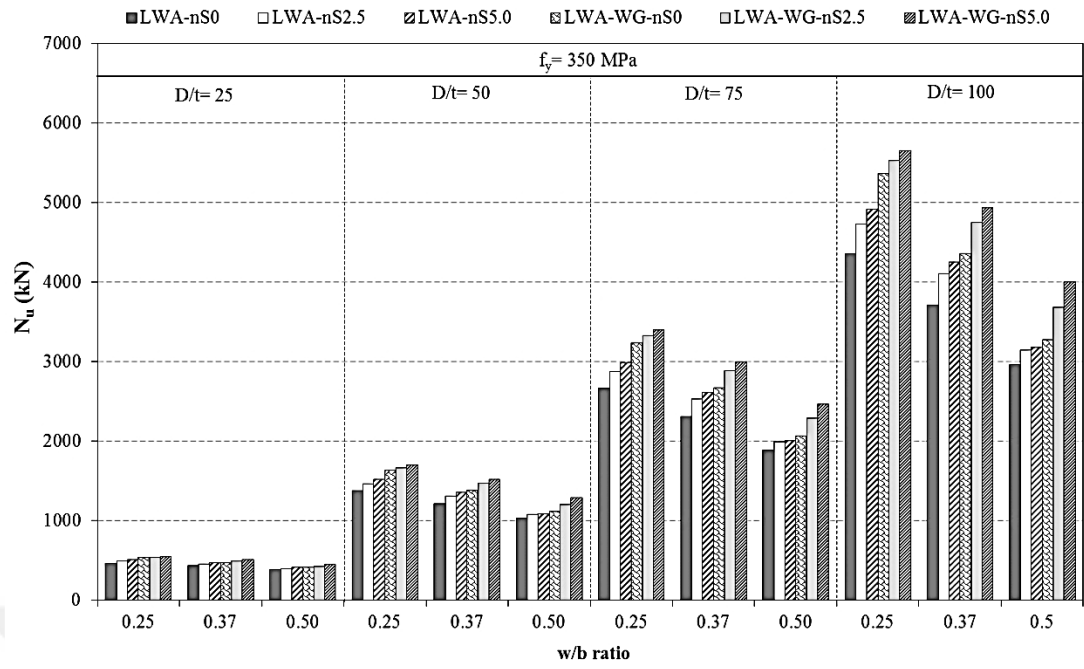


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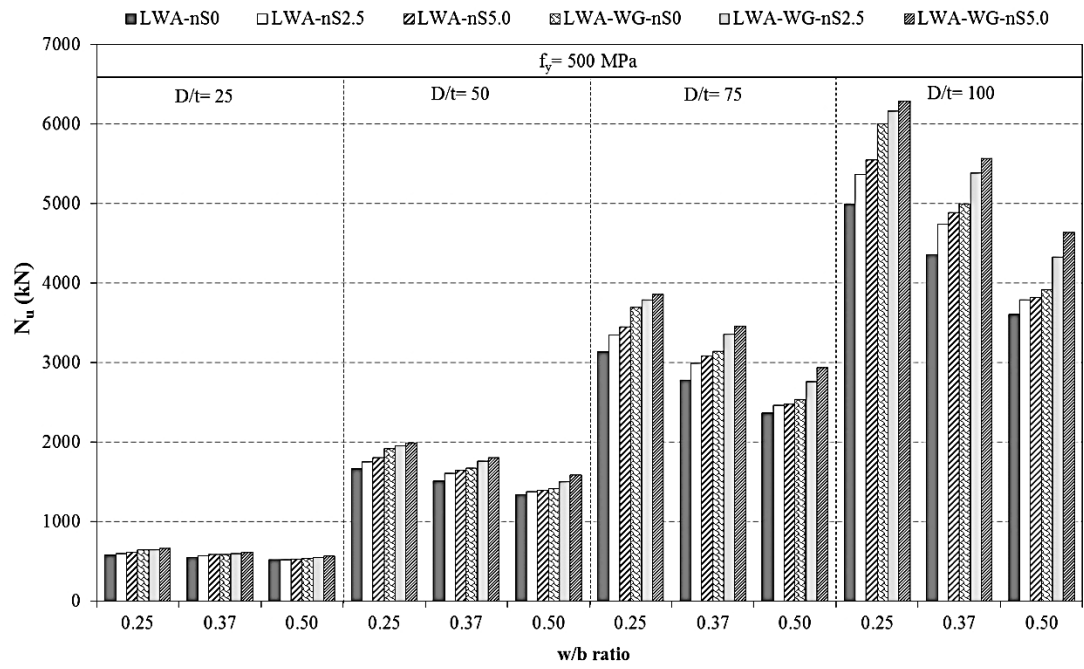
Figure 5.2 Axial capacity calculated via Eurocode 4 with respect to w/b ratio and steel yield strength; a, b, c, and d for 28 days and e, f, g, and h for 90 days



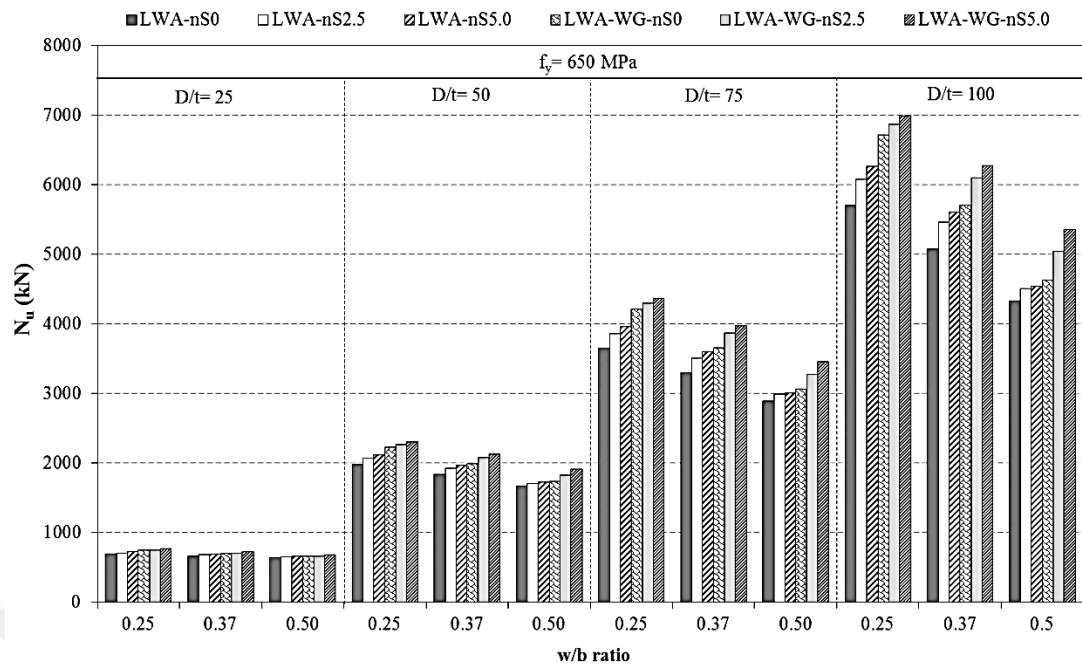
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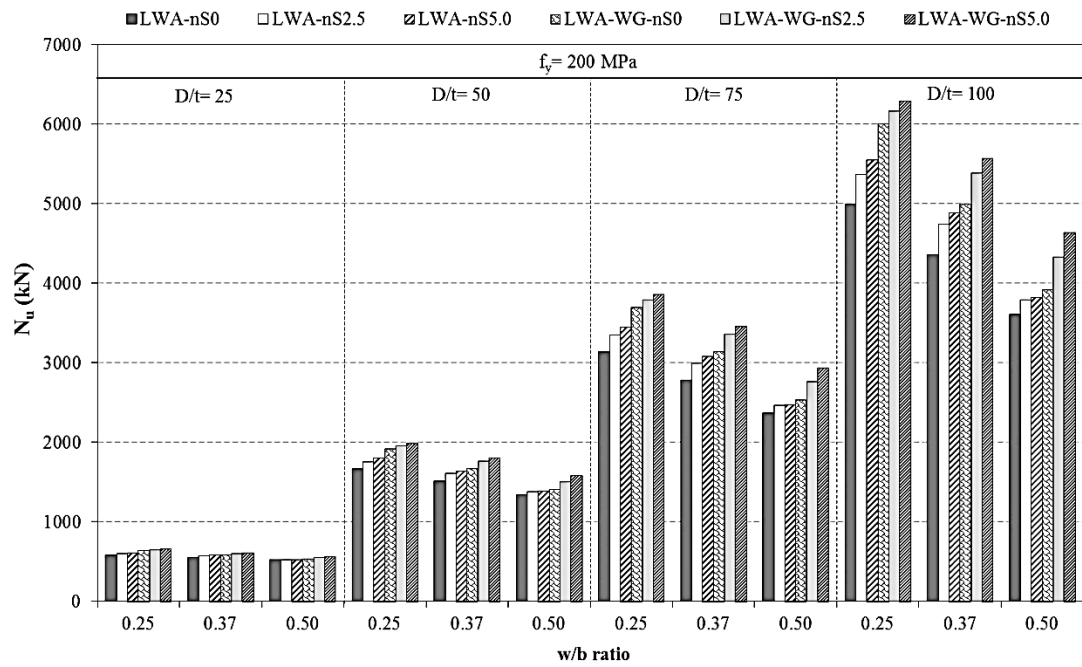
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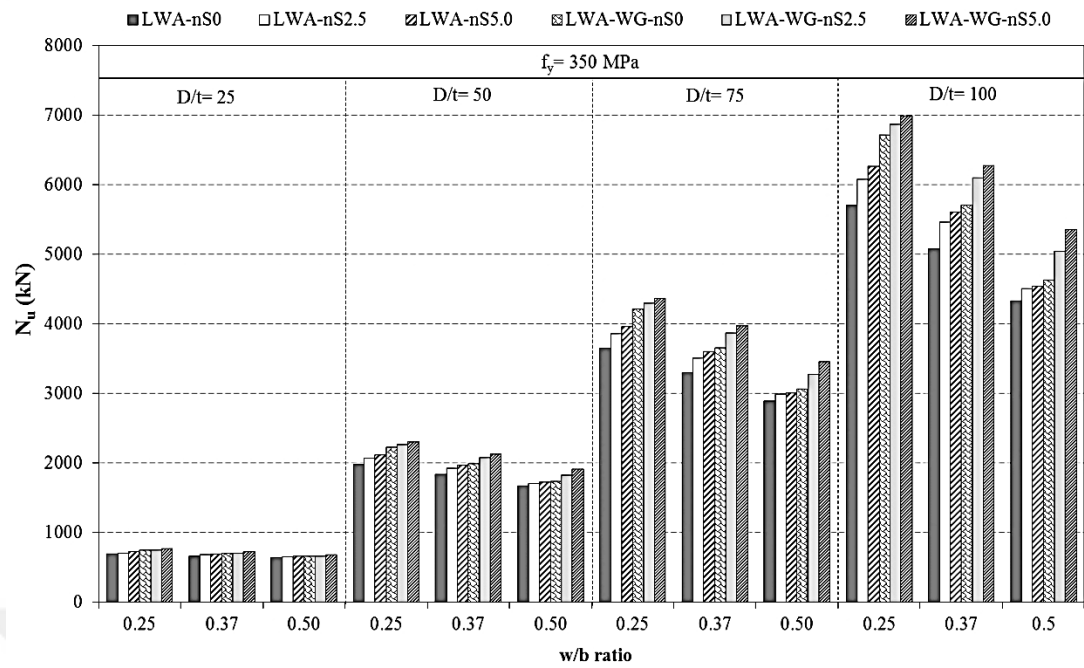
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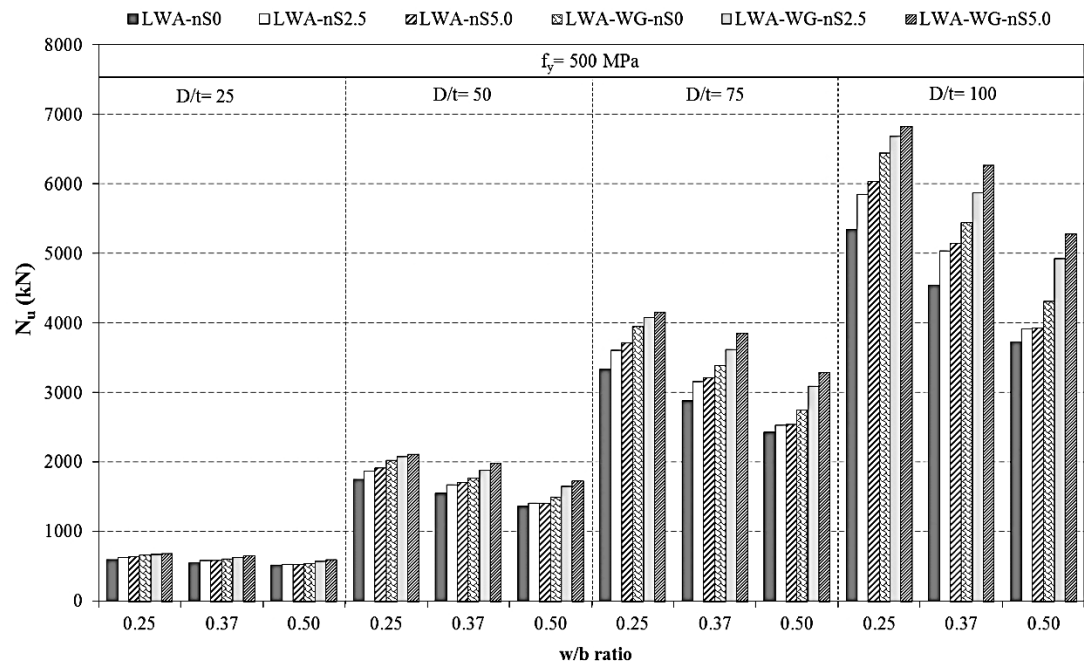
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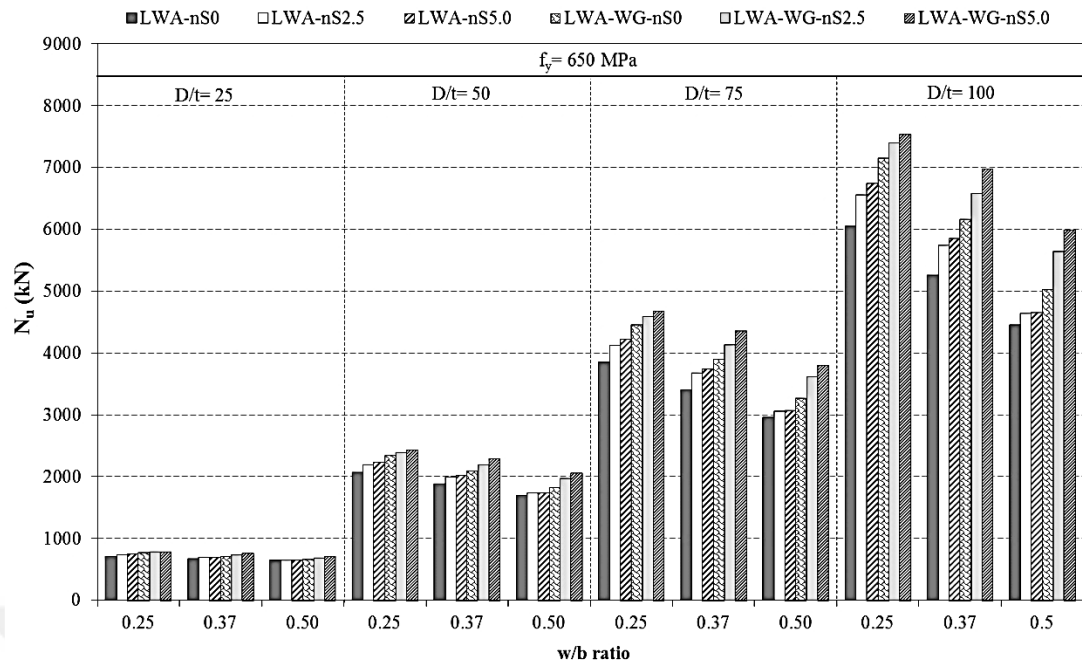
e)



