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Contribution of refractory brick aggregates to improving the behavior of concrete subjected to high temperature

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Abstract

Today, the use of recycled aggregates as a substitute for natural aggregates in concrete production is an important process for sustainable concrete. Moreover, one of the most important processes of physical deterioration in the concrete structures is the exposure to high temperatures that influences their durability and stability during service life.

This study aims, firstly, to investigate the possibility of using recycled refractory bricks (RBA) as coarse aggregate for concrete, and secondly, to find the ideal replacement percentage of coarse Natural Aggregate (NA) by RBA. Two types of RBA from two different sources were used, RBA-1 was obtained from the grinding of new refractory bricks and RBA-2 was obtained from refractory bricks used in the furnace recovered from the cement plant.

When the optimum replacement percentage of coarse NA by RBA was finding, three types of concrete formulations were prepared: concrete made with 20% of coarse RBA-1, concrete made with 20% of coarse RBA-2 and concrete made with 20% of coarse and fine RBA-2. For each of the mentioned concrete families, three water/cement ratios (w/c = 0.59, 0.47 and 0.38) were used. All studied concretes were subjected to heating-cooling cycles of 150°C, 250°C, 400°C, 600°C and 800°C. After cooling down to ambient temperature, the following properties were evaluated and compared with conventional concretes made with 100% of coarse and fine NA: density, porosity, mass loss, change of volume, ultrasonic pulse velocity, damage degree, compressive strength and dynamic elastic modulus.

Based on the experimental results obtained, it can be concluded that RBA is a potential alternative for natural aggregates in concrete manufacturing. Furthermore, concretes made with RBA show the best performance when subjected to high temperatures.

Keyword : refractory brick aggregates, recycled aggregates, high temperature, damage degree, dynamic elastic modulus, physical properties.

ملخص

في الوقت الحالي، يعد استخدام الركام المعاد تدويره كبديل للركام الطبيعي في إنتاج الخرسانة عملية مهمة للخرسانة المستدامة. علاوة على ذلك، فإن إحدى أهم عمليات التدهور المادي في الهياكل الخرسانية هي التعرض لدرجات حرارة عالية تؤثر على متانتها واستقرارها خلال فترة الخدمة.

تهدف هذه الدراسة، أولاً، إلى التحقق من إمكانية استخدام الطوب الحراري المعاد تدويره (RBA) كركام خشن للخرسانة، وثانيًا، لإيجاد نسبة الاستبدال المثالية للركام الطبيعي الخشن بواسطة (RBA). في هذه الدراسة، تم استخدام نوعين من RBA من مصدرين مختلفين، النوع الأول (RBA-1) تم الحصول عليه من تكسير الطوب الحراري الجديد والنوع الثاني (RBA-2) الذي تم الحصول عليه من الطوب الحراري المستخلص من مصنع الأسمنت بعد استخدامه في الفرن.

عندما تم إيجاد النسبة المئوية المثلى لاستبدال الركام الطبيعي الخشن بواسطة RBA، تم تحضير ثلاثة أنواع من التركيبات الخرسانية: الخرسانة مصنوعة من 20٪ من الركام الخشن RBA-1، الخرسانة مصنوعة من 20٪ من الركام الخشن RBA-2 والخرسانة المصنوعة من 20٪ من الركام الخشن والناعم RBA-2. لكل من العائلات المذكورة أعلاه، تم فحص ثلاث نسب ماء / أسمنت (ماء / أسمنت = 0.59، 0.47 و0.38). تم تعريض جميع الخرسانات المدروسة لدورات تسخين-تبريد إلى 150 درجة مئوية، 250 درجة مئوية، 400 درجة مئوية، 600 درجة مئوية و800 درجة مئوية. بعد التبريد إلى درجة الحرارة المحيطة، تم تقييم الخصائص التالية ومقارنتها بالخرسانة التقليدية المصنوعة بنسبة 100٪ من الركام الطبيعي الخشن والناعم: الكثافة، والمسامية، وفقدان الكتلة، وتغير الحجم، وسرعة النبض فوق الصوتي، ودرجة الضرر، وقوة الانضغاط، ومعامل المرونة الديناميكي.

بناءً على نتائج الدراسة التجريبية التي تم الحصول عليها، يمكن استنتاج أن RBA هو بديل محتمل للركام الطبيعي في تصنيع الخرسانة، وتظهر الخرسانة المصنوعة من RBA أداء أفضل في درجات حرارة عالية.

الكلمات المفتاحية: ركام الطوب الحراري، الركام المعاد تدويره، درجة الحرارة العالية، درجة الضرر، معامل المرونة الديناميكي، الخائص الفيزيائية.

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LIST OF ABBREVIATIONS

NA	**N**atural **A**ggregates
RBA	**R**ecycled **B**rick **A**ggregates
RBA-1	First type of recycled brick aggregates
RBA-2	Second type of recycled brick aggregates
CSH	**C**alcium **S**ilicate **H**ydrate
$Ca(OH)_2$/ CH	**C**alcium **H**ydroxide / Portlandite
ITZ	**I**nterfacial **T**ransition **Z**one
SEM	**S**canning **E**lectron **M**icroscopy
w/c	**W**ater/**C**ement
UPV	**U**ltrasonic **P**ulse **V**elocity
C1NA, C2NA and C3NA	**C**oncretes made with 100% of coarse and fine **N**atural **A**ggregates
C1-RBA1, C2-RBA1 and C3-RBA1	**C**oncretes made with 20% of coarse **R**ecycled **B**rick **A**ggregates (First type of RBA)
C1-RBA2, C2-RBA2and C3-RBA2	**C**oncretes made with 20% coarse **Recycled B**rick **A**ggregates (Second type of RBA).
C1'-RBA2, C2'RBA2 and C3'-RBA2	**C**oncretes made with 20% of coarse and fine **Recycled B**rick **A**ggregates (Second type of RBA).
f_s	Compressive strength
F_{max}	The max load applied
ν	The pulse velocity
L	The length of the sample
D	The damage degree
ν_T	The UPV after heating
ν_0	The UPV before heating
E_d	The dynamic modulus of elasticity
v	Poisson's ratio
M_1	The mass of the specimen before heating
M_0	The mass of the specimen after heating
V_1	The volume of the specimen before heating
V_0	The volume of the specimen after heating
P	Water porosity
Ms	Mass dried
Mw	Mass saturated
Mw'	Hydrostatic balance

INTRODUCTION

INTRODUCTION

1. Background

Concrete, owing to its availability, easy preparation and fabrication, is the most popular construction material. Normal concrete consists of coarse and fine aggregates, cement and water. Aggregates normally occupy 70% - 80% of concrete volume; therefore, the consumption of aggregates on such large scale can leads to depletion of natural resources. Moreover, the amount of construction and demolition waste has increased proportionately with population growth and continuous industrial development.

To reduce environmental burdens as well as to contribute to sustainable development of the construction industry, recycling construction and demolition waste as a new aggregates source for concrete products is an important step forward to a sustainable construction. Refractory brick is among of these wastes.

With the deepening of research and the practical needs of engineering applications, it is necessary to carry out research on physical and mechanical properties of concrete produced with recycled aggregates after being exposed to fire. In fact, the fire is a multi-risk and extremely dangerous disaster.

2. Problem statement

Refractory bricks are mainly used especially within the furnaces basin such as cement, glass and ceramics. When refractory bricks have served their service life, they will be demolished and replaced with a new one. In this regard, about 28 million tons of spent refractory wastes are generated each year [Horckmansa et al. 2019]. Therefore, it is interesting to investigate the potential of using Recycled Refractory Brick as coarse or/and fine aggregates to produce concrete.

High temperatures, in general, cause deterioration in physical and mechanical properties of concrete. The type of aggregates is an important factor affecting the performance of concrete at elevated temperatures. Thus, the performance at high temperature of concrete containing refractory brick aggregates RBA needs to be evaluated.

3. Objectives

The main objectives of this thesis can be summarized as follows:

- To achieve the possibility of using RBA as coarse aggregates for manufacturing concrete.
- To find the optimum replacement percentage of coarse Natural Aggregate (NA) by RBA that can produces concrete with acceptable properties.
- Evaluate the performance of concrete produced with recycled refractory brick as coarse or/and fine aggregates after exposure to elevated temperature.

4. Significance of the study

This study aims to provide important information on the performance of concrete containing RBA and encourages their application. Most importantly, the use of this waste as aggregates is environmentally and economically important and helps to reduce the use of natural aggregates. Specifically, this study offers emphasizes the important influence of RBA on the physical and mechanical properties of concrete after exposure to high temperatures.

5. Methodology

The methodology used to satisfy the objectives of the present study is as follows:

a. Literature review

An extensive survey was performed about all available topics related to recycling refractory brick wastes in order to improve a background about using this waste in manufacturing concrete.

The literature review concentrates on the following points:

- Historical uses and application of RBA in concrete at room temperature and after exposure to high temperature.
- Experimental tests were performed on recycling RBA to check the ability of using RBA in concrete production.
- Advantages and disadvantages of using RBA in concrete.

b. Recycled refractory brick collection

In this study, two types of RBA were employed (RBA-1 and RBA-2). The first type (RBA-1) was obtained from crushing news refractory bricks. The second type (RBA-2) was obtained by crushing the refractory bricks waste generated from the cement furnace.

c. Aggregates tests at room and high temperature

Experimental tests were performed on NA and RBA samples at room and high temperatures. The results of physical properties allow comparing and deducing the difference between the two types of aggregates.

d. Preparing concrete specimens

Concrete specimens were prepared using different percentage of coarse RBA and different cement dosages (different w/c ratio). The replacement levels by volume of the coarse RBA were taken as 0%, 10%, 20%, 30%, 40%, 50%, 70% and 100%.

After finding the optimum ratio of coarse RBA replacement (20%), other concrete mixtures were prepared. The first mixture was made with replacing NA by 20% of coarse RBA and the second mixture was made with replacing NA by 20% of coarse and fine RBA-2.

e. Experimental tests on concrete at room temperature and after exposed to high temperature

Standard tests were performed to find the mechanical and physical properties of concrete produced. Comparisons between concrete samples produced by different percentage of RBA at room temperature were performed to check the effect of using coarse RBA in concrete properties. This comparison was also done between three families of mixes at room temperature and after exposed to high temperature: the first mixes were made with 100% coarse and fine NA. The second mixes were made by replacing 20% of coarse NA by coarse RBA and the third mixes was made by replacing 20% of coarse and fine NA by coarse and fine RBA-2. The experimental procedures at room temperature and after exposed to high temperature are outlined in figure 1 and 2, respectively.

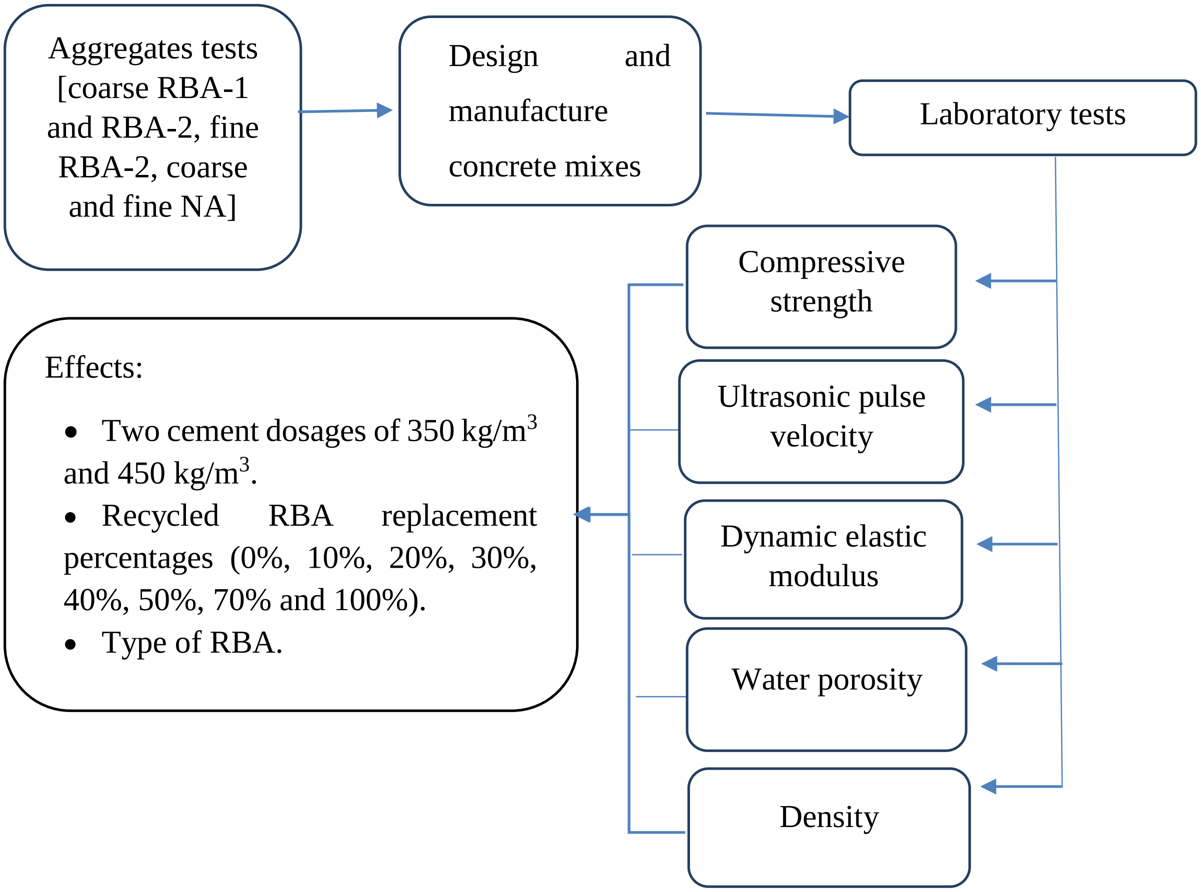


Figure 1. Experimental study programme at room temperature.

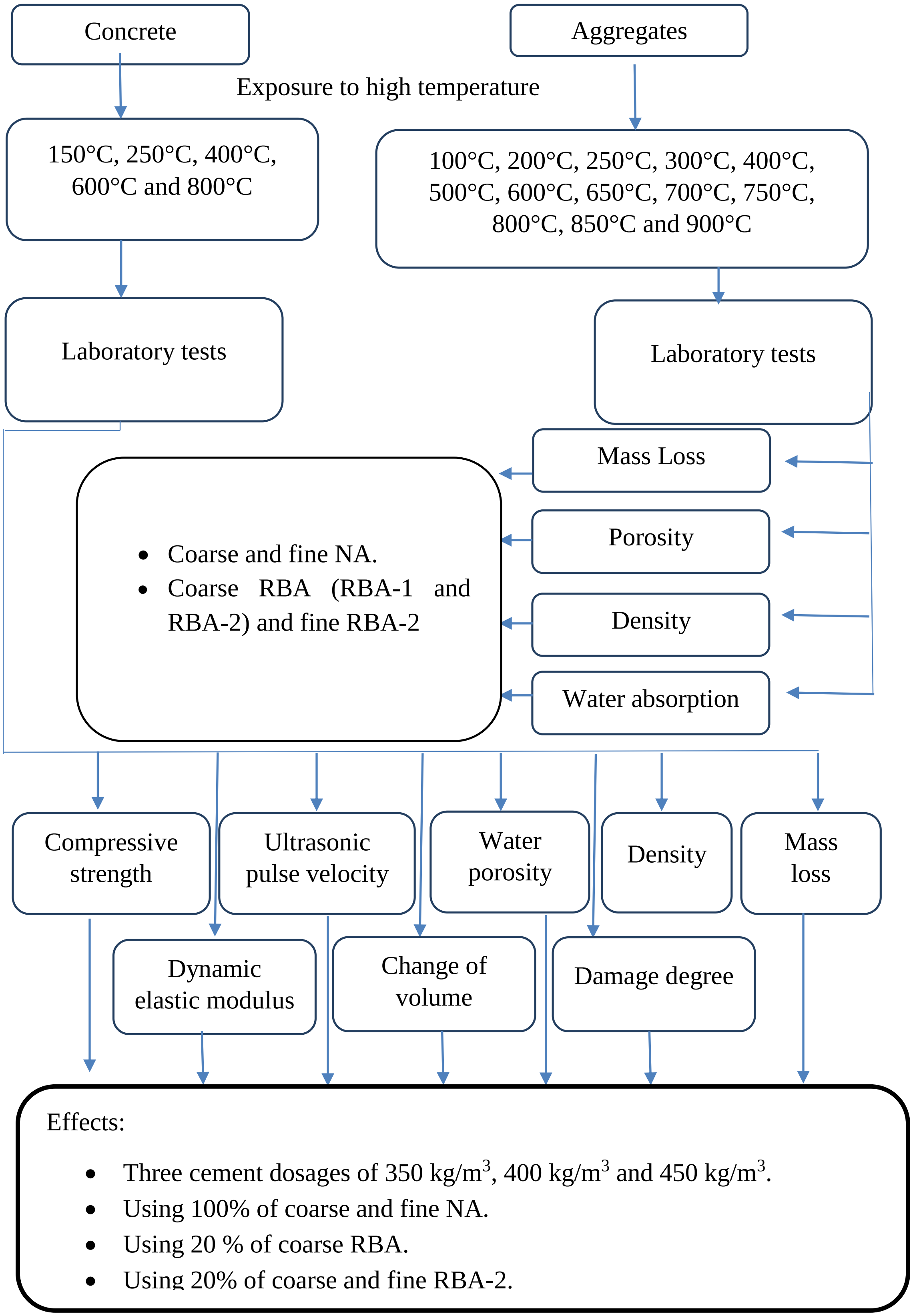


Figure 2. Experimental study programme at after exposed to high temperature.

6. Thesis organization

Thesis structure's is divided into **4** chapters; each chapter covers a certain area as follows:

Chapter I, presents a detailed literature review on concrete components: cement paste, aggregates and water. This chapter also covers a comprehensive review of the literature pertinent to the study of the physical, thermal and mechanical properties of concrete at high temperature. Finally, the behavior of concrete containing recycled refractory brick at different conditions is presented in details.

Chapter II, this chapter provides details of the experimental program conducted. A series of tests were carried out to evaluate the material used (cement, calcareous aggregates, siliceous sand and recycled refractory brick aggregates). This chapter also describes the mix proportions, preparation, curing conditions and testing of specimen's concrete. Additionally, measurement tools and heating equipment were discussed.

Chapter III, this chapter presents series of test results of this study. The first part of this chapter presents the physical and mechanical properties of concrete made with partial and total replacement of coarse NA by coarse RBA. The second part provides the physical properties of aggregates used (coarse NA, fine NA, coarseRBA-1 and RBA-2, and fine RBA-2) after heating. The third part presents the effect of high temperature on the performance of concrete having different characteristics: first, those made with coarse NA, second, those made by replacing 20% of coarse NA by coarse RBA, and third, those made by replacing 20% of coarse and fine NA by coarse and fine RBA. Compressive strength, dynamic elastic modulus, ultrasonic pulse velocity, damage degree, mass loss, porosity, density, and volume changes were evaluated.

Chapter IV, the first part of this chapter presents the compare the behavior of concrete having different compositions (concretes made with 100% of coarse and fine NA, concretes made with 20% of coarse RBA and concretes made with 20% of coarse and fine RBA-2) after exposure to high temperature. The second part, presents a thorough visual examination for concrete after heating and compared with the unheated specimens. The third part presents the relation between the physical and mechanical properties of different concretes after heating.

Chapter I: LITERATURE REVIEW

I.1 INTRODUCTION

Concrete is one of the most used materials in the world in the field of construction such as buildings, infrastructure, bridge, etc. The reason is due to its ease of formation, their strength increases over time and the main concrete components are widely available and inexpensive. Normal concrete consists of coarse and fine aggregates, cement, water and possibly additives. In this chapter, description of the concrete compounds and behaviour of concrete after exposed to high temperatures are discussed based on previous experimental observations. This chapter also presents a detailed summary of concrete containing recycled refractory brick when subjected to different conditions.

I.2 GENERAL INFORMATION ABOUT THE STRUCTURE OF CONCRETE

I.2.1 Cement

Cement can be described as adhesive and cohesive material, the cement commonly used in civil engineering is Portland cement, and the raw materials required for the manufacture of Portland cement are calcareous materials, such as limestone or chalk, and shale or clay so that it contains alumina and silica. Figure I.1 show the stages of cement production.

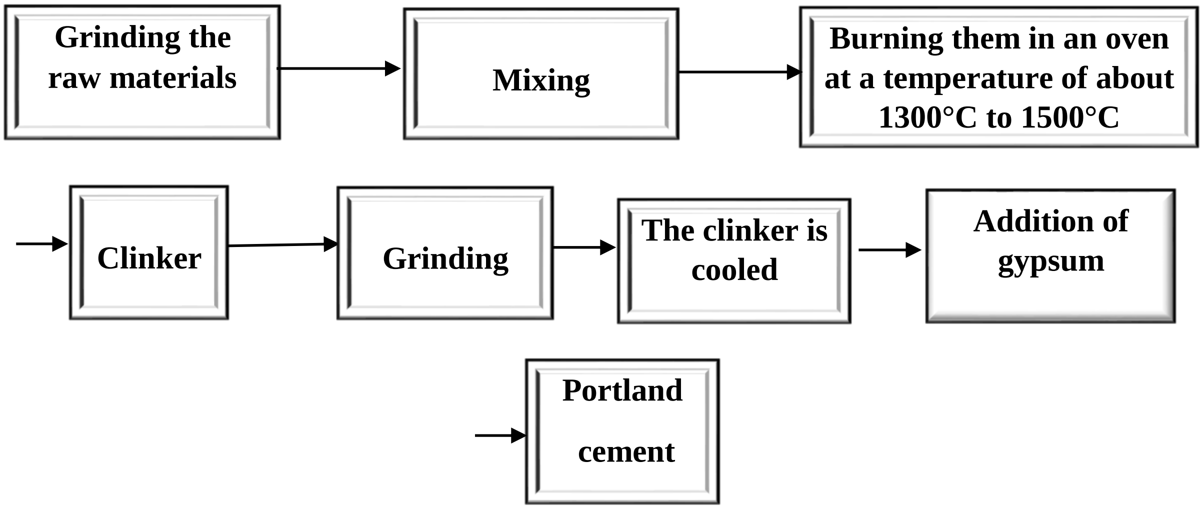


Figure I.1. Diagram of Portland cement production [Neville, 1987].

I.2.1.1 Chemical composition

The major compounds of Portland cement are listed in Table I. They are C3S, C2S, C3A, and C4AF. C3S is called tricalcium silicate ; C2S, dicalcium silicate ; C3A, tricalcium aluminate ; and C4AF, Tetracalcium aluminoferrite. In addition, there are a small amount of free calcium oxide, magnesium oxide, and alkali in cement the total amount should not be more than 10%.

Table I.1. Chemical composition of Portland cement [Zhang, 2011].

Name of compound	Oxide Composition	Abbreviation	Proportion
Tricalcium silicate	$3CaO.SiO_2$	C_3S	37%-60%
Dicalcium silicate	$2CaO.SiO_2$	C_2S	15%-37%
Tricalcium aluminate	$3CaO.Al2O_3$	C_3A	07%-15%
Tetracalcium aluminoferrite	$4CaO.Al2O_3.Fe2O_3$	C_4AF	10%-18%

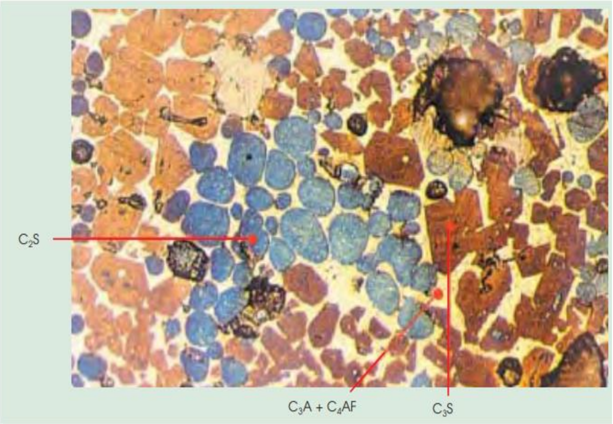


Figure I.2. Schematic portrayal of compounds in clinker [Shetty, 2005].

I.2.1.2 Hydration reactions

Chemical reactions between cement and water are indicated as cement hydration, the chemistry of the reaction between cement and water are the chemistry of concrete. The quantity, quality, stability and continuity of the hydration products play an important role in the properties of concrete. Cement compounds interact with each other when mixed with water, to form hydrated compounds **[**Shetty, 2005**]**.

- **Hydration of tricalcium silicate**

The reaction for the hydration of tricalcium silicate is given by:

Tricalcium silicate + water→ Calcium silicate hydrate + Calcium hydroxide +Heat

$$2\ (CaO.SiO2) + 6H_2O \rightarrow 3CaO.2\ SiO_2.\ 3H_2O + 3Ca\ (OH)_2 + 173.6kJ$$

- **Calcium Silicate Hydrate (CSH)**

CSH is considered one of the most important products and it is responsible for early strength of concrete. Moreover, it makes up 50-60 % of the volume of cement paste and determines the good mechanical properties of concrete, see figure I.3-a.

- **Calcium Hydroxide (Portlandite, Ca (OH) $_2$)**

It accounts for 20 to 25% of the volume of cement paste. It is an undesirable product because it reduces the durability of concrete due to its solubility in water which makes concrete more porous, see figure I.3-b.

- **Hydration of dicalcium silicate**

Dicalcium silicate reacts with water in a similar manner compared to tricalcium silicate, but much more slowly.

$$2\ (2\ CaO.SiO_2) + 4H_2O \rightarrow 3CaO.2SiO_2.\ 3H_2O + Ca\ (OH)_2 + 58.6\ kJ$$

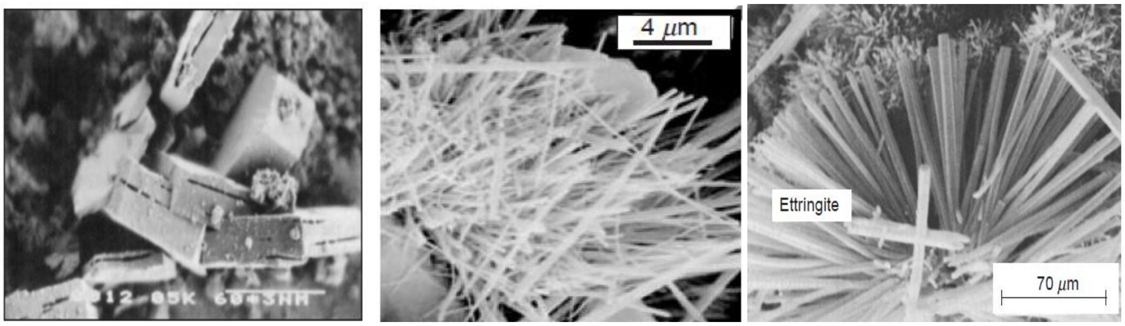
C_2S produces relatively more of (CSH) and less quantity of Ca $(OH)_2$ than that formed in C_3S hydration. It is responsible for the later strength of concrete.

- **Hydration of tricalcium aluminate**

The hydrated aluminates contribute a little to the early resistance, the reaction of C_3A with water is very rapid. Therefore, a large quantity of heat is released during hydration, with the addition of gypsum; rapid hydration of C_3A is slowed down.

- **Tetracalcium aluminoferrite**

The hydrates of C_4AF can resist the attack of the sulphates more than the hydrates of calcium, and does not contribute to the resistance (see figure I.4). When C_3A and C_4AF react with water in the presence of gypsum, the products produced are structurally similar and a hydrate of calcium aluminate trisulphate (Ettringite $C_6AS_3H_{32}$) is formed, see figure I.3-c.



(a) (b) (c)

Figure I.3. Image of hydrated cement: (a) calcium silicate hydrate (b) calcium hydroxide (c) crystal of ettringite **[**Shetty, 2005**]**.

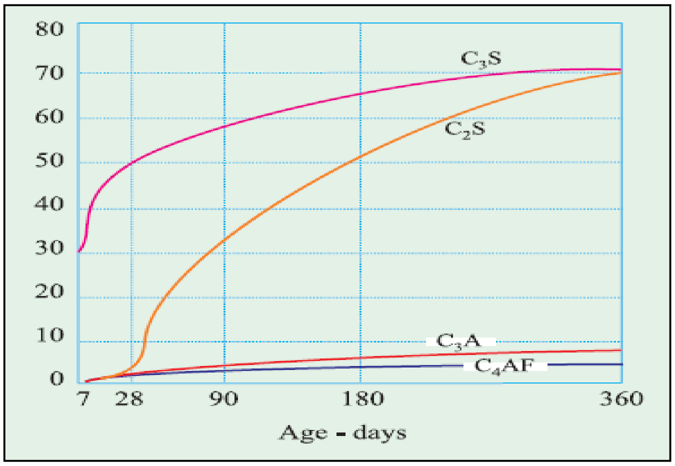


Figure I.4. Development of strength pure compounds **[**Shetty, 2005**]**.

I.2.2 Aggregates

Aggregates are occupied by between 70% and 80% of concrete volume and therefore its influence on the various properties of concrete is undoubtedly significant. The zone between cement paste and aggregates referred to as the interfacial transition zone (ITZ), and the thickness of this zone is about 50 µm. The ITZ between cement paste and aggregates are summarized in figure I.5.

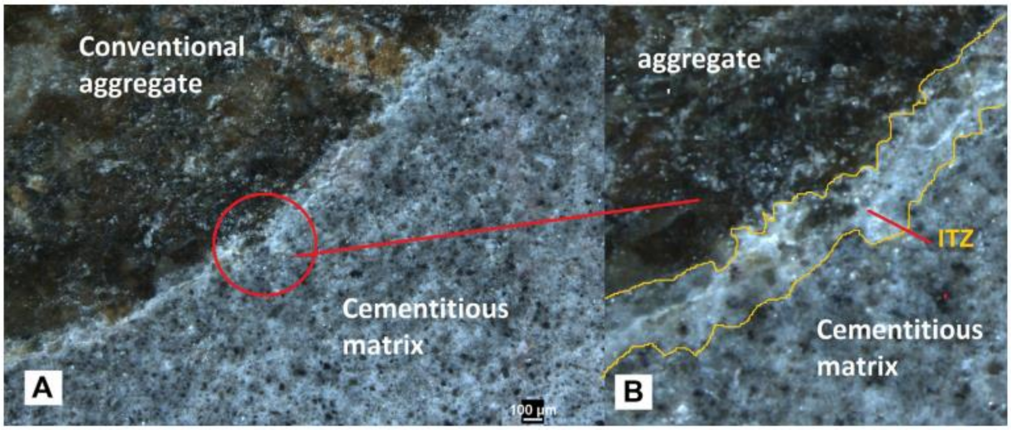


Figure I.5. The interfacial transition zone between cement paste and aggregates [Vargas et al., 2017].

The thickness of ITZ depends on the w/c ratio and curing age. In the case of high w/c, a thicker (ITZ) is generated, which leads to an increase in the porosity of the ITZ. As a results, the compressive strength of concrete decreases. In the case of low w/c, the thickness of ITZ becomes lower and more compact. On the other hand, the curing age has an effect on the thickness of ITZ, when the curing age increases from 56 days to 90 days, the thickness of ITZ slightly decreases compared to the age of curing at 28 days [Vargas et al., 2017], [Gao et al., 2014]. Furthermore, Tasong [Tasong et al.,1999] found that the porosity of ITZ depends on the natural of aggregates. The authors stated that the limestone aggregates produced a porous ITZ zone compared to the quartzite and basalt aggregates.

I.2.3 Water

Water is considered an important component in chemical reactions and contributes to the formation and growth of cement hydration products, as well as to the workability of the concrete mixture. The basic quality requirements for water used in concrete should not contain any impurities which affect the normal setting, hardening of cement and deterioration of strength and durability of concrete.

I.2.3.1 Water cases in concrete paste

- ***Capillary water:*** The capillary water is present in voids greater than about 50 Å, and this water can be sub-divided into two categories:

1- The water in large capillaries, which may be called free water (>50 ɳm).

2- Water in small capillaries (5 to 50 ɳm).

- ***Adsorbed water:*** This form of water is associated with gel particles under the influence of Van Der Waal forces. When the hydrated cement paste is dried, a large part of the absorbed water is lost, causing shrinkage in the hydrated cement paste.
- ***Interlayer water:*** This water is associated with the CSH structure by the hydrogen bond. When the interlayer water is lost, the CSH structure shrinks dramatically.
- ***Chemically combined water:*** This water is part of the microstructure of different cement hydration products. Different types of water associated with the CSH are illustrated in Figure I.6.

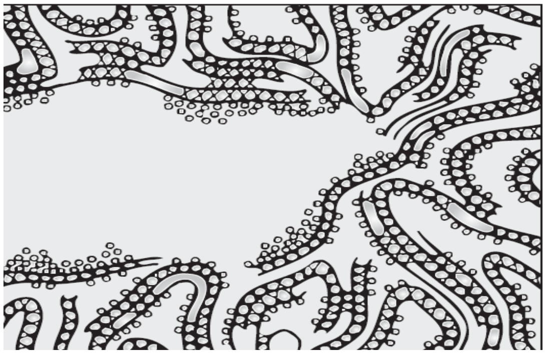


Figure I.6. Diagrammatic model of the types of water associated with the CSH hydrate [Mehta and Monteiro, 2006].

I.2.3.2 Porous structure of concrete

Concrete is considered a porous substance so that it contains a network of internal pores of different shapes and sizes. The shape and size of the pores vary depending on the type of concrete, the aggregates used and the w/c ratio. Porous structure plays an important role in the properties of concrete. When the pores are connected, the concrete is likely to interact with the external environment, which in turn leads to the deterioration of the concrete [Alonso et al., 2003-a].

- ***Pores of CSH gel:*** These pores are very small and do not affect negatively on the properties of cement paste, but they influence the shrinkage and creep. The width of the interlaminar spacing of the CSH gel varies from 5 Å to 25 Å and constitutes around 25 % of the total volume of gel.

- ***Capillary pores:*** They are the most important of the net porous microstructure of the concrete. The capillary porosity plays an important role in the durability of concrete, and it is depending on the w/c ratio. In the case of a low w/c ratio, the size of the capillary pores ranges from 10 to 50 ηm, and up to 5 µm in the case of a high ratio w/c.
- ***Air pores:*** This type of pores with a size >5 µm has a spherical shape. Moreover, the reason for its presence in concrete is due to two reasons.

1. Air pores formed during mixing, their size has even 3 mm.
2. Those that are generated by using the air-entraining additives in order to improve the performance of concrete; for example, to make it more resistant to freezing.

I.3 BEHAVIOR OF CONCRETE STRUCTURES DURING FIRE

Fire is one of the important disasters to destroy building structures. Concrete is an important material used in the construction of buildings. Concrete structures can experience fires during their service lifetime, exposing concrete to extremely high temperatures.

Figure I.7 shows the effect of fire on the “Address Downtown Dubai hotel”. Since New Year’s Eve 2016, an electrical short circuit set the 63-storey was tower ablaze.

(a) (b)

Figure I.7. (a) Address downtown hotel during the fire, (b) Address downtown hotel after the fire.

There are some cases of fire in the tunnels, such as the great belt tunnel fire in Denmark in 1994 and the channel tunnel fire linking from France and the United Kingdom in 1996. In the great belt tunnel, the oil spill from the tunneling machine caused the fire, resulting in temperatures of between 800°C -1000°C for 7 hours (see figure I.8). On the other hand, the channel tunnel fire was subjected to a fire for about 9 hours and a maximum temperature of about 1100°C [Khoury, 2003-a].

Figure I.8. Concrete spalling in the great belt tunnel fire (1994) [Khoury, 2003-a].

There are major factors that may contribute to the deterioration of properties of concrete under high temperatures have been identified as phase transformations taking place in cement paste, phase transformations taking place in aggregate, thermal incompatibility between the cement paste and aggregates and spalling of concrete [Willam et al., 2009].

I.3.1 Effects of high temperatures on concrete components

I.3.1.1 Effect of high temperature on hydrated cement paste

As mentioned before, water contributes to the formation and growth of cement hydration products. The main hydration products of common Portland cements include CH, CSH and ettringite. Under the effect of high temperatures, changes in the chemical composition of hydration products due to the removal of water lead to degradation of the mechanical and physical properties of concrete. Table I.2 presents the transformations in cement paste.

Table I.2. Decomposition of cement paste at different temperatures.

Temperature (ºC)	Transformations
(30 -120)°C	Loss of water (free and some adsorbed water) [Noumowé, 1995]
(120-300) °C	First stage of dehydration of hydrated calcium silicates (CSH). [Noumowé et al.,1996],
(450 -500)°C	Calcium hydroxide [$Ca(OH)_2$] decomposes and produces lime [CaO] and water [H_2O] [Divet et al.,2005].
(600 -700)°C	Second peak of (CSH) decomposition [Alonso et al., 2003-b].
(700 – 900)°C	De-carbonation of $CaCO_3$ into CaO and CO_2 [$CaCO_3 \Rightarrow CaO + CO_2$]($CaCO_3$ coming from the clinker) [Schneider et al.,1982], [Divet et al., 2005].
<900°C	Complete decomposition of the CSH [Mehta and Monteiro, 2006].

The neutron diffraction technique makes it possible to study the transformations of the cement phases during heating. Figure I.9 shows the dehydration processes of cement paste exposed to high temperature.

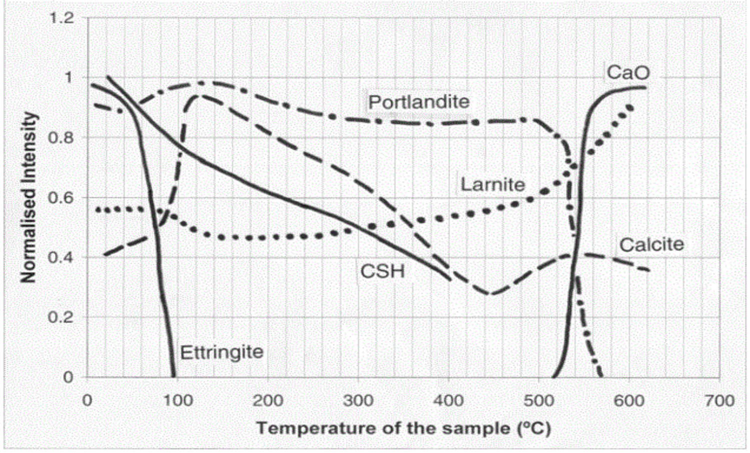


Figure I.9. Evolution of the cement paste during heating. [Alonso et al., 2003-b].