f)



g)



h)

Figure 5.3 Axial capacity calculated via DL/T with respect to w/b ratio and steel yield strength; a, b, c, and d for 28 days and e, f, g, and h for 90 days

5.1.1 Effect of curing period

The effect of age of curing on the axial capacity of SCLC filled CFST columns (N_u) was estimated as the axial load carrying capacity at 90 days divided by the axial load carrying capacity at 28 days. Figures 5.4, 5.5, and 5.6 depict the effect of age of curing on the axial capacity computed by ACI-318R (2005), Eurocode 4 (2004), and Chinese code DL/T (1999), respectively.

Figures 5.4 shows computed results of mixture made with w/b ratios of 0.25, 0.37, and 0.50. This figure revealed that the enhancement of axial capacity due to the increase of age ranging from 2.8% to 10.5% and 3.5% to 13.0%, 16.0% to 10.0% and 2.9 to 13.7%, and 0.8% to 4.3% and 2.6% to 23.0% for the concretes made with untreated and treated LWA, respectively, regardless the use of nS, D/t ratio, and steel grade.

It is evident that the percent of N_{u90}/N_{u28} affected by D/t ratio for all SCLC mixtures. During the application of axial loads, the concrete in circular CFST columns are undergone to confinement exerted by the stainless steel tube. The

compression strength of concrete as well as its ductility is revealed to rise due to the confinement effect. Xue and Gong (2016) reported that in the elastic-plastic stage, the sample with higher tube thickness has higher specimen stiffness, as well as the ultimate axial capacity of concrete filled CFST columns. This is due to higher thickness of tube can result in higher confinement effectiveness for the concrete, so as to attain higher mechanical performance (Liang, 2011). Patel et al. (2017) observed that the increase in the D/t ratio considerably reduces the flexural stiffness, significantly decreases the ultimate axial load, but obviously rises the displacement ductility. They related this result to the use of larger D/t ratio of a steel cross-section reduces both its cross-sectional area and confinement effect. It is noteworthy to mention that in Patel et al. (2017) study the diameter of the circular column was 600 mm, while the D/t ratio were set as 40, 70, and 100 mm. In the present work, because of increasing of concrete diameter while using the same steel thickness in all calculations, the higher the concrete diameter used, the higher the concrete cross section, consequently, the ultimate axial capacity enhances with increasing of D/t ratio.

Similar trends in computed N_{u90}/N_{u28} ratios were also found for the Eurocode 4 (2004), and Chinese code DL/T (1999), Figures 5.5 and 5.6. In this regard, the enhancement ratios calculated for Eurocode 4 (2004) and represented in Figures 5.5 were showed an improvement as 2.2% to 9.8% and 2.8% to 12.5%, 1.3% to 9.3% and 2.4% to 13.0%, and 0.6% to 3.9% and 2.0% to 21.0% for mixtures made with untreated and treated LWA, respectively. Figure 5.6 presents ratios slightly below or above N_{u90}/N_{u28} ratios with respect to the other two cods. Moreover, the lower ratios for all codes were obtained with mixtures made with untreated LWA at w/b ratio of 0.50.

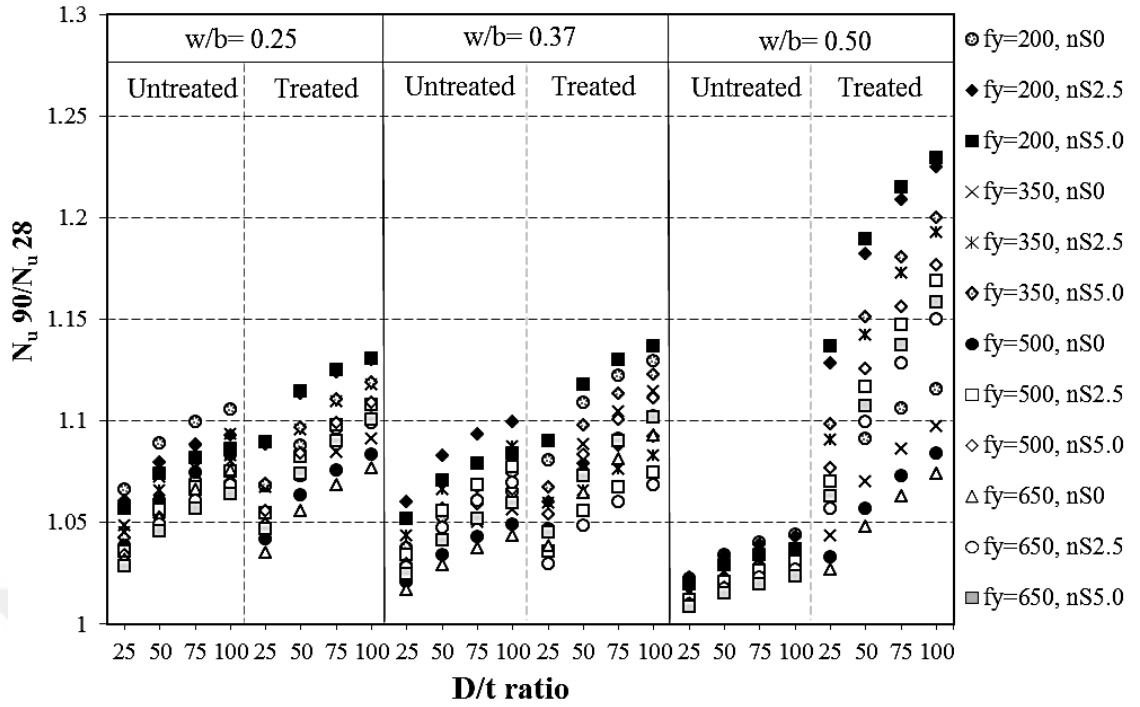


Figure 5.4 Effect of curing period on the predicted axial capacity calculated via ACI 318 with respect to diameter to thickness ratio

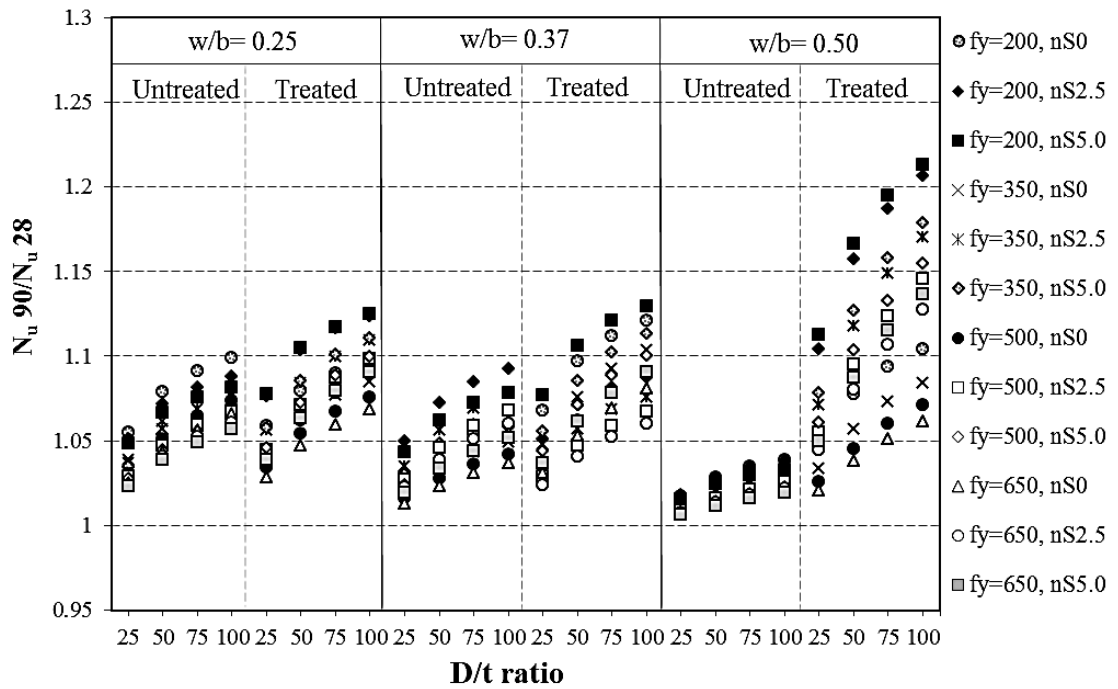


Figure 5.5 Effect of curing period on the predicted axial capacity calculated via Eurocode 4 with respect to diameter to thickness ratio

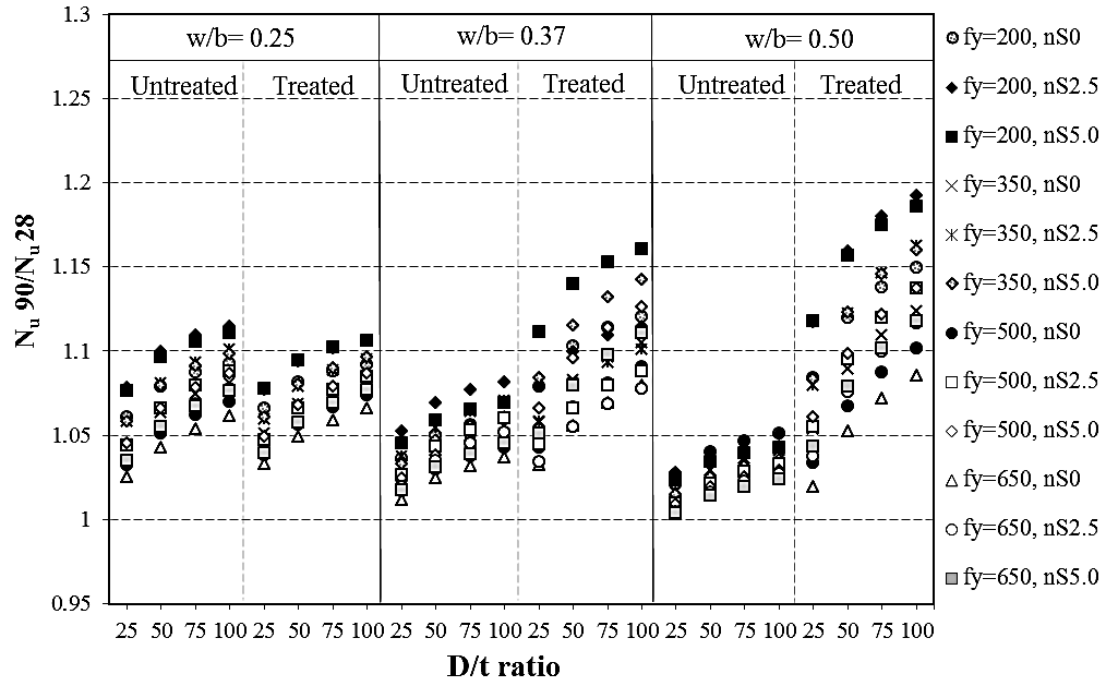


Figure 5.6 Effect of curing period on the predicted axial capacity calculated via DL/T code with respect to diameter to thickness ratio

5.3.2 Effect of using nano particles

The effect of using nS on the axial capacity of SCLC filled CFST columns (N_u) was assessed as the axial load carrying capacity for plain concrete divided by the axial load carrying capacity for concrete made with corresponding nS content. Figures 5.7 to 5.9 simulate the effect of nano particles incorporation on the axial capacity computed with respect to ACI-318R (2005), Eurocode 4 (2004), and Chinese code DL/T (1999), respectively. Figures 5.7, 5.8, and 5.9 illustrate the improvement ratios for the axial capacity of a circular CFST column at water-to-binder ratio of 0.25, 0.37, and 0.5 which have been computed with respect to ACI-318R (2005), Eurocode 4 (2004), and Chinese code DL/T (1999), respectively. These figures revealed that the lower enhancement in axial capacity due to incorporation nS resulted for LWA-WG-nS2.5 mixtures with steel columns having yield strengths of 650 MPa, whereas, the higher improvement yielded for mixes namely LWA-nS2.5 with circular steel columns having grade of 650 MPa. For instance, with respect to ACI-318R (2005) and w/b ratio of 0.25, the maximum improvements in axial capacity for LWA-nS2.5 are 15% and 13% at 28 and 90 days. On the other side, for both two ages 28 and 90 days, Figures 5.7, 5.8, and 5.9 showed that the higher enhancement due to the use of

nS occurred with LWA-WG-nS5.0 mixes and yield steel stress of 200 MPa, whereas the lower improvement computed for LWA-nS5.0 and yield strength of steel of 650 MPa compared to the other mixes.