From Figure I.9, it can be observed that:

- Ettringite disappears completely before reaching 100ºC, and the calcite increases at the same time [Alonso et al., 2003], [Alarcon-Ruiz et al., 2005].

- Dehydration of the CSH gel is gradual and begins before 100°C. The dehydration observed at this temperature is owing to the evaporation of water chemically [Alonso et al., 2003-b], [Alarcon-Ruiz et al., 2005].

- Dehydration for CSH starts before 100°C and continues up to 400°C. The CSH gel content decreases with increasing temperature [Castellote et al., 2004], [Alonso et Fernandez, 2004], [Pimienta et al., 2017].

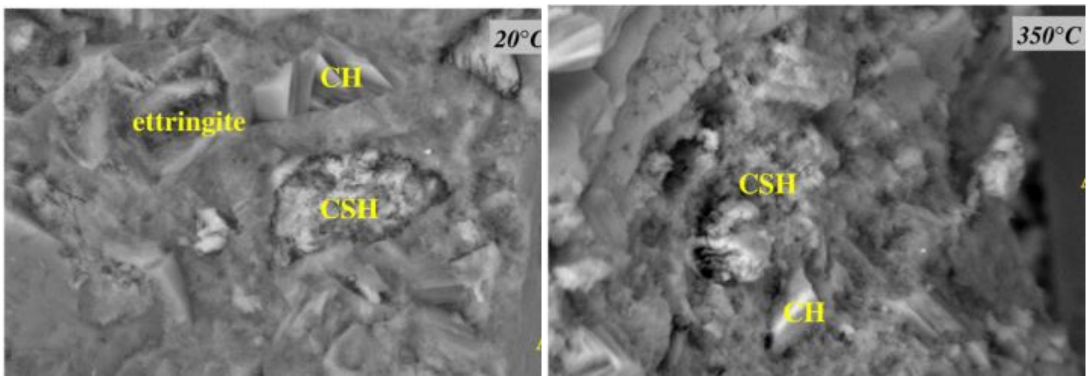
- Around 530ºC, the Portlandite disappear and CaO is produced [Schneider et al., 1982] [Alonso et al., 2003-b], [Alonso and Fernandez ,2004]. It can be seen, from figure 10, that the Portlandite content slightly increased around 250ºC. This increase of Portlandite is due to the progress of hydration of residual anhydrous components [Alonso et al., 2003-b], [Alonso and Fernandez ,2004].

- Calcite ($CaCO_3$) is believed to be present because of carbonation and decomposes at around 550-600ºC [Alonso et al., 2003].

- At 750°C, the transformation of CSH is complete into the nesosilicate form with a C_2S stoichiometry close to larnite but less crystalline [Alonso et al., 2003], [Fernandes et al., 2017]. The chemical reaction for the decomposition of CSH is given by [Willam et al., 2009]:

$$3.4CaO.2SiO_2.\ 3H_2O \rightarrow +\ 3.4CaO.2SiO_2 + 3H_2O\uparrow$$

Figure I.10 shows the microscopic structure of concrete components before and after heating by using Scanning Electron Microscopy (SEM). At 20°C the cement matrix is contain of ettringite, CH and CSH. At 350°C, the disappearance of ettringite is observed. The ettringite already disappears completely before reaching 100ºC [Alonso et al., 2003-b]. At 550°C, the decomposition of CH (>450°C) resulting in a cement matrix with a very porous structure.



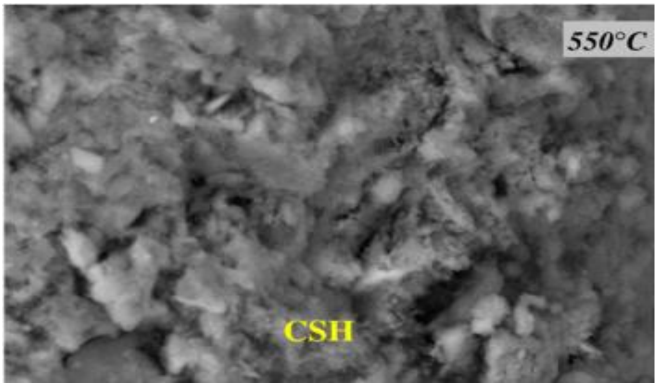


Figure I.10. Microstructural changes in the interfacial transition zone [Annerel and Taerwe, 2009].

I.3.1.2 Effect of high temperature on aggregates

As stated previously, aggregates occupy the largest proportion of the volume of concrete, and it is natural that the mineralogical nature of aggregates has a significant influence on the physical and mechanical properties of concrete exposed to high temperatures. According to their mineralogical nature, aggregates can be classified into two types: Siliceous and Calcareous. Siliceous aggregates containing quartzite in their composition include granite and sandstone. Carbonate aggregates include limestone and dolomite. Siliceous aggregates becomes unstable and undergoes significant increases in volume at about 573°C, which was explained by the phase transition of quartz from α to quartz β [Bodnarova et al., 2013], [Shen et al.,2018], [Biró et al.,2019]. On the other hand, carbonate aggregates present thermal instability at about 700°C, due to decomposes of $CaCO_3$ into free lime (CaO) and carbon dioxide (CO_2). [Hager et al., 2016], [Niry et al.,2016], [Biró et al.,2019].

Figure I.11 presents an example of thermally damaged mortar, which was made of quartz aggregates, and heated to 600°C. At 500°C, the SEM photo shows cracks with about 7 µm wide that are formed along the ITZ. When the specimens are exposed to 600°C, the SEM image shows that the width of the cracks increases especially along the ITZ where the width of the cracks was 25 µm. Niry [Niry et al.,2016] indicated that siliceous aggregates shows an important increase of the thermal strain between 500°C and 600°C corresponding to the transformation of quartz at about 573°C.

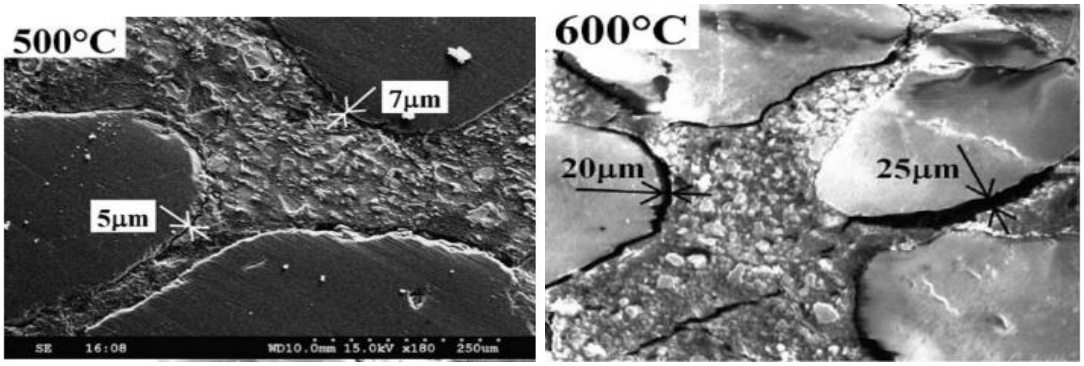


Figure I.11. The microstructure of mortar heated to 600°C [Matesová et al., 2006].

The author Hachemi [Hachemi, 2015] found that the thickness of the decarbonated layer increases with the increase of temperature. The decarbonated layer after exposure to 900°C is showed in the figure I.12. The results obtained by Hachemi [Hachemi, 2015] indicated that the thickness of the decarbonated layer after few minutes of cooling-down was about 1.0 mm. The author also found that the cracks extensively inside the grain increased at 900°C.

Figure I.12. Cracking of calcareous aggregate immediately after heating to 900°C [Hachemi, 2015].

I.3.2 Effects of high temperatures on physical properties of concrete

I.3.2.1 Mass loss

Mass loss is defined the difference between the mass of specimens before and after heating. The mass loss is an important parameter to reflect the changes of concrete after exposure to high temperature. In general, concrete mass loss increases with the increase of temperature, as shown in figure I.13.

From the figure I.13, three distinctive domains of mass loss of the concrete according to the temperature can be obtained:

- The first domain, from room temperature to 120°C (150°C), is characterised by a slight increase of mass loss. This increase of mass loss is due to the departure of free water from the capillaries [Noumowé et al., 2003], [Kanema et al., 2005].
- The second domain, from 120°C (150°C) to 300°C, is characterized by the high increase of loss of mass. The loss of mass in this domain was mainly attributed to the loss of water from the dehydration of the CSH [Noumowé et al., 2009],[Ling et al.,2012].
- The third domain, above 300°C, is characterized by a dramatic increase in mass loss. Two reasons can explain that situation: the first reason is due to the decomposition of $Ca(OH)_2$ [Khoury, 2003-b], [Laneyrie et al., 2016], [Molay et al., 2019] and the second reason is the decomposition of calcium carbonate $CaCO_3$ and the release of CO_2 [Kodur and Sultan, 2003], [Laneyrie et al., 2016], [Liu et al., 2018].

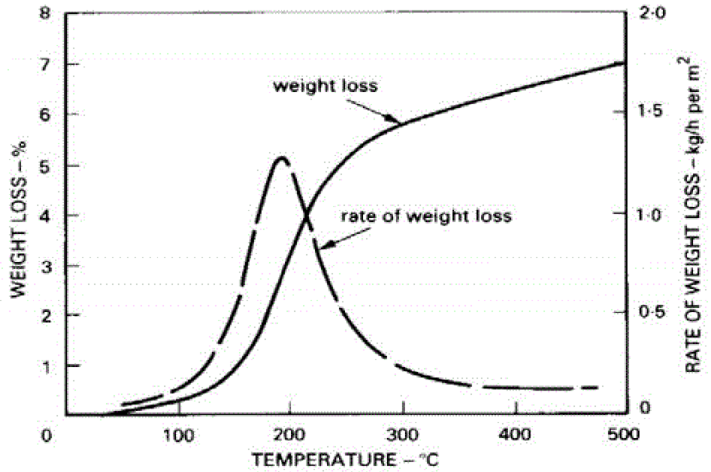


Figure I.13. Evolution of mass loss of concrete during heating [Khoury, 2003-b].

I.3.2.2 Evolution of porosity

Porosity of concrete is an important indicator of the properties of the concrete such as strength and durability. Several studies and experimental data are available regarding the evolution of porosity of concrete subjected to elevated temperatures [Fares et al., 2009], [Mydin et al., 2012], [Ling et al., 2012], [Xing et al., 2015], [Molay et al., 2019]. The results obtained by authors indicated that the porosity of concrete increases with temperature increasing.

Figure I.14 shows the evolution of porosity of foamed concrete subjected to elevated temperatures provided by Mydin [Mydin et al.,2012], two densities of foamed concrete (650 kg/m^3 and 1000 kg/m^3) were tested.

There are several reasons that can be given to explain the increases of porosity with the increase of temperature such as: removal of free water, dehydration of the CSH, decomposition of the hydration products and thermal incompatibility between aggregates and hardened cement paste creates an additional porosity by micro-cracks generate [Mydin et al.,2012], [Zhou et al.,2020].

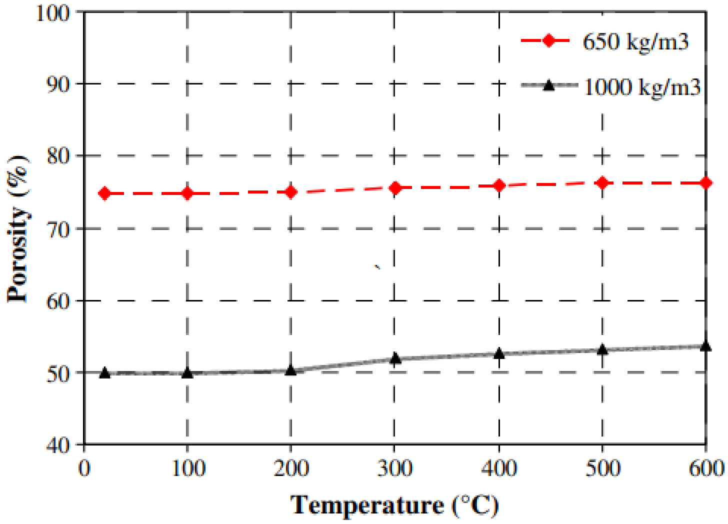
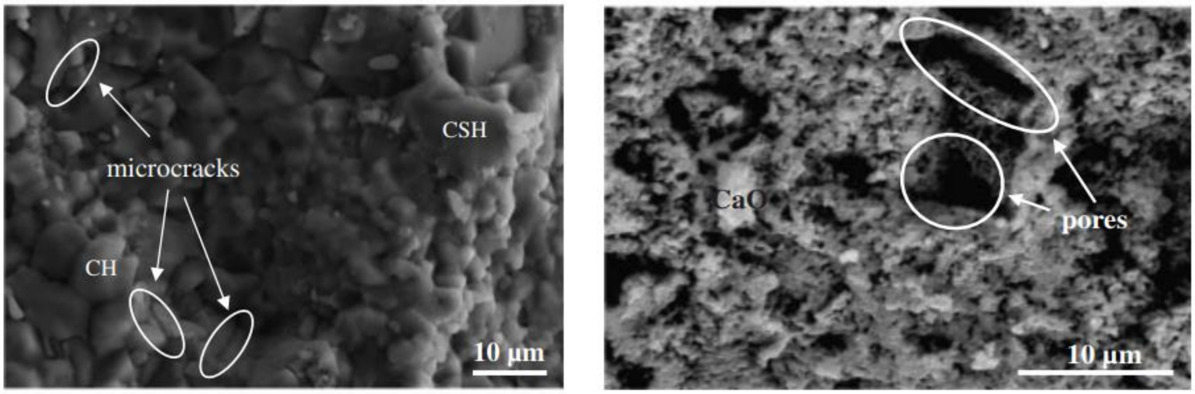


Figure I.14. Evolution of porosity of foamed concrete after exposure to elevated temperatures [Mydin et al., 2012].

Arioz [Arioz,2009] studies indicate, by a SEM, a significant changes in the microstructure of concrete after exposure to 800°C due to the micro-cracks and voids (figure I.15-a). Moreover, when concrete was subjected to 1200°C, the microstructure was completely deteriorated (figure I.15-b). In addition, large amount of voids was observed at that temperature level.



(a) (b)

Figure I.15. SEM micrographs of concrete specimens: (a) at 800°C; (b) at 1200°C [Arioz, 2009].

- **Porosity changes in cement after exposure to elevated temperatures**

Zhang [Zhang et al., 2013] studied the evolution of porosity of cement paste by using techniques Mercury Intrusion Porosimetry (MIP). Figure I.16 presents the results of porosity of heated cement paste after cooling. The results obtained by Zhang [Zhang et al., 2013] indicated that the total porosity of cement paste increases with the increase of temperature.

From figure I.16, two distinctive stages of evolution of cement paste porosity with temperature can be obtained. The first stage was corresponding to the temperature below 400°C, and was characterized by a slightly increases in total porosity. In this stage, the increase of porosity is due to the dehydration of CSH.

The second stage was corresponding to the temperature beyond 400°C, and was characterized by a significantly increased in total porosity. This could be attributed to the recrystallization of CSH.

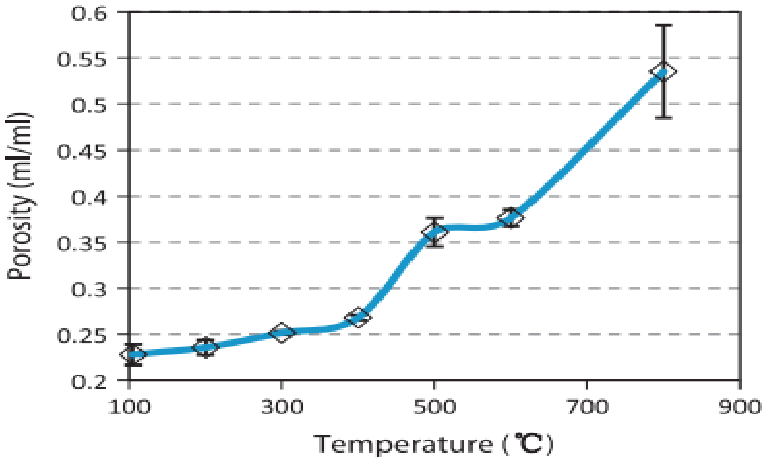


Figure I.16. Evolution of porosity of cement paste after cooling measured by MIP [Zhang et al., 2013].

I.3.2.3 Evolution of density

Several researchers studied the effects of high temperature on the density of concrete ([Fares et al., 2009], [Ling et al., 2012], [Hachemi and Ounis, 2015] , [Bui et al., 2018], [Varona et al., 2020], [Abed and Brito, 2020]). The outcomes indicated that the density of concrete reduces with the increase of temperature.

Fares [Fares et al., 2009] investigated the density of two self-consolidating concrete (SCC) and one vibrated concrete (VC) at high temperatures (compressive strength of SCC1=37 MPa, SCC2= 54 MPa and VC=41 MPa). The evolution of density of concrete with increasing temperature is shown in figure I.17. In the experiment study conducted by Fares [Fares et al., 2009] indicated that the mostly significant factor affecting concrete density at high temperature was the composition of the concrete.

The decrease of concrete density may be caused by the various physical and chemical transformations that occur during heating. These may include: loss of free and physically bound water and dehydration and the decomposition of hydrates like the CSH and CH [Fares et al., 2009], [Abed and Brito, 2020].

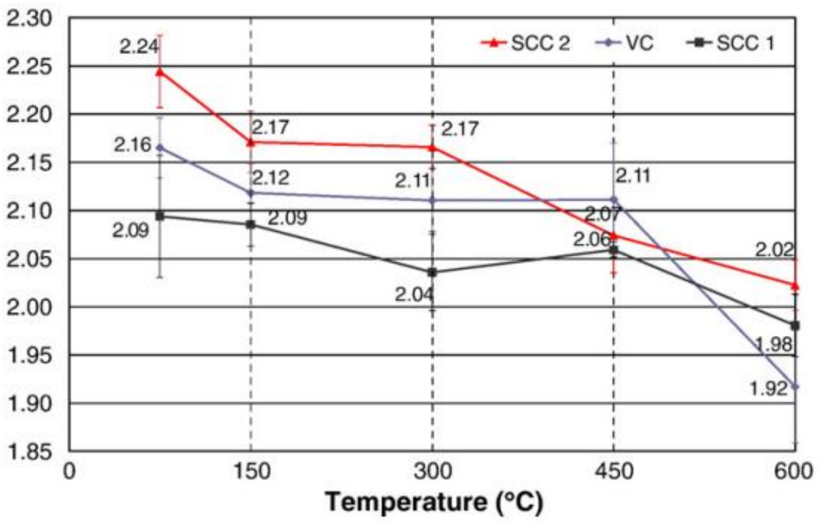


Figure I.17. Evolution of density of concrete after exposure to elevated temperatures [Fares et al., 2009].

I.3.2.4 Evolution of permeability

The permeability of concrete is the ability of this porous material to transfer fluids under of pressure. Many investigations have been undertaken to evaluate the relationship between the permeability of concrete and heating ([Kanema et al., 2005], [Fares et al., 2009], [Noumowé et al., 2009], [Pei et al., 2017], [Hageret al., 2019], [Sogbossi et al., 2020]). In general, the permeability of concrete increases with the increase of temperature.

Noumowé [Noumowé et al., 2009] summarized the values of permeability of concrete from 20°C to 600°C. Figure I.18 shows the variations of permeability of concrete with temperatures. In this study, three types of concrete mixtures were prepared: (B2) control high-performance concrete; (B3) high-performance concrete incorporating polypropylene fibers and (B4) high-performance concrete made with lightweight aggregates. According to Noumowé [Noumowé et al., 2009], the permeability of lightweight aggregate concrete was lower than that of the control concrete. This was probably because of the lower incompatibility between cement paste and aggregates due to the lower rigidity of lightweight aggregates.

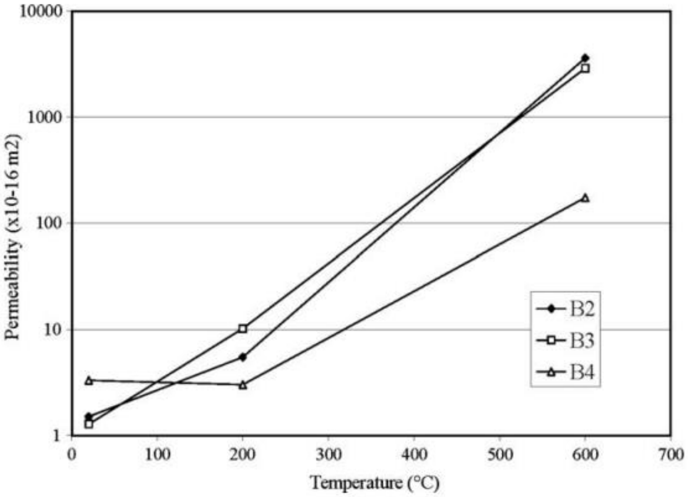


Figure I.18. Evolution of permeability of concrete with temperature [Noumowé et al., 2009].

It has been reported that the permeability of concrete is influenced by various physical and chemical transformations at elevated temperature [Kanema et al., 2005], [Fares et al., 2009], [Pei et al., 2017], [Hager et al., 2019]. The vaporization of free water affects the permeability between 75°C and 300°C. For temperatures higher than 300°C, decomposition of hydrated products leads to micro-cracking and an increase in permeability.

I.3.3 Effects of high temperatures on mechanical properties of concrete

This section highlights the mechanical properties of concrete at high temperatures, including the compressive strength (fc), modulus of elasticity (Ec) and tensile strength (ft). When concrete exposed to high temperatures causes various physical and chemical transformations, results in a reduction in mechanical properties in the long term.

Several influencing factors are known to have significant effects on the mechanical properties of concrete at high temperatures, including types of aggregates, additions fibres, additions mineral admixtures, etc. These factors are considered in the following sections.

I.3.3.1 Compressive strength

Testing compressive strength, as one of the destructive methods in place, is of high importance to evaluate the strength and durability of concrete structures under high temperature. Liu [Liu et al., 2018] compiled the values of compressive strength for different concretes under high temperature based on the experimental data reported in the literature. Figure I.19 shows the variations of compressive strength of concrete with temperatures.

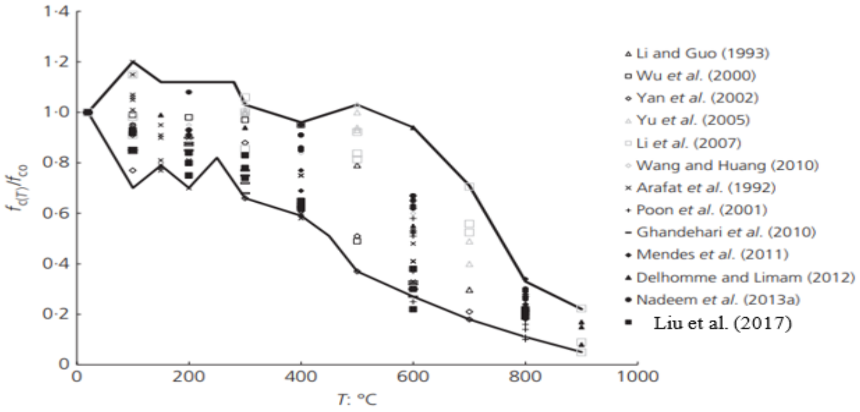


Figure I.19. Effect of high temperatures on compressive strength of various concrete [Liu et al., 2018].

It is observed from figure I.19 that whatever the mix, the compressive strength of concrete reduces with increasing temperature. There are several reasons that can be given to explain the decrease in compressive strength with the increase in temperature. For temperatures between 20°C and 400°C, the decrease of compressive strength in this range comes from the evaporation of free water, and dehydration of ettringite and CSH [Ou et al., 2012], [Ling et al., 2012], [Wu et al., 2019]. According to Ou [Ou et al., 2012], for temperature less than 300˚C, the CSH gel structure remains intact and will not be damaged.

In fact, up to 400°C, one opposite phenomena occur: the rehydrate of anhydrous cement in the capillary pores during the evaporation of free water, which leads to the cement gel layers moving closer to each other. This reaction increases Van der Waal’s forces forces between cement gel layers and increases the compressive strength of concretes [Cheng et al., 2004], [Chen et al., 2009], [Molay et al., 2019], [De Silva et al., 2020]

For temperatures higher than 400°C, the main reason for compressive strength loss can be attributed to the damage of CSH gel structure due to dehydration as well as to the decomposition of portlandite (CH). Additionally to those reasons, the formation and high development of cracks due to incompatible spreading and shrinkage at the interfaces between aggregates and cement matrix facilitate the decrease of compressive strength [Ou et al., 2012], [Chen et al., 2009], [Ling et al., 2012], [Camilo et al., 2019], [Sadrmomtazi et al., 2020], [Tang et al., 2021].

- **The effect of high temperatures of compressive strength for concrete with different nature of aggregates**

As mentioned earlier, aggregates occupy the largest proportion of concrete volume and, therefore the nature of aggregate will influence the concrete compressive behavior. Results in the literature ([Yoon et al., 2015], [Al-Jabri et al., 2015], [Hager et al., 2016], [Santos et al., 2016], [Tufail et al., 2017], [Hachemi and Ounis., 2019]) indicated that the nature of the aggregates strongly influences the compressive strength of concrete at high temperature.

Figure I.20 shows the results of residual compressive strength of concretes at different temperatures found by Hachemi [Hachemi and Ounis, 2019]. The authors used two types of natural sand, one is the siliceous sand (SS)، and the latter is calcareous sand (CS) to produce normal strength concrete (C1), high strength concrete (C2) and high performance concrete (C3). The authors found that CS-concretes with a high w/c (normal strength concrete) have the best residual compressive strength, while SS-concretes with a lower w/c (high performance concrete) has better residual behaviour. This observation was also noted by Xing [Xing, 2011]. Yoon [Yoon et al., 2015] indicated that concrete consists of lightweight aggregates show higher residual compressive strength at high temperatures than normal weight concrete.

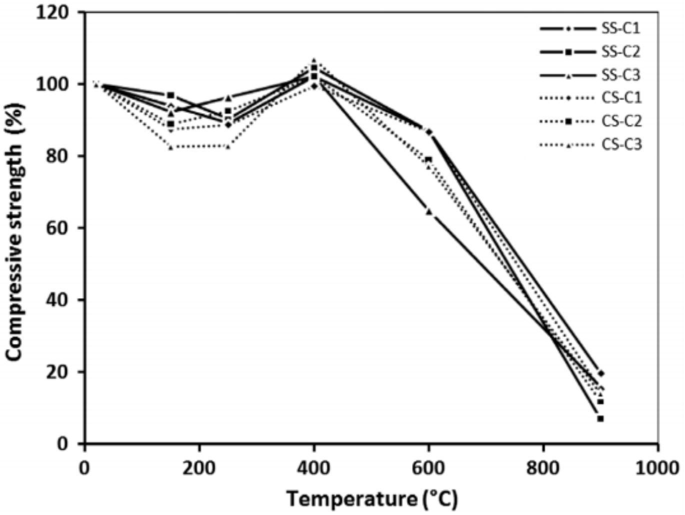


Figure I.20. The residual compressive strength of SS-concrete and CS-concrete after exposure to elevated temperature [Hachemi and Ounis, 2019].

- **The effect of high temperatures of compressive strength for concrete with fibers additions**

The behavior compressive of concrete with fibers additions under elevated temperatures has also been studied extensively in the literature. Table I.3 shows the variations of compressive strength of concrete with fibres additions compared to concrete without fibres additions under high temperatures.

From Table I.3, conflicting results were observed from the literature regarding the impact of additions fibers on the residual compressive strengths of concrete under high temperatures. This may be due to the difference in the composition of concrete and the quality of the fibers used.

Table I.3. Behaviour of concrete containing different type of fibers after elevated temperatures.

Types of fibers	**Variation in compressive strength as compared to natural concrete**	**Reference**
Polypropylene (PP), carbon or steel fiber	The residual compressive strengths of HSC with fibers was higher than those of normal HSC.	[Chena and Liub, 2004]
Polypropylene fibres (PP)	The addition of 2 kg/m^3 PP fibers to high performance concrete did not have significant effect on the compressive strengths	[Noumowé et al., 2009]
Polypropylene fibers (PP)	The residual compressive strengths of concretes containing of 2 kg/m^3 PP fibers were higher than those of concretes without PP fibers	[Behnood and Ghandehari, 2009]
Steel fibers	The residual compressive strengths of concretes containing steel fiber were higher than the values obtained for the concretes without steel fiber after exposure to a temperature of 500°C	[Bezerra et al., 2019]
High performance synthetic macro polypropylene fibers (HPP)	The addition of 1 kg/m^3 HPP fibers have improvement in the residual compressive strengths of concrete exposed to high temperatures	[Abaeian et al., 2018]
Steel fiber	The residual compressive strength of concrete exposed to high temperatures improved significantly by the addition of steel fibers	[Zhang et al., 2020]

❖ The effect of high temperatures of compressive strength for concrete with mineral admixtures

Mineral admixtures are materials which can be added to the concrete mixture to improve its properties. Among these materials, there are: Silica Fume (SF), Fly Ash (FA), Ground Granulated Blast Furnace Slag (GGBFS) and Meta-Kaoline (MK).

The influence of mineral admixtures on the evolution of the compressive strengths of concrete at high temperatures was studied by Li and Yuan [Li and Yuan, 2012], Bui [Bui et al., 2018], Rafi [Rafi and Aziz, 2019], Sadrmomtazi [Sadrmomtazi et al., 2020]. In general, the residual compressive strength of concrete was affected by the volume content of mineral admixtures and the temperature history of the samples, including the heating rate and cooling regime.

Figure I.21 presents the evolution of residual compressive strength of HSC concrete containing mineral admixtures subjected to elevated temperature [Zhou et al., 2020]. The residual strength characteristics of concretes were found to improve with the usage of mineral admixtures at elevated temperatures [Ma et al., 2015], [Zhou et al., 2020].

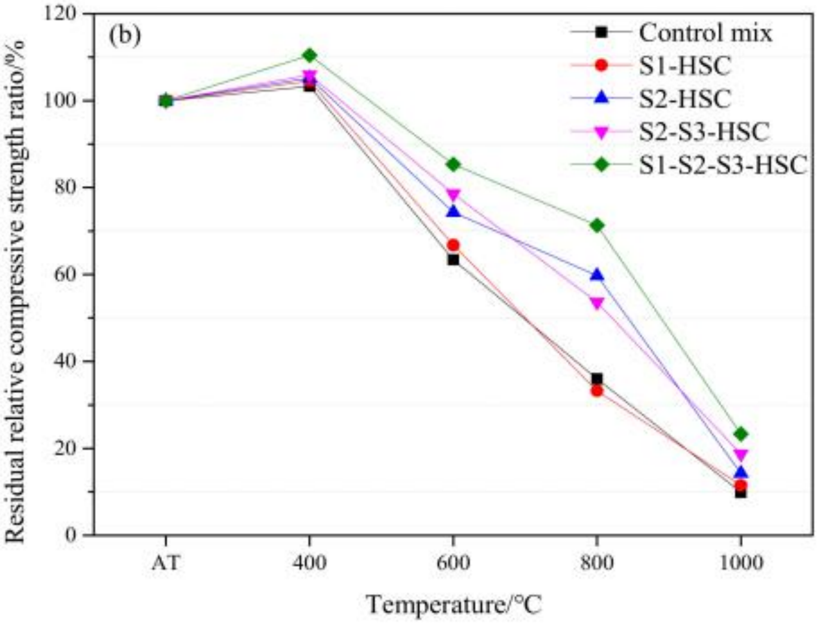
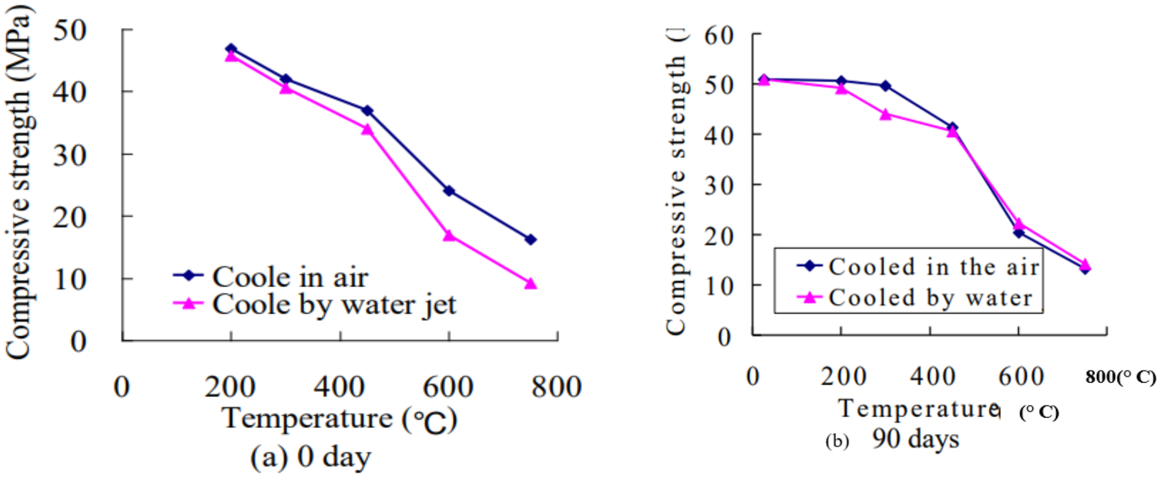


Figure I.21. Effect of temperature on the compressive strength of HSC concrete with different mineral admixtures Fly Ash (S1), ultra-fine fly ash (S2), and metakaolin (S3) [Zhou et al., 2020].

❖ Effects of cooling method or re-curing time on residual compressive strength of concrete after high temperature exposure

Many investigations have been performed to discuss the influence of cooling method or post-fire-curing on the residual compressive strength of concrete exposed to high temperature ([Bingol and Gül, 2009], [Li et al., 2017], [Carvalho et al., 2019], [Wang et al., 2019]). In general, the cooling method and re-curing time had a significant influence on the residual compressive strength. Based on previous experimental studies [Carvalho et al., 2019], the sudden cooling with water spraying was the most severe condition compared than hot tested or natural air cooling, resulting to a significant decrease in compressive strength. Similar results were obtained by Bingol [Bingol and Gül, 2009]. Wang [Wang et al., 2019] indicated that the lower residual compressive strength cooled with water could be explained by more severe thermal shock due to rapid cooling. Li [Li et al., 2017] found that the relative residual compressive strength of specimens cooled in the air ranges from about 77% to 83%, while the specimens cooled in the furnace ranges from about 65% to 87%. Figure I.22 shows the variation of the compressive strength of concretes with re-curing time after exposed to high temperatures. As shown in figure I.22, with the increase in the re-curing time, the degree of strength recovery in case of water-cooling was greater than in case of air-cooling, similar results were obtained by Akca [Akca and Özyurt, 2018].



(a) 0 day

(b) 90 days

Figure I.22. Effect of cooling method on compressive strength concrete exposed to high temperature: (a) after 0 days, (b) after 90 days [Zhuguo and Qingtao, 2010].

I.3.3.2 Tensile strength

The tensile strength is an important property for concrete under high temperature because the cracks are generally caused by tensile stresses in concrete. Research studies on tensile strength of concrete at elevated temperatures are limited. Figure I.23 shows the variations of tensile strength for concrete at elevated temperatures. The values were compiled by Bastami [Bastami et al., 2010] based on the experimental data reported in the literature.

As shown in figure I.23, the residual tensile strengths of concrete decreased similarly with the increase of temperature. Results in the literature ([Wang et al., 2019], [Al-Jabri et al., 2015]) indicated that the effect of heating on split tensile strength was greater than compressive strength. This phenomenon is mainly due to coalescence of crack which is more significant in tensile strength than in compressive strength [Zheng et al., 2013].

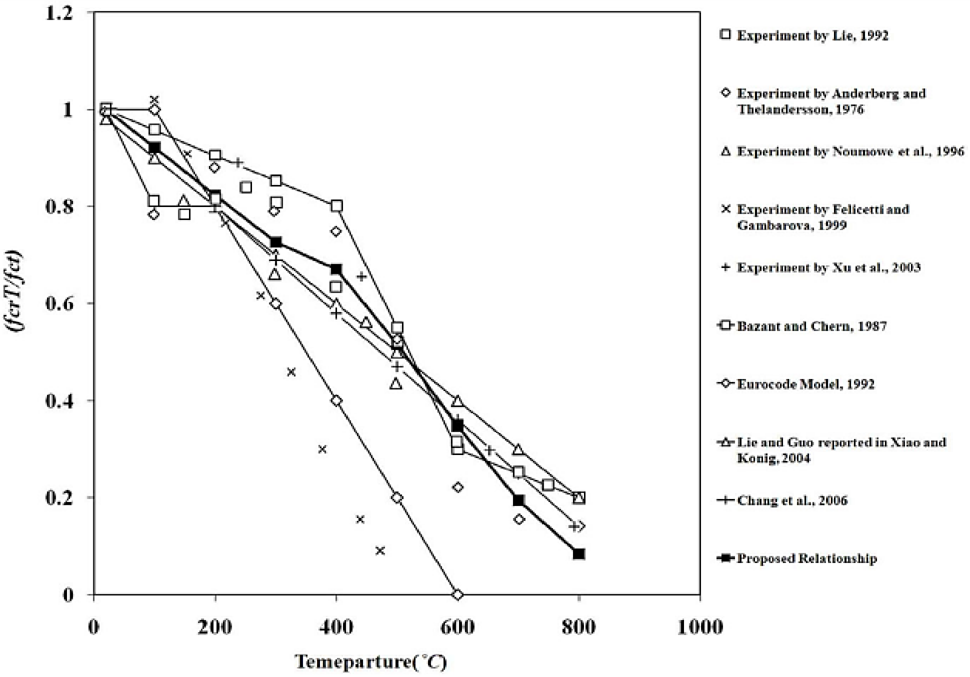


Figure I.23. The residual tensile strength of concretes as a function of heating temperature [Bastami et al., 2010].

Test results obtained by Santos [Santos et al., 2016], Al-Jabri [Al-Jabri et al., 2015] and Tufail [Tufail et al.,2017], showed in general that the nature of aggregate were considered as main factors impacting the concrete residual tensile strengths under high temperature. The loss of tensile strength with different nature of aggregate is shown in figure I.24. The authors found that the granite concrete presents a lower degradation rate of relative tensile strength compared to limestone and quartzite concrete as the temperature increases. Santos [Santos et al., 2016] has also reported this observation. According to Tufail [Tufail et al., 2017], the lower deterioration on tensile strength of the granite concrete was attributed to the low quartz fraction in granite aggregate (indicating low thermal expansion and low impact from phase transformation).

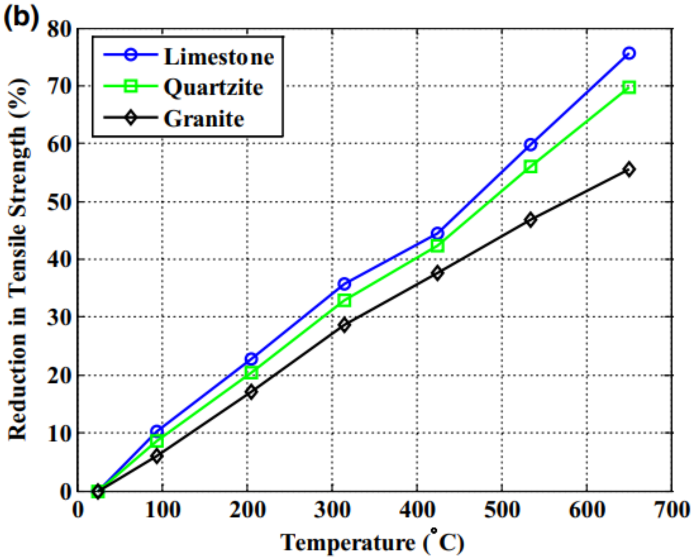


Figure I.24. Effects of aggregates nature on tensile strength of concrete under high temperature [Tufail et al., 2017].

In terms of the effect of high temperature on the residual tensile strength of concrete with fibers, several studies and experimental data are available regarding the use of fibers in concrete exposed to high temperature. A large number of studies ([Chena and Liub, 2004], [Behnood and Ghandehari, 2009], [Jameran et al., 2015], [Bezerra et al., 2019], [Abaeian et al., 2018], [Sadrmomtazi et al., 2020]) have demonstrated that fibers contributed positively to improving the residual tensile strength of concrete. For example, Jamerana [Jamerana et al., 2015] investigated the effects of steel fibers (ST) and polypropylene (PP) fiber to produce Fiber Reinforced Concrete (FRC). According to this study, the percentages of steel and propylene fiber were (100% and 0%), (75% and 25%), (50% and 50%), (25% and 75%), and (0% and 100%). The data results are shown in figure I.25.

It is evident from figure I.25 that the tensile strength of concrete with fiber were higher than control concrete (0-0), except for concrete ST-PP (0-100%) for all exposure temperature. This shows that the addition of fibers in the concrete mixture enhanced the tensile strength of the concrete.

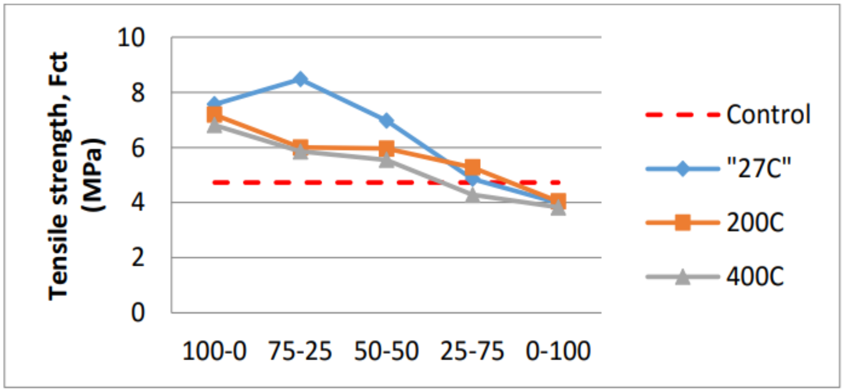


Figure I.25. Tensile strength of concrete with ST and PP fibers under high temperature [Jamerana et al., 2015].

With regard to the use of the additional materials on behaviour tensile strength of concrete after elevated temperature, several researchers ([Ahmad et al., 2017], [Afshoon and Sharifi, 2020], [Sadrmomtazi et al., 2020]) found that residual splitting tensile strength of concrete with additional materials exposed to high temperature was similar or even better than concrete without additional materials. In general, there are several factors are correlated with the residual tensile strength of concrete subjected to high temperature such as type and amount of additional used, composition of concrete and duration of temperature exposure.

Like residual compressive strength, researchers have come up with similar conclusions regarding the effects of cooling method on concrete residual tensile strength after exposed to high temperature. Figure I.26 shows the results of residual tensile splitting strength of concrete subjected to various cooling method.

The impact of cooling method on residual tensile strength of concrete after exposed to high temperature were studied by Peng [Peng et al., 2008] and Wang [Wang et al., 2019]. According to them, the cooling methods have significantly influences on residual tensile strength of concrete and water cooling caused more deterioration in tensile strength.

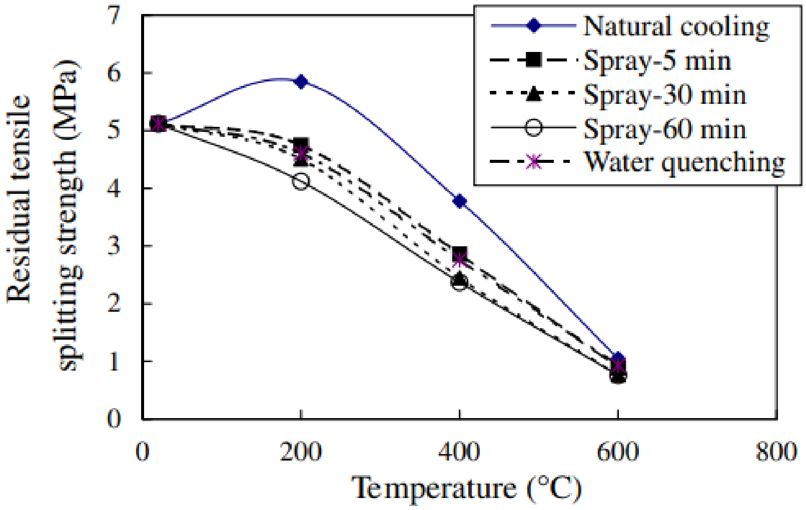


Figure I.26. The effects of cooling method on tensile strength of concrete after exposure to high temperature [Peng et al., 2008].

I.3.3.3 Modulus of elasticity

In fires, the excessive deformation of the structure will damage it. Elastic properties are important indicators of the ability of concrete to deform. Elastic modulus and Young's modulus are the indexes that are typically used for evaluating the elastic properties of concrete. At room temperature, modulus of elasticity value of concrete ranges from 5.0 × 10^3 MPa to 35.0 × 10^3 MPa [Kodur, 2014]. Drzymała [Drzymała et al., 2018] indicated that the modulus of elasticity of concrete is related to its compressive strength. Nguyen [Nguyen., 2013] compiled the values of elastic modulus for several of concretes under high temperature based on the experimental data reported in the literature. Figure I.27 shows the variations of concrete elastic modulus of with temperatures.

It can be seen from the figure that the modulus of elasticity decreases rapidly with the rise of temperature. According to the previous researches conducted by Li and Yuan [Li and Yuan, 2012] and Abid [Abid et al., 2019], the degradation in elastic modulus due to heating is more severe than that compressive strength. This can be attributed to various factors such as the departure of water during heating, decomposition of CSH and CH hydrates and generation of microcracks and therefore, the formation of a porous ITZ between the aggregates and cement matrix leads to a reduction in elastic modulus [Abid et al., 2019].

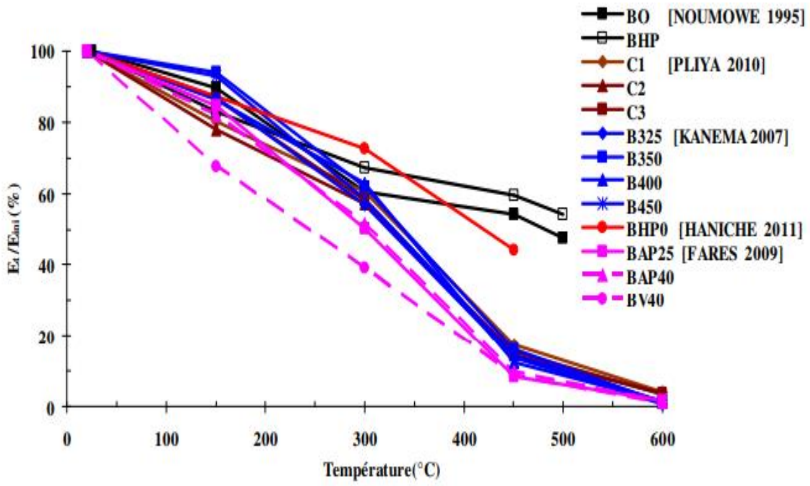


Figure I.27. Variation in elastic modulus of concrete as a function of temperature [Nguyen, 2013].

The effect of aggregates nature on concrete modulus of elasticity subjected to high temperature have been studied by Savva [Savva et al., 2005], Tufail [Tufail et al.,207], Santos [Santos and Rodrigues, 2016], Hager [Hager et al., 2016] and Xing [Xing ,2011]. According to them, the modulus of elasticity of concrete is linked to the nature of aggregate used at elevated temperatures.

Savva [Savva et al., 2005] tested limestone and siliceous ordinary concrete. They noticed that up to 300°C, the relative residual modulus of elasticity of siliceous concretes was higher than those of limestone mixtures. Above 300°C, the reduction in the modulus of elasticity ranged from 94% to 99%, no matter which type of aggregate was used.

Santos [Santos and Rodrigues, 2016] indicated that up to 400°C, the calcareous and granite aggregate concretes presented a similar decrease in modulus of elasticity. At 500°C, granite aggregate concretes presented a null value while the granite aggregate concretes still have around 10% of the value at ambient temperature, as shown in figure I.28.

Hager [Hager et al., 2016] analyzed four different aggregate types: basalt, granite, dolomite and riverbed gravel and found that the decrease of modulus of elasticity was almost identical for all the concretes.

According to study carried out by Tufail [Tufail et al., 2017], the rate of decrease of the relative modulus of granite concrete was lower than that of limestone and quartzite concrete.

Xing [Xing, 2011] found that the decrease in Young's modulus was almost the same for calcareous aggregate concrete and siliceous aggregate concrete.

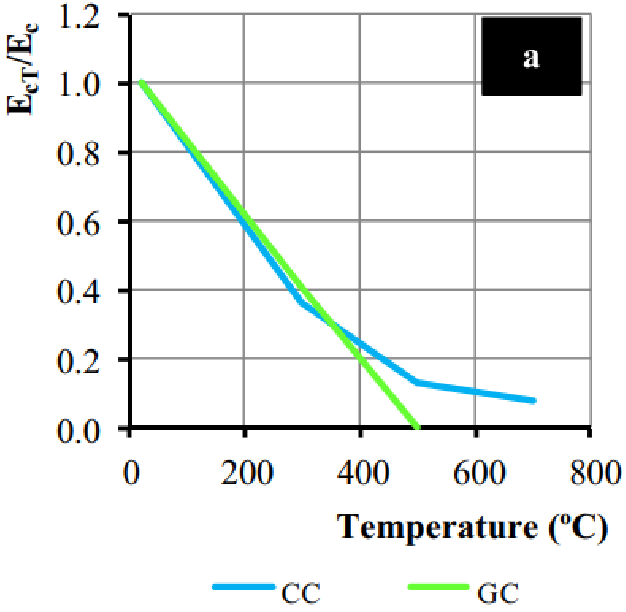


Figure I.28. Evolution of modulus of elasticity of calcareous aggregate concrete (CC) and granite aggregate concretes (GC) with temperature [Santos and Rodrigues, 2016].

In recent years, many researchers have conducted experiments to evaluate the modulus of elasticity of concrete containing fibers ([Khaliq and Kodur, 2011], [Drzymała et al., 2018], [Abid et al., 2019], [Bošnjak et al., 2019], [Zhang et al., 2020]). Khaliq [Khaliq and Kodur, 2011] investigated the effects of steel and polypropylene fiber on the modulus of elasticity of self-consolidating concrete (SCC). In their study, four types of SCC were produced: SCC without fibers, SCC reinforced with steel fiber (SCC-S), SCC reinforced with polypropylene fiber (SCC-P) and SCC reinforced with hybrid fibers (SCC-H). These concretes were subjected to the temperature range of 20°C–800°C. The relative modulus of elasticity of concretes after exposed to high temperature is presented in figure I.29.

It is evident from figure I.29 that modulus of elasticity for all four concretes decrease from 20°C–800°C temperature range. According to the authors, the addition of fibers leads to improving modulus of elasticity of concrete SCC under high temperature. Drzymała [Drzymała et al., 2018] found similar results in their experiments.

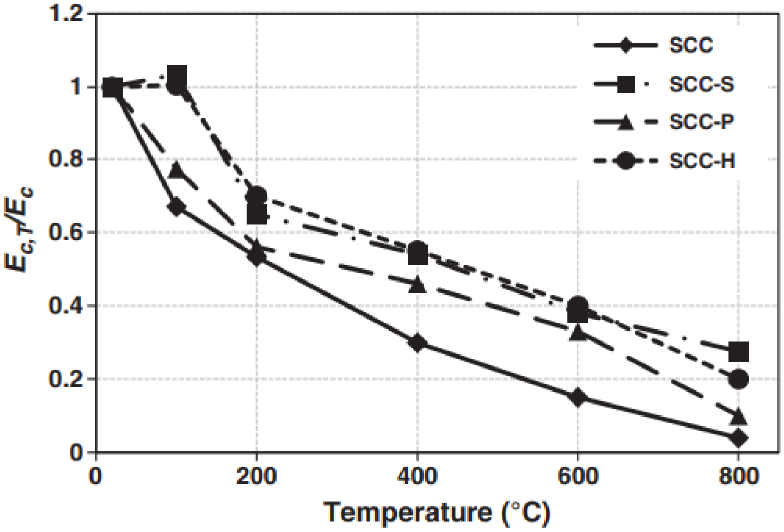


Figure I.29. Evolution of modulus of elasticity of concrete containing fibers after exposure to high temperature [Khaliq and Kodur, 2011].

Bui [Bui et al., 2018] evaluated the modulus of elasticity of concrete containing 100% recycled coarse aggregate at high temperatures. The authors used four different mineral admixtures (fly ash (FA), waste paper sludge ash (PSA), silica fume (SF) and metakaolin (MK). They concluded that the residual elastic modulus of concrete containing 100% recycled coarse aggregate was 43.55% lower than that natural aggregate concrete subjected to 500°C. On the other hand, the addition method enhanced the elastic modulus of recycled concrete aggregate better than the replacement method and the mineral admixtures contributed to increasing the residual elastic modulus of heated recycled concrete aggregate. Figure I.30 shown relative residual modulus of concrete subjected to 500°C.

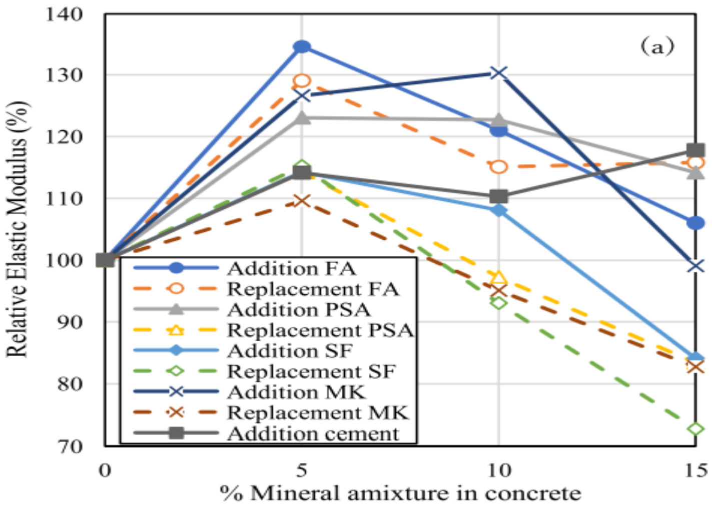


Figure I.30. Relative residual modulus of concrete with different mineral admixtures subjected to 500°C [Bui et al., 2018].

Hager [Hager, 2013] evaluated the modulus of elasticity of three concrete with w/c ratios of 0.3, 0.4 and 0.5 under high temperature. The values of modulus of elasticity of concrete with different w/c are presented figure I.31**.** Test result obtained by Hager [Hager, 2013] indicated that the decrease of modulus of elasticity for all concretes tests could be considered as quasi-identical, which is consistent with the results of Li [Li et al., 2017] who reported that the effects of w/c on the elastic modulus was not significant.

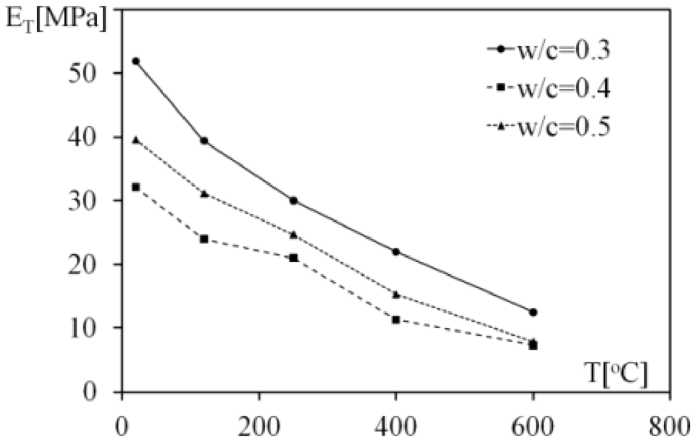


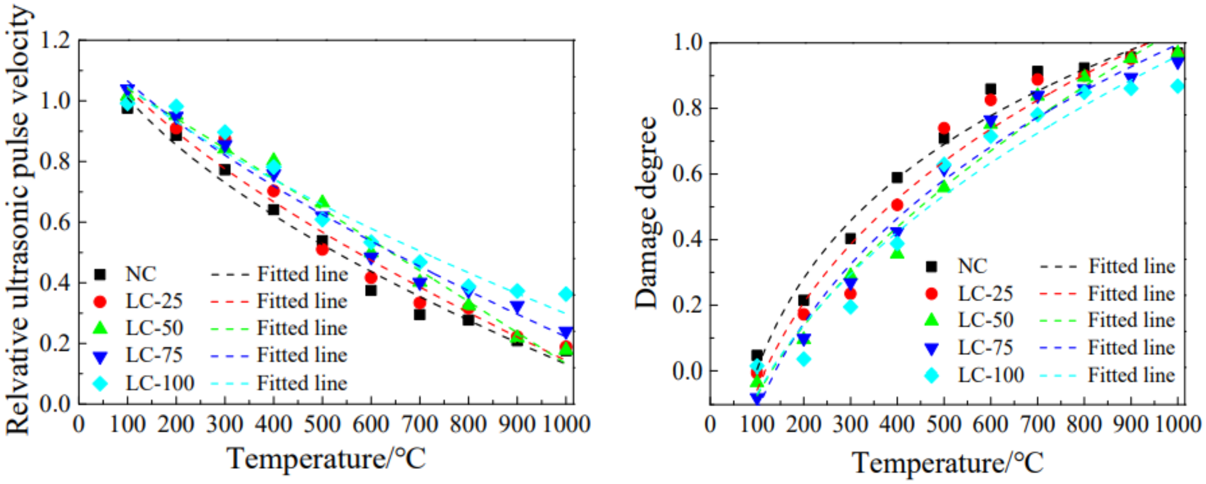
Figure I.31. Evolution of modulus of elasticity of concrete with different w/c subjected to high temperature [Hager, 2013].

I.3.4 Internal damage after Exposure to High Temperature

Testing ultrasonic pulse velocity, as one of the non-destructive methods in place, is of high importance to evaluate the quality and internal damage of concrete structures after exposure to high temperature.

Several studies and experimental data are available regarding the use of ultrasonic pulse velocity tests in order to evaluate the quality of concrete after exposure to high temperature ([Ghandehari et al., 2010], [Uysal et al., 2012], [Ling et al., 2012], [Sadrmomtazi et al., 2020], [Yao et al., 2020], **[**Hachemi, 2015], [Hachemi and Ounis, 2015], [Khattab et al.2021-a]). In general, the residual ultrasonic pulse velocity decreases and damage degree of concrete increases after exposure to high temperature.

Several influencing factors are known to have significant effects on the residual ultrasonic pulse velocity and damage degree of concrete at high temperatures, including type of concrete, type of aggregates, w/c ratio, used mineral admixtures and fibers and duration of temperature exposure. Figure I.32 shows the relative residual ultrasonic pulse velocity and damage degree of concrete after elevated temperature.



(a) (b)

Figure I.32. Evolution of UPV of concrete subjected to elevated temperature: (a) Relationship between relative UPV and temperature; (b) Relationship between damage degree and temperature [Yao et al., 2020].

As can be seen in figure I.32, the ultrasonic pulse velocity gradually decreases and damage degree increases continuously. By increasing the temperature from 20°C to 400°C, the decrease in the UPV in concrete comes from the loss of free physically absorbed water and the decomposition of hydration products [Ghandehari et al., 2010]. However, the decrease in UPV after 200°C was strongly influenced by micro-cracks. Therefore, the internal damage delays the concrete pulse velocity [Uysal et al., 2012], [Hachemi and Ounis, 2015], [Sadrmomtazi et al., 2020].

I.4 STATE OF THE ART ON THE USE OF RECYCLED REFRACTORY BRICK AT HIGH TEMPERATURE

Refractory bricks are solid materials that can withstand high temperatures. The walls and floor of furnaces are mainly covered with refractory bricks to provide favorable thermal insulation. As stated earlier, when the refractory bricks have served their service life, they are being demolished and replaced with a new one. Therefore, the removal and disposal of these refractory bricks result in a large amount of wastes. Thus, it is critical to find a way to reuse them so that this huge amount of waste can be turned into new resource for construction industry.

Refractory brick wastes can be a potential source of raw materials, these wastes have found their place as fine aggregates and cementitious materials in the manufacture of concretes or mortars. In fact, very few studies have been conducted on the use of recycled refractory brick in concrete.

A study has been initiated by Kavas [Kavas et al., 2006], who investigated the use of refractory bricks waste based of alumina and magnesium chromite as fine aggregates to produce high temperature resistant mortar. The result obtained by authors show that the compressive strength of mortars produced decreased with an increase in temperature. Figure I.33 shows the results of compressive strength of mortars containing refractory wastes based on alumina (AC), magnesium chromite (MC) and conventional mortars (PC) found by Kavas [Kavas et al.,2006]. From this figure, it is evident that mortars containing refractory wastes based of magnesium chromite have given the best results after exposure to elevated temperature.

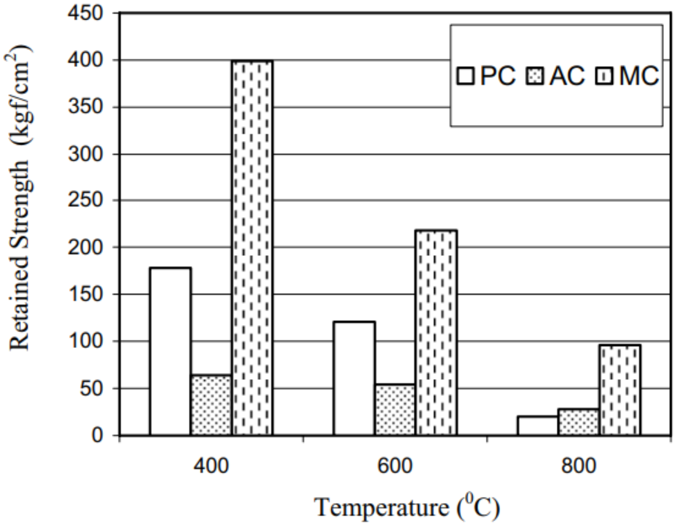


Figure I.33. Compressive strength of mortars cured for 28 days and heat treated at temperatures of 400°C, 600°C and 800°C [Kavas et al., 2006].

In a study carried by Saidi [Saidi et al., 2015] and has objective of reuse the refractory brick wastes as fine aggregates in the cement mortars production as well as to study the thermal behaviour at elevated temperature of these mortars. The replacement level by weight of fine refractory brick wastes were taken as 0%, 10%, 20%, 30% and 50%. Figure I.34 shows the test results of compressive strength and flexural strength as function replacement level of refractory brick wastes.

Test results by authors indicated that replacing up to 20% of natural sand with refractory brick waste, increased the compressive strength and flexural strength of the mortar (see Figure I.34). The authors also found that the mortar sample containing 20% of refractory brick waste shows lower mass loss compared to other mortars, indicating higher resistance at higher temperatures (Figure I.35). This study concluded that the refractory brick wastes could be used as a fine aggregate in mortars production.

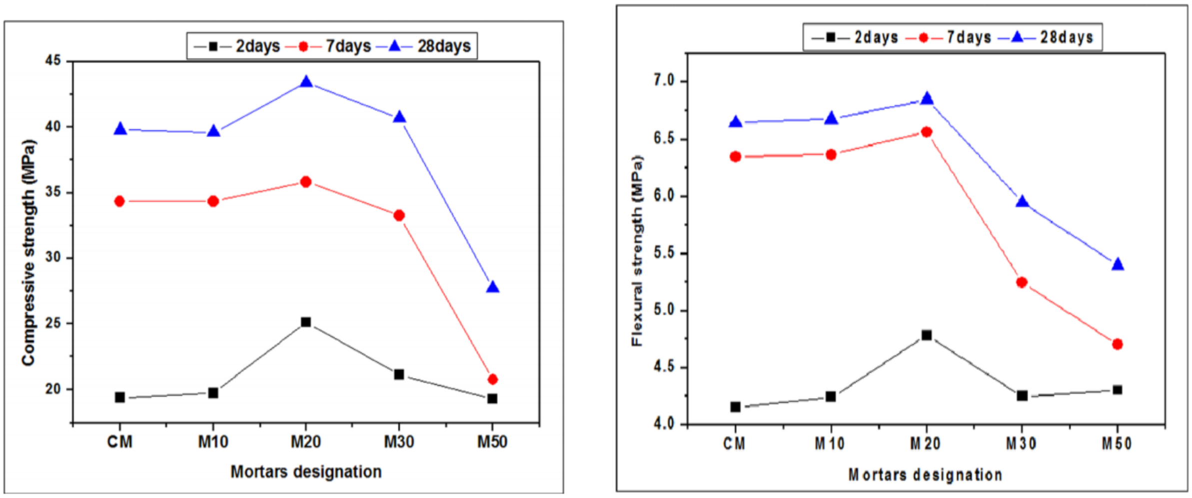


Figure I.34. The evolutions of compressive and flexural strength as function replacement level of refractory brick wastes [Saidi et al., 2015].

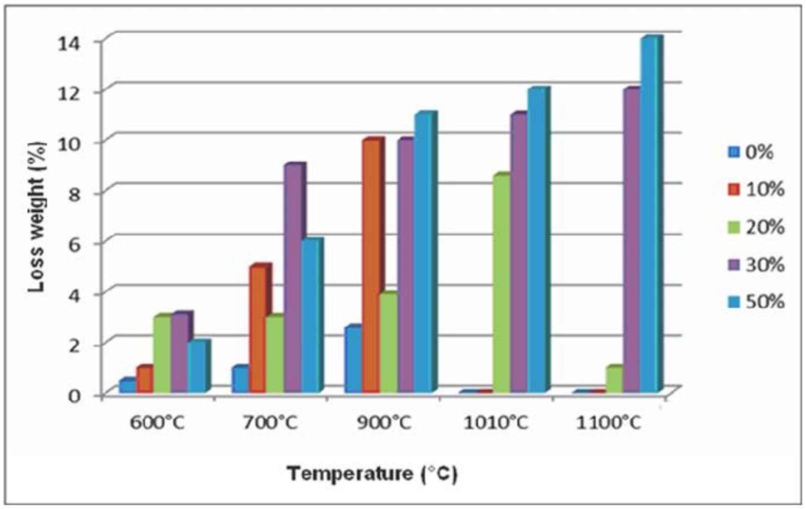
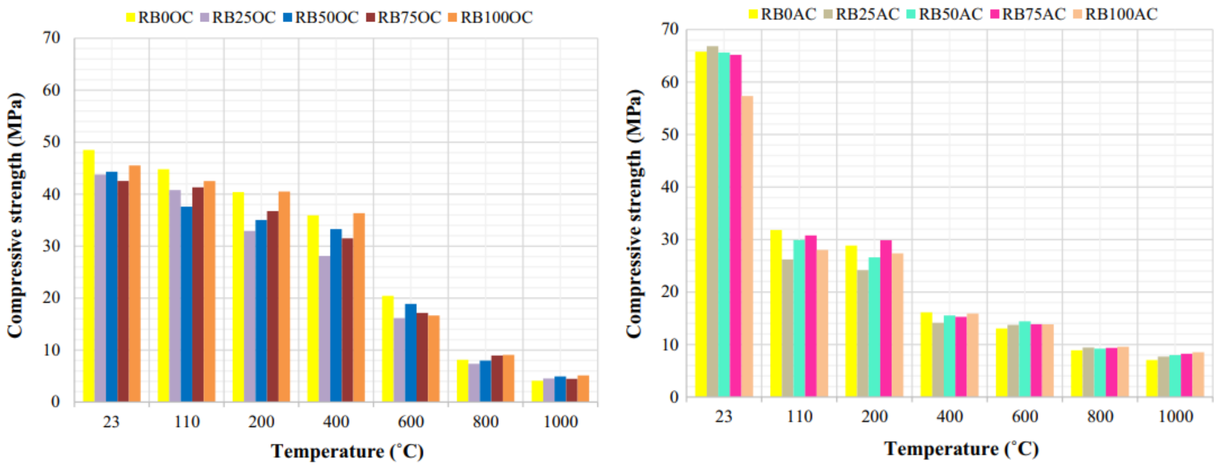


Figure I.35. The mass loss of mortar samples after exposure to elevated temperature [Saidi et al., 2015].

In another study, Aboutaleb [Aboutaleb et al., 2017] also examined the use of refractory brick waste as fine aggregate (by partial and total replacement of natural sand) to produce a self-compacting mortar. The result obtained by these authors showed that waste refractory bricks can be successfully used as fine aggregate in the production of self-compacted mortar, without affecting the essential properties of mortar.

Analogous to the above studies, Nematzadeh [Nematzadeh and Nasiri, 2018] carried out an experimental study in which concrete samples, prepared with partial and total replacements (0%, 25%, 50%, 75%, and 100%) of natural sand by refractory brick fine aggregate, were exposed to 110°C, 200°C, 400°C, 600°C, 800°C and 1000°C. The specimens were prepared in the two groups: the first group was made with ordinary Portland cement and the second group was made with calcium aluminate cement.

Figure I.36-a and I.36-b show the test results of compressive strength of specimens containing Portland cement and calcium aluminate cement exposed to elevated temperatures, respectively. The authors found that, for all the specimens, the compressive strength decreased with increasing temperature. However, there is a significant degradation in the compressive strength of specimens containing Portland cement at 400°C and for the specimens containing calcium aluminate cement at 110°C. Furthermore, an improvement in compressive strength of concrete containing 100% refractory brick and aluminate cement at temperatures above 800°C was observed.



a b

Figure I.36. Compressive strength of specimens containing (**a**) Portland cement (**b**) Calcium aluminate cement exposed to elevated temperatures [Nematzadeh and Nasiri, 2018].

In a related work, Nematzadeh [Nematzadeh et al., 2018] found that the performance of concretes containing fine refractory brick aggregate in terms of corrosion control against acid attack was a rather unsatisfactory.

Experiments to evaluate the reuse of refractory brick wastes as supplementary cementitious material (by a total replacement of silica fume) have also been conducted by Zeghad [Zeghad et al., 2017]. In their study, three types of refractory brick wastes were used (refractory bricks based alumina, magnesium and Silica-Zirconium) to produce an ultra-high performance concrete.

Test compressive and flexural strength results for all studied concrete at different ages of curing are shown in figure I.37. The results illustrated that the compressive and flexural strength were slightly decreased with the nature of the used brick type at whatever curing age. However, the study of Zeghad [Zeghad et al., 2017] led to the conclusion that refractory bricks finely ground has the potential to be used as a cementitious material for concrete manufacture

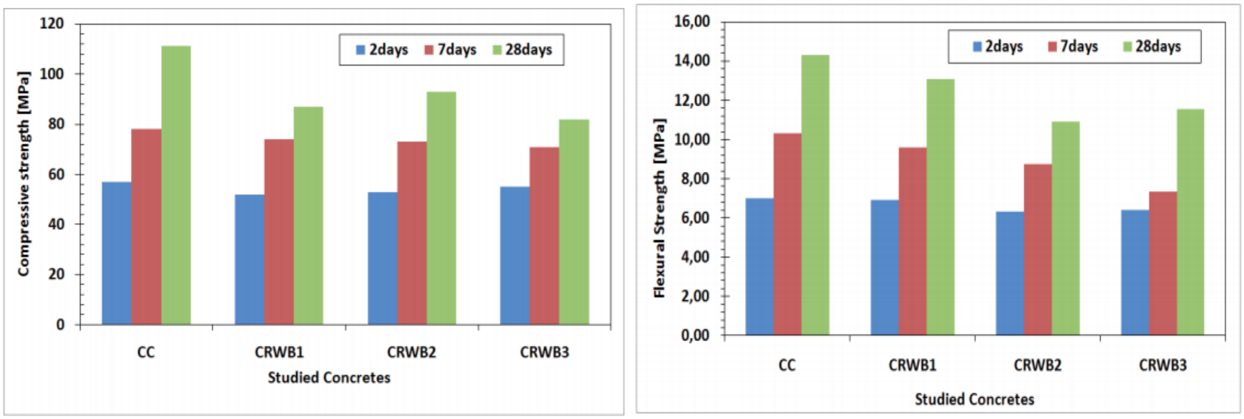


Figure I.37. Compressive and flexural strength of studied concretes [Zeghad et al., 2017].

I.5 CONCLUSIONS

This chapter has presented a detailed review of the state-of-the-art knowledge available on the behaviour of concretes at room and high temperatures. The experiments and observations carried out shown that exposing concrete to high temperature changes its physic-chemical properties, which in turn leads to modification of the microstructure as well as reduced mechanical properties and durability of the material. The degradation of concrete subjected to high temperature is strongly related to the cement matrix and aggregates. An increase in temperature causes the dehydration and decomposition of CSH, CH and decomposes of $CaCO_3$ in carbonate aggregate.

This chapter also provides an insight into the research activities and applications relating to use of wastes refractory brick. A limited number of researchers of the literature have found that recycled refractory brick can be incorporated into concrete as fine aggregate or supplementary cementitious material in order to improve certain properties of concrete. Therefore, it is interesting to investigate the potential of using refractory brick as coarse aggregate to produce concrete.

In the remainder of the thesis, we propose to study the potential of using refractory brick as coarse and fine aggregate to produce concrete. In particular, this study focused on the assessment of post-fire residual physical and mechanical properties of concrete.

Chapter II:
EXPERIMENTAL PROGRAM

II.1 INTRODUCTION

As mentioned in the literature review section, the natural of aggregates have a great affect on concrete behaviour at high temperature. Therefore, the choice of aggregates is a key factor for the thermal stability of concrete exposed to elevated temperatures. This requires finding other sources that have better thermal stability and environmentally friend to replace NA. These reasons have led us to use a new type of aggregates, which is Refractory Bricks Aggregate (RBA). The use of refractory brick wastes as aggregates can make significant contribution to environmental protection, and also improve the behaviour of concrete at elevated temperatures because this type of bricks used in applications requiring high temperatures.

Accordingly, the influence of RBA as aggregates (coarse or fine) on physical and mechanical properties of concrete at room and high temperatures has never been studied in the literature. Hence, it is interesting to investigate the potential of using coarse RBA or both coarse and fine RBA to produce concrete. Additionally, it is necessary to examine concrete made with RBA when exposed to high temperature.

Firstly, in order to find the optimum of coarse RBA replacement ratio to produce concrete with acceptable physical and mechanical properties, two series of concrete mixes prepared with two types of RBA as coarse aggregate by partial or total substitution of coarse NA at different content (0%, 10%, 20%, 30%, 40%, 50%, 70% and 100%) were prepared. For each series, five tests including water porosity, density, UPV, compressive strength and dynamic modulus of elasticity were conducted.

Secondly, after finding the optimal rate of substitution of the coarse RBA, three families of mixes were prepared and exposed to five temperatures (150°C, 250°C, 400°C, 600°C and 800°C). The first mix, of three reference mixtures (C1NA, C2NA and C3NA), was made with 100 % coarse and fine NA and determined according to Dreux mix design method [Dreux et Festa, 1998]. From the reference mixture, the second mix was developed by replacing 20% of coarse NA by coarse RBA. The third mix was also developed by replacing 20% of coarse and fine NA from the reference mixtures by coarse and fine RBA-2. For each of the mentioned families, a series of experiments including compressive strength, dynamic modulus of elasticity, UPV, damage degree, mass loss, change of volume, density and water porosity were investigated. In order to achieve an acceptable compressive strength and contribution to the general understanding of the impact of cement dosages (w/c ratios) on physical and mechanical properties of concrete at high temperatures, a different cement dosage was used 350 kg/m^3 (w/c=0.59), 400 kg/m^3 (w/c=0.47) and 450 kg/m^3 (w/c=0.38).