It is widely known that a transition zone constructs at the paste side between aggregate or steel bar and paste. On the other hand, it is believed that the transition zone holds a more porous structure and larger hexagonal calcium hydrate crystals than the bulk paste. The addition of pozzolans such as silica fume, fly ash, etc form denser microstructure. According to Bumrongjaroen (1999), Babu and Babu (2003), Zhang et al. (2010) and Boukendakdji et al. (2012), the presence of pozzolans produces a more homogeneous microstructure by balancing the Ca/Si molar ratio in the interfacial zone relative to that in the bulk paste, quite different from ordinary Portland cement concrete, where this ratio improves dramatically as the surface of aggregate is approached. The interface between concrete and steel tube has a similar structure as that of normal aggregate and paste (Li et al., 2006).

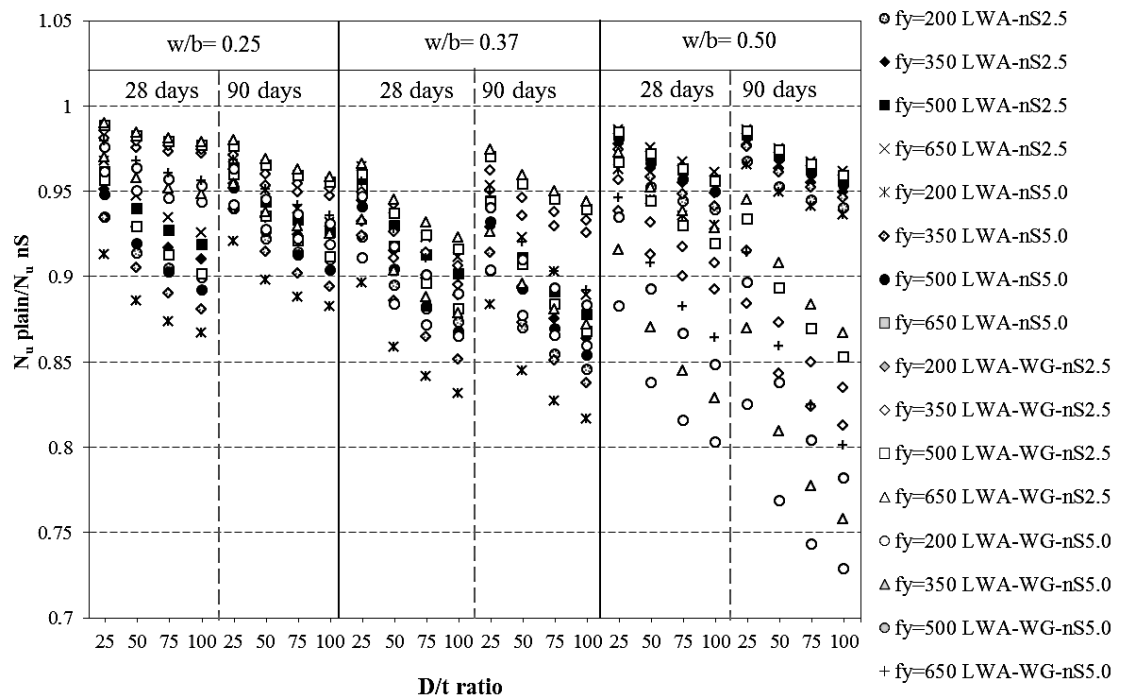


Figure 5.7 Effect of incorporation nS on $N_{u,ACI}$ of the composite columns having different SCLC mixtures

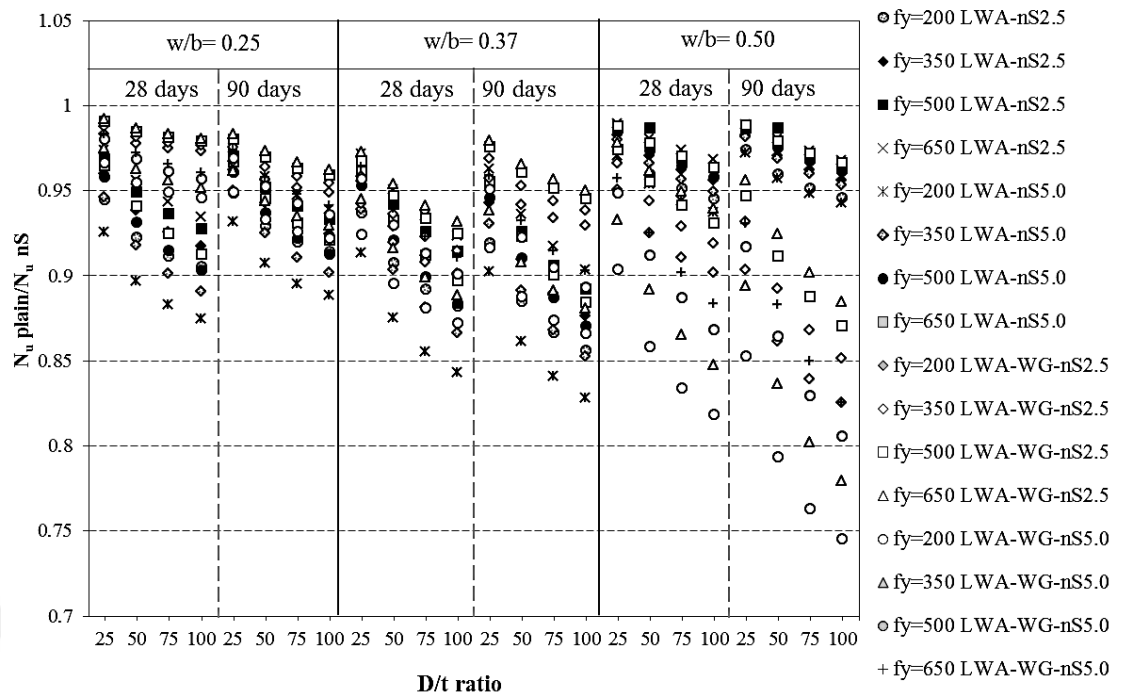


Figure 5.8 Effect of incorporation nS on $N_{u,EC4}$ of the composite columns having different SCLC mixtures

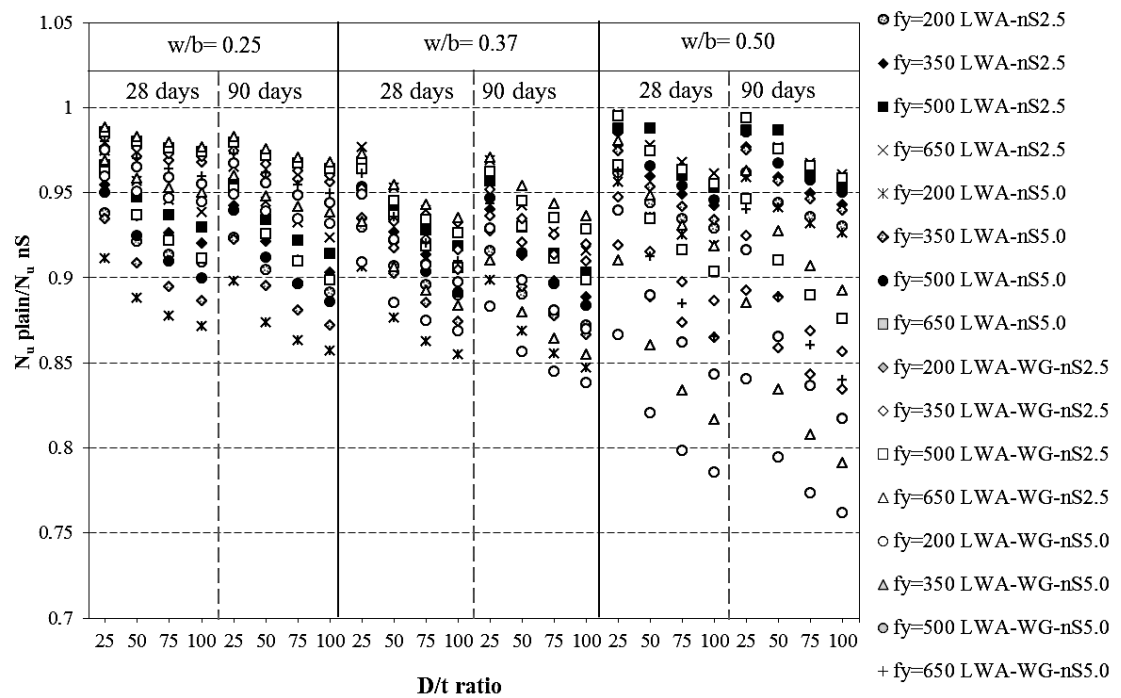


Figure 5.9 Effect of incorporation nS on $N_{u,DL/T}$ of different SCLC mixtures

5.3.3 Comparison of the design codes

In order to understand the behavior of the predicted ultimate axial strength of circular CFST columns using ACI-318R (2005), Eurocode 4 (2004), and Chinese code DL/T (1999) provisions, a comparative study of predictions of the evaluated data is conducted. To attain the objective of this section, three different presentations performed as follows:

- A. The ratios of the 28 and 90 days predicted axial capacity results by using the Eurocode 4 (2004) and Chinese code DL/T (1999) provisions for D/t ratio of 25, 50, 75, and 100 to those of ACI-318R (2005) are demonstrated in Figures 5.10 and 5.11, respectively. Similarly, Figure 5.12 presents the comparison between Eurocode 4 (2004) provision predicted values and Chinese code DL/T (1999) provision.
- B. The ratios of $N_{u, EC4} / N_{u, ACI}$, $N_{u, DL/T} / N_{u, ACI}$, and $N_{u, EC4} / N_{u, DL/T}$ versus the total number of specimens are shown in Figure 5.13 (a-c) and Figure 5.14 (a-c) for 28 and 90 days, respectively. For this, each mixture are presented by 48 ratios so as to involve all the selected parameters; w/b ratio, D/t ratio, and steel yield strength.
- C. Regression analysis between the predicted axial capacity results by using the different provisions are shown in Figure 5.15 (a-c) for 28 and 90 days.

From clause A, it can be revealed that the prediction of Eurocode 4 (2004) and Chinese code DL/T (1999) provisions for the axial capacity of SCLC filled CFST columns are generally higher than the predicted values obtained with ACI-318R (2005), Figures 5.10 and 5.11. This is mainly due to the fact that the composite action between the steel tube and the concrete core was not considered in ACI-318R (2005) (Lu and Zhao, 2010; Jamaluddin et al., 2013). Figure 5.10 (a-c) shows the variations of the predictions apparently become lower with the increase of D/t ratio. Although the Chinese code DL/T (1999) provision varies axial capacity results with D/t ratio, it can be indicated from Figure 5.11 that the lower variations usually accompanying a lower D/t ratio of 25, regardless w/b ratio and age of curing.

The differences between the results of Eurocode 4 (2004) and Chinese code DL/T (1999) provisions as presented in Figure 5.12 seem to be relatively similar with each other. This aforesaid note reflects the nature between these two codes. Wang et al.

(2014) adopted the design guidelines specified by Eurocode 4 (2004) and Chinese code DL/T (1999) provisions to determine the test strength for the CFST column. It was directed that Eurocode 4 (2004) and Chinese code DL/T (1999) were closer to the test column strength than those by the other design guidelines.

Observing Figures 5.13 (a-c) and 5.14 (a-c) one can note that, slight differences between the predictions of all the three provisions for SCLC mixtures are found. The reason may be related to the presence of randomness concrete strength, different steel grade, and various diameter to thickness ratio. The differences between the predicted results of SCLC appear to be decreases with increasing concrete strengths. Lu and Zhao (2010) states although the confinement effect is considered in the Chinese code DL/T (1999) formulas, it provided lesser predictions than those of ACI-318R (2005) for some specimens, particularly with very high-strength steel tube, which seems to be a deficiency of this code.

Regarding clause C described previously, it can be observed that there were considerably high relationship between the predicted values of Eurocode 4 (2004) and ACI-318R (2005), (Figure 5.15a), Chinese code DL/T (1999) and ACI-318R (2005), (Figure 5.15b), and Eurocode 4 (2004) and Chinese code DL/T (1999), (Figure 5.15c), respectively, so that a regression analysis provided correlation coefficient (R^2) of 0.99 for all codes and ages. Within the predicted axial capacity results by using the different provisions of this study, all models appeared to be well close to each other and provided higher prediction.