This chapter discusses the type of construction materials used in this experiment, proportioning of concrete mixtures, production of concrete specimens and test techniques and procedures.

II.2 METHODOLOGY AND EXPERIMENTAL DETAILS

II.2.1 Materials

II.2.1.1 Cement

The cement used in all studied concretes was Portland cement (CPJ CEM II/A 42.5). Table II.1 presents the chemical composition of this cement measured by the LPCMA laboratory. The mechanical properties of this cement were measured by the Biskria Cement and results obtained are reported in Table II.2.

Table II.1. Chemical composition of cement.

Chemical composition of cement (%)							
CaO	SiO_2	Fe_2O_3	Al_2O_3	MgO	SO_3	K_2O	Na_2O
60.10	16.64	3.39	4.34	1.29	3.29	0.68	0.19

Table II.2. Mechanical properties of cement.

Compressive strength (MPa)	2 days	18.20
	7 days	30.40
	28 days	35.00
Flexural strength (MPa)	2 days	1
	7 days	1.39
	28 days	2.7

From the results obtain in the Table II.2**,** It can be observed that the compressive strength of this cement, measured after 28 days (35 MPa), did not reach the required strength.

II.2.1.2 Water

The water used for mixing and curing of all concrete specimens was a tap water, which is available inside the laboratory. The water was believed to be drinkable and did not contain harmful impurities.

II.2.1.3 Superplasticizer

The superplasticizer (GLENIUM® 27) was used for mixing concretes with cement dosage of 450 kg/m^3(w/c=0.38) to provide the same workability without increasing the water content. Superplasticizer is characterized by a density of 1.05±0.02 and ph value of 6.0 ± 1. The superplasticizer range recommended by the manufacturer is 0.3% to 3% by weight of cement.

II.2.1.4 Aggregates

The aggregates employed in this study can be divided in two separate categories: natural aggregates and recycled refractory brick aggregates.

- **Natural Aggregates (NA)**

The coarse natural aggregate used in this study was limestone crushed stone with a diameter of 5/25 mm, while the fine natural aggregates used was siliceous sand with a diameter of 0/5 mm. Figure II.1 shows the different natural aggregates used.

(a) **(b)**

Figure II.1. Natural Aggregates (NA): (a) coarse NA (b) fine NA.

- **Recycled Refractory Brick Aggregates**

The recycled Refractory Brick Aggregates (RBA) used in this study can be divided into two types, based on its source. The first type, RBA-1, was obtained from crushing news refractory bricks (used for the manufacture of fireplaces), see figure II.2-a. The second type, RBA-2, was obtained from a big block of refractory brick after their uses in the furnace basin of cement factory (figure II.2-b).

(a) (b)

Figure II.2. Recycled Refractory brick before crushing: (a) RBA-1 (b) RBA-2.

❖ Preparation steps of RBA

The two types of RBA have undergone laboratory preparation processes, namely crushing and sieving, as shown in figure II.3. It is shown in this figure that first stage involved crushing the RBA-1 or RBA-2 using a small jaw crusher to obtain coarse RBA with a fraction of 5/15 and 15/25 mm. The second stage involved crushed a quantity of coarse RBA-2 using a Los Angeles abrasion machine to obtain fine RBA-2 with a fraction of 0/5mm. It should be noted that fine RBA-2 has the same chemical composition as coarse RBA-2 and come from the same source. Coarse RBA-1, coarse RBA-2 and fine RBA-2 are shown in figure II.4.

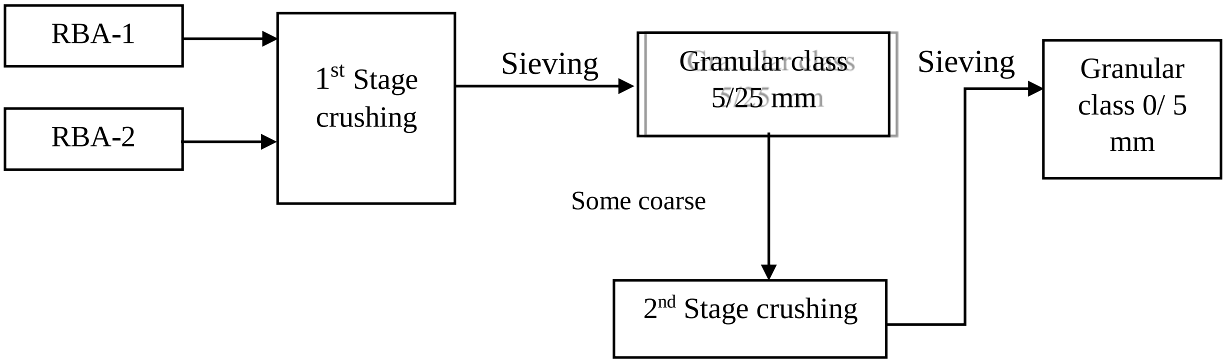


Figure II.3. Preparation steps of RBA-1 and RBA-2.

(a) (b) (c)

Figure II.4. Refractory brick aggregate after crushing: (a) coarse RBA-1 (b) coarse RBA-2 (c) fine RBA-2.

❖ Physical properties of aggregates used

Standard tests were performed to determine physical properties of coarse and fine NA, coarse RBA and fine RBA-2. A summary of these tests and physical properties are given in tables II.3 and II.4, respectively.

Table II.3. Test methods used to characterize natural aggregates and recycled refractory brick aggregates.

Property under evaluation	Standard used
Determination of density, porosity, absorption coefficient and water content of coarse aggregates	NF P 18–554
Determination of density, absorption coefficient and water content of fine aggregates	NF P 18–555
Determination of resistance to fragmentation. Los Angeles test	NF P 18–573
Determination of Sand equivalent	NF P 18–598

Table II.4. Physical properties of natural aggregates and recycled refractory brick aggregates.

Physical properties	**NA**	**NA**		**RBA-1**		**RBA-2**		**RBA-2**
	0/5 mm	15/25 mm	5/15 mm	15/25 mm	5/15 mm	15/25 mm	5/15 mm	0/5 mm
Apparent density (g/cm^3)	1.55	1.39	1.43	1.06	1.05	1.33	1.33	1.59
Absolute density (g/cm^3)	2.58	2.7	2.69	2.50	2.49	2.90	2.90	3.00
Water absorption (%)	0.67	0.26	0.22	7.45	7.12	3.53	2.88	4.32
Porosity (%)	-	1.56	1.16	17.52	16.99	9.57	9.30	-
Water content (%)	0.08	0.10	0.11	0.06	0.06	0.06	0.06	0.09
Los Angeles abrasion	-	30		61		58		-
Sand equivalent (%)	75.5	-		-		-		86.6
Finesse modulus	2.51	-		-		-		2.60

As seen in Table II.4, coarse RBA-2 presents a higher density, higher water porosity and higher water absorption than coarse NA. However, RBA-1 presents a lower density and higher water porosity and water absorption compared to coarse RBA-2 and coarse NA. Coarse RBA-1 and RBA-2, by comparison with coarse NA, have lower fragmentation resistance.

When comparing fine RBA-2 and fine NA of similar size, it is found that fine RBA-2 has a higher density and water absorption than fine NA. It is also found that fine RBA-2 has a comparable finesse modulus than fine NA. Finally, fine RBA-2 presents a higher sand equivalent value compared to fine NA.

Due to the high water absorption of coarse RBA (RBA-1 and RBA-2) and fine RBA-2, it was necessary to compensate for the water absorbed in order to prevent the absorption of the mixing water. In this study, in order to overcome the water absorption problems of coarse RBA, the following method was adopted: RBA aggregates were soaked in water then air-dried to a saturated dry condition before mixing with other ingredients of concrete. To estimate the coarse RBA water absorption through time, we have measured the water absorption of these aggregates for each hour; the results obtained are given in figure II.5.

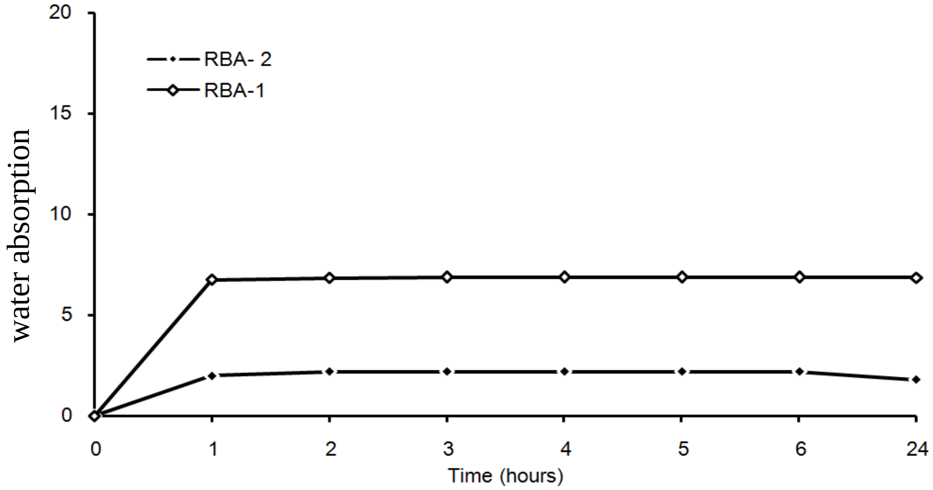


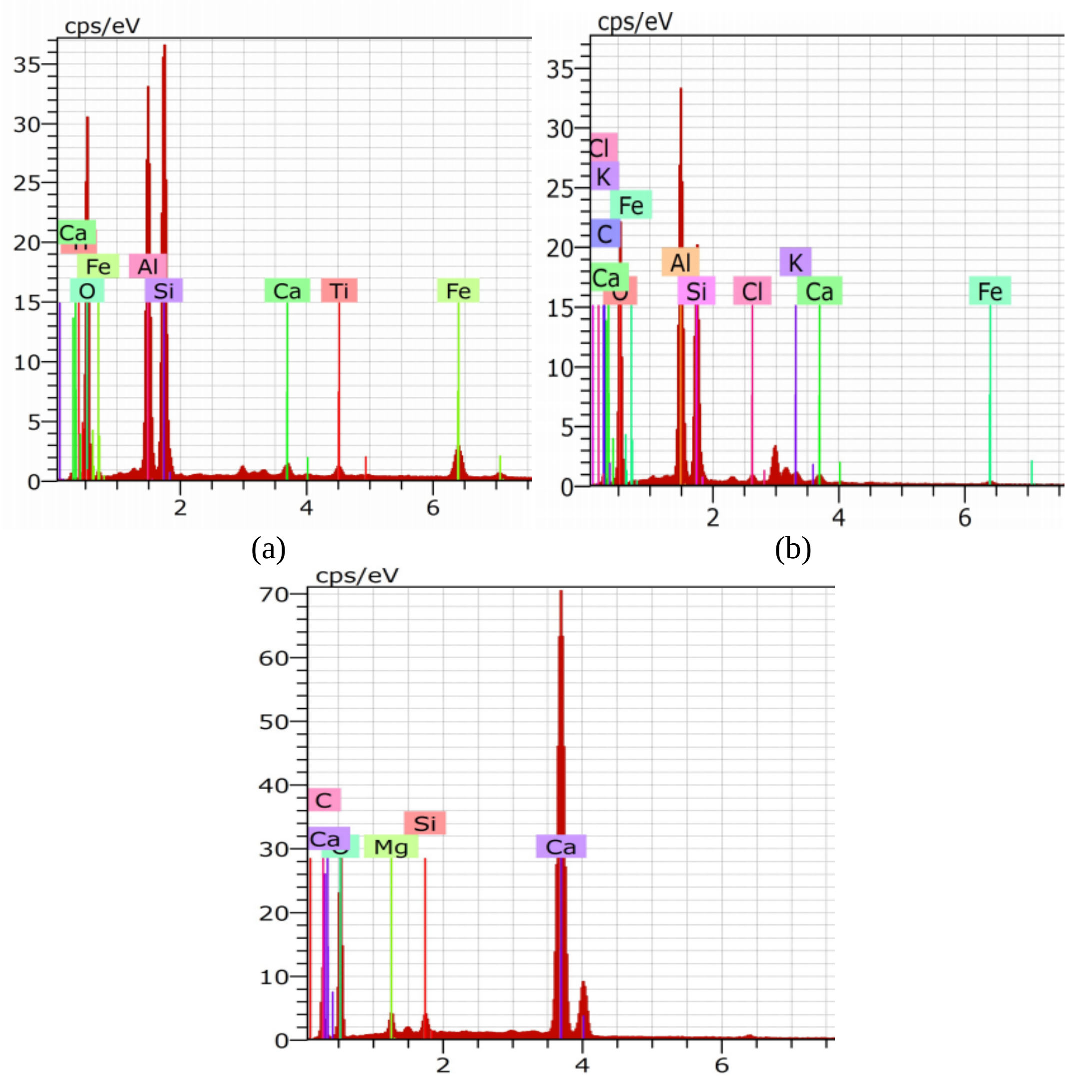
Figure II.5. Evolution of the water absorption of coarse RBA with time.

It can be observed from figure II.5 that most of the water absorption occurs during the two first hours of immersion. After 4 hours of immersion in water, the water absorption of the two types of coarse RBA stabilizes.

For mixture produced with fine RBA-2, additional water was added, which corresponding to the water absorbed by the fine RBA-2.

- **Chemical compositions and microstructure of aggregates used**

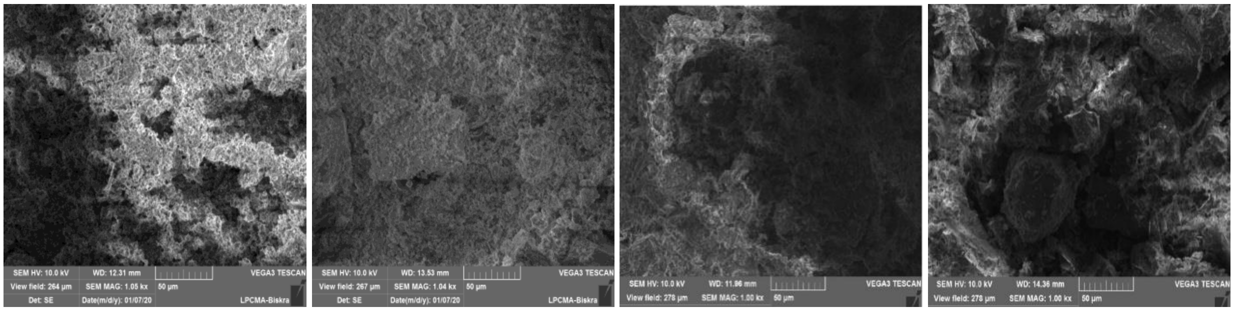
In this study, chemical characterization by Energy Dispersive Xray spectroscopy (EDX) was used to know the different chemical compounds of coarse NA and coarse RBA. The results obtained are shown in Figure II.6. Whereas, Figure II.7 shows the microstructure of the natural aggregates and recycled refractory brick aggregates which were performed by a Scanning Electron Microscopy (SEM).



(a) (b)

(c)

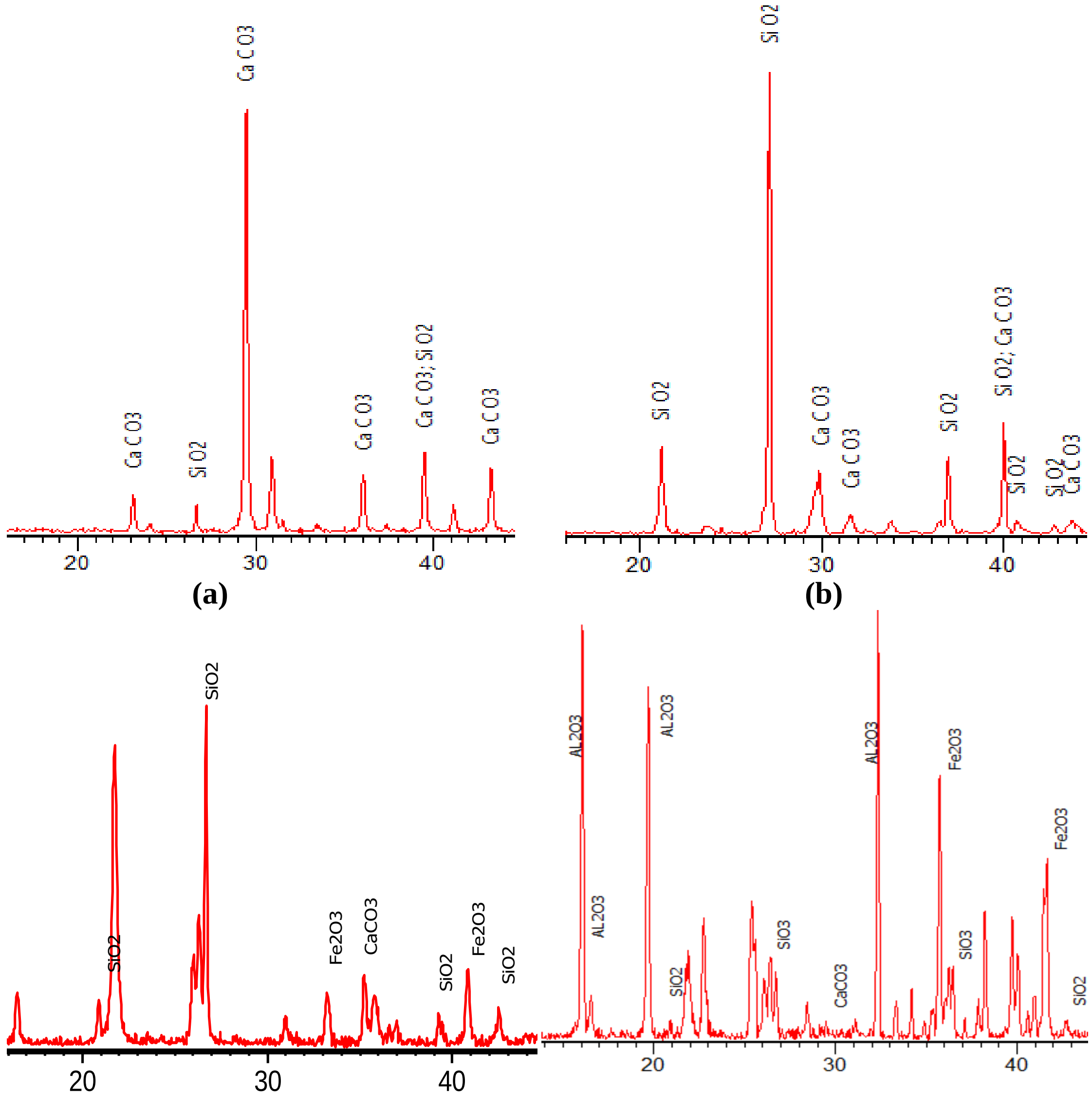
Figure II.6. Microanalysis (EDX) of aggregates used: (a) RBA-1 (b) RBA-2 (c) coarse NA.



(a) (b) (c) (d)

Figure II.7. SEM image of aggregates used: (a) coarse NA (b) Fine NA (c) RBA-1(d) RBA-2.

Moreover, the composition of natural aggregates and recycled refractory brick aggregates mineralogical in this study was analyzed by X-ray diffraction (XRD); the results are presented in figure II.8.



(a) **(b)**

(c) (d)

Figure II.8. X-ray diffraction of aggregates used: (a) coarse NA (b) Fine NA (c) coarse RBA-1 (d) coarse RBA-2.

As shown in figure II.8, results of mineralogy analysis of coarse NA, fine NA, RBA-1 and RBA-2 appear to indicate that calcite, quartz; quartz and alumina was the prevalent mineral, respectively.

II.2.2. Particle size distribution of aggregates

The particle size distribution was performed for natural aggregates and recycled refractory brick aggregates, which has an important influence on concrete manufacture. The grain size distribution of natural aggregates and recycled refractory brick aggregate was performed according to French Standardization P 18-560 [P 18-560,1990]; the results obtained are illustrated in figure II.9 and II.10, respectively.

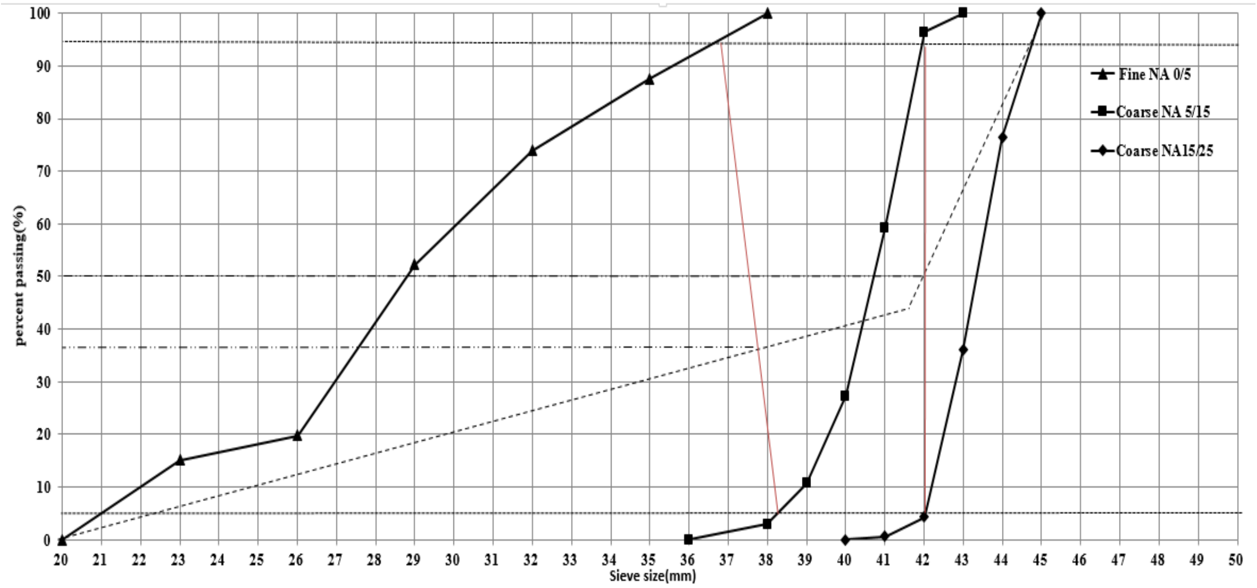


Figure II.9. Grain size distribution of natural aggregates.

The grain size distribution of natural aggregates (fine NA 0/5 mm, coarse NA 5/15 mm and coarse NA 15/25 mm) are given in figure II.9, on the same figures the optimum particle size curve according to the Dreux-Gorisse method [Dreux and Festa, 1998] is also plotted.

On a granulometric analysis graph, three-point (A, O and B) were drawn. The first point "O" corresponds to the origin of the coordinate (0, 0). The second point "B" (at 100%) corresponds to the maximum dimension of the greatest aggregate. The third point "A"is called the breaking point, and is located in the middle of the "gravel segment" limited by modulus of 38 (corresponding to 5 mm) and the corresponding modulus to D_{max}.

In order to obtain the absolute volume percentages of aggregates, the dividing lines between each aggregates type are drawn. This is performed by drawing lines between the point of 95% of the curve of the sand (0/5 mm) and the point of 5% of the curve of coarse aggregates (5/15 mm).

Another line was drawing between the point of 95% of the curve of the coarse aggregates (5/15 mm) and the point of 5% of the curve of coarse aggregates (15/25 mm), see figure II.9.

In this study, three cement dosages of 350 kg/m^3, 400 kg/m^3 and 450 kg/m^3 were investigated. Therefore, when cement dosages are increased, the water/cement ratio decreases (w/c ratio equal to 0.59, 047 and 0.38, respectively), and therefore the aggregates content is changed.

The crossing point of the curve with the dividing lines allows read the absolute volume percentage of each aggregates. According to figure II.9, we find the following results:

- For concrete mixture with cement dosages of 350 kg/m^3, the percentage of sand was 38%, 12% for aggregates of 5/15 mm fraction and 50% for aggregates of 15/25 mm fraction.

With the same previous work steps, we can find the volume of aggregates for concrete mixture with cement dosages of 400 kg/m^3 and 450 kg/m^3.

- For concrete mixture with cement dosages of 400 kg/m^3, we obtained 37% for sand, 13% for aggregates of 5/15mm fraction and 50% for aggregates of 15/25mm fraction.
- For concrete mixture cement dosages of 450 kg/m^3, we obtain 36% for sand, 12% for aggregates of 5/15mm fraction and 52% for aggregates of 15/25mm fraction.

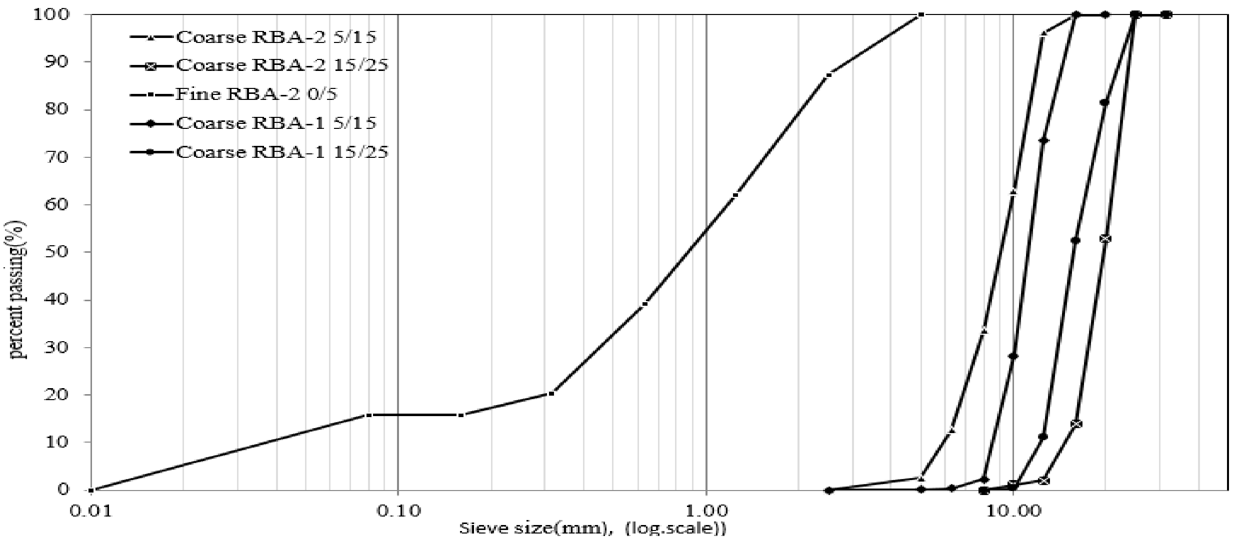


Figure II.10. Grading curves of RBA.

II.2.3 Concrete design

As mentioned before, the main purpose of this study is to determine the effect of the incorporation of RBA on the physical and mechanical properties of concrete at room temperatures and achieve a better understanding of the influence of elevated temperature on the performance of concrete containing this type of aggregates.

In this research, several concrete mixtures with different cement dosages (350 kg/m^3 (w/c=0.59), 400 kg/m^3 (w/c=0.47) and 450 kg/m^3 (w/c=0.38)) were prepared and tested at room temperatures and after exposed to elevated temperature. The present paragraph provides the details of mixtures designations and percentages of different materials in the mix.

II.2.3.1 Reference concrete mixtures

Three concrete mixtures with cement dosages of 350 kg/m^3 (C1NA), 400 kg/m^3 (C2NA) and 450 kg/m^3 (C3NA) were used as a reference mixture. These concrete mixtures were prepared according to the Dreux mix design method [Dreux et Festa, 1998] and made only of coarse and fine natural aggregates. In the following, the reference mixtures C1NA, C2NA and C3NA have compressive strength at 28 days of about 25 MPa, 33 MPa and 50 MPa, respectively. These mixtures were used as reference at ambient temperatures and after exposure to high temperature. The details of reference mixtures were illustrated in Table II.5.

II.2.3.2 Recycled bricks mixtures

In order to investigate the possibility of using RBA as coarse aggregates for manufacturing concrete, and find the optimum replacement ratio, a total of 28 mixtures were designed into two families of mixes and tested at room temperature. Its mix proportion are given in Table II.6.

- The first family, of fourteen mixtures with a cement dosage of 350 kg/m^3 (w/c=0.59) and 450 kg/m^3 (w/c=0.38), was prepared by substitution of coarse NA by coarse RBA-1 at the replacement ratio of 10%, 20%, 30%, 40%, 50%, 70% and 100% (by volume).
- The second family, of fourteen mixtures with a cement dosage of 350 kg/m^3 (w/c=0.59) and 450 kg/m^3 (w/c=0.38), was prepared by replacing coarse NA by coarse RBA-2 at the replacement ratio of 10%, 20%, 30%, 40%, 50%, 70% and 100% (by volume).

After finding the optimum coarse RBA replacement ratio, a total of nine mixures were designed into three series of concrete mixes and tested at high temperature. Its mix proportion are given in Table II.7.

- The first series, of three mixtures with cement dosages of 350 kg/m^3 (w/c=0.59), 400 kg/m^3 (w/c=0.47) and 450 kg/m^3 (w/c=0.38), were developed by replacing 20% of corse NA from the reference mixtures by coarse RBA-1 and were named C1-RBA1, C2-RBA1 and C3-RBA1, respectively.
- The second series, of three mixtures with cement dosages of 350 kg/m^3 (w/c=0.59), 400 kg/m^3 (w/c=0.47) and 450 kg/m^3 (w/c=0.38), were made by replacing 20% of corse NA from the reference mixtures by coarse RBA-2 and were named C1-RBA2, C2-RBA2 and C3-RBA2, respectively.
- The third series, of three mixtures with cement dosages of 350 kg/m^3 (w/c=0.59), 400 kg/m^3 (w/c=0.47) and 450 kg/m^3 (w/c=0.38), were developed by replacing 20% of corse and fine NA from the reference mixtures by coarse and fine RBA-2, which were named C1'-RBA2, C2'-RBA2 and C3'-RBA2, respectively. The percentage of fine RBA-2 (20%) used was taken from the results obtained in a previous study [Saidi et al.,2015].

For the mix designations in Tables II.5, 6 and 7, the numbers “0, 10, 20, 30, 40, 50, 70 and 100” in this naming method refer to the replacement ratio of NA by RBA. Letter “C” refer to concrete, “1” for cement dosage of 350 kg/m^3 (w/c=0.59), “2” for cement dosage of 400 kg/m^3 (w/c=0.47) and “3” for cement dosage of 450 kg/m^3 (w/c=0.38), “NA”’ refer to natural aggregates (coarse and fine).

Tables II.5. Mix proportion of reference mixtures (kg/m^3).

Mixtures	**Cement**	**Water**	**w/c**	**S**	**Sp (%)**	**Coarse NA (mm)**	
						5/15	**5/25**
C1NA	350	206.50	0.59	688.34	-	226.64	947.84
C2NA	400	190.48	0.47	654.83	-	239.88	926.06
C3NA	450	173.08	0.38	639.43	1.40	180.19	958.54

Tables II.6. The mix proportion of recycled brick mixtures "tested at room temperatures"(Kg/m^3).

Mixes	**C**	**W**	**w/c**	**S**	**NA(mm)**	**RBA-1 (mm)**	**RBA-2 (mm)**	**SP**

					15/25	5/15	15/25	5/15	15/25	5/15	(%)
C1NA	350	206.50	0.59	688.34	947.84	226.64	-	-	-	-	-
C1RBA1/10					853.05	203.97	87.76	21.00	-	-	
C1RBA1/20					758.27	181.31	175.53	42.00	-	-	
C1RBA1/30					636.48	158.65	263.27	63.01	-	-	
C1RBA1/40					568.70	135.98	351.05	84.01	-	-	
C1RBA1/50					473.92	113.32	438.82	105.02	-	-	
C1RBA1/70					284.35	67.99	614.34	147.03	-	-	
C1RBA1/100					-	-	877.63	210.04	-	-	
C1NA					947.84	226.64	-	-	-	-	
C1RBA2/10					853.05	203.97	-	-	101.94	24.47	
C1RBA2/20					758.27	181.31	-	-	203.90	48.93	
C1RBA2/30					636.48	158.65	-	-	305.83	73.40	
C1RBA2/40					568.70	135.98	-	-	407.78	97.86	
C1RBA2/50					473.92	113.32	-	-	509.72	122.33	
C1RBA2/70					284.35	67.99	-	-	713.61	171.27	
C1RBA2/100					-	-	-	-	1019.5	244.67	
C3NA	450	173.08	0.38	639.43	958.54	180.19	-	-	-	-	1.4
C3RBA1/10					862.69	162.17	88.75	16.70	-	-	
C3RBA1/20					766.83	144.15	177.51	33.40	-	-	
C3RBA1/30					670.98	126.13	266.26	50.10	-	-	
C3RBA1/40					575.12	108.11	355.02	66.80	-	-	
C3RBA1/50					479.27	90.09	443.77	83.77	-	-	
C3RBA1/70					383.42	54.06	621.28	116.89	-	-	
C3RBA1/100					-	-	887.54	166.54	-	-	
C3NA					958.54	180.19	-	-	-	-	
C3RB2/10					862.69	162.17	-	-	103.10	19.45	
C2RB2/20					766.83	144.15	-	-	206.19	38.90	
C3RB2/30					670.98	126.13	-	-	309.29	58.36	
C3RBA2/40					575.12	108.11	-	-	419.39	77.81	
C3RBA2/50					479.27	90.09	-	-	515.48	97.26	
C3RBA2/70					383.42	54.06	-	-	721.67	136.17	
C3RBA2/100					-	-	-	-	1030.6	194.52	

Tables II.7. Mix proportion of RBA concrete mixtures tested at high temperatures (kg/m^3).

	C1-RBA1	C2-RBA1	C3-RBA1	C1-RBA2	C2-RBA2	C3-RBA2	C1'-RBA2	C2'-RBA2	C3'-RBA2

Cement	350	400	450	350	400	450	350	400	450
Water	206.50	190.48	173.08	206.50	190.48	173.08	206.50	190.48	173.08
Extra water	-	-	-	-	-	-	6.95	6.58	6.42
w/c	0.59	0.47	0.38	0.59	0.47	0.38	0.59	0.47	0.38
NA 0/5	688.34	654.83	639.43	688.34	654.83	639.43	550.67	523.86	511.54
RBA-2 0/5	-	-	-	-	-	-	160.08	152.29	148.70
NA 5/15	181.31	191.91	144.15	181.31	191.91	144.15	181.31	191.91	144.15
NA 15/25	758.27	740.85	766.83	758.27	740.85	766.83	758.27	740.85	766.83
RBA-1 5/15	42.00	44.46	33.40	-	-	-	-	-	-
RBA-1 15/25	175.53	171.49	177.51	-	-	-	-	-	-
RBA-2 5/15	-	-	-	48.93	51.79	38.90	48.93	51.79	38.90
RBA-2 15/25	-	-	-	203.90	199.21	206.19	203.90	199.21	206.19
Sp (%)	-	-	1.40	-	-	1.40	-	-	1.40

II.2.4 Production of concrete specimens

II.2.4.1 Mixing of specimens

Concrete specimens were mixed in a vertical axle mixer. Before starting the mixing process, the mixer was cleaned and moistened with water to avoid absorbing the mixture water. The mixing procedure of concrete specimens is presented in figure II.11.

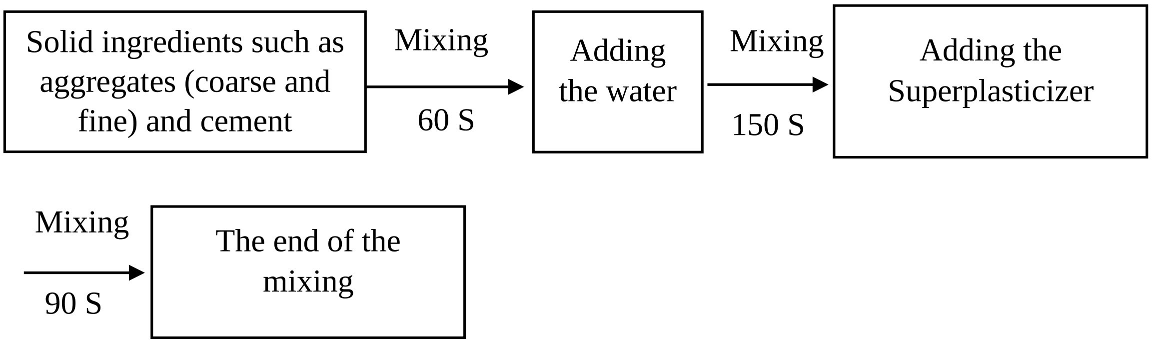


Figure II.11. Forming process of concrete.

After the end of the mixing procedure, the mixer is emptied into steel molds in two layers. Each layer was compacted on a vibrating table for 10 seconds. When the manufacture was finished, the surface of all samples was covered with plastic bags for 24 hours.

II.2.4.2 Curing conditions of specimens

Curing can be described as keeping the concrete moist enough so that the hydration of cement can continue. The curing method of different concretes was performed in accordance with the standard NF P 18-404 [NF P 18-404,1981].

After 24 hours, all specimens were removed from the molds and were cured in a water-curing tank at a constant temperature of 20±2°C for 28 days. The curing of concrete specimens is given in figure II.12.

Figure II.12. Curing of concrete specimens in water for 28 days.

II.2.4.3 Description of test molds

The experimental program consisted of using cubes molds with dimensions of 10×10×10 cm^3 and 4×4×4 cm^3. These molds are made of metal (steel) and characterized by an impermeable, non-absorbent and non-reactive with cement. Cubic specimens of 10×10×10 cm^3 were used for measuring compressive strength, mass loss, UPV, dynamic modulus of elasticity, damage degree and change of volume before and after heating. Cubic specimens of 4×4×4 cm^3 were used for measuring density and water porosity before and after heating.

II.2.5 Exposure to elevated temperatures

As stated earlier in the literature review section, when concrete is exposed to high temperatures, several transformations of physical and chemical properties can occur. In the present study, six temperatures (150°C, 250°C, 400°C, 600°C and 800°C) were adopted as they represent key temperatures responsible for several degradation phenomena in the cement matrix. In particular, these temperatures were chosen because they correspond to evaporation of free and physically bound water, decomposition of ettringite and the first stage of decomposition of CSH (150°C). On the other hand, the CSH decomposition becomes faster at 200°C and disappears at a temperature of 400°C, while the CH decomposes between 450°C and 550°C. Regarding the aggregates, at 573°C, the crystal quartz suffers transformation from α-quartz to β-quartz. Temperature of 800°C corresponds to the decomposition of $CaCO_3$ in cement paste and carbonate aggregates.