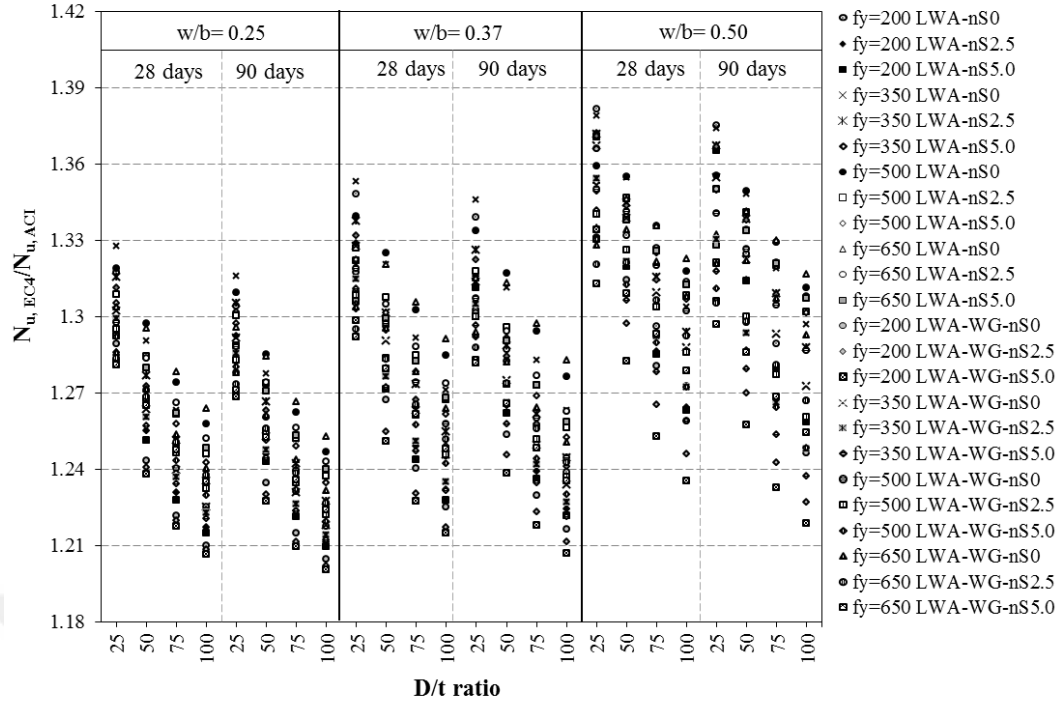


Figure 5.10 Influence of the diameter-to-thickness fraction on ratio of predicted axial capacity results by using the Eurocode 4 and ACI 318 provisions

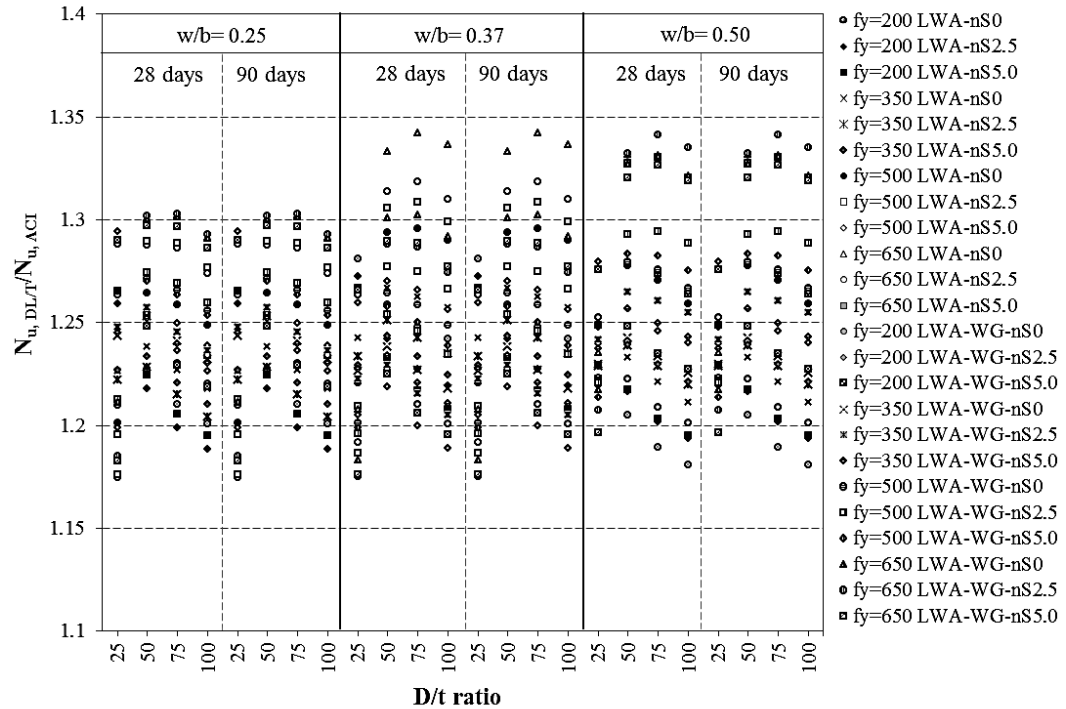


Figure 5.11 Influence of the diameter-to-thickness fraction on ratio of predicted axial capacity results by using the Chinese code DL/T and ACI 318 provisions

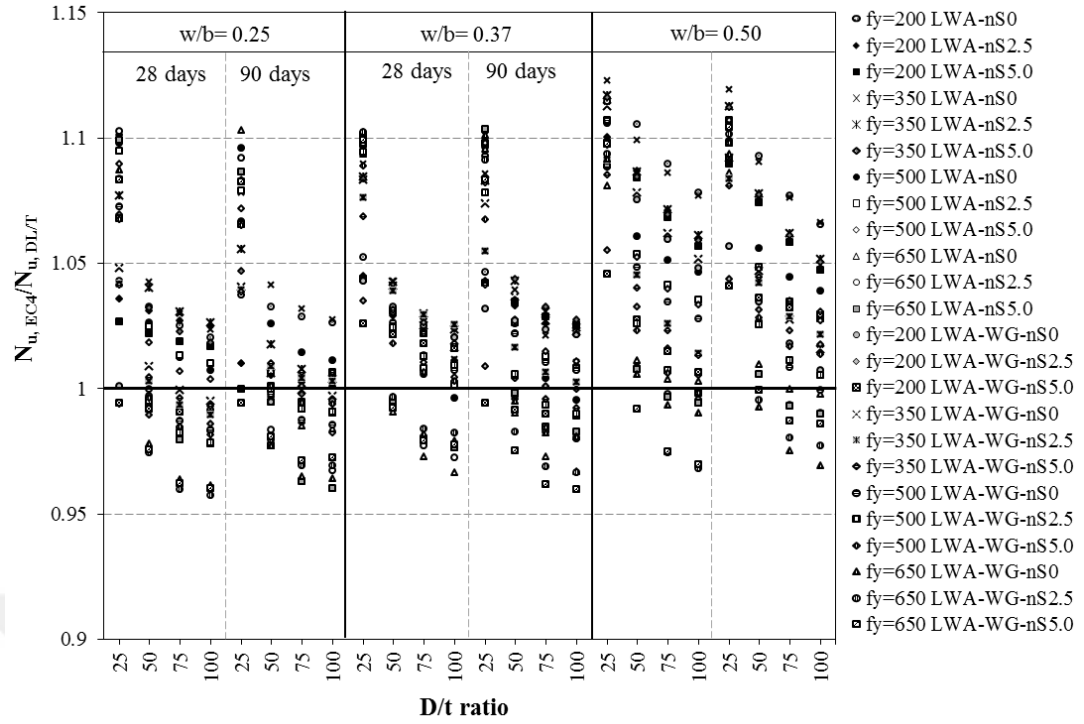
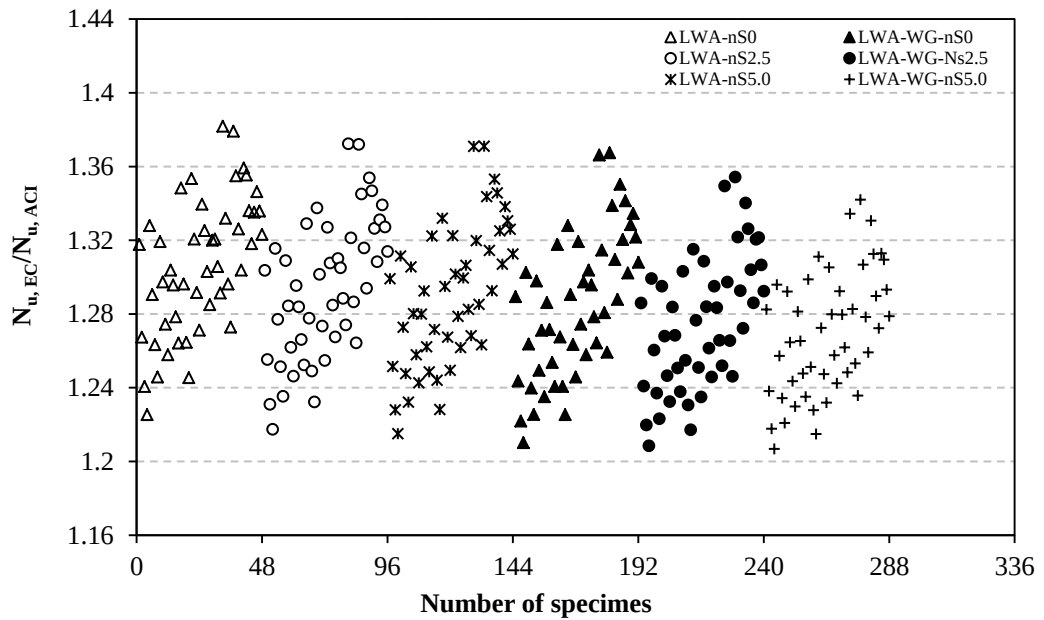
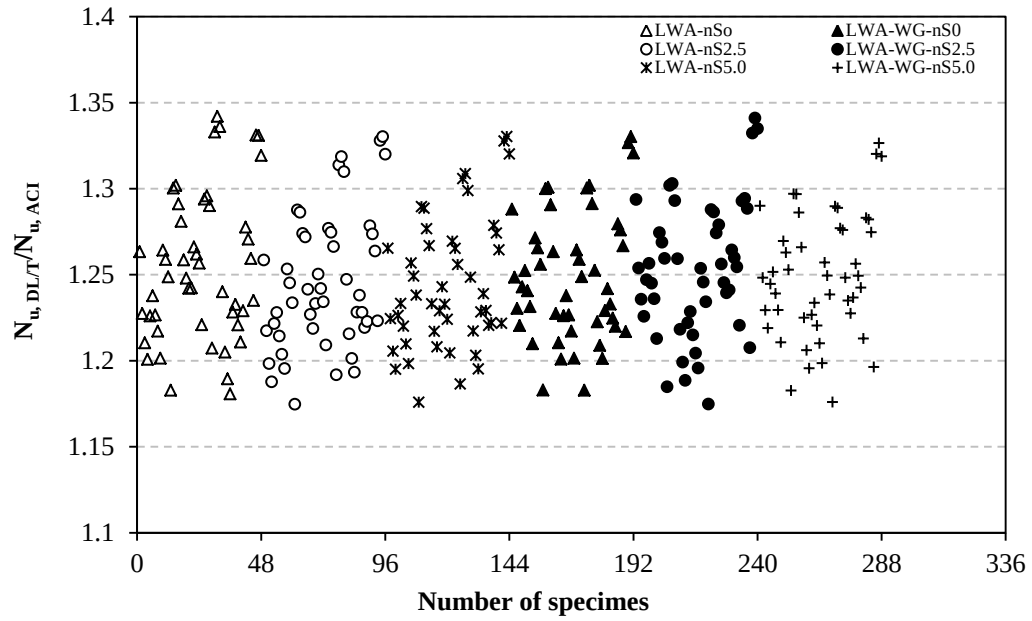


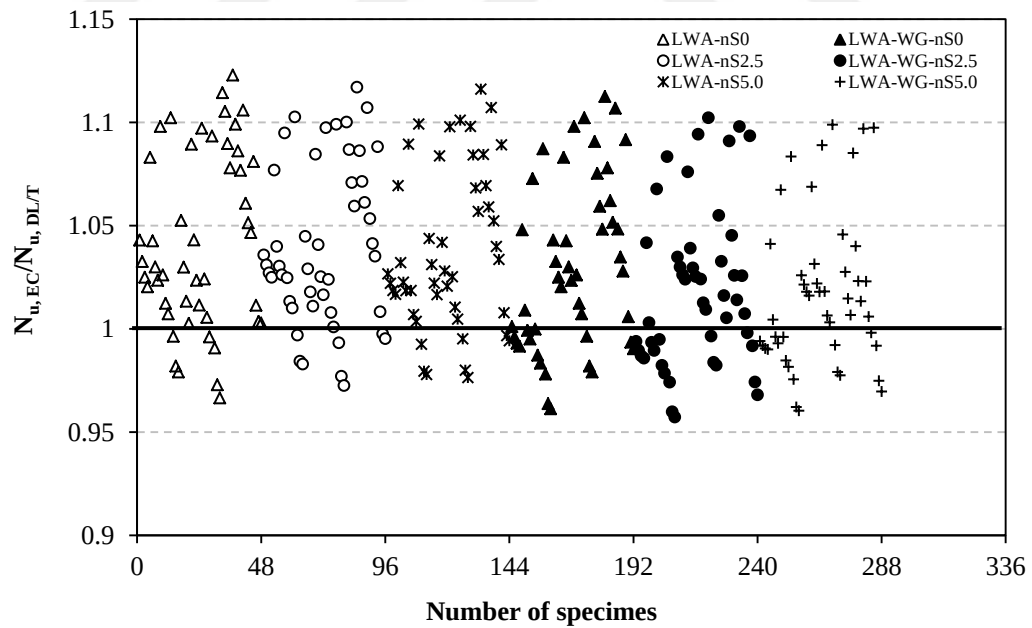
Figure 5.12 Influence of the diameter-to-thickness fraction on ratio of predicted axial capacity results by using the Eurocode 4 and Chinese code DL/T provisions



a)