II.2.5.1 Heating equipment

In this study, two electric furnaces were used to heat the specimens as shown in figure II.13. The first furnace has internal dimensions of 25 cm × 25.5 cm × 35.5 cm (wide × long × high) with the maximum temperature of 1200°C (figure II.13-a). The second furnace has internal dimensions of 380 cm × 380 cm × 480 cm (wide × long × high) with the maximum temperature of 800 °C (figure II.13-b). The temperature inside the furnace chamber is controlled by a control panel connected to a thermocouple in the furnace. As shown in figure II.14**,** the test specimens were stacked with sufficient space between two adjacent specimens to obtain uniform heating in each specimen.

(a) (b)

Figure II.13. Furnaces used for heating concrete specimens.

Figure II.14. View of concrete specimens inside the furnace.

II.2.5.2 Heating and cooling method

After curing, the specimens were kept under laboratory condition for 2 days heating in order to reduce moisture content, as can be seen in figure II.15. In the experimental program, three specimens from each mixture were heated from room temperature up to 150°C, 250°C, 400°C, 600°C and 800°C with a constant rate of heating (3°C/min). Room temperature (20 ± 2) °C is considered as reference temperature. The rate of heating refers to the recommendations of the ISO/TR 15655 [ISO/TR 15655, 2003]. Heating phase was followed by the temperature stabilization phase for 1 h to maintain thermal stability inside the specimens and achieve good temperature homogeneity. When the heating process was completed, heated specimens were naturally cooled inside the electrical furnace at a rate of about 1°C/min without opening the furnace door. The time–temperature curves for the furnace are given figure II.16.

Figure II.15. Concrete specimens in a laboratory condition for 2 days before exposure to high temperature.

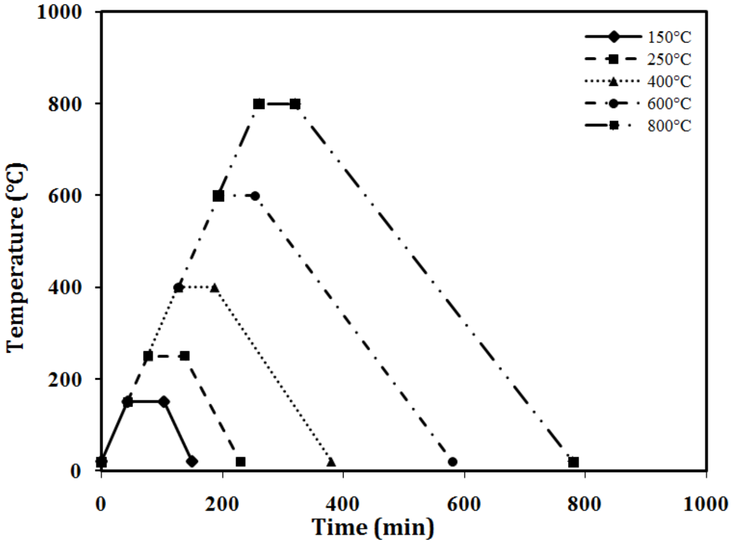


Figure II.16. Heating and cooling curves.

II.2.6 Test plan

In order to study the effect of replacing coarse NA by coarse RBA on the mechanical and physical properties of concretes, several tests on concrete made with such aggregates were conducted. Table II.8 presents the tests carried out as part of this work depending on the type of RBA and replacement ratios.

Moreover, the influence of temperature on mechanical and physical properties of concrete mixes prepared with different combinations of aggregates was investigated. Namely, mixes made with 100% of NA, mixes made with 20% of coarse RBA (RBA-1or RBA-2) and mixes made with 20% of coarse and fine RBA-2. The test plan is presented in Table II.9 includes the characteristics of the specimens and the maximum temperature that the specimens were subjected to.

More details on the different experimental tests carried out in this study are given in the following paragraphs.

❖ Tests performed on hardened concrete specimens

On the basis of the data reported in Table II.8, a total of 112 cubic specimens with the dimensions of 10×10×10 cm^3 were tested to determine the UPV, dynamic modulus of elasticity and compressive strength. Moreover, a total of 96 cubic specimens with the dimensions of 4×4×4 cm^3 were tested to determine the water porosity and density.

As presented in Table II.9**,** 216 specimens concrete with the dimensions of 10×10×10 cm^3 were tested to study the mechanical and physical properties of each type of concrete after exposure to high temperatures. Furthermore, mass loss, volume change and UPV, damage degree and dynamic modulus of elasticity were conducted on the same specimens before the compressive strength tests were performed. In addition, 216 specimens concrete with the dimensions of 4×4×4 cm^3 were also tested to determine water porosity and density of each type of concrete after exposure to high temperatures.

As shown in Tables II.8 and II.9, three tests as a minimum, for each combination of parameters, were carried out.

Table II.8. Quantity and program of tests carried out during the work at room temperature.

Mixes	T (°C)	water porosity and density	UPV, E_d, compressive strength
C1NA	20	3	3
C1RB1/10	20	3	3
C1RB1/20	20	3	3
C1RB1/30	20	3	3
C1RB1/40	20	3	3
C1RB1/50	20	3	3
C1RB1/70	20	3	3
C1RB1/100	20	3	3
C1NA	20	3	4
C1RB2/10	20	3	4
C1RB2/20	20	3	4
C1RB2/30	20	3	4
C1RB2/40	20	3	4
C1RB2/50	20	3	4
C1RB2/70	20	3	4
C1RB2/100	20	3	4
C3NA	20	3	3
C3RB1/10	20	3	3
C3RB1/20	20	3	3
C3RB1/30	20	3	3
C3RB1/40	20	3	3
C3RB1/50	20	3	3
C3RB1/70	20	3	3
C3RB1/100	20	3	3
C3NA	20	3	4
C3RB2/10	20	3	4
C2RB2/20	20	3	4
C3RB2/30	20	3	4
C3RB2/40	20	3	4
C3RB2/50	20	3	4
C3RB2/70	20	3	4
C3RB2/100	20	3	4

Table II.9. Quantity and program of tests carried out during the work at high temperature.

Properties	T (°C)	C1NA	C2NA	C3NA	C1-RBA1	C2-RBA1	C3-RBA1	C1-RBA2	C2-RBA2	C3-RBA2	C1'-RBA2	C2'-RBA2	C3'-RBA2
mass loss+ volume variation+ UPV+ E_d + damage degree+ Compressive strength	20	3	3	3	3	3	3	3	3	3	3	3	3
	150	3	3	3	3	3	3	3	3	3	3	3	3
	250	3	3	3	3	3	3	3	3	3	3	3	3
	400	3	3	3	3	3	3	3	3	3	3	3	3
	600	3	3	3	3	3	3	3	3	3	3	3	3
	800	3	3	3	3	3	3	3	3	3	3	3	3
water porosity and density	20	3	3	3	3	3	3	3	3	3	3	3	3
	150	3	3	3	3	3	3	3	3	3	3	3	3
	250	3	3	3	3	3	3	3	3	3	3	3	3
	400	3	3	3	3	3	3	3	3	3	3	3	3
	600	3	3	3	3	3	3	3	3	3	3	3	3
	800	3	3	3	3	3	3	3	3	3	3	3	3

- **Microscope observations**

In order to study the evolution of surface cracks of each type of concrete after exposure to high temperatures, a microscopic study was carried out after removing the samples from the furnace.

II.2.7 Concrete testing

As mentioned above, several tests were performed on hardened concretes. Hardened tests involved compressive strength, UPV, damage degree, dynamic modulus of elasticity, mass loss, volume variation, water porosity and density.

II.2.7.1 Compressive strength tests

The compressive strength of concrete can be defined as the measured maximum resistance of concrete to axial loading. The compressive strength of each concrete type prepared in Tables II.5, II.6 and II.7 was obtained by using equation (II.1).

$$f_s = F_{max}/ S \qquad \text{(II.1)}$$

Where f_s is the compressive strength; F_{max} is the max load applied; and S is the area of the test specimen to which the load is applied.

The max load applied (F_{max}) was obtained using a hydraulic press machine with a loading capacity of 3000 KN according to the standard NF EN 12390-3 [NF EN 12390-3,2003], while the length and width of the specimens were measured using Caliper Measurement to determine the surface area (S). When performing a compressive strength test, there are a few observations that are taken into account:

- At room temperatures, a max load applied and specimen dimension (length and width) were measured after 28 days.
- At high temperatures, a max load applied and specimen dimension (length and width) were measured after each heating/cooling cycles.
- The loading rate applied in the compressive strength tests was kept at 0.5 MPa/s until specimen rupture, according to standard NF EN 12390-4 [NF EN 12390-4,2000], see figure II.17.

Figure II.17. Test sample placed on the hydraulic press to determine the compressive strength.

II.2.7.2 Ultrasonic Pulse Velocity (UPV)

The UPV was considered as a non-destructive method, it is widely used for evaluation of quality of concrete. The main aim in performing this test was: (a) to evaluate homogeneity of concrete due to the incorporation of RBA, (b) to evaluate the quality of concrete after exposure to high temperatures and (c) to examine the damage degree of concrete due to the internal deterioration caused by heating. Yao et al. [Yao et al., 2020] reported in their studies that the use of UPV test to evaluate the performance degradation of concrete after exposure to elevated temperature is reasonable and feasible. The UPV tests were carried out in accordance to P 18-418 standard [AFNOR P 18-418,1989]. The UPV test was based on the propagation of sound waves inside a material through a pulse transmitter on one face of a specimen and receiving the wave on the opposite face, which leads to measuring the time needed to spread the wave through the material. The UPV can be measured from the path length divided by the transit time using the following equation:

$$v = l/s \qquad \text{(II.2)}$$

Where v is the pulse velocity, l is the length of the sample and s is the time.

In addition, the data obtained in the ultrasonic tests was also used to examine the damage degree of concrete due to heating following equation (II.3) [Yao et al., 2020], [khattab et al.,2022-b]:

$$D = 1 - (\mathrm{v}_T/\mathrm{v}_0)^2 \quad \text{(II.3)}$$

Where D is the damage degree, v_T is the UPV after heating, and v_0 is the UPV before heating.

It should be noted that the path length was measured using the Caliper Measurement and grease was used to ensure perfect contact between the surface of specimens each transducer (transmitter and the receiver). The device of UPV used in this work is shown in figure II.18.

Figure II.18. UPV measuring device.

II.2.7.3 Modulus of elasticity

Dynamic modulus of elasticity is one of the most important material properties of concrete, as it is always used throughout the structural design process. In the experimental program, the values of dynamic modulus of elasticity were obtained using the values of UPV according to the standard ASTM C 597–16 [ASTM C 597–16,2016]. The dynamic modulus of elasticity from UPV values can be calculated using the following equation [Pal, 2019], [Laneyrie et al., 2017].

$$E_d = \frac{\rho \mathrm{v}^2 \ (1+v)(1-2v)}{(1-v)} \qquad \text{(II.4)}$$

Where E_d is the dynamic modulus of elasticity, v is Poisson's ratio (taken as $v = 0.2$ for concrete), **ρ** is the oven-dry density, and v is the UPV value.

II.2.7.4 Concrete's mass loss

The mass loss is an important parameter used to characterize concrete under high temperature. Mass loss is defined as the difference between the mass before and after temperature exposure. To determine mass loss, the mass of each specimen before being placed in the furnace (before heating) and after being removed from it (after heating) were weighed using an electronic digital balance with an accuracy of ±0.01 g. The average mass loss values were taken from three specimens for each concrete and each temperature cycle. The mass loss was determined according to equation (II.5).

$$M_{loss} = \frac{M_1 - M_0}{M_0} \times 100\% \qquad \text{(II.5)}$$

Where M_1 and M_0 represent the mass of specimen before and after heating, respectively.

II.2.7.5 Variation of specimen's volume

Concrete, like the majority of materials, changes volume when subjected to high temperature. In order to obtain the volume variation of concrete, the dimension of the specimens (length, width and height) before and after heating were measured. Specimen dimension was measured using the digital Caliper Measurement, which is capable to measure 300 mm long and had a measurement resolution of 0.01 mm.

For each dimension, three values were taken and average values were recorded as the final test results. Figure II.19 shows the digital Caliper Measurement used in this work.

The change of volume was determined according to equation (II.6).

$$V_{change} = \frac{V_1 - V_0}{V_0} \qquad \text{(II.6)}$$

Where V_1 and V_0 represent the volume of the specimen before and after heating, respectively.

Figure II.19. Digital Caliper Measurement.

II.2.7.6 Water porosity and density

Water porosity and density are regarded as the main properties of concrete; they play a major role in the durability of concrete. The density and water porosity tests were carried out according to the standard NF EN 12390-7 [NF EN 12390-7,2001].

- **Experimental methodology**

The concrete porosity and density were tested by a technique of water immersion. The principle of the technique is based on inferring the volume of voids from the mass difference between the dry and saturated state. To measure the concrete porosity and density, cubic specimens of 4×4×4 cm^3 were used, and the testing conditions were similar to those recommended by AFPC-AFREM [AFPC-AFREM, 1997].

- **Specimen's dry mass (Ms)**

For specimens tested at room temperature (20 ± 2) °C dry mass was obtained by drying in an oven at 60°C ±2°C until the weight stabilized. To estimate the dry mass of specimens through time, the mass of specimens were measured each 24 hour. The mass can be considered constant, when the difference between two successive weights separated by 24 hours does not exceed 0.1%. Figure II.20-a, shows the oven used to obtain dry mass.

For specimens tested at high temperature (150°C, 250°C, 400°C, 600°C and 800°C), dry mass was obtained after each heating/cooling cycle.

- **Specimen's saturation mass weighed in air (M_w)**

After obtaining a dry mass, all specimens were immersed into water. First, the specimens were placed in a container and then they were immersed in water up to half height of specimens for 15 minutes. This process allows air contains in the core of specimens to escape [Hager, 2004], [Hachemi, 2015]. After 15 minutes, the specimens are completely submerged in water, as shown in figure II.20-b.

Second, the specimens were weighted regularly until they were completely saturated. The mass can be considered saturated when the difference between two successive weights separated by 24 hours do not exceed 0.1%. Once the specimens are completely saturated, they were weighed at the surface-dried state (Mw).

- **Specimen's immersed mass (Mw')**

After the total saturation of specimens, the immersed mass (Mw') was determined with a hydrostatic balance (see figure II.20-c).

Water porosity (P) was determined according to the below equation:

$$P=\frac{Mw-Ms}{Mw-\mathrm{Mw}`} \qquad \text{(II.7)}$$

Density (D) was determined according to the following equation:

$$D=\frac{Ms}{Mw-\mathrm{Mw}`} \qquad \text{(II.8)}$$

(a) (b)

(c)

Figure II.20. Concrete porosity and density tests: (a) Drying oven (b) Specimens immersed into water (c) Device for measuring specimen immersed mass.

II.2.7.8 Microscope concrete testing

In order to observe the damage to the specimen’s surface by the appearance of new cracks in the concrete and aggregates caused by heating, a microscope study was carried out. This microscope (MPB-2 MAGNET 24×) can magnify approximately 24× and measures crack widths with measurement resolution of 0.05 mm. The microscope used in this work are shown in figure II. 21.

Figure II.21. Microscope MPB-2 MAGNET 24×.

II.2.8 Physical properties of aggregates after exposure to high temperatures

As mentioned earlier, five types of aggregates were used to prepare concrete. Two representatives of the natural aggregates group: siliceous sand and calcareous crushed stone and three from the recycled refractory brick aggregates group: coarse RBA-1, coarse RBA-2 and fine RBA-2.

In order to understand the influence of high temperature on concrete made from these aggregates, the effect of high temperature on the physical properties of aggregates used should be investigated.

The physical properties of different studied aggregates were evaluated in terms of mass loss, density, water porosity and absorption coefficient. Additionally, a microscopic study was carried out to note the development of aggregates degradation.

II.2.8.1 Heating and cooling method for aggregates

Two hundred grams (g) for each aggregate type were subjected to fourteen independent heating-cooling cycles with maximum temperatures fixed at 100°C, 200°C, 250°C, 300°C, 400°C, 500°C, 550°C, 600°C, 650°C, 700°C, 750°C, 800°C, 850°C and 900°C. Each cycle consists of three phases: heating at a rate of 10°C/min, stabilization at constant temperature for 1 minute and finally cooling down without opening the door of furnace. Figure II.22 presents the temperature developments as a function of time for the fourteen temperature cycles.

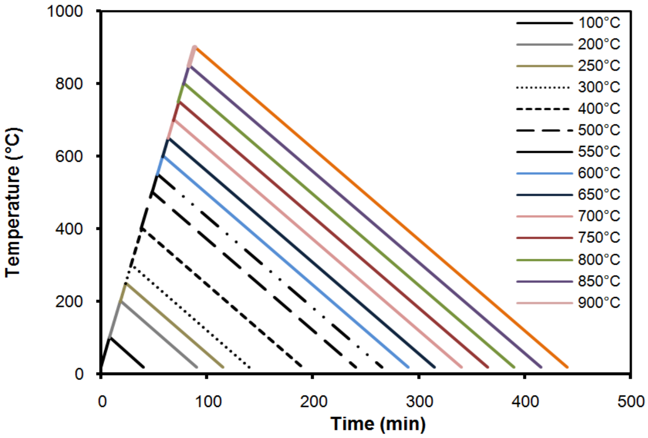


Figure II.22. Heating cycles for NA and RBA aggregates.

The heating was performed in the electric furnace mentioned in figure II.13-a. This furnace allows heating specimens up to 1200°C. Figure II.23 presents the arrangement of different aggregates in the furnace.

Figure II.23. View of aggregate inside furnace.

II.2.8.2 Aggregate mass loss

In order to obtain the aggregates mass loss, they were weighed before and after each temperature heating cycle using an electronic digital balance with an accuracy of ±0.01 g. Aggregates mass loss was determined according to Equation II.9.

$$M_{loss} = \frac{M_{1AG} - M_{0AG}}{M_{0AG}} \times 100\% \qquad \text{(II.9)}$$

M_{loss} : is aggregates mass loss.

M_{1AG}: is the initial mass of aggregates (g) (before heating).

M_{0AG} : is the heated mass of aggregates (g) (after heating).

II.2.8.3 Density and absorption coefficient of aggregates

The density and absorption coefficient of aggregates were measured according to the French standards NF P 18-554 [NF P 18-554,1990]. These properties were determined by a technique of water immersion. The dry mass of aggregates ($Ms_{,AG}$) was obtained after each heating/cooling cycle. Then, the aggregates samples were immersed in water for 24 hours. After that, the aggregates simples were removed from water and weighed after surface drying to obtain mass saturation ($Mw_{,AG}$). Finally, the immersed mass ($Mw'_{,AG}$) was measured using a hydrostatic balance.

The density (D) and absorption coefficient (A_b) of aggregates were calculated according to the following equations (II.10) and (II.11), respectively:

$$D_{AG} = \frac{Ms_{,AG}}{Mw_{,AG} - \mathrm{Mw}'_{,AG}} \qquad \text{(II.10)}$$

$$Ab_{\ AG} = \frac{Mw_{,AG} - \mathrm{Mw}'_{,AG}}{Ms_{,AG}} \qquad \text{(II.11)}$$

II.2.8.4 Water porosity

The water porosity was measured from the same test, but mass saturation ($Mw_{,AG}$) and the immersed mass ($Mw'_{,AG}$) were obtained after 7 days of immersion in water. Water porosity of aggregates was calculated according to the following equation (II.12):

$$P_{AG} = \frac{Mw_{,AG} - Ms_{,AG}}{Mw_{,AG} - \mathrm{Mw}'_{,AG}} \qquad \text{(II.12)}$$

Where $Mw_{,AG}$ and $\mathrm{Mw}'_{,AG}$ are the saturated mass of aggregate samples measured in the air and in the water, respectively.

II.3 CONCLUSION

This chapter considered the materials used in the production of concrete and some of the most important their physical properties. The different mixes' composition and the different processes of production are also identified and explained in this chapter. In addition, this chapter also discusses several tests which were performed on the different types of concretes at room temperature and after exposure to elevated temperature.

Twenty-eight concrete mixtures were produced to evaluate the mechanical and physical properties of concrete made with partial and total replacement of coarse NA by RBA (RBA-1 and RBA-2).

After finds the ideal replacement percentage of coarse NA by coarse RBA, several concrete mixtures were designed: mixes made with 20% of coarse RBA (RBA-1 and RBA-2), and mixes made with 20% of coarse and fine RBA-2. The experimental results of physical and mechanical properties of these concrete mixtures were compared with those obtained on conventional concretes made with 100 % of coarse and fine NA before and after exposed to high temperature. The results obtained are presented in Chapter III and IV.

Chapter III: RESULTS AND DISCUSSIONS

III.1 INTRODUCTION

In the previous chapter, materials used, several tests and procedures were described. This chapter presents the discussion of the results of tests carried out on aggregates and concretes. The results of all experiments are displayed in the form of tables or graphs.

The first part of this chapter presents the discussion of the results of physical and mechanical properties of concrete mixtures containing coarse RBA (RBA-1 and RBA-2) at room temperatures described in paragraph **II.2.3.2.** In the second part, we presented the discussion of the results of physical properties and macroscopic observations of aggregates used described in paragraph **(II.2.8)** (coarse NA, fine NA, coarse RBA-1, coarse and fine RBA-2) before and after heating. Additionally, the physical and mechanical properties of concretes having different characteristics described in paragraph **(II.2.3.2)** (concretes made with 100% of coarse and fine NA, concretes made with 20% of coarse RBA and concretes made with 20% of coarse and fine RBA-2) before and after exposure to high temperature are discussed.

III.2 INFLUENCE OF THE INCORPORATION OF COARSE RBA-1 OR RBA-2 ON PHYSICAL AND MECHANICAL PROPERTIES OF CONCRETE (20 ± 2°C)

This part presents the discussion of physical and mechanical properties of concrete made with coarse RBA-1 or RBA-2 obtained at room temperature.

III.2.1 Water porosity

Figure III.1 illustrates the water porosity of different concretes mixes. As can be seen in this figure, the water porosity increased with the increase of the substitution level of coarse RBA, which can be explained by the high porosity of coarse RBA.

It was observed that water porosity of concrete containing RBA-1 with percentage replacement of 10% 20% 30% 40% 50% 70% and 100% were increased by 14%, 18%, 24%, 18%, 23%, 31% and 42% for C1 and by 35%,36%, 37%, 43%, 45%, 60% and 68% for C3, respectively, compared to concrete prepared with natural aggregates. However, concretes containing coarse RBA-2 show an increase in water porosity lower than that of concrete containing coarse RBA-1. Water porosity of concrete containing RBA-2 was increased by 10%, 10%, 15%, 18%, 21%, 22% and 32% for C1 and by 28%, 24%, 32%, 35%, 35%, 47% and 49% for C3, for the same percentage replacement of 10% 20% 30% 40% 50% 70% and 100%, respectively. This might be explained by the lower porosity of RBA-2 compared to RBA-1 (RBA-2 porosity was 55% lower than that RBA-1).

It should be noticed from figure III.1 that concrete mixtures with cement dosage of 350 kg/m^3 (w/c=0.59) have a higher water porosity compared to the mixtures with a cement dosage of 450 kg/m^3 (w/c=0.38).

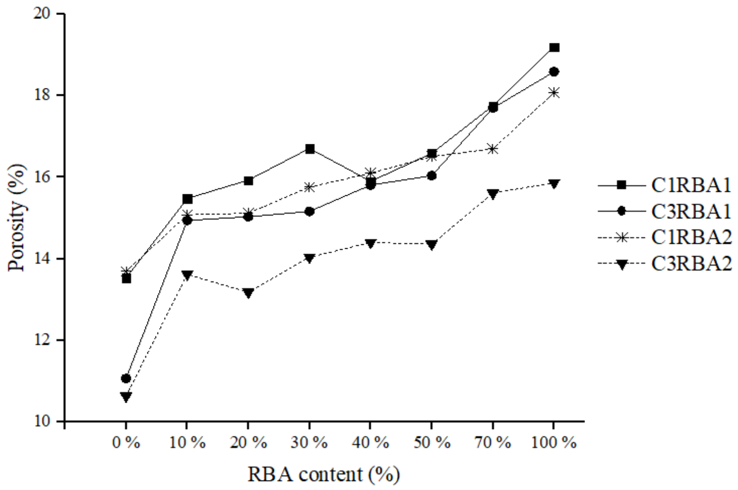


Figure III.1. Effect of coarse RBA-1 and RBA-2 content on water porosity of concrete.

III.2.2 Concrete density

Figure III.2 shows the replacement level effect of coarse NA by coarse RBA on concretes density. It can be observed that concretes prepared with coarse RBA have a lower density compared to concrete prepared with natural aggregates**.** The density of concretes was reduced by 6%, 7%, 9%, 8%, 10%, 12% and 16% for C1 concrete and by 6%, 6%, 6%, 7%, 10%, 12% and 18% for C3 concrete when coarse NA is replaced with coarse RBA-1 by 10%, 20%, 30%, 40%, 50%, 70% and 100%, respectively.

The inclusion of coarse RBA-2 in concrete results in a higher density when compared with that of concrete containing coarse RBA-1 at the same replacement levels, which can be explained by the high density of RBA-2 compared to RBA-1 density. When the percentage of replacement of coarse RBA-2 was 10%, 20%, 30%, 40%, 50%, 70% and 100%, the density shows a decrease of 5%, 6%, 5%, 4%, 4%, 4% and 6% for C1 concrete and 2%, 1%, 2%, 0.3%, 1%, 2% and 5% for C3 concrete, respectively, compared to concrete prepared with NA.

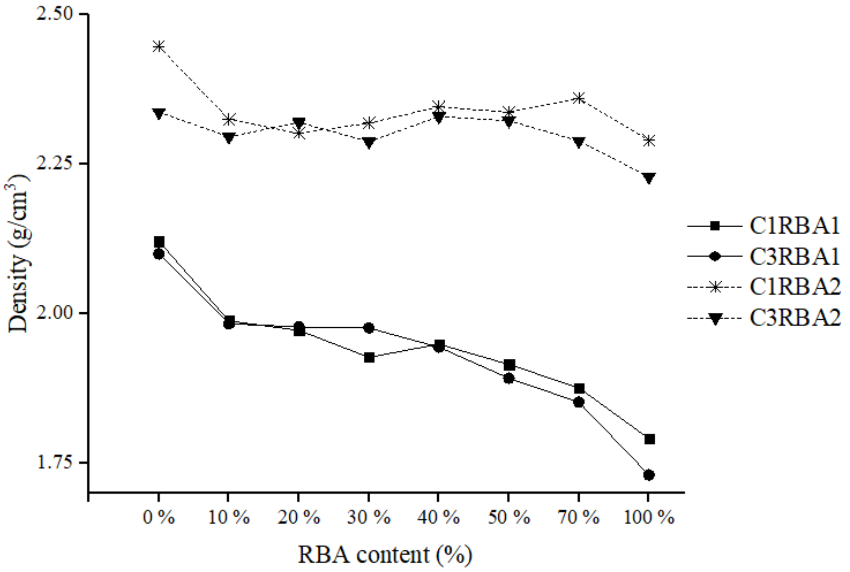


Figure III.2. Effect of coarse RBA-1 and RBA-2 content on concrete density.

Figure III.2 shows that the effect of cement dosage (w/c) on concrete density can be ignored since both concrete series (C1 with cement dosage of 350 kg/m^3 and C3 with cement dosage of 450 kg/m^3) exhibited almost the same density reduction rate with the increase of replacement percentage of coarse RBA. Generally, the inclusion of the coarse RBA has induced a slight decrease in concretes density. These results are similar to those of Aboutaleb [Aboutaleb et al., 2017] which observed that replacement of the natural sand with refractory brick has not a significant effect on concrete density.

III.2.3 UPV

The UPV test was used in order to assess the uniformity and quality of concrete as a non-destructive method. The UPV results of various concretes are given in figure III.3. The results obtained shows that UPV decreases for concrete containing either coarse RBA-1 or RBA-2 which can be explained by the higher porosity and inferior quality of coarse RBA.

Concretes containing coarse RBA-1 show a reduction of UPV about 5%, 9%, 11%, 6%, 12%, 16% and 24% for C1 concrete and 1%, 1%, 3%, 6%, 14%, 18% and 22% for C3 concrete when NA was replaced with RBA-1 by 10%, 20%, 30%, 40%, 50%, 70% and 100%, respectively, compared to concrete prepared with NA. Concrete mixtures containing RBA-2 with replacement percentage of 10%, 20%, 30%, 40%, 50%, 70% and 100% had a decrease of UPV by about 7%, 7%, 7%, 9%, 12%, 12% and 19% for C1 concrete and about 6%, 7%, 5%, 5%, 5%, 11% and 14% for C3 concrete, respectively, compared to concrete prepared with NA.

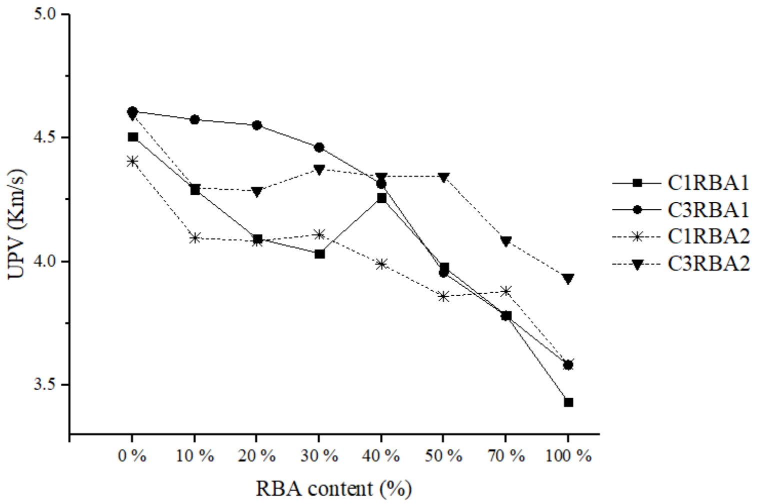


Figure III.3. Effect of coarse RBA-1 and RBA-2 content on UPV of concrete.

The results obtained in this test indicate that UPV values of all concretes containing RBA were greater than 3.5 km/s which can be classified as good quality concrete, except C1RBA1/100 (about 3,4 km/s).

It should be noticed from figure III.3 that the first series of concrete mixtures C1 with a cement dosage of 350 kg/m^3 had lower UPV values than that of the second concrete mixtures C3 with a cement dosage of 450 kg/m^3.

III.2.4 Compressive strength

Debieb and Kenai [Debieb and Kenai ,2008] reported in their studies that the substitution of NA with RBA should be done with caution as it can adversely affect the compressive strength. Figure III.4 presents the effect of replacement percentage of RBA on compressive strength for concrete mixes C1 and C3.

When coarse RBA-1 were replaced NA in C1 by 10%, 20%, 30%, 40%, 50%, 70% and 100%, the compressive strength tends to decrease by 22%, 18%, 21%, 27%, 29%, 30% and 18%, respectively. For C2 concrete, the decrease was about 26%, 11%, 11%, 22%, 22%, 23% and 20% for 10%, 20%, 30%, 40%, 50%, 70% and 100% of substitution, respectively.

The inclusion of coarse RBA-2 in concrete results in a lower decrease in compressive strength when compared to concrete made with coarse RBA-1. In case of C1, for RBA-2 replacement of

10%, 20%, 30%, 40%, 50%, 70% and 100%, compressive strength decreased by 20%, 6%, 10%, 23%, 11%, 28% and 24%, respectively. On the other hand, in case of C3 and for the same percentage of replacement of RBA-2, compressive strength decreased by 16%, 9%, 17%, 24%, 19%, 14% and 17%, respectively. It is clearly seen that coarse RBA are more likely to decrease compressive strength in hardened concrete [khattab and Hchemi,2020].

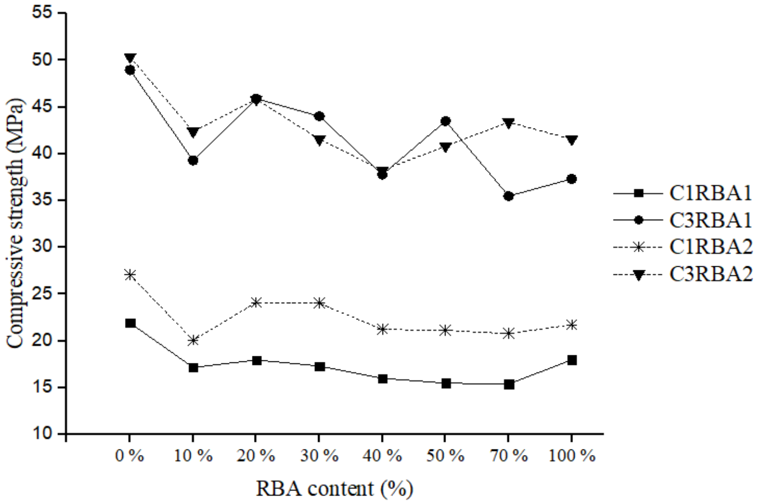


Figure III.4. Effect of coarse RBA-1 and RBA-2 content on compressive strength.

There are several possible reasons that can be given to explain the decrease in compressive strength for concrete mixtures made with coarse RBA. The main reason for such a loss in strength is attributed to the higher porosity and Los Angeles abrasion of RBA as well as to the moisture state of these aggregates. Poon [Poon et al., 2004] stated that the moisture states of the aggregates can change the workability and the uniformity of the concrete mixes and hence would affect the properties of the hardened concrete. Moreover, it is important to note that, based on observations during mixing of concrete containing coarse RBA, the external layer of coarse RBA was wrecked and transformed to fine aggregate. Furthermore, the diameter size of these aggregates was reduced [khattab and Hachemi,2021].

The evolution of compressive strength with the inclusion of coarse RBA shows that coarse RBA-1 has greater impact on the reduction of compressive strength. One explanation for that may be due to the fact that RBA-1 has porosity much higher than that RBA-2 which affects negatively concrete compressive strength. Another explanation relates to fact that RBA-2 has mechanical strength higher than that RBA-1. The results obtained show that concrete mixtures with 20% RBA replacement have the minor compressive strength loss.

III.2.5 Dynamic modulus of elasticity

Results of the dynamic modulus of elasticity of all concrete mixes are presented in figure III.5. It can be clearly observed that concretes prepared with coarse RBA present low dynamic modulus of elasticity compared to NA concrete. The results presented in figure III.5 show that the dynamic modulus of elasticity of concretes were significantly affected by the increase of replacement percentage of coarse NA by coarse RBA, which can be explained by the lower mechanical strength of this type of aggregates (see Table II.4).

The increase of replacement percentage of RBA-1 in C1 has led to the loss of dynamic modulus of elasticity. The loss was about 15%, 23%, 27%, 18%, 30%, 38% and 50% for C1, while for C3 the loss was about 7%, 8%, 12%, 19%, 34%, 41% and 50% for 10%, 20%, 30%, 40%, 50%, 70% and 100% substitutions respectively, compared to concrete prepared with natural aggregates.

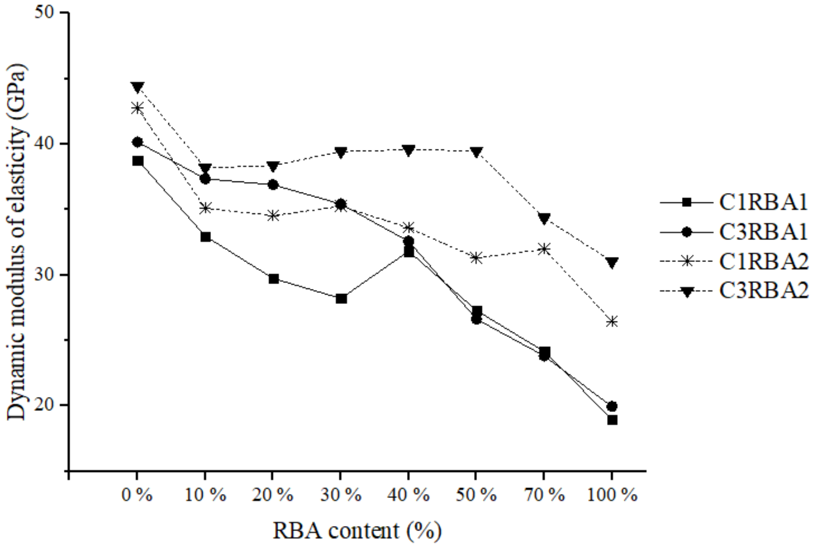


Figure III.5. Effect of coarse RBA-1 and RBA-2 content on dynamic modulus of elasticity of concrete.

However, the inclusion of RBA-2 in concrete results in a lower loss of dynamic modulus of elasticity when compared to concrete made with RBA-1 at the same replacement levels, which can be explained by the high mechanical strength of RBA-2. The dynamic modulus of elasticity was reduced by 18%, 19%, 18%, 21%, 27%, 25% and 38% for C1 and by 14%, 14%, 11%, 11%, 11%, 23% and 30% for C3 when coarse NA was replaced with RBA-2 by 10%, 20%, 30%, 40%, 50%, 70% and 100%, respectively. The results obtained in this test indicate that concrete mixture made with 100% of RBA represents the worst dynamic modulus of elasticity.

It is interesting to note that the replacement of coarse NA by RBA might have a negative effect on the dynamic modulus of elasticity. According to González [González et al., 2017], the decrease in dynamic modulus of elasticity is due to the lower rigidity of RBA. As can be seen from figure III.5, the effect of inclusion RBA on the loss in modulus of elasticity is more obvious than that of compressive strength [Hachemi et al.,2022].

III.2.6 Summary

The first part of this chapter was carried out to investigate the physical and mechanical properties of concrete made with coarse RBA at room temperatures. The results obtained show that, whatever the type of coarse aggregates (RBA-1 and RBA-2), the increase of coarse RBA content has a negative effect on the physical and mechanical properties of concrete. However, it would be acceptable to use RBA as coarse aggregates with a percentage of substitution of 20% to produce concrete with acceptable properties and good quality.