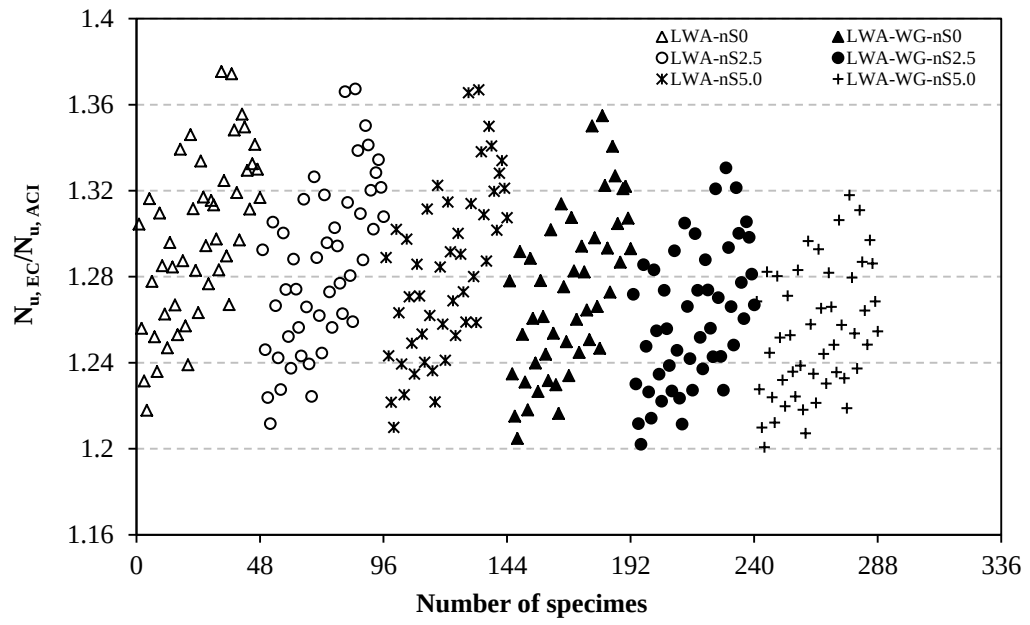


b)

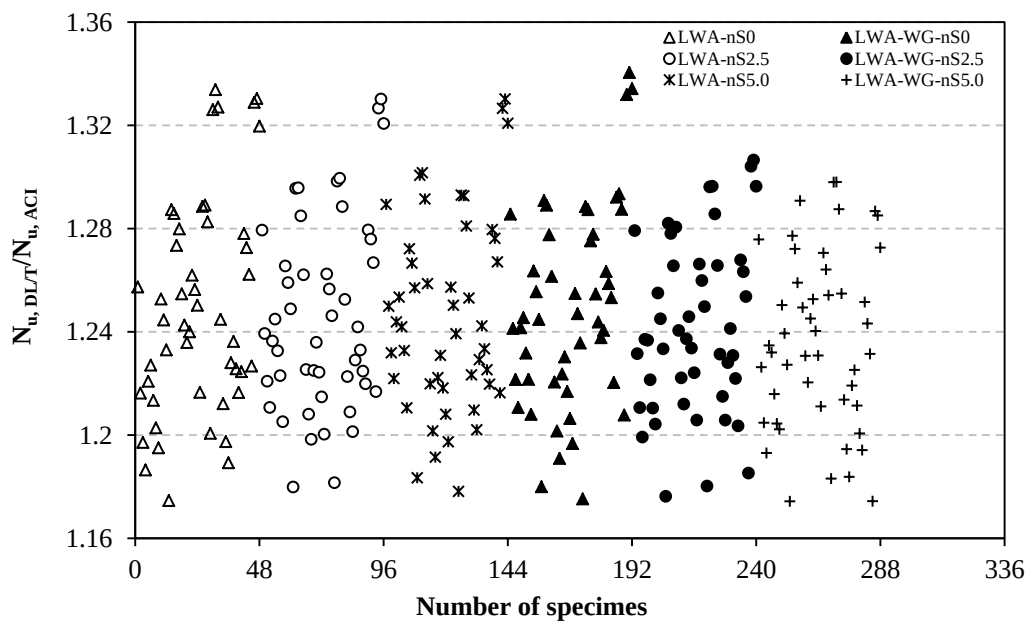


c)

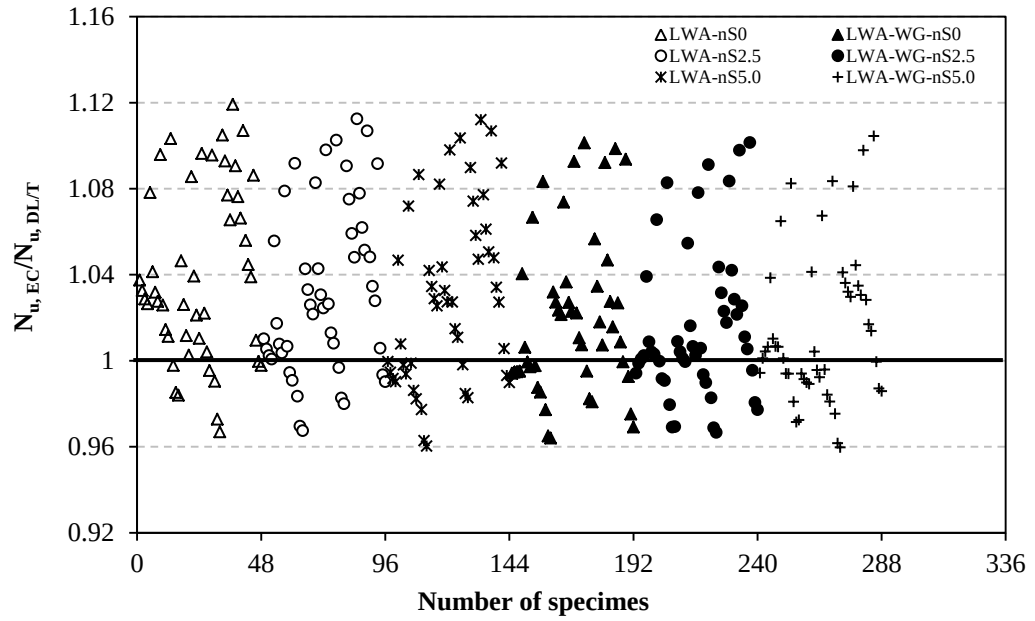
Figure 5.13 Ratio of predicted axial capacity results by using the Eurocode 4, ACI 318, and Chinese code DL/T provisions for different SCLC at 28 days: a) $w/b = 0.25$, b) $w/b = 0.37$ and c) $w/b = 0.50$



a)

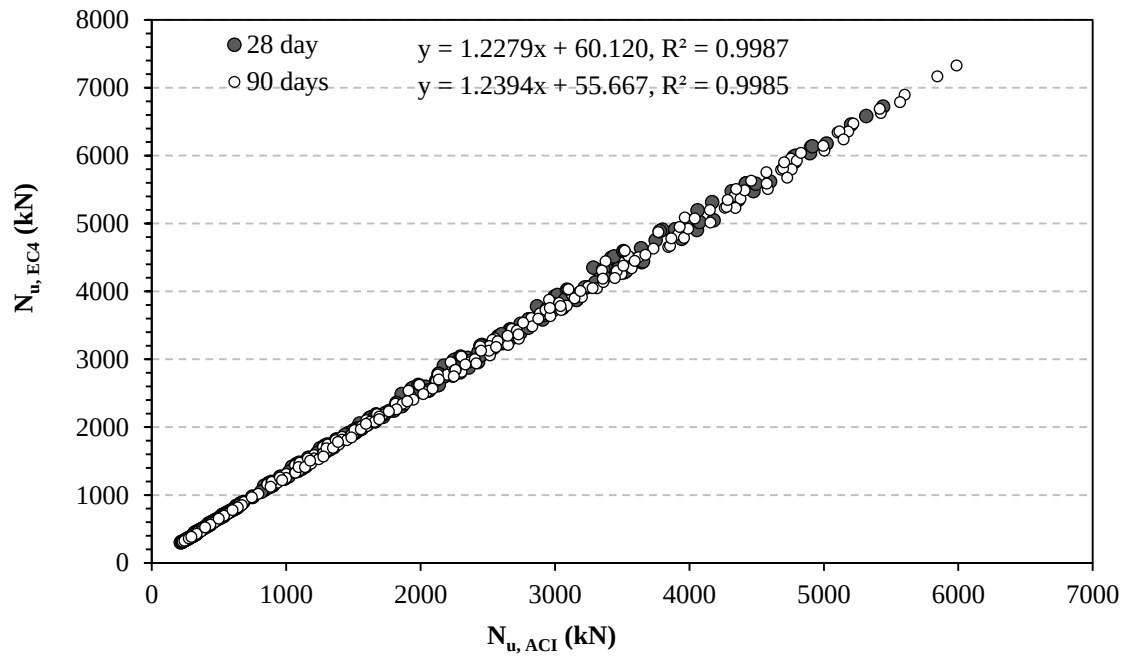


b)

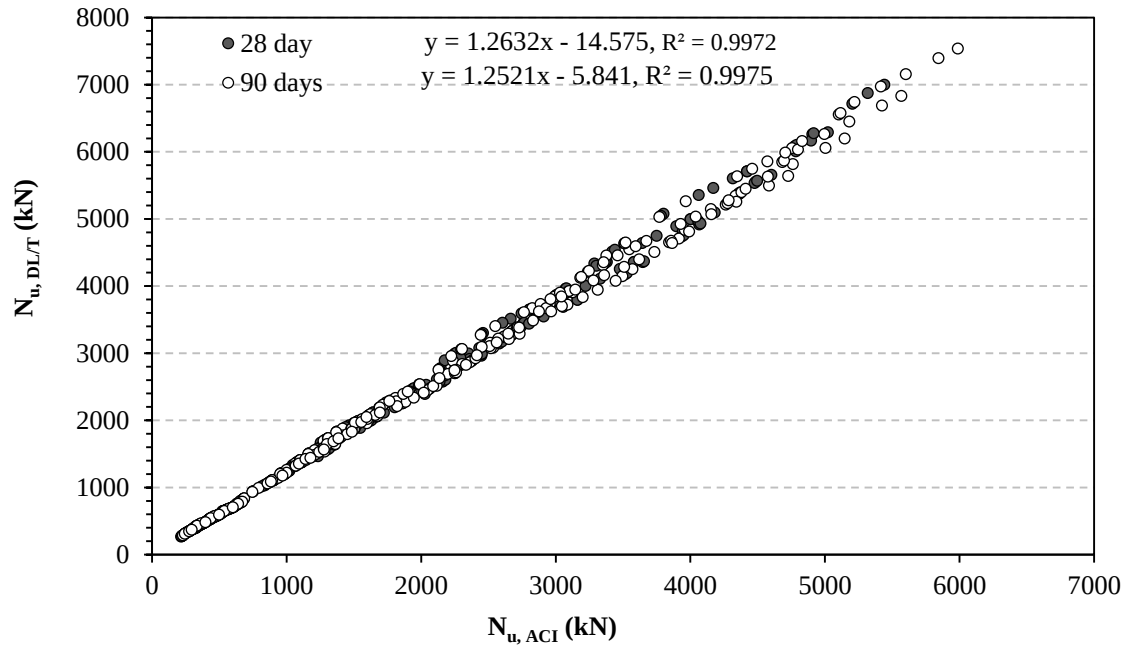


c)

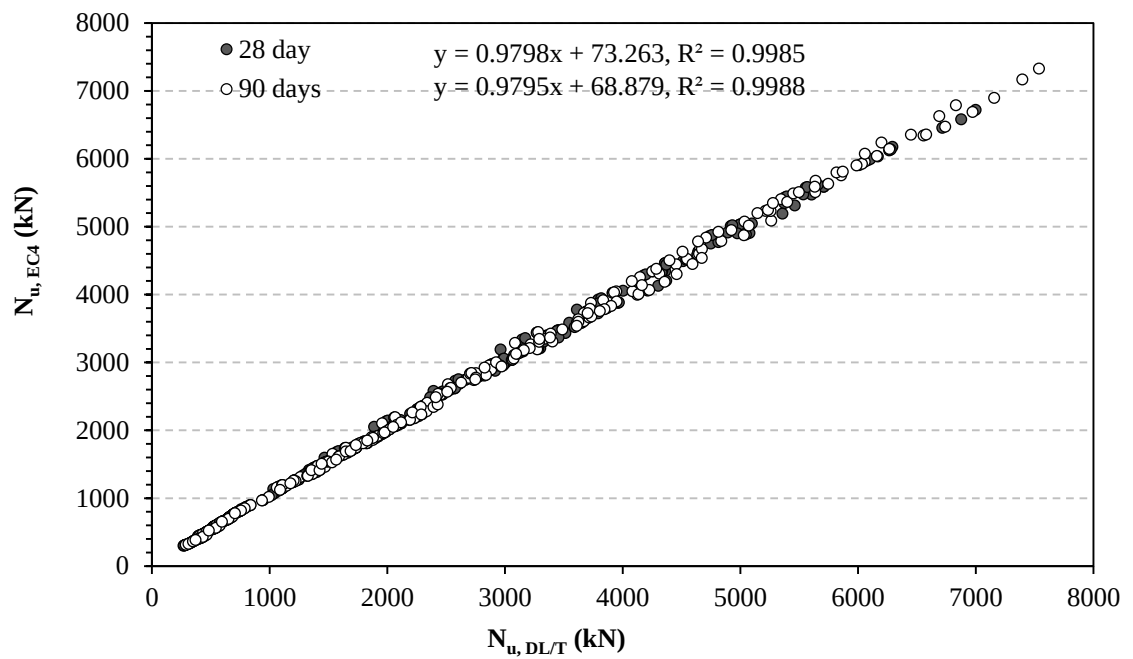
Figure 5.14 Ratio of predicted axial capacity results by using the Eurocode 4, ACI 318, and Chinese code DL/T provisions for different SCLC at 90 days: a) $w/b = 0.25$, b) $w/b = 0.37$, and c) $w/b = 0.50$



a)



b)



c)

Figure 5.15 Regression model of predicted axial capacities of the composite columns

CHAPTER 6

CONCLUSIONS

In the first part of the thesis, an experimental study was adopted to study and enhance the properties of self compacting lightweight concrete (SCLC) produced by the use of cold bonded fly ash (FA) lightweight aggregate (LWA). Two special strategies were selected to enhance the inferior properties of such concrete. These strategies are the treatment of LWA by soluble solution of sodium silicates (water glass) and the replacement of cement by different levels of nano silica (nS). On the other hand, in the second part of the thesis, the axial compressive behavior of SCLC filled steel tube columns having different diameter to thickness ratio and steel yield strengths have been analytically investigated by using American code (ACI), Eurocode 4 (EC4), and Chinese design code (DL/T). Based on the fresh and hardened properties investigated through a comprehensive experimental program and the analytical study presented in this thesis, the following conclusions can be listed:

- The experiments indicated that the crushing strength of water glass treated aggregate higher than untreated aggregate. The results obtained through water absorption test showed that the lower absorption would have for treated LWAs of about 4.0%. Moreover, the treatment technique resulted in a slight increment of the specific gravity of LWA without influencing on the SCLC density. The treatment changed the appearance of surface texture of the aggregate particle as lustrous due to the form a glassy cover which can be clearly detected by visual observation.
- All of the concrete mixtures had fresh density in the range of 1836-1961 kg/m³. The replacement of cement with nS decreased the fresh density of SCLCs. All of the mixtures produced in this study were considered as lightweight or semi-lightweight self-compacted concretes.
- The SCLC mixes were designed to give a slump flow diameter of 700±50 mm, which was achieved by adjusting the dosage of HRWRA.