III.3 PHYSICAL PROPERTIES OF DIFFERENT AGGREGATES AFTER HEATING

Under high temperatures, aggregates exhibit thermal instabilities (mineralogical changes and thermal expansion). Therefore, mechanical and physical properties of concrete at high temperatures are strongly influenced by the type aggregates used. This part presents the discussion on the results of physical properties and macroscopic observation of the aggregates used in this study after exposure to high temperatures.

III.3.1 Aggregate mass loss

It is important to know the mass loss of aggregates at high temperature in order to understand the mass loss of concrete when subjected to high temperature. The mass loss of coarse NA, fine NA, coarse RBA and fine RBA-2 are presented in figure III.6. It can be seen from this figure that mass loss of coarse and fine NA and RBA increases with the increase of temperature.

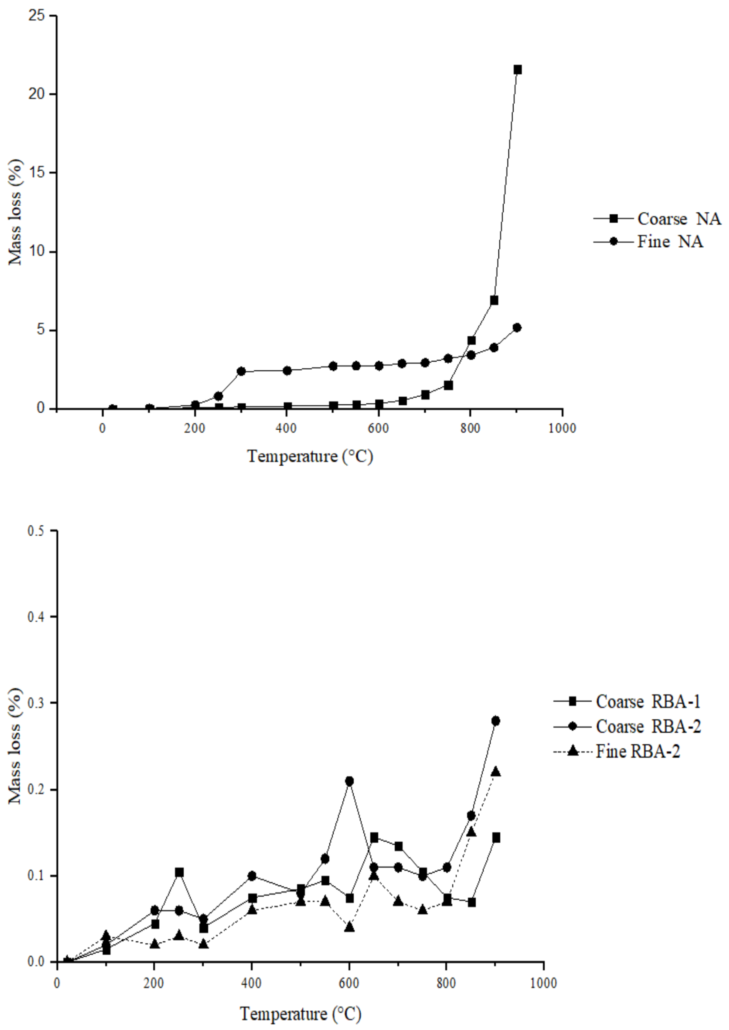


Figure III.6. Mass loss of the different types of aggregate subjected to elevated temperature.

It is observed that, the mass loss of fine NA from 100 to 200°C is negligible (<0.3%). After heating to 250°C, a slight increase in mass loss is recorded, but the values were less than 1%. This increase can be explained by the departure of moisture. According to figure III.6, the mass loss of fine NA was nearly stable from 300°C to 700°C. Over 700°C, a significant increase in mass loss is observed and reaches a value of about 5.2% after heating to 900°C.

For the heating-cooling cycles from 100 to 700°C, the mass loss of coarse NA was stable (less than 1%). Beyond 700°C, a gradual increase in mass loss is observed; it is more significant at 900°C (about 22%). The increase in mass loss of coarse NA can be explained by the decomposition of the calcite and release of CO_2 between 700°C and 900°C [Niry Razafinjato et al.2016].

In the case of the recycled refractory brick aggregates (RBA-1 and RBA-2), there was no significant increase in mass loss, it remained negligible in the range of 100°C - 900°C. The maximum increase was recorded after heating to 900°C, this increase was about 0.15% for coarse RBA-1, 0.28% for coarse RBA-2 and 0.22% for fine RBA-2. This confirms that RBA is more thermally stable when exposed to high temperatures.

III.3.2 Density of aggregates

The density values of coarse NA and RBA are grouped together in figure III.7. The results of density show that density of coarse NA decrease with the increase of temperature. However, from 100 to 600°C, the decrease of density was low (about 2%). Beyond 600°C, the density of coarse NA decreases strongly.This decrease can be explained by the decomposition of calcium carbonate ($CaCO_3$). The decrease in density is more significant at 900°C, it reachs 22%.

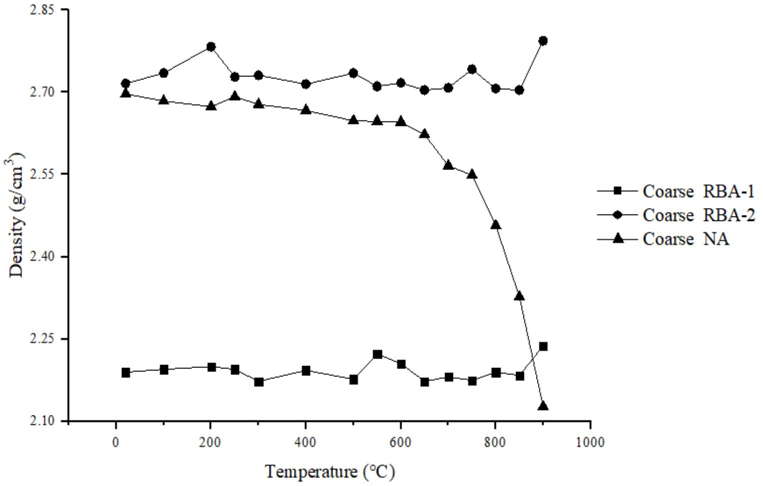


Figure III.7. Evolution of density of aggregates according to the temperature.

Figure III.7 shows that the density of coarse RBA-1 and RBA-2 undergoes a slight variation when they are heated up to 900°C. The results obtained show a moderate increase in density after heating to 900°C,this increase is about 2.2% for coarse RBA-1 and about 2.8% for coarse RBA-2.

III.3.3 Aggregates water absorption

Figure III.8 presents the evolution of water absorption of coarse NA and RBA obtained before and after heating. It is observed that water absorption of coarse NA increases with the increase of temperature.

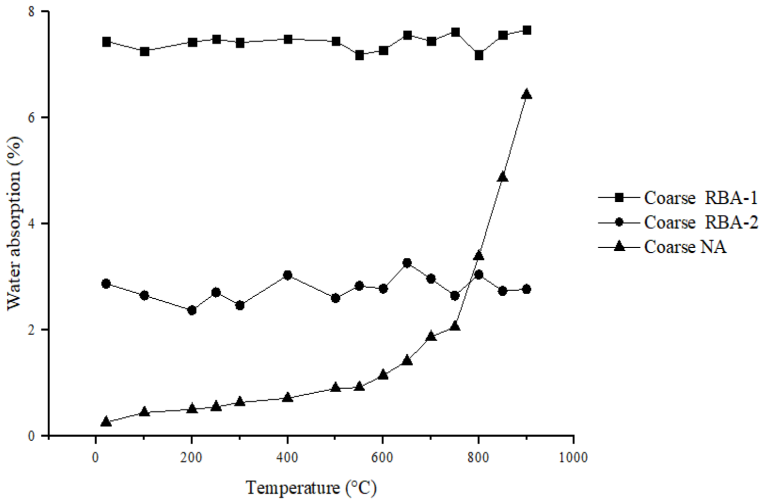


Figure III.8. Evolution of water absorption of aggregates according to the temperature.

The results obtained show that the variation of water absorption for coarse NA is weak in the temperature range of 100°C-600°C. Beyond 600°C, water absorption of coarse NA increases significantly. Heating to the temperature of 900°C causes an increase in water absorption of about 25% then that measured at room temperature.

We noticed that, for temperature heating from 100 to 900°C, the variation of water absorption of coarse RBA-1 and RBA-2 recorded was negligible.

III.3.4 Aggregate water porosity

Figure III.9 presents the evolution of water porosity of the three coarse aggregates, NA, RBA-1 and RBA-2, before and after heating. When temperature rose from 100 to 850°C, a gradual increase of water porosity was observed for NA. The maximum increase in water porosity was recorded after heating to 850°C, this increase is about 85%. We noticed that the porosity of NA could not be accurately measured after heating to the temperature of 900°C because the external layer of some NA were wrecked and transformed to fine aggregates. Figure III.10 shows the condition of coarse NA after heating to 900°C and immersing in water for 24 h.

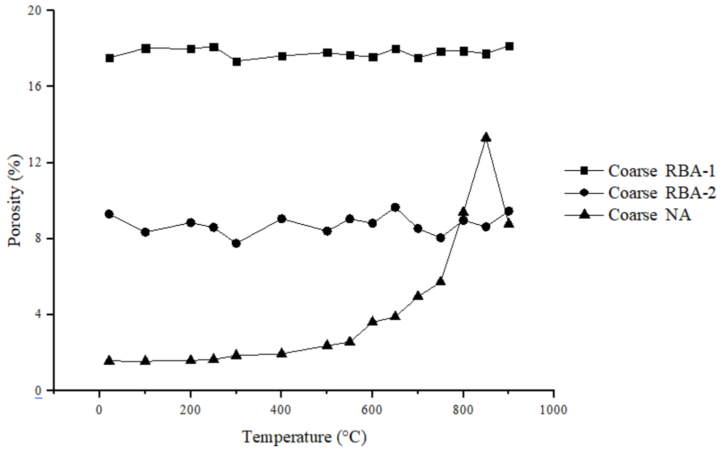


Figure III.9. Evolution of water porosity of aggregates according to the temperature.

For RBA-1and RBA-2, it should be noted that the variation of water porosity was nearly stable up to 900°C. This means that the porous structure of each coarse RBA is not significantly modified. The evolution of water porosity is correlated with the evolution of water absorption; because the water porosity and the water absorption are normally related to each other.

Figure III.10. Coarse NA after a heating to 900°C and immersing in water for 24 h.

III.2.5 Macro and mesoscopic degradation of aggregates after heating

The description of surface cracks and color change of aggregates as well as the photographs of samples before and after heating are given below.

- **Coarse and fine NA**

When aggregates are exposed to heat, expansion of aggregates due to crack development and discolouration due to transformations of mineral compounds are the main harmful factors that occur for aggregates. Table III.1 and 2 show the change of color of coarse and fine NA at different heating temperatures.

For coarse NA, a slight color change is observed in the temperature range of 100°C –300°C. Beyond 300°C, coarse NA became darker red with the increasing temperature up to 750°C (Table III.1). While in the case of fine NA, a gradual reddening occurs after being subjected to temperatures from 650 to 900°C. The appearance of red coloration is caused by oxidation of the iron content [Biró and Lublóy, 2020]. According to Niry [Niry Razafinjato et al., 2016], the dehydrates of the goethite FeO(OH) and produces hematite (Fe_2O_3) which are responsible for the red colour after 750 °C heating/cooling.

At 800°C, the color of coarse NA changes to gray and became darker grey after heating to 900°C. This changed was due to the decomposition of the calcium carbonate ($CaCO_3$) to lime (CaO) [Hager et al., 2015].

Table III.1. Pictures of coarse NA before and after exposure to different temperatures.

20°C

100°C-300°C

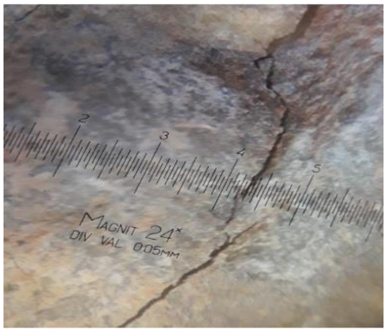
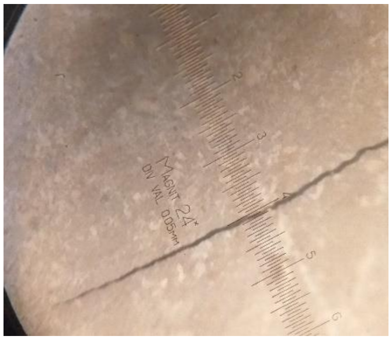
400°C -650°C

		
700°C -750°C	800°C -850°C	900°C

Table III.2. Pictures of fine NA before and after exposure to different temperatures.

		
20°C	100°C -300°C	400°C -650°C
		
700°C -750°C	800°C -850°C	900°C

Table III.3. Cracks development of coarse NA from 700°C to 900°C.

T (°C)	Mesoscopic observations	Crakcs	Width of crakcs
700			0.03 mm
750			0.05mm
800			0.15-0.05mm
850			0.15-0.05mm

900			0.025-0.25mm

No significant cracks or degradation were observed at macroscopic scale for most of coarse NA before heating up to 600°C. As seen in table III.3, the formation of cracks on the surfaces of coarse NA was observed after heating to 700°C. Additionally, these cracks were visible with the naked eye. According to results obtained by microscope, the widths of cracks increase with the increase of temperature. After the heating cycles of 800°C, 850°C and 900°C, cracks become very pronounced. This was attributed to the expansion of these aggregates.

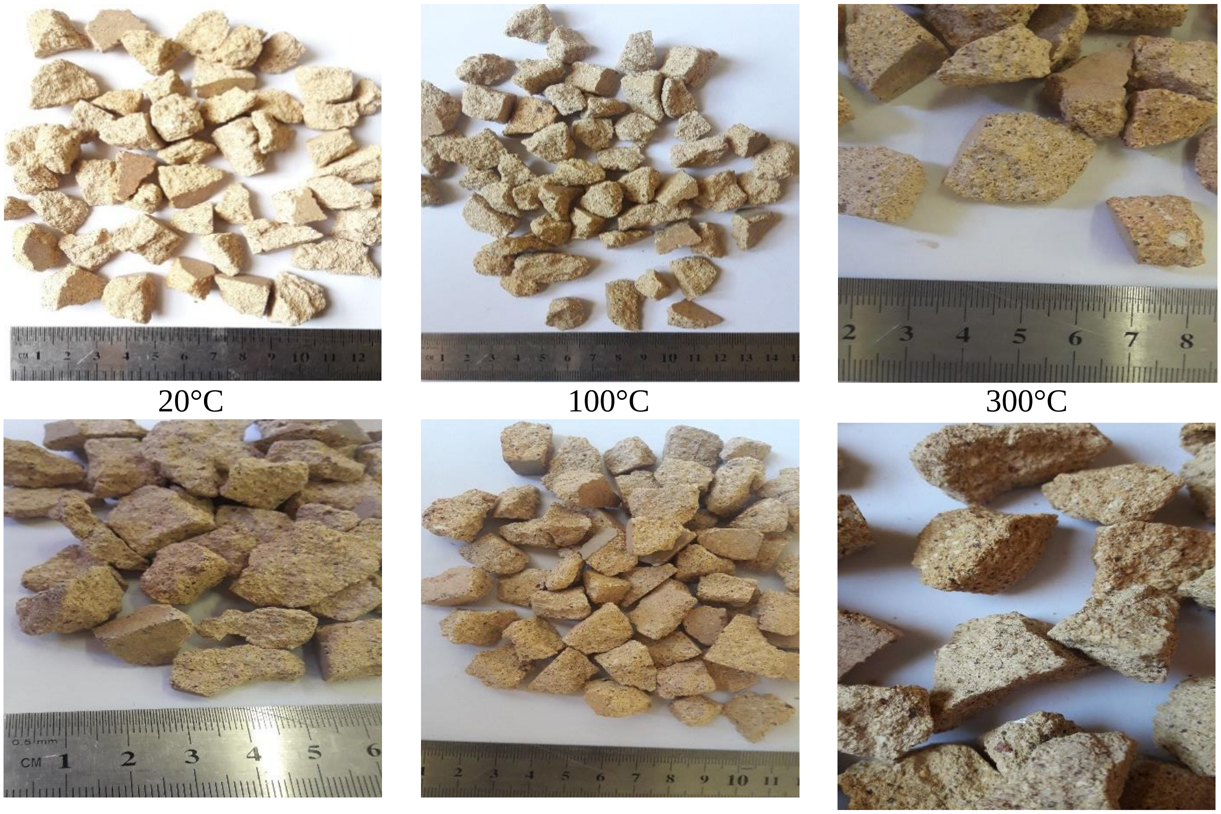
❖ Coarse RBA-1 and RBA-2 and fine RBA-2

Pictures of coarse RBA-1 and RBA-2 and fine RBA-2 before and after exposure to different temperatures are shown in figure III.11, III.12 and III.13, respectively.

As can be seen in these figures, no change in color was observed in the 20–900°C temperature range.

As seen in Figure III.11 and III.12, no cracks were visible with the naked eye on coarse RBA-1 and RBA-2 up to 900°C. Moreover, no degradation was noticed at macroscopic scale for coarse RBA-1 and RBA-2. This confirms that coarse RBA-1 and RBA-2 and fine RBA-2 are more stable after exposure to high temperature compared to natural aggregates (coarse and fine).

As mentioned before, between 100°C and 900°C, the degradation of mass loss, density, water absorption and water porosity of coarse RBA-1 and RBA-2 and fine RBA-2 were negligible compared to natural aggregates (coarse and fine). In this domain, the physical properties of coarse RBA-1 and RBA-2 and fine RBA-2 remained intact, which indicates that the internal structure of these aggregates is not modified between 100°C and 900°C.



20°C 100°C 300°C

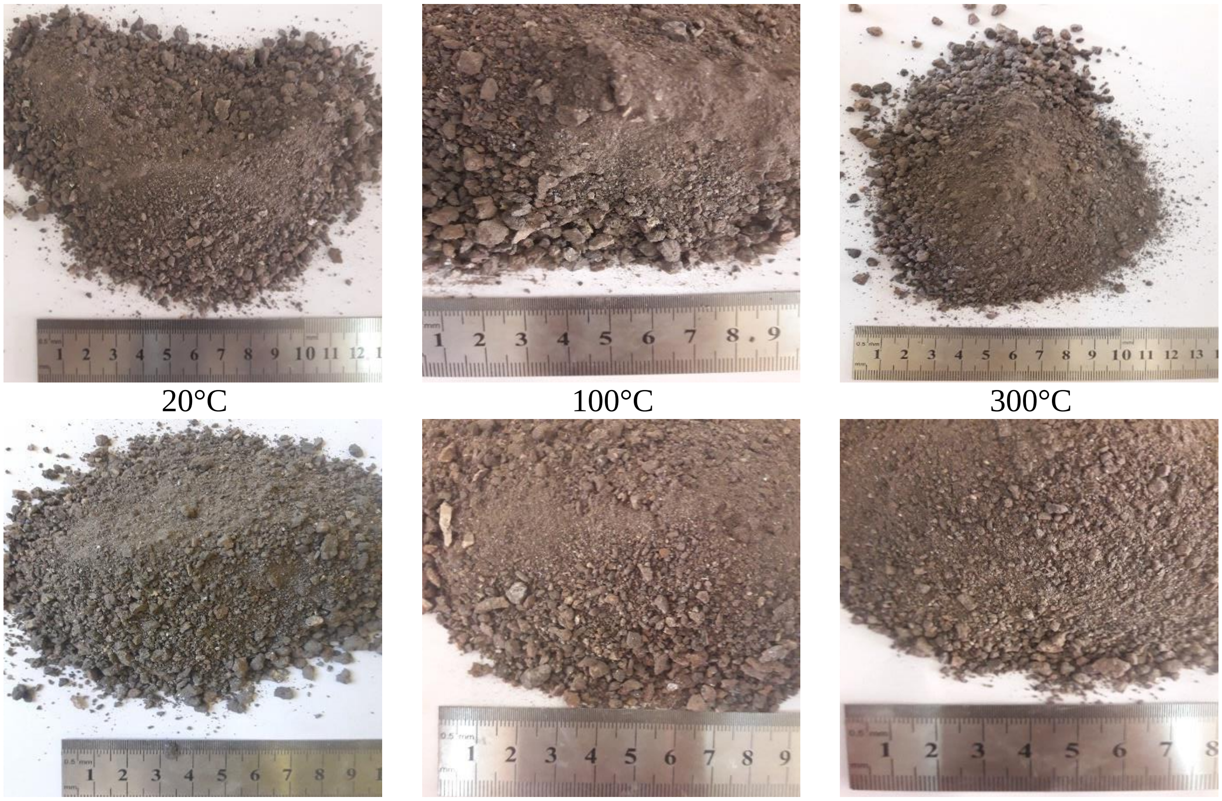
500°C 700°C 900°C

Figure III.11. Pictures of coarse RBA-1 before and after exposure to different temperatures.

20°C 100°C 300°C

500°C 700°C 900°C

Figure III.12. Pictures of coarse RBA-2 before and after exposure to different temperatures.



20°C 100°C 300°C

400°C 500°C 600°C

Figure III.13. Pictures of fine RBA-2before and after exposure to different temperatures.

III.4 INFLUENCE OF HIGH TEMPERATURE ON THE PHYSICAL AND MECHANICAL PROPERTIES OF CONCRETE MADE WITH NATURAL AGGREGATES

III.4.1 Water porosity

Figure III.14 displays the average values of water porosity obtained from conventional concrete specimens before and after exposure to high temperatures. It can be seen that water porosity of conventional concrete increased when their exposure to temperatures was increased.

The results obtained show that, between 150°C and 400°C, there was an important increase in concrete porosity. In this domain, porosity value ranges were about 20% for C1NA, 19% for C2NA and 17% for C3NA after heating to 400°C. The increased porosity in this domain corresponds to the departure of water and decomposition of cement hydrates, for instance, the CSH and the $Ca(OH)_2$ ([Zhou et al., 2020], [Xing et al, 2015], [Moley et al., 2019]).

Between 400°C and 600°C, water porosity of conventional concrete continued to increase due mainly to the creation of several cracks which occur as a result of thermal incompatibility between the aggregates and the cement paste.

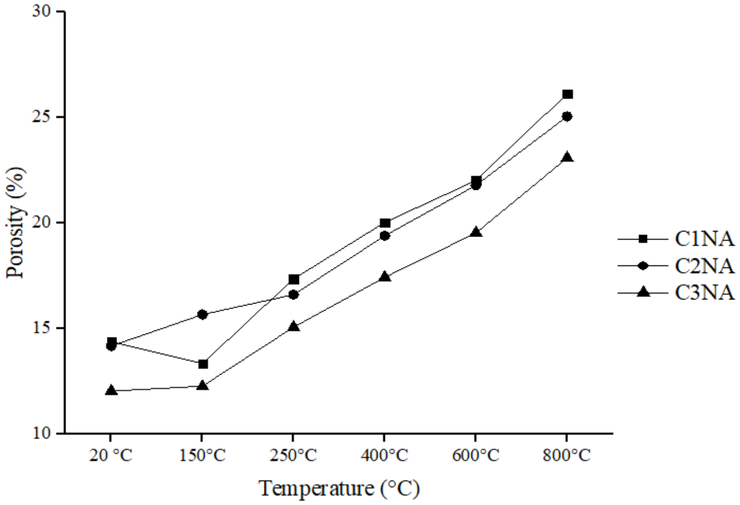


Figure III.14 Evolution of water porosity of conventional concretes according to the temperature.

When the specimens are exposed to 800°C, the increase of porosity becomes more significant. C1NA, C2NA and C3NA suffered an increase in porosity of 26%, 25% and 23%, respectively.

That situation could be due to the decrease of the amount of portlandite, owing to its rapid dehydration and decomposition. These process caused additional void spaces in structure of concrete ([Zhou et al., 2020], [Liu et al., 2018], [Mydin and Wang, 2012]).

III.4.2 Concrete density

The effect of elevated temperatures on the density of conventional concrete specimens is presented in figure III.15. The results show that an increase in the testing temperature leads to a decrease in the density of concrete.

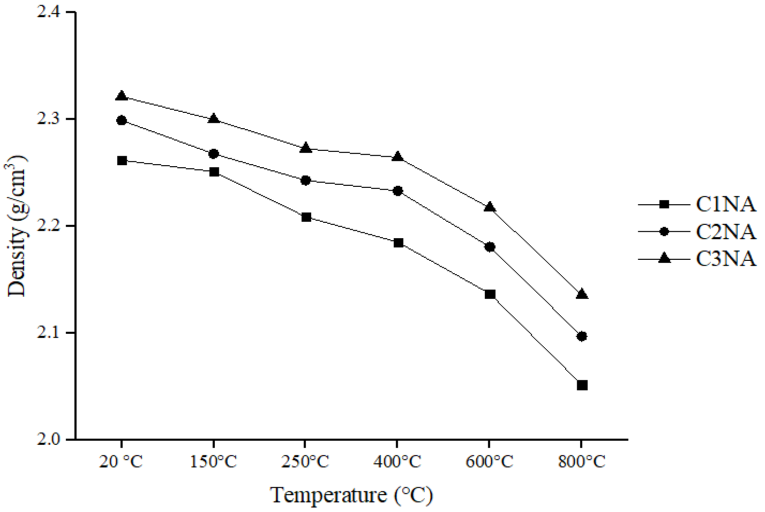


Figure III.15 Evolution of conventional concretes density with temperature.

The results indicate that, after exposure to a temperature of 250°C, the density of the conventional concretes (C1NA, C2NA and C3NA) decreased by approximately 2% . When the temperature rose from 250 to 400°C, the density loss increased; it reaches 3%. The decrease in density observed up to 300°C is mainly due to the remove of both capillary water and gel water ([Fares et al., 2015], [Bui et al., 2018]).

Beyond 400°C, a continuous decreases of density was observed for all the mixes. This decreases can be explained by the decomposition of CSH and CH ([Varona et al., 2020], [Abed and Brito., 2020], [Fares et al., 2009], [Mydin and Wang., 2012]). After heating to 800°C, the decrease in density of all concrete mixes was less than 10%. As shown in figure III.15, a decrease in concrete density of about 8% and 9% was observed for conventional concretes (C1NA, C2NA and C3NA). This means that, at 800°C, the structure of concretes is significantly modified.

III.4.3 Concrete mass loss

The experimental results of the evolution of the mass loss as a function of temperature for the conventional concrete tested are shown in figure III.16. As presented in figure III.16, it can be observe that the mass loss of the concrete increases as the exposure temperature increased.

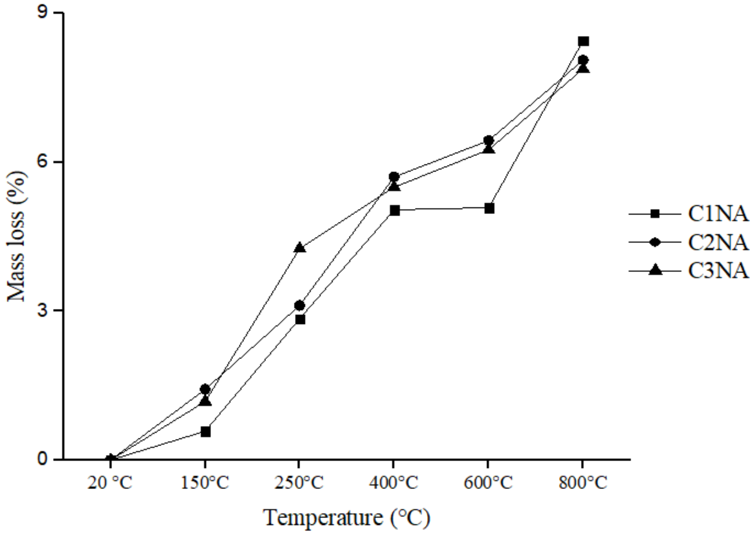


Figure III.16. Evolution of conventional concretes mass loss as function of temperature.

From figure III.16, three domains phases of the evolution of mass loss of different concretes according to the temperature can be observed. The first domain, which set from the 20 to 400°C, is characterized by a fast loss of mass of concretes. Therefore, the mass loss was about 5% for C1NA, 5.7 % for C2NA and 5.5 % for C3NA. In this domain, the loss mass of concrete is owing to the evaporation of the free water and chemically bound water ([Fares et al., 2009], [Tan et al., 2017], [Chen et al., 2018]).

The second domain was corresponding to the temperature range from 400 to 600°C, and is characterized by a low mass loss of concretes. In addition, conventional concrete presents a similar mass loss, about 5.1% for C1NA, 6.4% for C2NA and 6.3% for C3NA at 600°C. Beyond 800°C, the thired domain is characterized by the significantly increased in loss mass. The loss of mass obtained was 8.4% for C1NA, 8.1% for C2NA and 7.9% for C3NA.

After 400°C, the mass loss was governed by the destruction of CSH and the decomposition of CH ([Salahuddin et al., 2019], [Laneyrie et al., 2016], [Zhou et al., 2020]). Additionally to those factors, for temperatures higher than 700°C, the decomposition of calcareous aggregates and the release of CO_2 led to the increase of mass loss ([Xing, 2011], [Xing et al., 2015], [Rafi and Aziz, 2019]).

III.4.4 Change of volume

The relative changes in volume of conventional concretes are presented in figure III.17. It is observed from the figure that the volume of concrete changes as its temperature changes. The results indicate that for C1NA and C2NA, specimen's volume decreased between 20°C and 400°C. In this domain, the range of decrease in the volume was from 0.1 to 0.5% for C1NA and from 0.1 to 0.4% for C2NA in comparison with the initial volume before heating. Dehydration of CSH gel in concrete occurs at high temperatures, which causes shrinkage of concretes ([Khaliq, 2018], [Yao et al., 2020]). As can be seen in figure III.17, a slight increase in volume is observed for C3NA after the heating from 20 to 400°C.

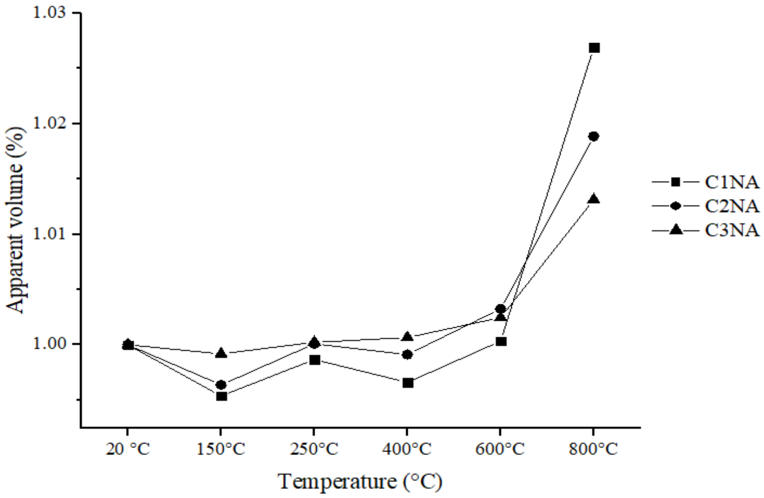


Figure III.17. Relative volume of conventional concretes as function of elevated temperatures.

Beyond 400°C, an important increase in specimen's volume was observed for all the mixes. This increase can be explained by the generation of cracks due to the expansion of aggregates ([Fernandes et al., 2017], [Varona et al. 2020]). The increase in volume is more significant at 800°C; it ranges from 0.03 to 2.7% for C1NA, from 0.3 to 1.9 % for C2NA and from 0.2 to 1.3% for C3NA.

Arioz [Arioz, 2009] studies indicate that, by a (SEM), several microcracks generally intermingled with voids after heating to 800°C. The presence of these microcracks explains the increase in the volume of the specimens.

III.4.5 UPV

Figure III.18 shows the results of UPV of conventional concretes specimens after being subjected to the elevated temperatures. It can be seen in figure III.18 and Table III.4 that the UPV for all of the mixes decreased with the increase of temperature.

Between 20°C and 250°C, all the specimens have a UPV higher than 3.5 km/s which indicates a good quality condition of the concrete in this domain. As the heating temperature was increased to 400°C, the UPV dropped significantly when compared to the conventional value (measured at 20°C). The reduction was about 28% for C1NA, 30% for C2NA and 24% for C3NA.

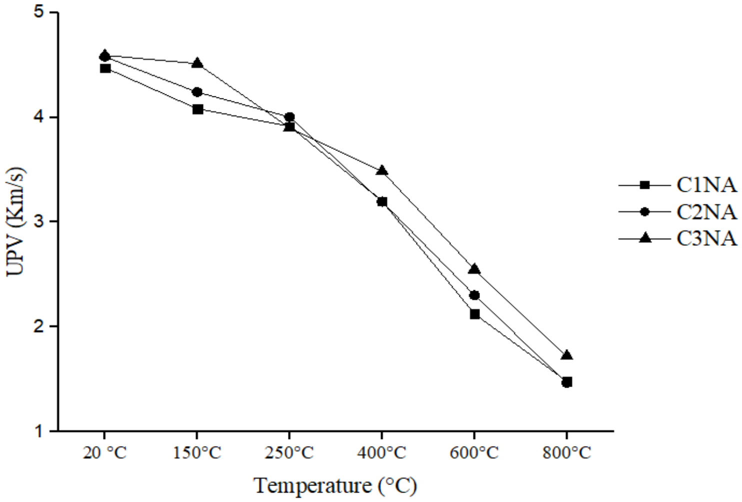


Figure III.18. UPV of conventional concrete after exposure to elevated temperatures.

Beyond 400°C, an important decrease of UPV was observed for all the mixes. This could be attributed to the evaporation of free water within aggregates and the hydration products. This process causes rapid deterioration of concrete structure [Uysal et al., 2012]. After heating to 600°C, conventional concrete (C1NA, C2NA and C3NA) shows a severe deterioration and the quality can be classified as poor quality (2.0 km/s < UPV < 3.0 km/s). This could be attributed to the development of cracks and extension of concrete and, therefore, velocity of the past pulse would be delayed ([Hachemi and Ounis, 2015], [Salahuddin et al., 2019]). These cracks led to the formation of gaps in concrete. At 800°C, the reduction of UPV was about 67% for C1NA, 68% for C2NA and 63% for C3NA. The quality of conventional concrete was classified as very poor after heating to 800°C (UPV less than 2.0 km/s).

Table III.4. UPV value of conventional concrete after exposure to elevated temperatures.

Temperature		20°C	150°C	250°C	400°C	600°C	800°C
C1NA	UPV (m/s)	4474	4082	3914	3200	2126	1482
	Quality	good	good	good	doubtful	poor	very poor
C2NA	UPV (m/s)	4577	4241	4003	3196	2304	1467
	Quality	excellent	good	good	doubtful	poor	very poor
C3NA	UPV (m/s)	4591	4512	3900	3486	2549	1722
	Quality	excellent	excellent	good	doubtful	poor	very poor

III.4.6 Damage Degree

Yao [Yao et al., 2020] indicated that the use of damage degree as an evaluation parameter to examine the performance degradation of concrete after heating was reasonable and feasible. Figure III.19 shows the concrete damage degree of conventional concrete specimens as a function of temperature. As expected, the damage degree of concrete specimens increased as the temperature increased. After heating to 150°C, the increase of damage degree for C1NA and C2NA was about 16% and 14%, respectively, while in the case of C3NA, the increase in damage degree was negligible (about 3%).

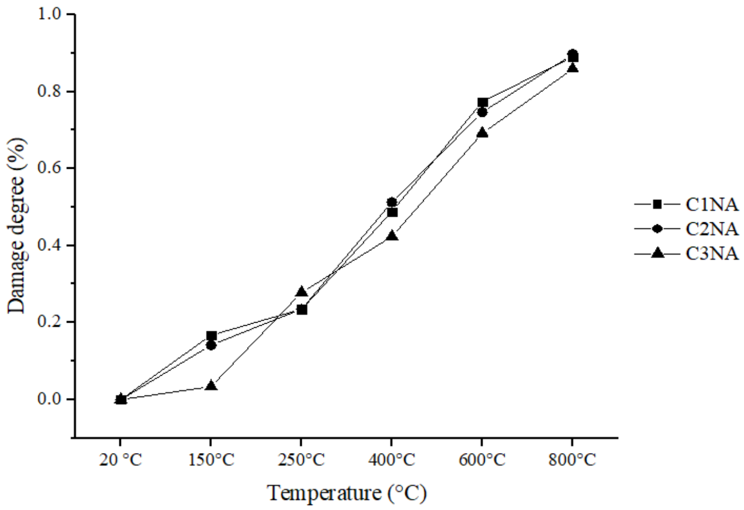


Figure III.19 Evolution of damage degree of the conventional concretes as function of temperature.

The results obtained show that, between 150°C and 400°C, there is a significant increase in damage degree of all concretes. For instance, after 400°C exposure, the damage degree of C1NA, C2NA and C3NA increased by about 49%, 51% and 42%, respectively. The main reason for this increase of damage degree in the range of 20–400°C is attributed to the dehydration and decomposition of ettringite, gypsum and CSH, which causes the structure of the cement paste to loosen ([Liu et al ., 2018], [Khaliq, 2018]).

For temperatures higher than 400°C, the increase in damage degree of conventional concrete becomes more prominent. When the specimens were exposed to 600°C, the damage degree increased by 77%, 75% and 69% for C1NA, C2NA and C3NA, respectively. At 800°C, all tested concretes suffered from thermal deterioration and the increase in damage degree was very high and reaches 89%, 90% and 86% for C1NA, C2NA and C3NA, respectively.

Above 400°C, the increase in damage degree in concrete is attributed to the alteration in the microstructure. The CH content starts to decrease due to the loss of crystal water [Zhou et al., 2020]. Additionally to those factors, the development of cracks at the interfaces between aggregates and cement matrix becomes more sharply. As result, the cement paste and aggregates was separated [Liu et al., 2018].

III.4.7 Compressive strength

Figure III.20 presents the residual compressive strength of conventional concretes specimens after exposure to elevated temperature. It is observed from the figure that the increase in temperature results in a decrease of compressive strength .

The results obtained show that, between 20°C and 400°C, there was a moderate decrease in residual compressive strength of all concretes. In this domain, the compressive strength of C1NA reduced by about 5% to 11% whereas C3NA suffered a strength reduction of about 22% to 25%. The strength reduction observed up to 400°C was mainly attributed to the evaporation of capillary pore water (25–105°C) and dehydration of ettringite, CSH and calcium carboaluminate hydrates (105–300°C) ([Ling et al., 2012], [Khaliq, 2018]).

After exposure to 150°C, C2NA showed an improvement in residual compressive strength compared to that measured after exposure to 20°C. This phenomenon has been observed by several authors ([Yang et al., 2017], [Moley et al., 2019]) and has sometimes been interpreted as a consequence of CSH re-hydration betwwen 150°C and 400°C, when water migrates and condenses within the colder areas of the sample.

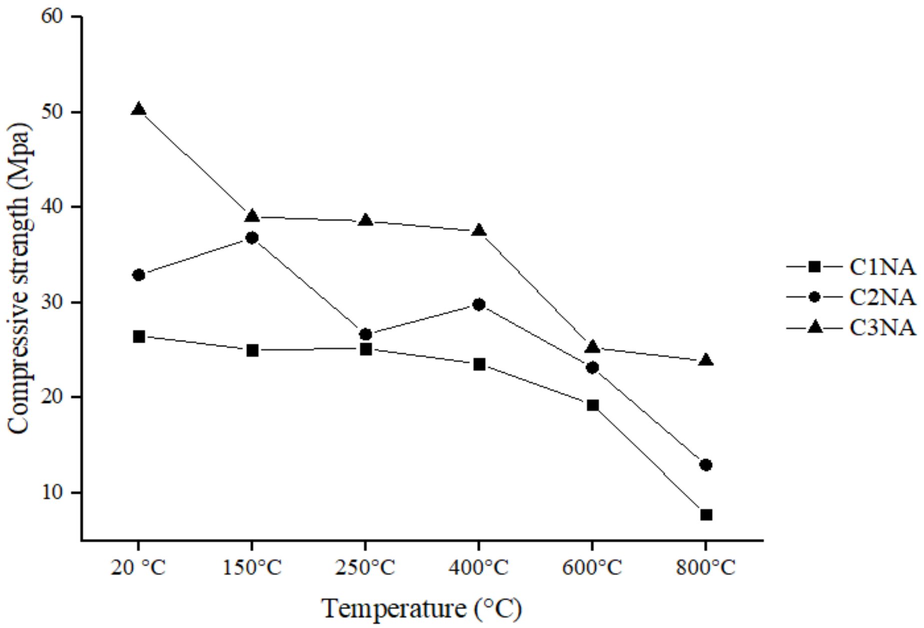


Figure III.20. Residual compressive strength of conventional concrete after exposure to elevated temperatures.

As presented in figure III.20, the compressive strength dropped significantly for all concretes for temperatures higher than 400°C, which indicates that the internal structure of concrete has significantly deteriorated. For the temperature range between 400°C and 600°C, it has been observed that the loss of compressive strength was less than 30% for C1NA and C2NA. For C3NA concrete mixtures, the loss of compressive strength was approximately 50% in comparison with the initial values. The lowest strength was recorded after heating up of the samples to the temperature of 800°C; it reaches 61%, 71% and 53% for C1NA, C2NA and C3NA, respectively.

There are several possible reasons that can be given to explain the decrease in compressive strength after exposed to 400°C. The main reason for the loss of strength is attributed to the weakening and breakdown of the bonds in the CSH gel which is strongly related to the dehydration ([Yang et al., 2017], [Wu et al., 2019], [Powęzka et al., 2020]). According to Shaikh [Shaikh 2017], the decrease in compressive strength is due to the decomposition of $Ca(OH)_2$ in the interfaces between aggregates and cement matrix at high temperatures between 430°C and 460°C. Moreover, This thermal incompatibility between aggregates and cement paste in concrete at elevated temperatures also results in excessive microcracking leading to strength loss ([Yang et al., 2017], [Khaliq, 2018], [Wu et al., 2019], [Zhao et al., 2020]).

III.4.8 Dynamic modulus of elasticity

The dynamic modulus of elasticity of conventional concrete after thermal exposure is illustrated in figure III.21. The results obtained confirm that modulus of elasticity is influenced by temperature and decreases with the increase of temperature. Further, the loss of dynamic modulus of elasticity is more critical compared to compressive strength which is attributed to increased internal cracking resulting from crack coalescence. This observation was also noted by Li and Yuan [Li and Yuan,2012].

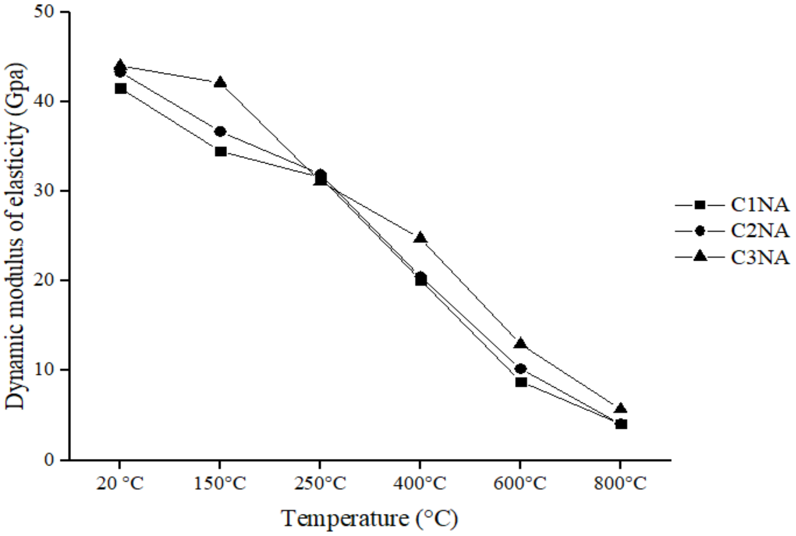


Figure III.21. Residual dynamic modulus of elasticity of conventional concrete after exposure to elevated temperatures.

The results obtained show that, between 20°C and 250°C, there is a moderate decrease in dynamic modulus of elasticity of all concretes, it reaches 24% for C1NA, 26% for C2NA and 29% for C3NA at 250°C. It can be seen that dynamic modulus of elasticity continues to decrease with the increase of temperature with a loss of about 52% for C1NA, 53% for C2NA and 44% for C3NA at 400°C. Up to 400°C, the loss in dynamic modulus of elasticity can be attributed to the chemical changes and deterioration in microstructure resulting from loss of water caused by the applied elevated temperature on the specimens ([Sarhat and Sherwood, 2013], [Khaliq, 2018]). Subsequently, the reduction of dynamic modulus of elasticity was very sharp above 400°C. The reduced elastic modulus of C1NA, C2NA and C3NA at 800°C was 4.05 GPa, 4.06 GPa and 5.7 GPa, respectively. These values present 10%, 9% and 13% of its respective original unheated value. Beyond 600°C, the loss in dynamic modulus of elasticity can be attributed to various factors such as physical and chemical changes (trapped vapor; decomposition of $Ca(OH)_2$ and CSH hydrates), cracking of interfacial transition zone due to thermal incompatibilities and calcination of $CaCO_3$ that occurs at 650°C onwards ([Abid et al., 2019], [Aslani et al., 2019], [Khaliq and Waheed, 2017]).

III.5 INFLUENCE OF HIGH TEMPERATURE ON THE PHYSICAL AND MECHANICAL PROPERTIES OF CONCRETE MADE WITH 20% OF COARSE RBA

III.5.1 Water porosity

Figure III.22 shows the average values of water porosity obtained from the concrete specimens prepared with 20% of coarse RBA before and after exposure to high temperatures. It can be seen from figure III.22 that concretes porosity presents a little decrease after heating to 400°C. This observation was also noted by Hachemi [Hachemi and Ounis, 2015] and was explained by the shrinkage of cement paste in the range temperature of 120°C -180°C.

When the temperature rose from 150 to 400°C, a significant increase of water porosity was observed, it reached a value of 25% for C1-RBA1, 19% for C2-RBA1 and C3-RBA1 after heating to 400°C. According to the results obtained for the concrete made with coarse RBA-2, a porosity value corresponding to around 22% for C1-RBA2, 20% for C2-RBA2 and 19% for C3-RBA2 was observed.

The heating from 600°C up to 800°C of concrete made with RBA leads to an important increment of their porosity. The maximum porosity value was recorded after heating to 800°C; it reaches 28% for C1-RBA1, 26% for C2-RBA1 and 25% for C3-RBA1. For concrete made with RBA-2, the porosity value of 27% for C1-RBA2, 26% for C2-RBA2 and 24% for C3-RBA2 is observed.

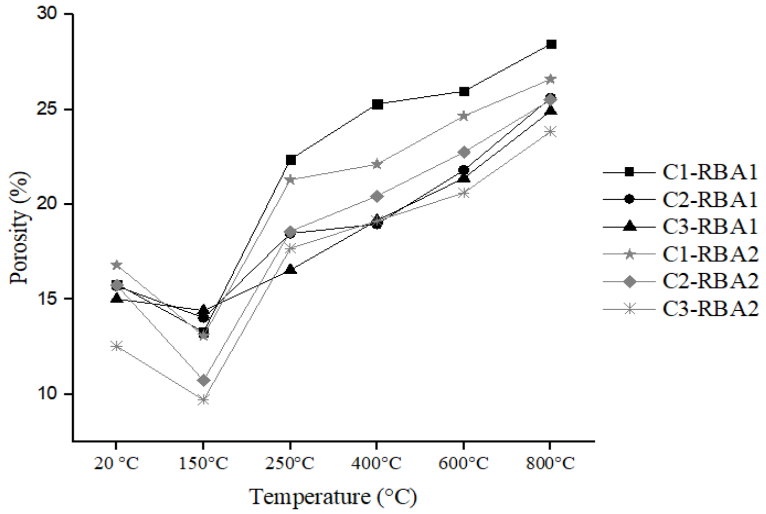


Figure III.22. Evolution of water porosity of the concretes prepared with 20 % of coarse RBA-1 and RBA-2 according to the temperature.

III.5.2 Concrete density

The effect of elevated temperatures on the density of the concrete specimens made with 20% of coarse RBA-1 or RBA-2 is presented in figure III.23. It is noted that with increasing temperature, a decrease in density was observed in concrete made of coarse RBA-1 or RBA-2.

When the specimens are exposed to 250°C, the density of the concrete made with coarse RBA-1 shows a decrease of 7%, 3% and 1% for C1-RBA1, C2-RBA1 and C3-RBA1, respectively. Beyond 250°C, the density of concrete made with coarse RBA-1 continued to decrease , this decrease was about 8% for C1-RBA1 and 2% for C2-RBA1 and C3-RBA1 after heating to 400°C. At 800°C, the decrease of density reaches a value of about 8%, 9% and 7% for C1-RBA1, C2-RBA1 and C3-RBA1, respectively. The concrete mixture C1-RBA1 represents the worst density after exposure to high temperature.

The inclusion of RBA-2 in concrete results in a higher density when compared with that of concrete containing RBA-1 after the same temperature exposure. The reduction reached 1% for C1-RBA2 and 2% for C3-RBA2 at 250°C. From this temperature, the reduction of density increased, this decrease was about 2% for C1-RBA2 and C3-RBA2 after heating to 400°C. Among all specimens, the density of C2-RBA-2 after being exposed to the effect of different temperatures are higher than that of the original density (measured at 20°C): 5% at 150°C, 1% at 250°C, 1% at 400°C and 1% at 600°C. The heating of concrete made with coarse RBA-2 from 600°C up to 800°C leads to an important decrease of density. The maximum decrease was recorded after heating to 800°C, this decrease was about 7% for C1-RBA2, about 3% for C2-RBA2 and about 8% for C3-RBA2.

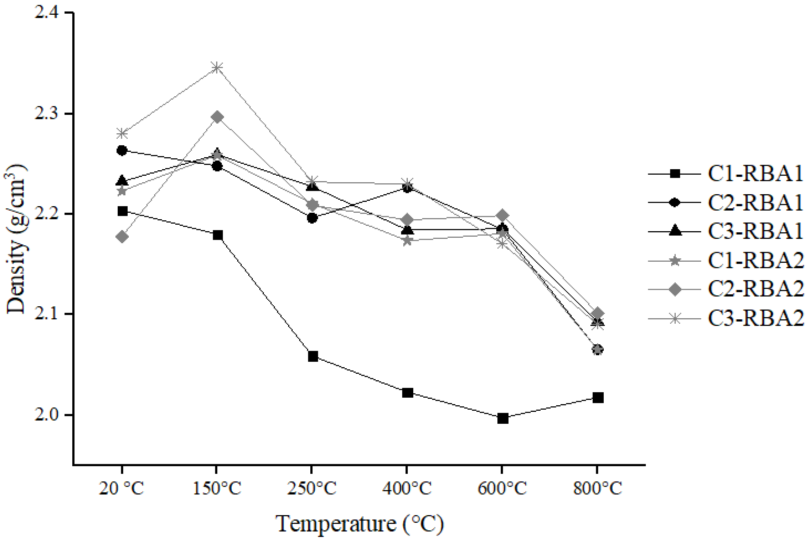


Figure III.23. Evolution of density of the concretes prepared with 20 % of coarse RBA-1 and RBA-2 with temperature.

III.5.3 Mass loss

The experimental results of the evolution of mass loss as a function of temperature for the concretes made with 20% of coarse RBA-1 or RBA-2 tested are shown in figure III.24. As the case in the conventional concrete, the mass loss of concrete made with coarse RBA-1 or RBA-2 increase as the exposure temperature increased.

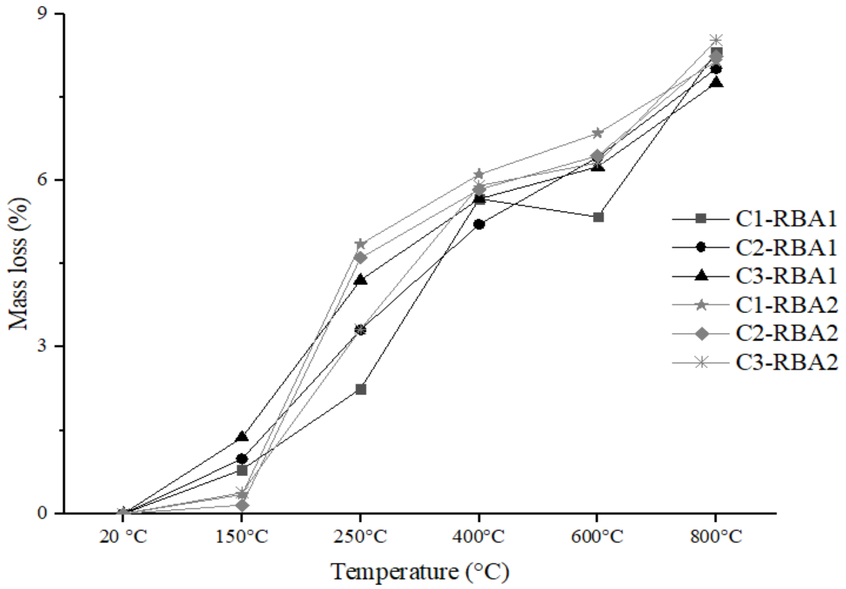


Figure III.24. Evolution of mass loss of the concretes prepared with 20 % of coarse RBA-1 and RBA-2 with temperature.

In the range temperature of 20°C –400°C, an important increase in mass loss was observed for all the mixes. At 400°C, compared with the initial mass, the loss mass of specimens increased by 5.7% for C1-RBA1, 5.2% for C2-RBA1 and 5.7% for C3-RBA1. For the concretes made with coarse RBA-2, the increase was 6.1%, 5.8% and 5.9% for C1-RBA2 , C2-RBA2 and C3-RBA2, respectively.

Moreover, it is important to note that, based on observations during removing concrete specimens from the furnace after the heating-cooling cycle of 150°C, the evaporation was obvious and some drops of water appeared on the surface of the concrete containing coarse RBA-1or RBA-2, as seen in Table III.5. This means that, until to 150°C, concrete prepared with coarse RBA-1or RBA-2 still contains more free water.

Between 400°C and 600°C, concrete made with coarse RBA-1 or RBA-2 present a similar mass loss. The heating of concrete from 600°C up to 800°C leads to an important increase of their mass loss. The maximum increase was recorded after heating to 800°C, this increase was about 8.3% for C1-RBA1, 8% for C2-RBA1, 7.8% for C3-RBA1, 8.1% for C1-RBA2, 8.2% for C2-RBA2 and 8.5% for C3-RBA2.

Table III.5. The seepage of free water on concrete prepared with coarse RBA after heating to 150°C.

Conventional concrete

Concretes prepared with coarse RBA-1

Concretes prepared with coarse RBA-2


III.5.4 Change of specimen's volume

Results of relative changes in volume of concretes made with 20% of coarse RBA-1 or RBA-2 after exposure to high temperature are shown in figure III.25. The results obtained show that, between 20°C and 250°C, there is a moderate decrease in volume of all concretes. In this domain, the volume of cncretes made with 20% of coarse RBA-1 or RBA-2 reduced by about 0.1 to 0.4%. Beyond 250°C, a gradual increase in volume of all concretes specimens was observed. Between 400°C and 800°C, the changes of the initial volume ranged from -0.2 to +2.4% for C1-RBA1, -0.2 to +1.6% for C2-RBA1, -0.1 to +0.9% for C3-RBA1, -0.2 to +1% for C1-RBA2, -0.2 to +1.3% for C2-RBA2 and -0.1 to +1.5% for C3-RBA2.

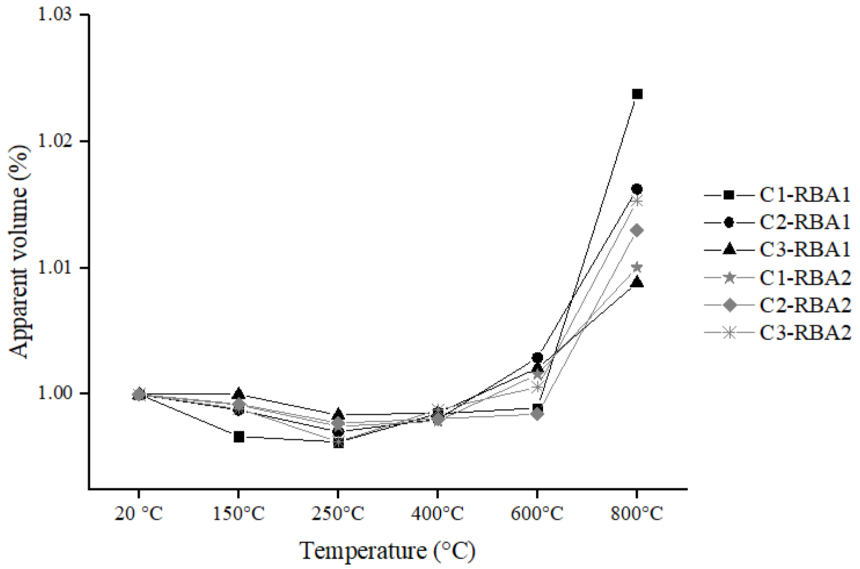


Figure III.25. Relative volume of concretes prepared with 20 % of coarse RBA-1 and RBA-2 as function of elevated temperatures.

III.5.5 UPV

Figure III.26 and table III.6 shows the results of residual UPV of concretes specimens made with 20% coarse RBA-1 or RBA-2 after being subjected to the elevated temperatures. As seen in the results, the UPV of the concretes made with coarse RBA decreased with the increase in temperature.

The results indicate that, between 20°C-150°C, the loss of UPV recorded was negligible (less than 0.08%). After heating to 250°C, the loss of UPV continued to decrease, while all specimens have a UPV value higher than 3.5 km/s. In another word, the quality of concrete made with RBA-1 can be classified as good for the range temperature of 20°C-250°C (3.5 km/s < UPV < 4.5 km/s). Beyond 250°C, an accelerating process in the decrease of UPV was observed, it is about 24% for C1-RBA1, 24% for C2-RBA1, 22% for C3-RBA1, 19% for C1-RBA2, 18% for C2-RBA2 and 21% for C3-RBA2 after heating to 400°C.

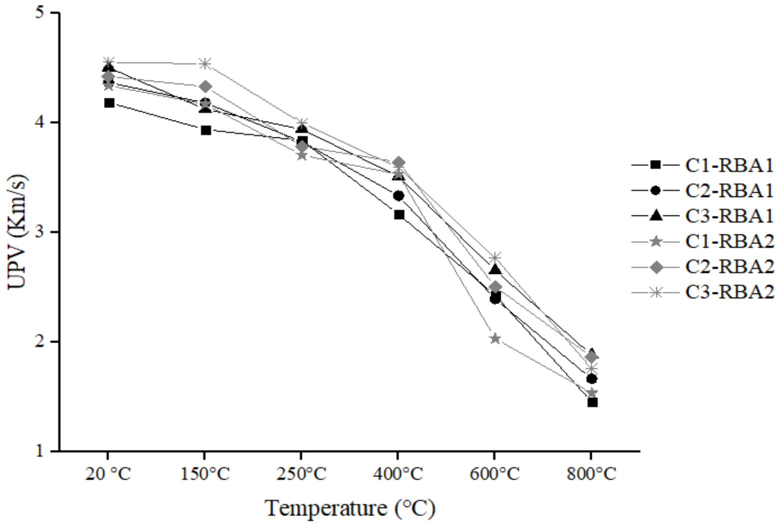


Figure III.26. UPV of concretes specimens made with 20 % of coarse RBA-1 and RBA-2 after exposure to elevated temperature**.**

It is noticed that, for the heating-cooling cycles of 150°C and 400°C, the UPV values of concretes made with coarse RBA-2 recorded was greater than that 3.5 km/s. This means that the quality of concrete made by coarse RBA-2 can be classified as good quality for the range temperature of 20°C-400°C (3.5 km/s < UPV < 4.5 km/s).

Beyond 400°C, a significant decrease occurred in UPV for all the mixes. Moreover, concretes made with coarse RBA-1 or RBA-2 present a similar loss of UPV after exposure to 600°C. Then, at 800°C, the decrease of UPV was very high and reaches 65% for C1-RBA1, 62% for C2-RBA1 and 58% for C3-RBA1, 65% for C1-RBA2, 58% for C2-RBA2 and 61% for C3-RBA2.

Table III.6. UPV value of concrete made with coarse RBA after exposure to elevated temperatures.

Temperature		20°C	150°C	250°C	400°C	600°C	800°C
C1-RBA1	UPV (m/s)	4186	3938	3839	3165	2427	1449
	Quality	good	good	good	doubtful	poor	very poor
C2-RBA1	UPV (m/s)	4327	4182	3827	3334	2392	1663
	Quality	good	good	good	doubtful	poor	very poor
C3-RBA1	UPV (m/s)	4504	4122	3940	3510	2652	1879
	Quality	excellent	good	good	good	poor	very poor
C1-RBA2	UPV (m/s)	4340	4170	3705	3531	2031	1532
	Quality	good	good	good	good	poor	very poor
C2-RBA2	UPV (m/s)	4422	4330	3787	3640	2503	1863
	Quality	good	good	good	good	poor	very poor
C3-RBA2	UPV (m/s)	4554	4538	3997	3590	2766	1757
	Quality	excellent	good	good	good	poor	very poor

III.5.6 Damage Degree

Figure III.27 illustrates the damage degree of concrete specimens containing coarse RBA-1 or RBA-2 after different temperatures exposure. From figure III.27, it can be seen that damage degree increased with the increased of temperature. The range temperature of 20°C–150°C was characterized by a slight rise in damage degree. In this range, the damage degree of concretes made with coarse RBA-1 increase by about 13% ,8% and 16% for C1-RBA1, C2-RBA1 and C3-RBA1, respectively. However, the increase of damage degree of concretes made with coarse RBA-2 was low; it was about 8% for C1-RBA2, 4% for C2-RBA2 and 1% for C3-RBA2.

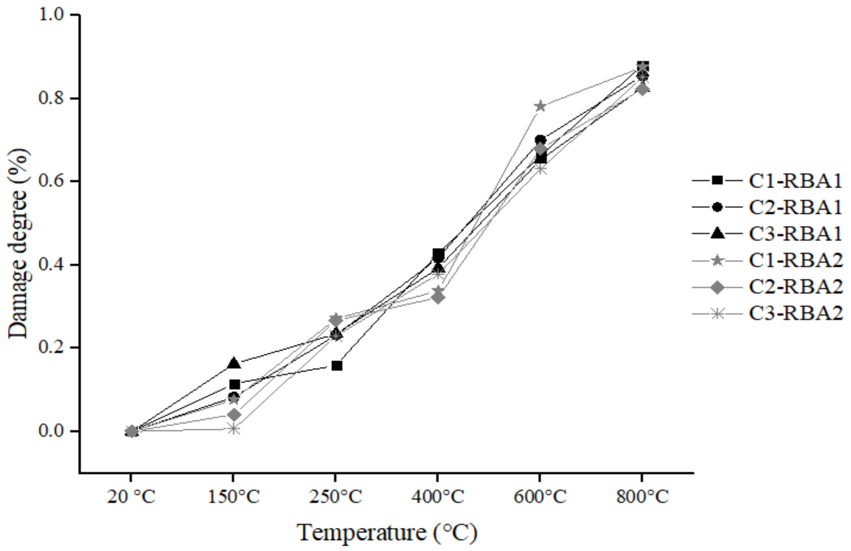


Figure III.27. Evolution of damage degree of concretes prepared with 20% of coarse RBA-1 and RBA-2 with temperature.

Heating concretes to 400°C causes an increase in damage degree less than 50%; it was about 41% for concretes containing coarse RBA-1. However, concretes made with coarse RBA-2 have the lowest degree of damage, about 35%. Coarse RBA-2 as used bricks (previously heated to 1400°C) acquires a good thermal stability which gives concrete better resistance when subjected to high temperature. The increase in damage degree is more significant at 800°C. Moreover, all the mixes present a similar damage degree, about 88% for concrete made with coarse RBA-1 or RBA-2.

III.5.7 Compressive strength

The average residual compressive strength of unheated and heated specimens prepared with 20% of coarse RBA-1 or RBA-2 is presented in figure III.28. The results obtained show that, after exposure to high temperature, the use of 20% of coarse RBA in concrete influences the evolution of compressive strength with temperature.

In the range of 150°C–600°C, a moderate increase of compressive strength (2% − 13%) was observed for concrete made with coarse RBA-2. For concrete containing coarse RBA-1, the increase of compressive strength, in the range temperature of 150°C to 400°C, was higher than that concrete made with coarse RBA-2. An important increase was observed by C3RBA-1 at 150°C (about 38%) and for C2RBA-1 at 250°C (about 29%).

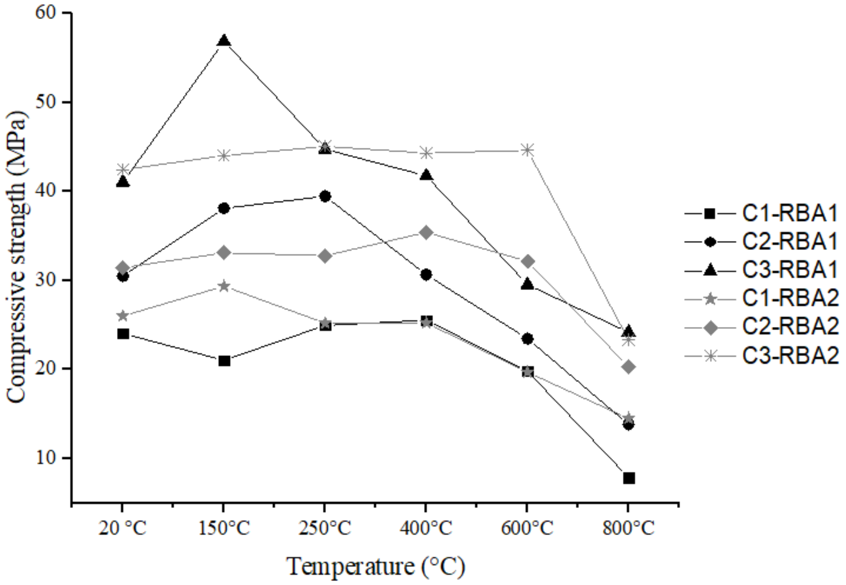


Figure III.28. Residual compressive strength of concretes prepared with 20% of coarse RBA-1 and RBA-2 after exposure to elevated temperatures.

Several reasons can explain the gain in compressive strength for concrete made with RBA compared to conventional concrete. RBA is one of the best materials that have good resistance to fire, when used as aggregate for concrete, it performs well. However, limestone aggregates undergo visual degradation following the decarbonation of limestone, $CaCO_3$ transformed into CaO following the departure of CO_2. These thermal instabilities promote further degradation of concrete [Xing et al., 2015], [Rafi and Aziz, 2019]. As mentioned in figure III.11 and III.12 , coarse RBA-1 or RBA-2 does not show any macroscopic degradation after heating to 800°C, such as bursting, cracking or color change. On the other hand, a significant degradation was observed for coarse NA (cracks and color change). When the deterioration of aggregates is important, the compressive strength of concrete becomes lower.

After heating to 800°C, concretes containing coarse RBA-1 lose a less important amount of initial strength (about 54%) when compared to conventional concrete. However, concretes prepared with coarse RBA-2 have the best compressive strength; they were able to retain more than 58% of initial compressive strength.

III.5.8 Dynamic modulus of elasticity

Dynamic modulus of elasticity of concretes prepared with 20% of coarse RBA-1 or RBA-2 as function of temperature is plotted in figure III.29. As shown in this figure, the dynamic modulus of elasticity decreases continuously with the increase of temperature, this can explain the increase of damage degree and the decrease in UPV which were observed in figure III.27 and figure III.22, respectively. In addition, as can be seen in figure III.29, the dynamic modulus of elasticity decreased linearly for all mixtures in the temperature range of 20°C –800°C.

When the temperature was increased from 20 to 250°C, the dynamic modulus of elasticity were decreased by 21% for C1-RBA1, 25% for C2-RBA1, 24% for C3-RBA1, 30% for C1-RBA2, 26% for C2-RBA2 and 25% for C3-RBA2. For temperatures higher than 250°C, the decrease of dynamic modulus of elasticity is relatively more important. By increasing the temperature up to 400°C, the residual dynamic modulus of elasticity decreases by 48%, 43% and 41% for C1-RBA1, C2-RBA1 and C3-RBA1, respectively. However, the dynamic elastic modulus of concrete made with coarse RBA-2 was higher than that of concrete made with RBA-1. Coarse RBA-2 has better absolute density, better abrasion resistance and lower porosity when compared to coarse RBA-1.

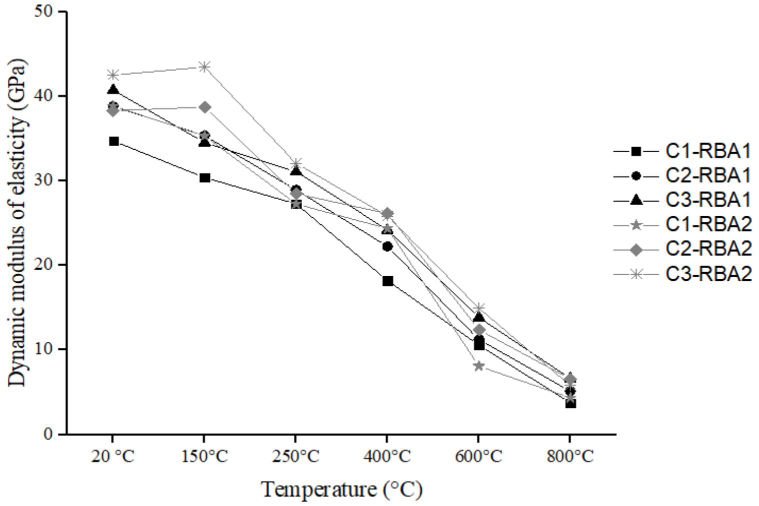


Figure III.29. Dynamic modulus of elasticity of concretes prepared with 20% of coarse RBA-1 and RBA-2 after exposure to elevated temperatures.

Between 400°C and 600°C, the dynamic modulus of elasticity of all specimen concretes continued to decrease. Moreover, the decrease of dynamic modulus of elasticity was similar of all concretes mixtures. At 800°C, a dramatic decrease in dynamic modulus of elasticity was obsvered for all mixes, it reaches 89% for concretes made with coarse RBA.

III.6 INFLUENCE OF HIGH TEMPERATURE ON THE PHYSICAL AND MECHANICAL PROPERTIES OF CONCRETE MADE WITH 20% OF COARSE AND FINE RBA-2

Using recycled refractory bricks as fine aggregates in cement mortar was investigated by [Saidi et al.2015]. The results obtained by the authors showed that the mechanical tests (compressive strength and flexural strength) increase with an increase in the replacement level (up to 20%) of recycled refractory bricks by sand substitution. Moreover, the authors found that the mortar with 20% of recycled refractory bricks shows good thermal stability since its mass loss is lower compared with other compositions.

Therefore, it is interesting to investigate the potential of using refractory brick as coarse and fine aggregate to produce concrete. For this reason, this part aims to produce concrete by using RBA-2 as coarse and fine aggregate by partial substitution of coarse and fine natural aggregate at content 20%. The physical and mechanical properties of specimens produced before and after heating are discussed.

III.6.1 Water porosity

Figure III.30 shows the average values of absolute water porosity obtained from the concrete specimens prepared with 20% of coarse and fine RBA-2 before and after exposure to high temperatures. The results obtained show that, at room temperature, the inclusion of 20% coarse and fine RBA-2 in concrete gives it higher porosity (from 14% to 32%) than that of conventional concrete prepared with natural aggregates. This can be explained by the high porosity of coarse and fine RBA-2 (see Table II.4).

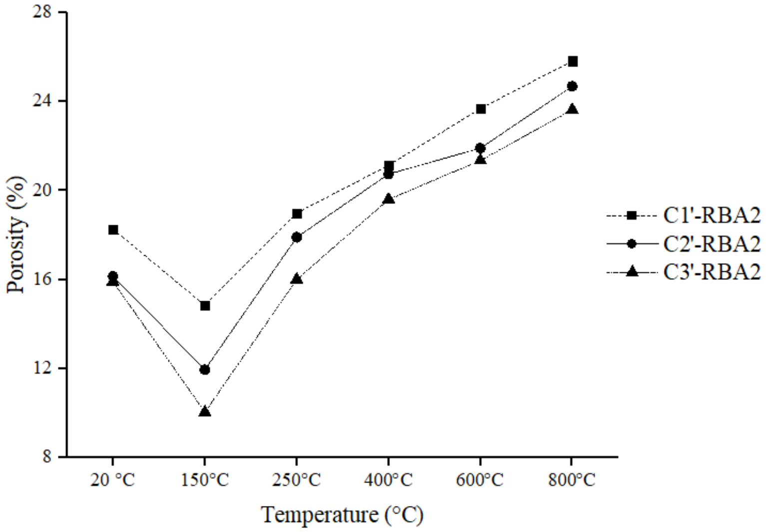


Figure III.30. Evolution of water porosity for concretes prepared with 20% of coarse and fine RBA-2 according to the temperature.

As shown in figure III.30, it can be note that the porosity value of the concretes after exposure to 150°C was lower with respect to its value at 20°C. From this temperature, it is observed that porosity of these concretes increases significantly. Heating to the temperature of 400°C causes an increase in water porosity about 21% for C1'-RBA2, 21% for C2'-RBA- and 20% for C3'-RBA2. Between 400°C and 600°C, the porosity of concrete made with 20% of coarse and fine RBA-2 continued to increase. After heating to 800°C, the porosity reaches a value of about 26%, 25% and 24% for C1'-RBA2, C2'-RBA2 and C3'-RBA2, respectively.

III.6.2 Concrete density

The effect of elevated temperatures on the density of concrete specimens made with 20% of coarse and fine RBA-2 is presented in figure III.31. The inclusion of 20% of coarse and fine RBA-2 in concrete results in a higher density when compared to concrete containing 20% of coarse RBA-2, which can be explained by the high density of fine RBA-2 (Table II.4). Moreover, concretes made with 20% of coarse and fine RBA-2 show a slightly lower density compared to conventional concretes. The reduction ranges from 1 to 2% at a replacement level of 20%.

The results indicate that, up to 250°C, the density was not greatly affected by temperature. In addition, as shown in figure III.31, the evolution of concrete density with temperature depends on the type of aggregates (natural or recycled). Heating concretes to 400°C causes a decrease in concrete density less than 3%; it was about 2% for concrete specimens made with 20 % of coarse and fine RBA-2 (C1'-RBA2, C2'-RBA2 and C3'-RBA2).

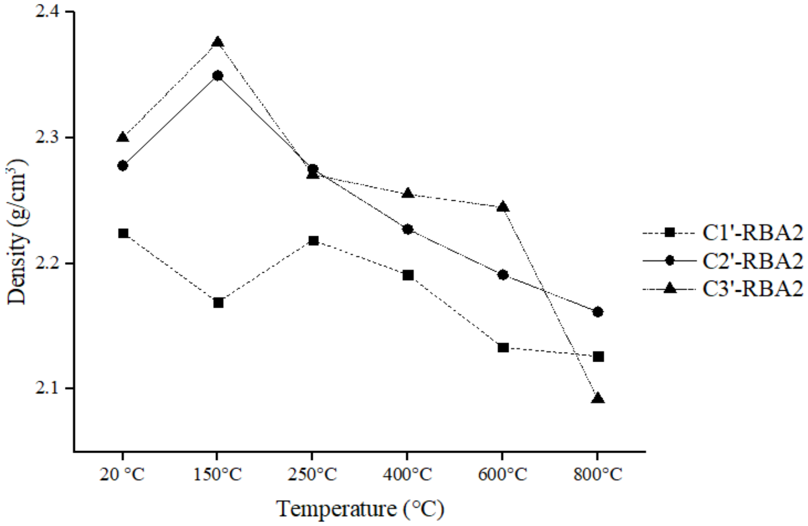


Figure III.31. Evolution of density of concretes prepared with 20% of coarse and fine RBA-2 with temperature.