- The SCLC mixes were designed to give a slump flow diameter of 700 ± 50 mm, which was achieved by adjusting the dosage of HRWRA. To satisfy the desired slump flow diameter, the amount of HRWRA was increased with the increase in nS replacement level whereas the use of treated aggregates decreased the dosage of HRWRA. All of the mixtures manufactured in this study can be classified as SF2.
- The smoother treated aggregate particles increased flowability of SCLC. The slump flow diameters of SCLC containing LWA implemented with the treated LWA are higher than of SCLC including natural surface LWA.
- The time required to reach 500 mm (T_{500}) increased with the use of nS in the slump flow test. On the contrary, the glassy layer created from water glass treatment caused to increase the value of T_{500} when treated LWAs used instead of untreated LWAs. A similar trend was also observed in the V-funnel flow times of the SCLCs. Depending on w/b ratio, the SCLCs can be classified as VF2 or VF1. However, incorporating nS and untreated aggregates in SCLCs at w/b ratio of 0.25 exceeded slightly the upper limit of VF2 class.
- All the mixes had blocking ratio between 0.8 and 1.0 therefore they were classified as PA2 in terms of passing ability class. The variation in H_2/H_1 ratio had been lessened and approached to ideal ratio (1.0) as the nS and treated aggregates used.
- Irrespective of type of SCLCs, the shrinkage of SCLC prism samples increase with rising w/b ratios. The use of treated instead of untreated aggregates reduces the negative effect of w/b ratio increments on shrinkage rates of SCLCs. However, the shrinkage rates of SCLC specimens containing treated aggregates which having lower water absorption were observed to be higher than those with untreated aggregates during the first week of drying.
- Keeping the same w/b ratio, the use of 2.5% and 5.0% nS in the SCLC specimens decreased shrinkage after 61 days as much as 10% and 14%, respectively. Furthermore, it was observed that the efficiency of nS in reducing shrinkage fairly improved with high w/b ratio and impermeable LWA.

- The results showed that weight loss was affected by w/b ratio, aggregate porosity and nS incorporation. Development of weight loss showed larger losing trend coinciding with w/b ratio increment. Whereas by replacing untreated with treated aggregates, it can be considered that the weight loss upon drying is a function of aggregate porosity, taking into account the use of absorbent aggregates resulted in higher weight loss.
- SCLCs made with treated LWAs have lower sorptivity indexes than samples containing natural surface LWAs. This behavior is due to the rate of water absorption of treated aggregate being about 5 times less than that of untreated aggregates.
- The findings of the gas permeability test confirmed that the refines of pore structure due to the addition of nS and age advancement are below the lower permeability coefficient values irrespective of w/b ratio and type of aggregates. At maximum w/b ratio, LWA-WG-nS5% after 90 days achieved a 54% coefficient less than the same mix produced with binary blends. In addition, the reduction in aggregate porosity after treatment process reduced gas penetration.
- The concretes faced remarkable resistance against water penetration when applying nS and/or treated LWAs. The minimum water depth observed through water permeability tests was reduced up to 1.7% for LWA-WG-nS5.0 with a w/b ratio of 0.25 and cured 90 days.
- Irrespective of LWA type and nS content, as well as curing age, it is pointed out that the higher w/b ratio resulted in the lower compressive strength. The result indicates that the compressive strength of 0.25 w/b ratio concrete mix made with FA and natural LWAs decreases from 55 MPa to 31 MPa due to the increase in w/b ratio to 0.50. Additionally, the negative effect of high w/b ratio on compressive strength was eliminated by the incorporation of nS and treated LWAs.
- The improvements of strength (compressive and splitting) were detected up to 5.0% of nS such that the maximum enhancement in strength of 40% was observed at w/b ratio of 0.50 within 90 days compressive strength of LWA-WG-nS5.0 compared to that of the concrete without nS.

- It was observed at a given w/b ratio, the treated LWAs dominant the raise in the splitting strength when used instead of untreated LWAs.
- The use of treated LWAs provided an excellent enhancement in the static elastic modulus of SCLC compared to those modified with untreated LWAs. The key role of LWA quality in improving elastic modulus of concrete reduced the efficiency of incorporation nS on that of SCLC made with untreated surface aggregate.
- The axial compressive load capacities of CFST columns were very sensitive to D/t ratio and steel yield strength of steel. It clearly appears that the axial capacity increased with the D/t ratio and steel yield strength increasing. The higher the concrete diameter used, the higher the concrete cross section area, consequently, the ultimate axial capacity value enhanced with increasing of D/t ratio as the increase of cross section area of concrete core in all the three codes contribute to the ultimate axial capacity. CFST columns with high strength steel obtained higher axial capacity in all SCLC types.
- Increasing period of curing upgraded the predicted resistance of CFST column under axial compression. It was seen that the higher effect resulted for the column made with SCLC having treated lightweight aggregate due to the increment in curing period. Moreover, the increase of D/t ratio of composite short column has almost influence on the variation ratio of axial capacity of 90 days to 28 days. In addition, an increase in the period of curing for concrete made with w/b ratio of 0.50 and untreated LWA has a slight effect on the variation of ultimate axial capacity since the core concrete compressive strength is relatively low.
- The ultimate capacity of the CFST column increases systematically with the incorporation of nano particles. The lower enhancement in axial capacity due to the use of nS resulted for LWA-WG-nS2.5 mixtures with steel columns having yield strengths of 650 MPa, whereas, the higher improvement yielded for mixes namely LWA-nS2.5 with circular steel columns having grade of 650 MPa.
- The use of treated lightweight aggregate with lower w/b ratio as well as higher grade of steel tube improves the ultimate strength of CFST column

and the enhancement of strength leads to a higher stiffness and a better performance for concrete filled steel tubular columns.

- Comparisons between the predictions of ultimate axial capacity of circular CFST short columns show that the ACI and DL/T codes provide lower predictions with respect to Eurocode 4. This is due to the fact that Eurocode 4 takes account mainly the composite action between the tube and the core of the concrete. For all the three codes, the lower variations usually accompanying a lower D/t ratio of 25. The predictions by the corresponding codes have almost the same trend especially those of Eurocode 4 and DL/T.
- The key regression equations between Eurocode 4 and the others are also developed at different curing periods to establish the relationship between them. Even though each design code used different parameters and followed different ways to calculate the capacity of the columns under axial compression, linear regression models with significantly high coefficient were observed between them in the current study.
- It is also worthy to mention that it seems possible to design the composite columns having SCLC with satisfactory strength level (up to 7537 kN) and lighter weight (up to 25% reduction in self-weight). This is very crucial issue in the structural design concept to achieve the earthquake-resistance structures.

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APPENDIX A

PREDICTED RESULTS OF AXIAL CAPACITY (ACI, EC4, AND DL/T)



Table A1 Predicted axial load carrying capacity $N_{u,ACI}$, $N_{u,EC4}$, and $N_{u,DL/T}$ (kN)

Mixture code	Age of curing (day)	D/t	f_y (MPa)	Predicated $N_{u,ACI}$			Predicated $N_{u,EC4}$			Predicated $N_{u,DL/T}$		
				w/b (0.25)	w/b (0.37)	w/b (0.50)	w/b (0.25)	w/b (0.37)	w/b (0.50)	w/b (0.25)	w/b (0.37)	w/b (0.50)
LWA-nS0	28	25	200	279.05	244.81	216.56	367.68	330.08	299.25	352.52	313.60	268.53
LWA-nS2.5	28	25	200	298.57	265.18	223.83	389.24	352.42	307.16	375.76	337.29	279.19
LWA-nS5.0	28	25	200	305.72	273.20	224.94	397.15	361.24	308.37	386.85	346.08	280.83
LWA-WG-nS0	28	25	200	322.14	279.02	228.82	415.34	367.65	312.60	414.93	352.48	286.60
LWA-WG-nS2.5	28	25	200	328.40	299.34	243.76	422.29	390.09	328.92	424.84	376.95	311.78
LWA-WG-nS5.0	28	25	200	335.23	306.27	259.24	429.88	397.75	345.89	432.46	387.69	330.78
LWA-nS0	28	50	200	901.22	752.12	629.06	1142.18	974.91	837.82	1106.14	946.59	758.00
LWA-nS2.5	28	50	200	986.25	840.84	660.71	1238.03	1074.31	872.98	1200.72	1043.92	803.21
LWA-nS5.0	28	50	200	1017.41	875.74	665.56	1273.22	1113.52	878.37	1245.75	1079.87	810.14
LWA-WG-nS0	28	50	200	1088.91	901.08	682.47	1354.10	1142.03	897.19	1359.48	1106.00	834.37
LWA-WG-nS2.5	28	50	200	1116.15	989.62	747.54	1384.97	1241.83	969.78	1399.56	1205.58	939.05
LWA-WG-nS5.0	28	50	200	1145.93	1019.79	814.94	1418.72	1275.90	1045.25	1430.33	1249.18	1017.25
LWA-nS0	28	75	200	1861.95	1517.09	1232.45	2309.92	1918.48	1597.42	2253.63	1893.24	1465.89
LWA-nS2.5	28	75	200	2058.62	1722.29	1305.66	2534.07	2151.13	1679.79	2467.02	2113.16	1568.55
LWA-nS5.0	28	75	200	2130.69	1803.01	1316.87	2616.35	2242.87	1692.41	2568.59	2194.34	1584.28
LWA-WG-nS0	28	75	200	2296.05	1861.63	1356.00	2805.37	2309.55	1736.50	2825.00	2253.31	1639.20
LWA-WG-nS2.5	28	75	200	2359.07	2066.40	1506.48	2877.50	2542.95	1906.48	2915.32	2477.98	1876.19
LWA-WG-nS5.0	28	75	200	2427.94	2136.18	1662.40	2956.37	2622.62	2083.13	2984.68	2576.33	2052.93
LWA-nS0	28	100	200	3161.23	2539.73	2026.73	3873.40	3162.86	2579.62	3796.06	3154.52	2393.00
LWA-nS2.5	28	100	200	3515.67	2909.54	2158.68	4280.04	3585.22	2729.29	4175.76	3546.04	2576.06
LWA-nS5.0	28	100	200	3645.56	3055.01	2178.88	4429.27	3751.72	2752.23	4356.47	3690.54	2604.11
LWA-WG-nS0	28	100	200	3943.58	3160.65	2249.39	4772.05	3872.74	2832.33	4812.61	3795.48	2701.99
LWA-WG-nS2.5	28	100	200	4057.15	3529.69	2520.61	4902.81	4296.15	3141.06	4973.28	4195.27	3124.15
LWA-WG-nS5.0	28	100	200	4181.27	3655.46	2801.59	5045.78	4440.65	3461.79	5096.66	4370.24	3438.83
LWA-nS0	28	25	350	380.88	346.64	318.39	505.75	469.13	439.10	466.96	430.61	391.04

Table A1 continued

Mixture code	Age of curing (day)	D/t	f_y (MPa)	Predicated $N_{u,ACI}$			Predicated $N_{u,EC4}$			Predicated $N_{u,DL/T}$		
				w/b (0.25)	w/b (0.37)	w/b (0.50)	w/b (0.25)	w/b (0.37)	w/b (0.50)	w/b (0.25)	w/b (0.37)	w/b (0.50)
LWA-nS2.5	28	25	350	400.40	367.01	325.65	526.74	490.90	446.81	489.12	452.60	399.97
LWA-nS5.0	28	25	350	407.55	375.03	326.77	534.45	499.48	447.99	499.78	460.88	401.38
LWA-WG-nS0	28	25	350	423.97	380.84	330.65	552.17	505.72	452.11	526.92	466.93	406.38
LWA-WG-nS2.5	28	25	350	430.22	401.17	345.59	558.94	527.57	468.01	536.55	490.27	428.94
LWA-WG-nS5.0	28	25	350	437.06	408.10	361.07	566.34	535.04	484.53	543.97	500.59	446.51
LWA-nS0	28	50	350	1109.12	960.02	836.96	1431.24	1267.80	1134.01	1372.79	1215.50	1031.74
LWA-nS2.5	28	50	350	1194.15	1048.74	868.61	1525.00	1364.89	1168.31	1466.42	1311.33	1075.44
LWA-nS5.0	28	50	350	1225.31	1083.64	873.46	1559.45	1403.22	1173.57	1511.07	1346.83	1082.16
LWA-WG-nS0	28	50	350	1296.81	1108.98	890.37	1638.64	1431.09	1191.94	1623.98	1372.65	1105.71
LWA-WG-nS2.5	28	50	350	1324.05	1197.52	955.44	1668.89	1528.72	1262.80	1663.81	1471.23	1208.09
LWA-WG-nS5.0	28	50	350	1353.83	1227.69	1022.84	1701.96	1562.07	1336.50	1694.41	1514.47	1285.03
LWA-nS0	28	75	350	2175.92	1831.07	1546.42	2748.80	2364.90	2050.60	2668.80	2310.57	1887.85
LWA-nS2.5	28	75	350	2372.59	2036.27	1619.63	2968.96	2592.98	2131.18	2881.28	2529.05	1989.06
LWA-nS5.0	28	75	350	2444.66	2116.98	1630.84	3049.82	2682.99	2143.53	2982.48	2609.80	2004.60
LWA-WG-nS0	28	75	350	2610.02	2175.60	1669.97	3235.71	2748.45	2186.69	3238.09	2668.48	2058.87
LWA-WG-nS2.5	28	75	350	2673.04	2380.37	1820.46	3306.67	2977.68	2353.14	3328.18	2892.20	2293.64
LWA-WG-nS5.0	28	75	350	2741.91	2450.15	1976.37	3384.29	3055.99	2526.29	3397.37	2990.19	2469.17
LWA-nS0	28	100	350	3581.27	2959.77	2446.78	4461.34	3762.45	3189.96	4358.93	3719.52	2962.53
LWA-nS2.5	28	100	350	3935.72	3329.58	2578.72	4861.91	4177.71	3336.75	4737.75	4109.63	3144.18
LWA-nS5.0	28	100	350	4065.60	3475.06	2598.93	5009.01	4341.55	3359.26	4918.09	4253.70	3172.03
LWA-WG-nS0	28	100	350	4363.62	3580.70	2669.44	5347.07	4460.68	3437.86	5373.46	4358.36	3269.28
LWA-WG-nS2.5	28	100	350	4477.20	3949.74	2940.65	5476.10	4877.78	3741.03	5533.90	4757.21	3689.27
LWA-WG-nS5.0	28	100	350	4601.31	4075.50	3221.64	5617.21	5020.23	4056.31	5657.11	4931.84	4002.76
LWA-nS0	28	25	500	482.70	448.47	420.21	636.76	600.74	571.14	579.92	547.53	516.44
LWA-nS2.5	28	25	500	502.23	468.84	427.48	657.39	622.15	578.74	600.40	566.89	522.72

Table A1 continued

Mixture code	Age of curing (day)	D/t	f_y (MPa)	Predicated $N_{u,ACI}$			Predicated $N_{u,EC4}$			Predicated $N_{u,DL/T}$		
				w/b (0.25)	w/b (0.37)	w/b (0.50)	w/b (0.25)	w/b (0.37)	w/b (0.50)	w/b (0.25)	w/b (0.37)	w/b (0.50)
LWA-nS5.0	28	25	500	509.38	476.85	428.60	664.97	630.59	579.91	610.39	574.37	523.77
LWA-WG-nS0	28	25	500	525.80	482.67	432.48	682.38	636.73	583.97	636.09	579.89	527.58
LWA-WG-nS2.5	28	25	500	532.05	503.00	447.42	689.03	658.21	599.63	645.29	601.48	546.10
LWA-WG-nS5.0	28	25	500	538.89	509.93	462.90	696.30	665.55	615.89	652.39	611.15	561.45
LWA-nS0	28	50	500	1317.02	1167.92	1044.86	1708.53	1547.76	1416.11	1665.04	1511.24	1334.93
LWA-nS2.5	28	50	500	1402.05	1256.64	1076.51	1800.77	1643.26	1449.87	1757.20	1604.76	1376.30
LWA-nS5.0	28	50	500	1433.21	1291.54	1081.36	1834.67	1680.96	1455.05	1801.26	1639.55	1382.71
LWA-WG-nS0	28	50	500	1504.71	1316.88	1098.27	1912.62	1708.38	1473.12	1912.90	1664.90	1405.21
LWA-WG-nS2.5	28	50	500	1531.95	1405.42	1163.34	1942.39	1804.43	1542.83	1952.35	1761.95	1504.04
LWA-WG-nS5.0	28	50	500	1561.73	1435.59	1230.74	1974.95	1837.25	1615.33	1982.67	1804.62	1579.03
LWA-nS0	28	75	500	2489.89	2145.04	1860.39	3173.06	2794.83	2485.35	3134.35	2779.46	2363.90
LWA-nS2.5	28	75	500	2686.56	2350.24	1933.60	3390.11	3019.51	2564.68	3345.42	2995.73	2462.87
LWA-nS5.0	28	75	500	2758.63	2430.96	1944.81	3469.85	3108.20	2576.85	3446.05	3075.80	2478.10
LWA-WG-nS0	28	75	500	2923.99	2489.57	1983.94	3653.23	3172.71	2619.34	3700.45	3134.03	2531.37
LWA-WG-nS2.5	28	75	500	2987.02	2694.34	2134.43	3723.26	3398.71	2783.25	3790.17	3356.27	2762.74
LWA-WG-nS5.0	28	75	500	3055.88	2764.12	2290.34	3799.86	3475.94	2953.80	3859.10	3453.72	2936.39
LWA-nS0	28	100	500	4001.32	3379.81	2866.82	5032.74	4343.09	3778.71	4996.47	4360.33	3610.36
LWA-nS2.5	28	100	500	4355.76	3749.63	2998.77	5428.37	4752.78	3923.38	5373.91	4748.27	3789.81
LWA-nS5.0	28	100	500	4485.65	3895.10	3018.97	5573.71	4914.49	3945.57	5553.70	4891.68	3817.36
LWA-WG-nS0	28	100	500	4783.66	4000.74	3089.48	5907.84	5032.10	4023.05	6007.88	4995.90	3913.63
LWA-WG-nS2.5	28	100	500	4897.24	4369.78	3360.69	6035.42	5444.05	4321.96	6167.95	5393.31	4330.28
LWA-WG-nS5.0	28	100	500	5021.35	4495.54	3641.68	6174.96	5584.79	4632.97	6290.91	5567.40	4641.94
LWA-nS0	28	25	650	584.53	550.30	522.04	762.06	726.39	697.03	691.38	664.36	644.75
LWA-nS2.5	28	25	650	604.06	570.67	529.31	782.47	747.60	704.58	709.61	680.16	647.44
LWA-nS5.0	28	25	650	611.21	578.68	530.42	789.96	755.96	705.73	718.68	686.56	648.00

Table A1 continued

Mixture code	Age of curing (day)	D/t	f_y (MPa)	Predicated $N_{u,ACI}$			Predicated $N_{u,EC4}$			Predicated $N_{u,DL/T}$		
				w/b (0.25)	w/b (0.37)	w/b (0.50)	w/b (0.25)	w/b (0.37)	w/b (0.50)	w/b (0.25)	w/b (0.37)	w/b (0.50)
LWA-WG-nS0	28	25	650	627.63	584.50	534.31	807.17	762.03	709.77	742.43	691.35	650.20
LWA-WG-nS2.5	28	25	650	633.88	604.83	549.25	813.74	783.28	725.30	751.04	710.58	663.25
LWA-WG-nS5.0	28	25	650	640.72	611.76	564.72	820.93	790.53	741.40	757.72	719.38	675.59
LWA-nS0	28	50	650	1524.92	1375.82	1252.76	1975.77	1816.85	1686.59	1982.89	1833.80	1667.59
LWA-nS2.5	28	50	650	1609.95	1464.54	1284.41	2066.92	1911.27	1720.02	2073.07	1924.20	1705.80
LWA-nS5.0	28	50	650	1641.11	1499.44	1289.26	2100.41	1948.53	1725.14	2116.32	1958.04	1711.78
LWA-WG-nS0	28	50	650	1712.61	1524.78	1306.17	2177.42	1975.62	1743.03	2226.25	1982.76	1732.87
LWA-WG-nS2.5	28	50	650	1739.85	1613.32	1371.24	2206.83	2070.53	1811.98	2265.18	2077.73	1826.89
LWA-WG-nS5.0	28	50	650	1769.63	1643.49	1438.64	2239.01	2102.96	1883.67	2295.13	2119.63	1899.23
LWA-nS0	28	75	650	2803.86	2459.01	2174.36	3584.43	3210.48	2904.42	3650.28	3299.91	2894.03
LWA-nS2.5	28	75	650	3000.53	2664.21	2247.58	3799.05	3432.61	2982.90	3859.44	3513.18	2989.98
LWA-nS5.0	28	75	650	3072.60	2744.93	2258.79	3877.91	3520.30	2994.93	3959.31	3592.35	3004.80
LWA-WG-nS0	28	75	650	3237.97	2803.54	2297.91	4059.28	3584.08	3036.95	4212.06	3649.96	3056.71
LWA-WG-nS2.5	28	75	650	3300.99	3008.31	2448.40	4128.55	3807.55	3199.03	4301.29	3870.21	3283.46
LWA-WG-nS5.0	28	75	650	3369.85	3078.10	2604.31	4204.33	3883.92	3367.65	4369.86	3966.92	3454.59
LWA-nS0	28	100	650	4421.36	3799.86	3286.86	5589.27	4906.94	4348.70	5708.68	5076.97	4336.48
LWA-nS2.5	28	100	650	4775.80	4169.67	3418.81	5980.85	5312.24	4491.80	6084.25	5461.97	4512.97
LWA-nS5.0	28	100	650	4905.69	4315.14	3439.01	6124.74	5472.25	4513.74	6263.29	5604.49	4540.11
LWA-WG-nS0	28	100	650	5203.70	4420.78	3509.52	6455.60	5588.63	4590.38	6715.85	5708.11	4635.06
LWA-WG-nS2.5	28	100	650	5317.28	4789.82	3780.74	6581.95	5996.37	4886.04	6875.44	6103.56	5047.19
LWA-WG-nS5.0	28	100	650	5441.39	4915.58	4061.72	6720.16	6135.71	5193.71	6998.04	6276.93	5356.36
LWA-nS0	90	25	200	297.43	253.93	221.43	387.97	340.06	304.55	373.96	324.98	275.64
LWA-nS2.5	90	25	200	316.57	281.15	228.89	409.16	370.00	312.68	405.03	354.83	286.70
LWA-nS5.0	90	25	200	322.99	287.35	229.34	416.29	376.84	313.17	416.45	361.67	287.37
LWA-WG-nS0	90	25	200	344.04	301.56	243.12	439.68	392.54	328.22	442.28	380.40	310.64

Table A1 continued

Mixture code	Age of curing (day)	D/t	f_y (MPa)	Predicated $N_{u,ACI}$			Predicated $N_{u,EC4}$			Predicated $N_{u,DL/T}$		
				w/b (0.25)	w/b (0.37)	w/b (0.50)	w/b (0.25)	w/b (0.37)	w/b (0.50)	w/b (0.25)	w/b (0.37)	w/b (0.50)
LWA-WG-nS2.5	90	25	200	357.39	317.31	274.98	454.55	409.99	363.20	457.19	406.33	348.04
LWA-WG-nS5.0	90	25	200	365.28	333.80	294.66	463.34	428.29	384.91	466.00	430.86	369.74
LWA-nS0	90	50	200	981.27	791.82	650.28	1232.40	1019.33	861.39	1193.43	993.45	788.22
LWA-nS2.5	90	50	200	1064.64	910.36	682.77	1326.62	1152.47	897.52	1319.41	1115.56	834.79
LWA-nS5.0	90	50	200	1092.61	937.37	684.75	1358.29	1182.89	899.73	1365.63	1143.42	837.63
LWA-WG-nS0	90	50	200	1184.29	999.27	744.73	1462.25	1252.73	966.65	1470.00	1219.58	934.36
LWA-WG-nS2.5	90	50	200	1242.45	1067.87	883.50	1528.33	1330.29	1122.24	1530.14	1324.71	1087.87
LWA-WG-nS5.0	90	50	200	1276.80	1139.70	969.22	1567.38	1411.65	1218.81	1565.67	1423.89	1176.28
LWA-nS0	90	75	200	2047.09	1608.90	1281.55	2520.91	2022.46	1652.64	2450.58	1999.15	1534.53
LWA-nS2.5	90	75	200	2239.92	1883.09	1356.68	2741.17	2333.99	1737.28	2734.68	2274.90	1640.17
LWA-nS5.0	90	75	200	2304.62	1945.55	1361.26	2815.18	2405.13	1742.44	2838.87	2337.75	1646.59
LWA-WG-nS0	90	75	200	2516.67	2088.73	1500.00	3058.05	2568.43	1899.14	3074.06	2509.58	1865.57
LWA-WG-nS2.5	90	75	200	2651.20	2247.40	1820.95	3212.37	2749.73	2263.27	3209.58	2746.61	2212.39
LWA-WG-nS5.0	90	75	200	2730.63	2413.53	2019.22	3303.57	2939.85	2489.12	3289.61	2970.17	2411.89
LWA-nS0	90	100	200	3494.89	2705.19	2115.22	4256.18	3351.66	2679.96	4146.51	3343.09	2515.41
LWA-nS2.5	90	100	200	3842.41	3199.33	2250.63	4655.63	3917.07	2833.74	4651.95	3833.91	2703.71
LWA-nS5.0	90	100	200	3959.02	3311.90	2258.88	4789.82	4046.15	2843.11	4837.28	3945.75	2715.16
LWA-WG-nS0	90	100	200	4341.17	3569.94	2508.92	5230.09	4342.38	3127.73	5255.62	4251.49	3105.24
LWA-WG-nS2.5	90	100	200	4583.62	3855.90	3087.34	5509.77	4671.16	3788.74	5496.66	4673.18	3722.65
LWA-WG-nS5.0	90	100	200	4726.78	4155.29	3444.67	5675.02	5015.85	4198.51	5638.99	5070.83	4077.67
LWA-nS0	90	25	350	399.25	355.76	323.26	525.51	478.86	444.27	487.40	441.11	396.96
LWA-nS2.5	90	25	350	418.40	382.97	330.72	546.15	508.01	452.19	517.33	469.16	406.47
LWA-nS5.0	90	25	350	424.82	389.17	331.17	553.09	514.67	452.67	528.40	475.66	407.06
LWA-WG-nS0	90	25	350	445.87	403.39	344.95	575.88	529.96	467.33	553.55	493.58	427.90
LWA-WG-nS2.5	90	25	350	459.22	419.14	376.81	590.38	546.95	501.39	568.11	518.59	462.73

Table A1 continued

Mixture code	Age of curing (day)	D/t	f_y (MPa)	Predicated $N_{u,ACI}$			Predicated $N_{u,EC4}$			Predicated $N_{u,DL/T}$		
				w/b (0.25)	w/b (0.37)	w/b (0.50)	w/b (0.25)	w/b (0.37)	w/b (0.50)	w/b (0.25)	w/b (0.37)	w/b (0.50)
LWA-WG-nS5.0	90	25	350	467.11	435.63	396.49	598.96	564.79	522.53	576.74	542.42	483.36
LWA-nS0	90	50	350	1189.17	999.72	858.18	1519.49	1311.18	1157.00	1459.19	1261.58	1060.91
LWA-nS2.5	90	50	350	1272.54	1118.26	890.67	1611.73	1441.30	1192.26	1584.18	1382.11	1106.12
LWA-nS5.0	90	50	350	1300.51	1145.27	892.65	1642.75	1471.05	1194.41	1630.09	1409.67	1108.88
LWA-WG-nS0	90	50	350	1392.19	1207.17	952.63	1744.63	1539.39	1259.74	1733.85	1485.12	1203.48
LWA-WG-nS2.5	90	50	350	1450.35	1275.77	1091.40	1809.43	1615.32	1411.74	1793.70	1589.44	1354.73
LWA-WG-nS5.0	90	50	350	1484.70	1347.60	1177.12	1847.75	1695.04	1506.19	1829.06	1688.00	1442.20
LWA-nS0	90	75	350	2361.06	1922.88	1595.52	2956.03	2466.81	2104.61	2864.90	2415.74	1955.49
LWA-nS2.5	90	75	350	2553.89	2197.06	1670.65	3172.56	2772.43	2187.44	3148.03	2689.97	2059.82
LWA-nS5.0	90	75	350	2618.59	2259.52	1675.23	3245.36	2842.29	2192.49	3251.93	2752.53	2066.17
LWA-WG-nS0	90	75	350	2830.64	2402.70	1813.97	3484.39	3002.73	2345.95	3486.54	2923.68	2283.11
LWA-WG-nS2.5	90	75	350	2965.17	2561.38	2134.92	3636.38	3180.98	2703.01	3621.77	3159.93	2627.75
LWA-WG-nS5.0	90	75	350	3044.60	2727.50	2333.19	3726.23	3368.04	2924.80	3701.65	3382.89	2826.36
LWA-nS0	90	100	350	3914.94	3125.23	2535.26	4838.39	3948.01	3288.36	4708.55	3907.36	3083.96
LWA-nS2.5	90	100	350	4262.45	3619.37	2670.67	5232.23	4504.33	3439.24	5213.05	4396.69	3270.98
LWA-nS5.0	90	100	350	4379.06	3731.94	2678.92	5364.61	4631.46	3448.44	5398.10	4508.25	3282.37
LWA-WG-nS0	90	100	350	4761.21	3989.98	2928.96	5799.17	4923.35	3727.94	5815.87	4813.32	3670.44
LWA-WG-nS2.5	90	100	350	5003.66	4275.95	3507.38	6075.40	5247.54	4377.99	6056.63	5234.25	4285.73
LWA-WG-nS5.0	90	100	350	5146.83	4575.33	3864.71	6238.68	5587.67	4781.57	6198.81	5631.32	4639.87
LWA-nS0	90	25	500	501.08	457.59	425.09	656.18	610.31	576.24	598.80	556.67	520.53
LWA-nS2.5	90	25	500	520.22	484.80	432.55	676.47	638.98	584.04	626.97	581.93	527.65
LWA-nS5.0	90	25	500	526.65	491.00	433.00	683.29	645.52	584.52	637.50	587.91	528.11
LWA-WG-nS0	90	25	500	547.70	505.22	446.77	705.68	660.56	598.95	661.59	604.57	545.21
LWA-WG-nS2.5	90	25	500	561.05	520.97	478.63	719.93	677.26	632.47	675.63	628.17	576.05
LWA-WG-nS5.0	90	25	500	568.94	537.46	498.32	728.35	694.78	653.25	683.96	650.90	595.04

Table A1 continued

Mixture code	Age of curing (day)	D/t	f_y (MPa)	Predicated $N_{u,ACI}$			Predicated $N_{u,EC4}$			Predicated $N_{u,DL/T}$		
				w/b (0.25)	w/b (0.37)	w/b (0.50)	w/b (0.25)	w/b (0.37)	w/b (0.50)	w/b (0.25)	w/b (0.37)	w/b (0.50)
LWA-nS0	90	50	500	1397.07	1207.62	1066.08	1795.35	1590.43	1438.74	1750.08	1556.12	1362.50
LWA-nS2.5	90	50	500	1480.44	1326.16	1098.57	1886.13	1718.43	1473.44	1873.52	1674.20	1405.61
LWA-nS5.0	90	50	500	1508.41	1353.17	1100.55	1916.66	1747.69	1475.56	1918.96	1701.30	1408.25
LWA-WG-nS0	90	50	500	1600.09	1415.07	1160.53	2016.97	1814.93	1539.82	2021.79	1775.65	1499.56
LWA-WG-nS2.5	90	50	500	1658.25	1483.67	1299.30	2080.78	1889.66	1689.35	2081.17	1878.72	1647.31
LWA-WG-nS5.0	90	50	500	1692.60	1555.50	1385.02	2118.53	1968.13	1782.26	2116.28	1976.32	1733.34
LWA-nS0	90	75	500	2675.03	2236.85	1909.49	3377.36	2895.21	2538.53	3329.13	2883.48	2429.99
LWA-nS2.5	90	75	500	2867.86	2511.03	1984.62	3590.92	3196.35	2620.08	3610.79	3155.36	2532.31
LWA-nS5.0	90	75	500	2932.56	2573.49	1989.20	3662.75	3265.22	2625.06	3714.23	3217.49	2538.55
LWA-WG-nS0	90	75	500	3144.61	2716.67	2127.94	3898.67	3423.41	2776.17	3947.95	3387.57	2752.33
LWA-WG-nS2.5	90	75	500	3279.14	2875.35	2448.89	4048.75	3599.23	3127.93	4082.74	3622.63	3093.61
LWA-WG-nS5.0	90	75	500	3358.58	3041.47	2647.17	4137.50	3783.82	3346.56	4162.36	3844.67	3290.84
LWA-nS0	90	100	500	4334.98	3545.27	2955.31	5405.13	4526.13	3875.69	5344.81	4547.05	3730.27
LWA-nS2.5	90	100	500	4682.49	4039.42	3090.72	5794.32	5075.20	4024.41	5847.85	5034.07	3915.32
LWA-nS5.0	90	100	500	4799.10	4151.98	3098.96	5925.18	5200.73	4033.48	6032.46	5145.21	3926.59
LWA-WG-nS0	90	100	500	5181.25	4410.02	3349.00	6354.93	5489.07	4309.05	6449.35	5449.24	4311.58
LWA-WG-nS2.5	90	100	500	5423.71	4695.99	3927.43	6628.23	5809.45	4950.46	6689.68	5869.00	4923.57
LWA-WG-nS5.0	90	100	500	5566.87	4995.37	4284.76	6789.82	6145.74	5349.00	6831.62	6265.17	5276.36
LWA-nS0	90	25	650	602.91	559.41	526.92	781.27	735.87	702.09	708.16	671.65	646.36
LWA-nS2.5	90	25	650	622.05	586.63	534.38	801.33	764.25	709.84	733.94	693.14	650.25
LWA-nS5.0	90	25	650	628.48	592.83	534.83	808.07	770.73	710.31	743.75	698.41	650.53
LWA-WG-nS0	90	25	650	649.53	607.05	548.60	830.19	785.60	724.63	766.40	713.38	662.56
LWA-WG-nS2.5	90	25	650	662.88	622.80	580.46	844.26	802.11	757.81	779.73	735.06	688.01
LWA-WG-nS5.0	90	25	650	670.77	639.29	600.15	852.58	819.42	778.38	787.68	756.31	704.78
LWA-nS0	90	50	650	1604.97	1415.52	1273.98	2061.56	1859.04	1709.00	2066.08	1877.07	1692.98

Table A1 continued

Mixture code	Age of curing (day)	D/t	f_y (MPa)	Predicated $N_{u,ACI}$			Predicated $N_{u,EC4}$			Predicated $N_{u,DL/T}$		
				w/b (0.25)	w/b (0.37)	w/b (0.50)	w/b (0.25)	w/b (0.37)	w/b (0.50)	w/b (0.25)	w/b (0.37)	w/b (0.50)
LWA-nS2.5	90	50	650	1688.34	1534.06	1306.47	2151.25	1985.55	1743.34	2187.43	1991.83	1733.24
LWA-nS5.0	90	50	650	1716.31	1561.07	1308.45	2181.42	2014.47	1745.44	2232.22	2018.31	1735.73
LWA-WG-nS0	90	50	650	1807.99	1622.97	1368.43	2280.52	2080.90	1809.01	2333.79	2091.17	1822.59
LWA-WG-nS2.5	90	50	650	1866.15	1691.57	1507.20	2343.58	2154.74	1956.81	2392.55	2192.55	1965.59
LWA-WG-nS5.0	90	50	650	1900.50	1763.40	1592.92	2380.88	2232.27	2048.63	2427.32	2288.86	2049.67
LWA-nS0	90	75	650	2989.00	2550.82	2223.46	3786.44	3309.72	2957.03	3843.29	3402.38	2958.04
LWA-nS2.5	90	75	650	3181.83	2825.00	2298.60	3997.65	3607.46	3037.69	4122.94	3671.08	3057.63
LWA-nS5.0	90	75	650	3246.53	2887.47	2303.17	4068.69	3675.55	3042.61	4225.76	3732.62	3063.72
LWA-WG-nS0	90	75	650	3458.58	3030.64	2441.91	4302.10	3831.98	3192.03	4458.27	3901.27	3273.23
LWA-WG-nS2.5	90	75	650	3593.11	3189.32	2762.86	4450.60	4005.87	3539.81	4592.46	4134.71	3609.97
LWA-WG-nS5.0	90	75	650	3672.55	3355.44	2961.14	4538.44	4188.46	3755.98	4671.76	4355.50	3805.32
LWA-nS0	90	100	650	4755.02	3965.32	3375.35	5957.85	5088.01	4444.62	6055.28	5262.16	4454.35
LWA-nS2.5	90	100	650	5102.54	4459.46	3510.76	6343.17	5631.28	4591.72	6556.35	5746.07	4636.72
LWA-nS5.0	90	100	650	5219.15	4572.03	3519.01	6472.77	5755.53	4600.69	6740.35	5856.62	4647.85
LWA-WG-nS0	90	100	650	5601.30	4830.07	3769.05	6898.46	6040.94	4873.27	7156.07	6159.25	5028.65
LWA-WG-nS2.5	90	100	650	5843.75	5116.03	4347.47	7169.26	6358.16	5507.85	7395.80	6577.42	5636.19
LWA-WG-nS5.0	90	100	650	5986.91	5415.42	4704.80	7329.40	6691.22	5902.28	7537.42	6972.38	5987.14

PERSONAL INFORMATION

Full name: Oday Ali AZEZ

Nationality: Iraqi

Birth place and date: Baghdad, 1977

Marital status: Married, 2 children

Phone number: 5314992248

Email: oday.azez@gantep.edu.tr, dr.odayaltaie@hotmail.com

EDUCATION

<u>Degree</u>	<u>Graduate school</u>	<u>Year</u>
Master	Building & Construction Engineering Dep., University of Technology, Baghdad, Iraq	2005
Bachelor	Building & Construction Engineering Dep., University of Technology, Baghdad, Iraq	2000
High school	Algeihadd for boys	1996

WORK EXPERIENCE

2006-2009	Assistant lecturer	Civil Engineering Dep., Tikrit University, Saladin, Iraq
2009-Present	Lecturer	Civil Engineering Dep., Kirkuk University, Kirkuk, Iraq

PUBLICATIONS

A. International Journals

- [1] Erhan Güneyisi, Mehmet Gesoğlu, **Oday Ali Azez**, H. Öznur Öz, (2015) “Physico-mechanical properties of self-compacting concrete containing treated cold-bonded fly ash lightweight aggregates and SiO₂ nano-particles” Construction and Building Materials, Vol. 101 Iss. 1 pp. 1142-1153.
- [2] Erhan Güneyisi, Mehmet Gesoğlu, **Oday Ali Azez**, H. Öznur Öz, (2016) “Effect of nano silica on the workability of self-compacting concretes having untreated

and surface treated lightweight aggregates” Construction and Building Materials Vol. 115 pp. 371–380.

B. National Journals and Conference

- [1] **Oday Ali Azez** (2012) “Re-tempering effect on concrete properties” International Review of Civil Engineering, Vol. 3 Iss. 1 pp. 59-66.
- [2] Waleed M. Sh. Alabdraba, **Oday Ali Azez**, Bakhtiar A. Muhaidin, (2012) “Studying the properties of concrete containing natural bio productive EM1” Tikrit Journal of Engineering Sciences, Vol. 19 Iss. 3 pp.1-13.
- [3] Hussein Hameed Karim, Hisham K. Ahmed, **Oday Ali Azez**, (2012) “Using quartzite rocks for manufacturing refractory silica bricks” Implementation of postgraduate's researches to serve the Society, at abstracts of research engineering conference comprehensive research theses (7-8 May 2012,), Al-Mustansiriya University, College of Engineering, Baghdad, Iraq.
- [4] Hussein Hameed Karim, Hisham K. Ahmed, **Oday Ali Azez**, (2012) “Using quartzite rocks for manufacturing refractory silica bricks to resist concentrated acids” Implementation of postgraduate's researches to serve the Society, at abstracts of research engineering conference comprehensive research theses (7-8 May 2012,), Al- Mustansiriya University, College of Engineering, Baghdad, Iraq.

FOREIGN LANGUAGE

Arabic (Mother language), English, and Turkish

HOBBIES

Photography, Traveling, Reading