Beyond 400°C, a moderate decrease in density was observed for all the mixes. It can be note that the decrease in density measured after heating to 800°C was about 4%, 5% and 9% for C1'-RBA2, C2'-RBA2 and C3'-RBA2, respectively.

III.6.3 Mass loss

The experimental resuts of the evolution of the mass loss as a function of temperature for the concretes prepared with 20% of coarse and fine RBA-2 tested are shown in figure III.32. It can be observed, from this figure, that the mass loss of all concretes increases when the temperature increased.

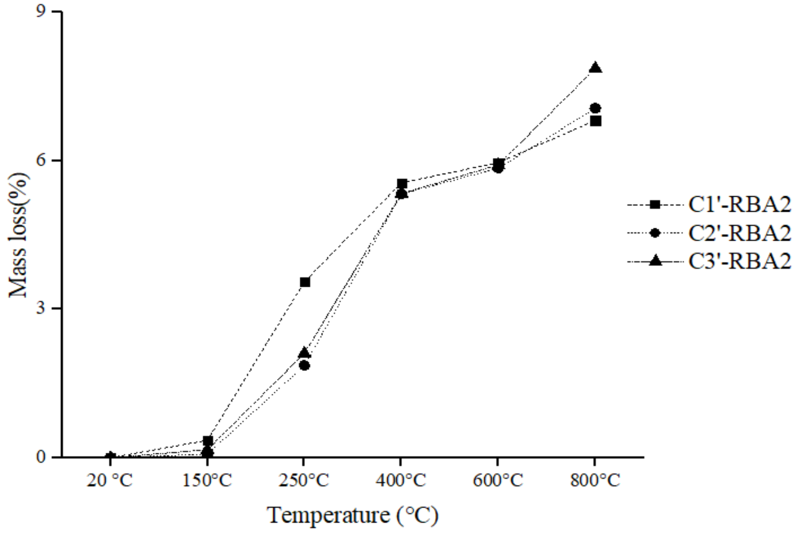


Figure III.32. Evolution of mass loss of concretes prepared with 20% of coarse and fine RBA-2 with temperature.

When the temperature rises from 20 to 400°C, a mass loss corresponding to 5.6%, 5.3% and 5.3% of the initial mass for C1'-RBA2, C2'-RBA2 and C3'-RBA2 was observed, respectively. Between 400°C and 600°C, C1'-RBA2, C2'-RBA2 and C3'-RBA2 present a similar mass loss, about 6% for C1'RBA-2 and 5.9% for C2'RBA-2 and C3'RBA-2 at 600°C. The mass loss increased again beyond 600°C heat exposure. At 800 °C, the increase of mass loss was about 6.8%, 7.1% and 7.9% for C1'-RBA2, C2'-RBA2 and C3'-RBA2, respectively.

III.6.4 Change of specimens' volume

The relative changes in the volume of tested concrete are summarized in figure III.33. It can be noted that the volume of specimens changes as the temperature increases. After temperature exposure, the volume of C1'RBA-2 and C3'RBA-2 decreases between 150°C and 400°C, while the volume of the C2'-RBA2 specimens gradually decreases between 150°C and 600°C. The range of decrease in the volume was from 0.03 to 0.11% for C1'-RBA2, from 0.07 to 0.4% for C2'-RBA2 and from 0.1 to 0.2% for C3'-RBA2. The results obtained in this test indicate that the type of aggregates (natural or recycled) has no influence on the volume variation; the aggregates undergo transformation at temperatures above 500°C. Heating concrete specimens from 600 to 800°C increases their volume. Maximum volume increases was recorded after heating to 800°C; this increase was about 0.96% for C1'-RBA2, 1.03% for C2'-RBA2 and 1.7% for C3'-RBA2.

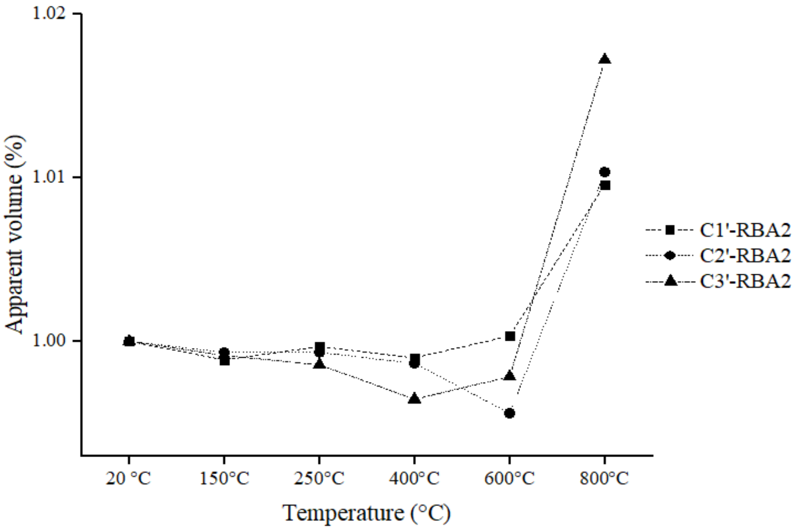


Figure III.33. Relative volume of concretes prepared with 20 % of coarse and fine RBA-2 as function of elevated temperatures.

III.6.5 UPV

Figure III.34 and Table III.7 shows the results of residual UPV of concretes specimens made with 20% of coarse and fine RBA-2 after exposure to high temperature. The results obtained show that, at room temperature, the inclusion of coarse and fine RBA-2 in concrete reduces the UPV when compared to conventional concrete prepared with natural aggregates (from 3 to 7%).

In the range of 20°C–250°C, the relative changes in UPV are weak of all specimen concretes; this suggested that temperature below 250°C has a negligible impact on the UPV of concrete made with 20% of coarse and fine RBA-2. Moreover, a small reduction in UPV was observed for these concretes after heating to 400°C. For instance, the UPV values were decreased by 14%, 11% and 18% for the C1'-RBA2, C2'-RBA2, and C3'-RBA2 ,respectively. On the other hand, all specimens could be categorised as good quality at 400°C; their UPV was higher than 3.5 km/s. Beyond 400°C, the reduction in UPV increases continuously for all the mixes. The reduction was 46%, 37% and 35% for C1'-RBA2, C2'-RBA2 and C3'-RBA2, respectively. After heating to 800°C, a moderate decrease in UPV was observed for all concretes and reaches 59%, 55% and 55% for C1'-RBA2, C2'-RBA2 and C3'-RBA2, respectively.

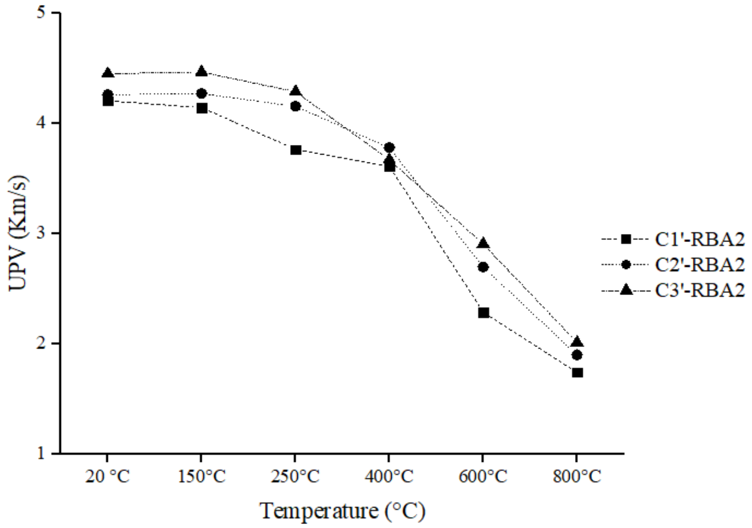


Figure III.34. Residual UPV of concretes specimens made with 20% of coarse and fine RBA-2 after exposure to elevated temperature**.**

Table III.7. UPV value of concrete made with 20% of coarse and fine RBA-2 after exposure to elevated temperatures.

Temperature		20°C	150°C	250°C	400°C	600°C	800°C
C1'RBA-2	UPV (m/s)	4207	4142	3764	3611	2285	1738
	Quality	good	good	good	good	poor	very poor
C2'RBA-2	UPV (m/s)	4260	4271	4155	3779	2697	1879
	Quality	good	good	good	good	poor	very poor
C3'RBA-2	UPV (m/s)	4451	4467	4288	3670	2905	2010
	Quality	good	good	good	good	poor	very poor

III.6.6 Damage degree

Figure III.35 illustrates damage degree for concrete specimens containing 20% of coarse and fine RBA-2 after different temperatures exposure.

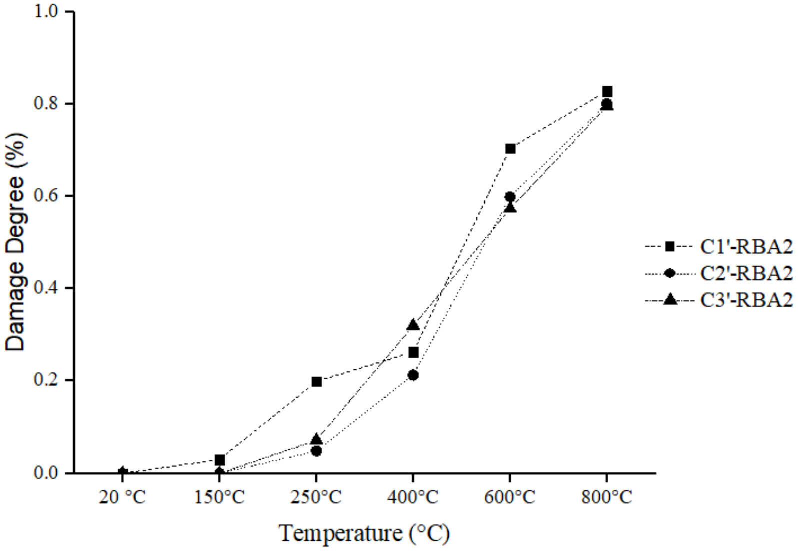


Figure III.35. Evolution of damage degree of concretes prepared with 20% of coarse and fine RBA-2 with temperature.

The results indicate that, after exposure to a temperature of 150°C, the damage degree of C1'-RBA2 specimens increased by approximately 3%, whereas the damage degree of C2'-RBA2 and C3'RBA-2 were close to zero. It is observed from figure III.35 that whatever the mix, the damage degree slightly increased between 150°C and 400°C. Moreover, C1'RBA-2, C2'RBA-2 and C3'-RBA2 present a similar damage degree, about 26% for C1'RBA-2, 21% for C2'RBA-2 and 32% for C3'RBA-2 at 400°C. As shown in figure III.35, the damage degree continues to increase with the increase of temperature with an increase of about 71% for C1'-RBA2, 60% for C2'-RBA2 and 57% for C3'RBA-2 at 600°C. Beyond 600°C, the damage degree starts to be important. At 800°C, the increase in damage degree reaches 83% for C1'-RBA2 and 80% for C2'-RBA2 and C3'-RBA2.

III.6.7 Compressive strength

Figure III.36 present the residual compressive strengths of concrete specimens made with 20% of coarse and fine RBA-2 after exposure to elevated temperatures. The results obtained show that, at room temperature, the compressive strengths was higher (from 2 to 9%) than that of conventional concrete prepared with natural. This means that the incorporation of fine RBA-2 imposes a positive effect on the concrete compressive strength, which might be attributed to the high density of RBA-2.

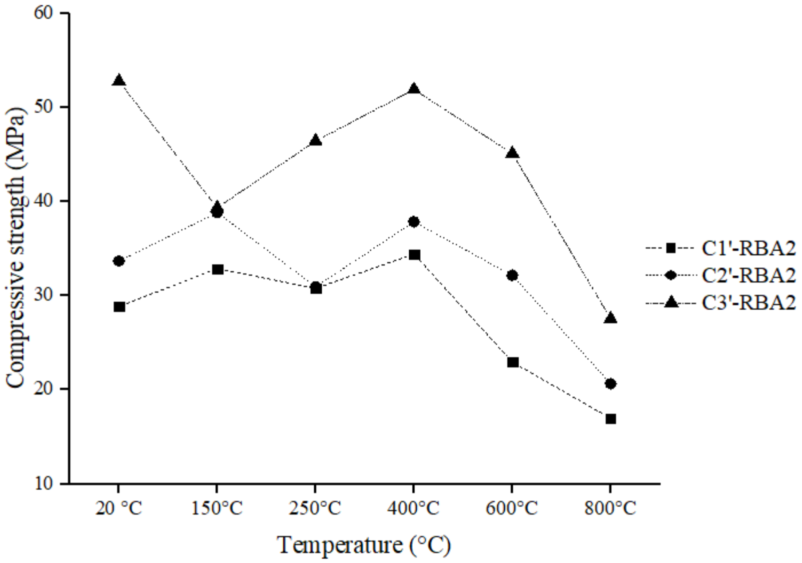


Figure III.36. Residual compressive strength of concretes prepared with 20% of coarse and fine RBA-2 after exposure to elevated temperatures.

As shown in figure III.36, concretes containing 20% of coarse and fine RBA-2 show different evolution of compressive strength with temperature. In the 150 to 400°C temperature range, the strength of C1'-RBA2 remains higher (7%-19%) than that the initial values (unheated specimens). An important increase was observed for C2'-RBA2 at 150°C and 400°C (about 15%). On other hand, the compressive strengths of C3'-RBA2 decreased at 150°C and then increased at 250°C and 400°C.

Beyond 400°C, compressive strength of all the mixes started to decrease. At 600°C, the reduction of compressive strength was about 20% for C1'-RBA2, 12% for C2'-RBA2 and 5% for C3'-RBA2. After heating to 800°C, concretes prepared with 20% of coarse and fine RBA-2 were able to retain more than 60% of initial compressive strength.

III.6.8 Dynamic modulus of elasticity

Figure III.37 shows the evolution of dynamic modulus of elasticity of concretes made with 20% of coarse and fine RBA-2 after exposure to high temperature. The results obtained show that dynamic modulus of elasticity of concretes made with 20% of coarse and fine RBA-2 obtained at room temperature was lower (from 7% to 14%) than that of conventional concrete prepared with natural aggregates. This can be explained by the high water absorption of fine RBA-2.

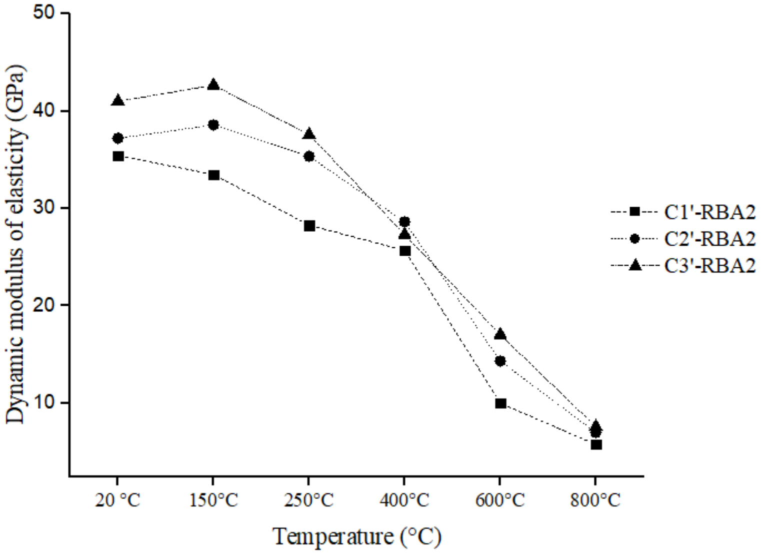


Figure III.37. Residual dynamic modulus of elasticity of concretes made with 20% of coarse and fine RBA-2.

As presented in figure III.37, the dynamic modulus of elasticity decreased for all concretes as the temperatures increased. When the specimens are exposed to 250°C, the residual dynamic modulus of elasticity of concrete shows a decrease of 20% for C1'-RBA2, while in the case of C2'-RBA2 and C2'RBA-2, the decrease recorded was low (about 5% for C2'-RBA2 and 8% for C3'-RBA2). Above 250°C, a gradual decrease in residual dynamic modulus of elasticity were observed, it reaches 27% for C1'-RBA2, 23% for C2'-RBA2 and 33% for C3'-RBA2 after heating to 400°C.

By raising the temperature from 400 to 800°C, an important decline in the dynamic modulus of elasticity of all specimens occurred. After exposure to 600 °C, the dynamic modulus of elasticity was reduced by 72% for C1'-RBA2, 61% for C2'-RBA2 and 58% for C3'-RBA2. At 800 °C, the reduction of dynamic modulus of elasticity was about 84% for concretes made with 20% of coarse and fine RBA-2.

III.7 CONCLUSIONS

The first part of this chapter devoted to evaluating the mechanical and physical properties of concrete made with partial and total replacement of coarse NA by coarse RBA at room temperature. The results obtained show that the performance of concretes decreases with the increase in the replacement ratio of coarse RBA. However, it would be acceptable to use RBA as coarse aggregate with a percentage of substitution of 20% to produce concrete with acceptable properties and good quality.

The effect of high temperature (up to 900°C) on different types of coarse and fine aggregates used for manufacturing concrete was presented in the second part of this chapter. Five types of aggregates were tested: coarse NA, fine NA, coarse RBA-1, coarse RBA-2 and fine RBA-2. Natural aggregates (coarse and fine) show a significant degradation (cracks and color change) compared to RBA (coarse RBA-1, coarse RBA-2 and fine RBA-2). RBA is one of the best materials that have good resistance to fire. Moreover, RBA does not show any macroscopic degradation after heating to 900°C.

Finally, four families of mixes were prepared and tested: the first mixes was made with coarse and fine NA (as reference concrete), the second mixes made by replacing 20% of coarse NA by coarse RBA-1, the third mixes made by replacing 20% of coarse NA by coarse RBA-2 and the fourth mixes was made by replacing 20% of coarse and fine NA by coarse and fine RBA-2. For each of the mentioned families, three cement dosages of 350 kg/m^3 (w/c=0.59), 400 kg/m^3 (w/c=0.47), 450 kg/m^3 (w/c=0.38) were investigated. The specimens obtained from these compositions were subjected to different heating-cooling cycles. This part of this chapter is devoted to the characterization of the physical and mechanical properties of the various concretes before and after heating up to 150°C, 250°C, 400°C, 600°C and 800°C.

The loss of mass is an important parameter to analyze the concrete behavior under high temperature. Its importance resides into the capacity to quantify the different forms of water in concrete. In the range of 20°C–400°C, the loss of mass of all concretes increases strongly. This result is due to the departure of both capillary and gel water from the dehydration of the CSH gel. Between 400°C and 600°C, the loss mass stabilized with temperature. The increase in loss of mass is more significant beyond 600°C. The mass loss is owing to the complete decomposition of the CSH gel due to dehydration.

The evolution of porosity, one of the important properties characterizing the durability of concrete, is associated with the physicochemical changes and microcracks that occur in the cement paste. The evolution of porosity is correlated with the evolution of density. The increase in temperature causes an increase in porosity and a decrease in density.

The increase in porosity is also accompanied by a decrease in the values of UPV and an increase in internal damage of concrete. The measurement of dimensions of the specimens before and after exposure to high temperature is reasonable and feasible to study the change in volume of concrete specimens. The removal of the moisture from the cement paste causes a decrease in the volume of specimens. However, the development of cracks due to the expansion of aggregates causes an increase in the volume of the concrete specimens.

The compressive strength is the most important parameter used to characterize concrete. The results obtained after exposure to high temperature are diverse. That diversity could be attributed to the difference of concrete’s composition. Residual dynamic modulus of elasticity of different types of concrete was also determined. A continuous decrease in dynamic modulus of elasticity was observed with the increase of temperature. In addition, the effect of temperature on the loss of dynamic modulus of elasticity is more evident than that of compressive strength.

Chapter IV:

Comparative analysis and relationship between the properties of different concretes

IV.1 INTRODUCTION

The objective of the first part in this chapter is to compare the behavior of concrete having different compositions (concretes made with 100% of coarse and fine NA, concretes made with 20% of coarse RBA and concretes made with 20% of coarse and fine RBA-2) after exposure to high temperature. This comparative study makes it possible to characterize the influence of refractory brick wastes as aggregates on the evolution of the physical and mechanical properties of concrete at high temperatures.

In the second part, specimens were subjected to a thorough visual examination after their removal from the furnace and compared with the unheated specimens. Several visual inspections were carried out on concrete to observe any changes in color, spalling and cracks. Factors that affect the change in the surface texture of the specimens at elevated temperatures are the physical and chemical changes, microstructural deterioration, initiation and propagation of microcracks, decomposition of $Ca(OH)_2$ and disintegration of CSH and decomposition and calcination if limestone aggregates are used [Hager, 2013]. Finally, in order to better understand the behavior of concrete at high temperatures, the aim of the last part of this chapter is to find the relation between the physical and mechanical properties of different concretes.

IV.1 COMPARATIVE ANALYSIS OF THE PHYSICAL AND MECHANICAL PROPERTIES OF DIFFERENT CONCRETE

IV.2.1 Water porosity

Figure IV.1 illustrates the evolution of water porosity of the different concretes studied as a function of the temperature. At 20°C, figure IV.1 shows that the inclusion of 20% of coarse RBA-1 or RBA-2 in concrete gives it higher porosity (from 4% to 25%) than that of conventional concrete prepared with coarse and fine NA. On the other hand, the use of 20% of coarse and fine RBA-2 leads to an increase in porosity (from 14% to 32%) than that of conventional concrete prepared with coarse and fine NA. This can be explained by the high porosity of refractory brick wastes (see Table II.4).

As stated previously, after heating to 150°C, concretes porosity presents a little decrease about 4% to 32% for concrete made with 20% of coarse RBA and about 19% to 37% for concrete made with 20% of coarse and fine RBA, except C2NA and C3NA. If we compare the porosity of the different concretes, it appears that concrete made with 20% of coarse RBA-2 presents similar or even lower porosity than concrete made with 20% of coarse RBA-1 (see figure IV.1). One explanation for that may be due to the fact that coarse RBA-2 has porosity lower than that coarse RBA-1 (see Table II.4). However, the inclusion of 20% of coarse RBA-1or RBA-2 results in a higher porosity as compared with that of concrete containing natural aggregates after exposure to high temperature. This confirms that coarse RBA-1or RBA-2 are more porous than those of coarse NA. On the other hand, it appears that the concretes made with 20% of coarse and fine RBA-2 presents a slightly higher porosity compared to conventional concretes. This might be due to high porosity of RBA-2. Furthermore, it also found that the porosity of concrete made with 20 % of coarse and fine RBA-2 is similar or even lower than that of concrete made with 20 % of coarse RBA-2.

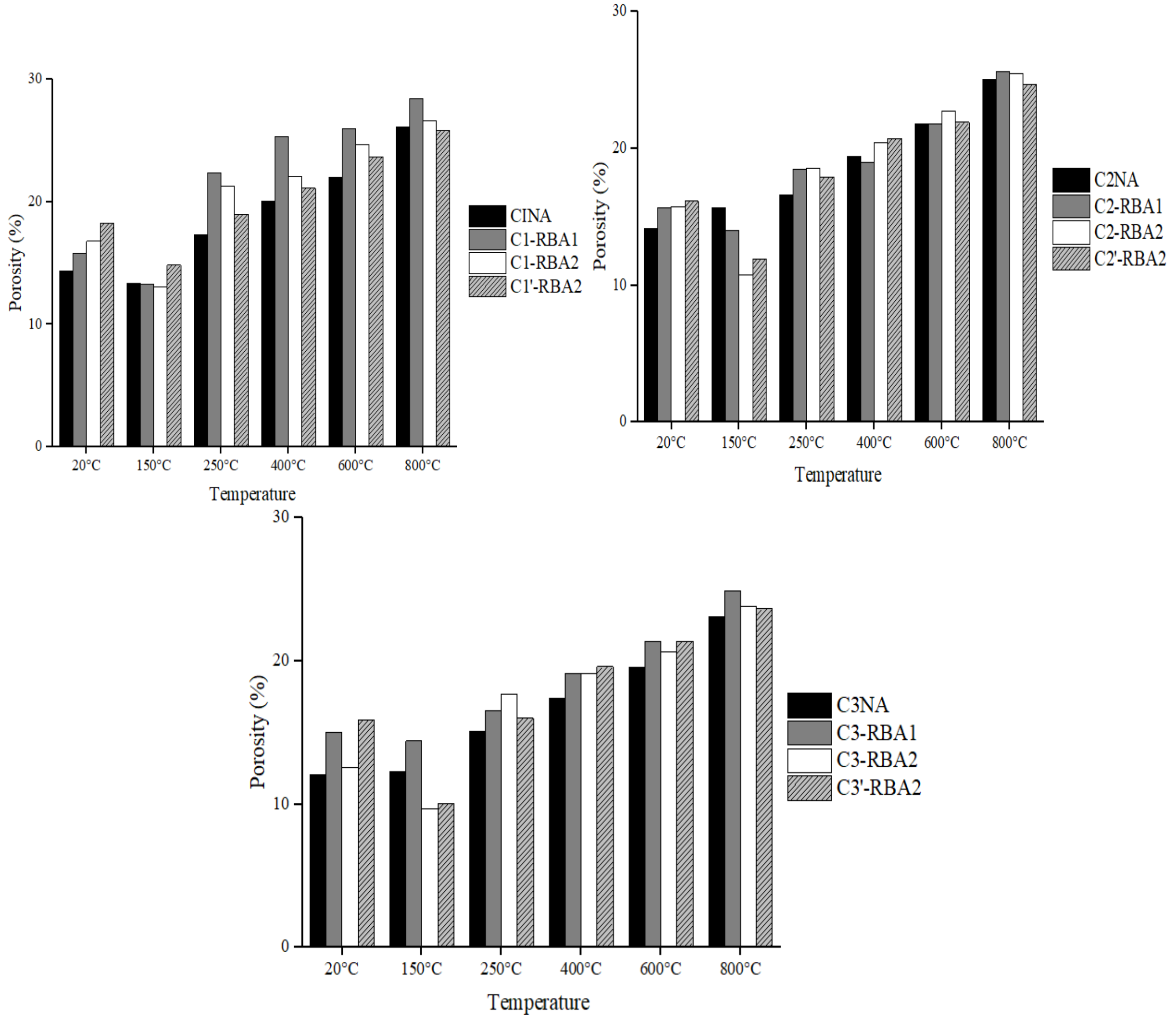


Figure IV.1. Evolution of water porosity of different types of concrete after exposure to elevated temperature.

The heating of specimens up to 800°C causes deterioration of concrete material. Regardless of the type of aggregates used, concrete porosity measured after heating to 800°C was similar for conventional concrete, concrete made with 20% of coarse RBA-1 (2%-8% of difference), concrete made with 20% of coarse RBA-2 (2%-3% of difference) and concrete made with 20% of coarse and fine RBA-2 (1%-2% of difference).

It should be noticed from figure IV.1 that, whatever the mix, the effect of cement dosage on concrete porosity is important. The results obtained show that, between 20 and 800°C, the concrete mixtures prepared with cement dosage of 350 kg/m^3 (w/c=0.59) had higher water porosity than that the concrete mixtures prepared with cement dosage of 400 kg/m^3 (w/c=0.47) and 450 kg/m^3 (w/c=0.38).

IV.2.2 Concrete density

Figure IV.2 shows the evolution of density of conventional concrete (C1NA, C2NA and C3NA) compared to that of concrete made with 20% of coarse RBA (C1-RBA1, C2-RBA1, C3-RBA1, C1-RBA2, C2-RBA2 and C3-RBA2) and that of concrete made with 20% of coarse and fine RBA2 (C1'-RBA2, C2'-RBA2 and C3'-RBA2). The results obtained confirm that concrete density is influenced by temperature and decreases with the increase of temperature.

After exposed to 400°C, conventional concrete (C1NA,C2NA and C3NA) shows a decrease in density of about 3%. The decrease in density for C2-RBA1 (3%) and C3-RBA1 (2%) was similar to conventional concrete. However, C1-RBA1 specimens experienced a density reduction of approximately 8%. Furthermore, in the rang temperature of 150°C-600°C, the concrete mixture C1-RBA1 has the highest decrease of density than that of conventional concrete (C1NA). For concrete prepared with 20 % of coarse RBA2, a decrease in density of about 2% was recorded for C1-RBA2 and C3-RBA2. C2-RBA2 shows no decrease in density.

Whereas for C1'-RBA2, C2'-RBA2 and C3'-RBA2, the density decreases by 2% after heating to 400°C. It should be noticed from figure IV.2 that the concrete mixture (C1'-RBA2, C2'-RBA2 and C3'-RBA2) have the best residual density than that of conventional concretes and concrete made with 20 % of coarse RBA-2 after exposure to high temperature. This is in accordance with the fact that the density of fine RBA-2 was higher than that of fine NA, which seems to indicate that concrete density is affected by the density of fine aggregates used. After heating to 800°C, concrete specimens undergo a high reduction in density about 8% to 9% for conventional concrete,about

6% to 8% for concrete made with 20% of coarse RBA1, about 8% was recorded for C1-RBA2 and C3-RBA2 and about 9% for C3'-RBA2. However, concrete mixtures C2-RBA2, C1'-RBA2 and C3'-RBA2 were presented low decrease in density (about 5%).

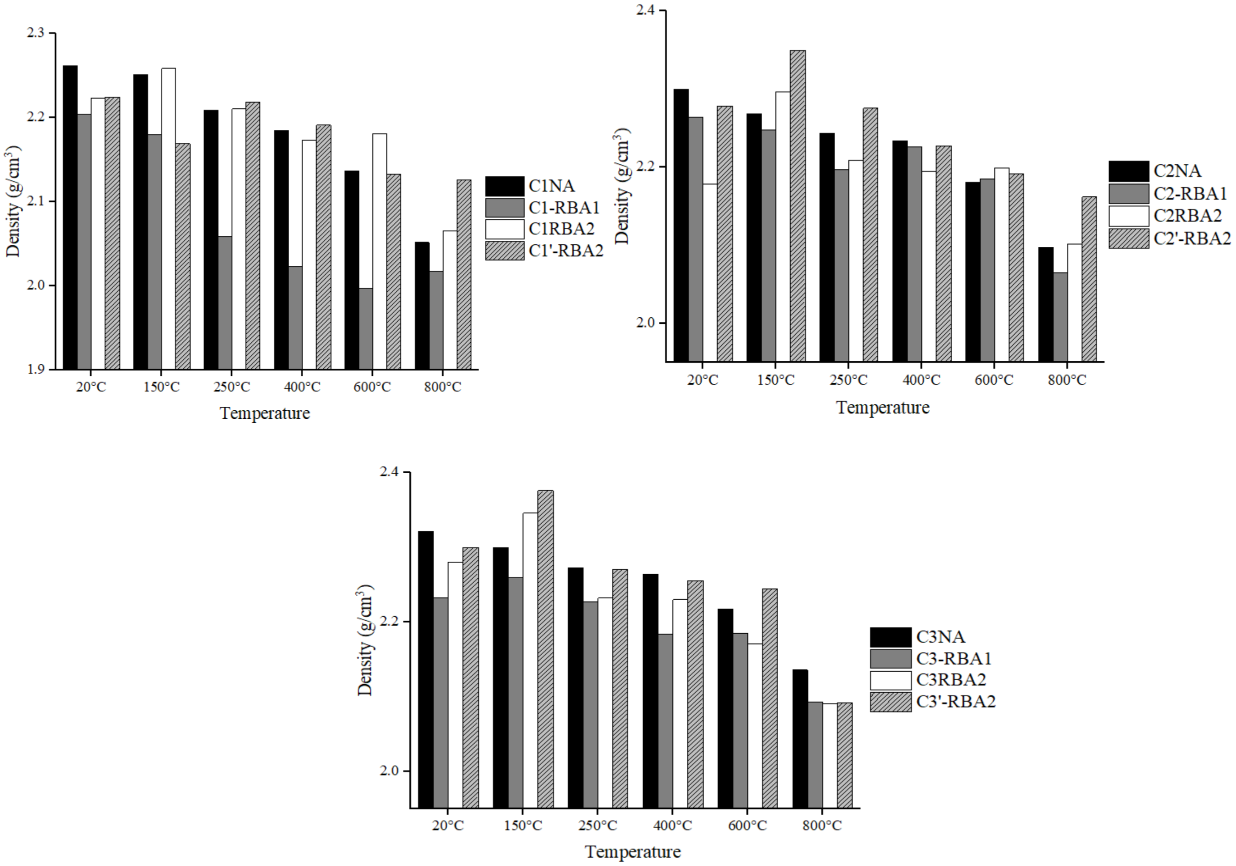


Figure IV.2. Evolution of density of different types of concrete after exposure to elevated temperature.

It can be seen that concretes made with 20% of coarse RBA-1 has the highest decrease of density, followed by conventional concrete and then concretes made with 20% of coarse RBA-2. The comparison of the density evolution in different concretes shows that the density of concrete made with 20 % of coarse and fine RBA-2 is higher than that of other concretes. As can be seen in figure IV.2, the effect of the cement dosage on concrete density is negligible whatever the type of aggregates used (natural or recycled).

IV.2.3 Mass loss

Figure IV.3 shows the average values of the mass loss obtained from concrete simples after exposure to high temperatures. It can be seen from figure IV.3 that the increase of mass loss of concrete made with coarse RBA (RBA-1 or RBA-2) was higher than that of conventional concrete. Further, the greatest mass loss was recorded by concretes containing 20% of coarse RBA2 in the temperature range of 250°C to 800°C. The pre-saturation of coarse RBA provides to concrete an additional quantity of water which escapes during the heat treatment; this can explains the increase of mass loss of concrete made with coarse RBA compared to conventional concrete.

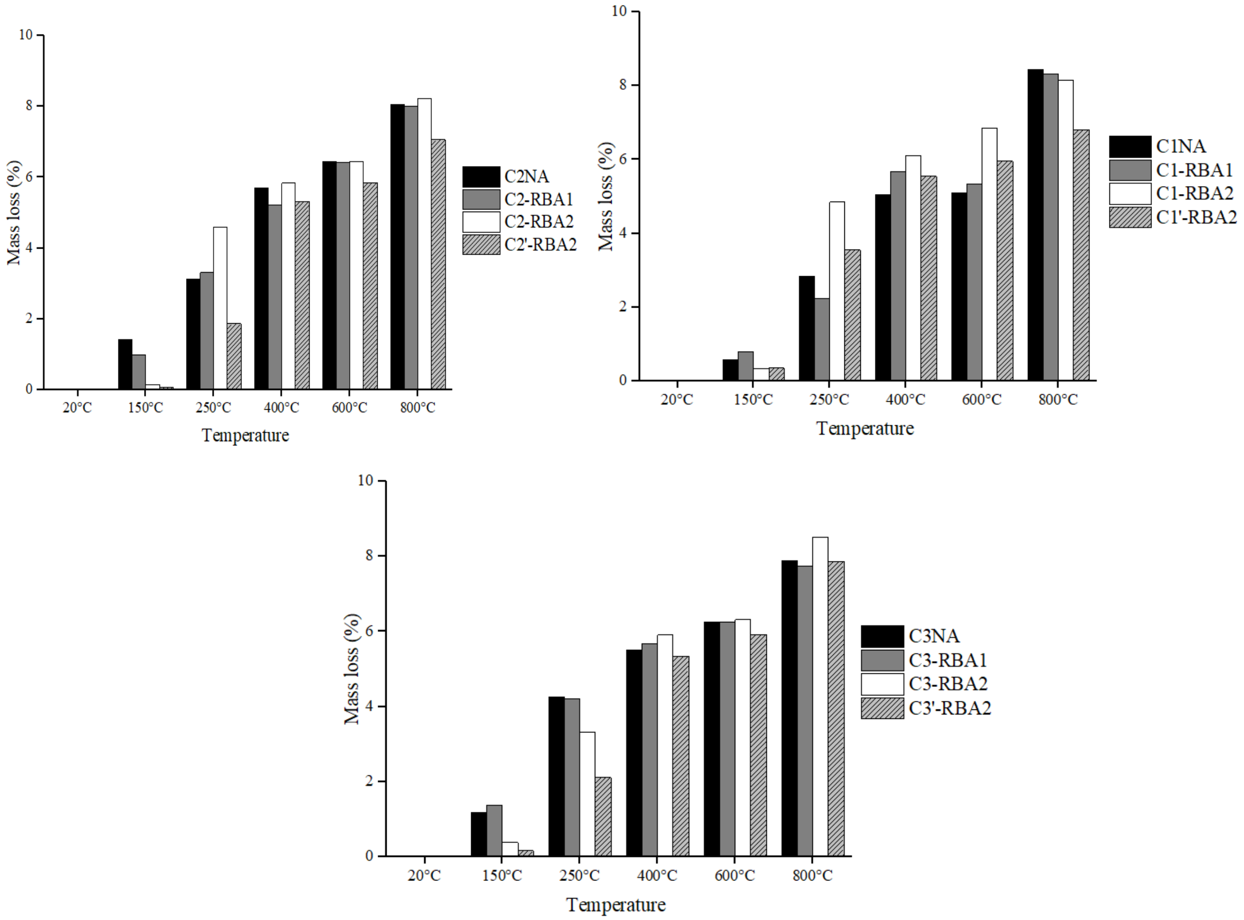


Figure IV.3. Evolution of mass loss for the different concretes according to the temperature.

Based on the experimental results, concretes made with 20 % of coarse and fine RBA-2 presents lower mass loss than that of conventional concretes and concrete made with 20 % coarse RBA-2. This can be explained by the replacement of fine NA by fine RBA-2 which leads to reduce the amount of fine NA subject to transformation at elevated temperatures. As mentioned in section

III.3.1, fine NA present greater mass loss than fine RBA-2 between 100°C and 900°C. This means that the use of RBA-2 as fine aggregates contributed to the lowest mass loss.

As can be seen in figure IV.3, the effect of the cement dosage on the mass loss of concrete is negligible.

IV.2.4 Change of specimens volume

The relative changes in the volume of different types of concrete are summarized in figure IV.4. In the temperature range of 20°C-400°C, all concrete specimens undergo a slight reduction in volume (less than 0.5%). In this temperature range, the type of aggregates has no influence on the volume variation; the aggregates undergo transformation at temperatures above 500°C.

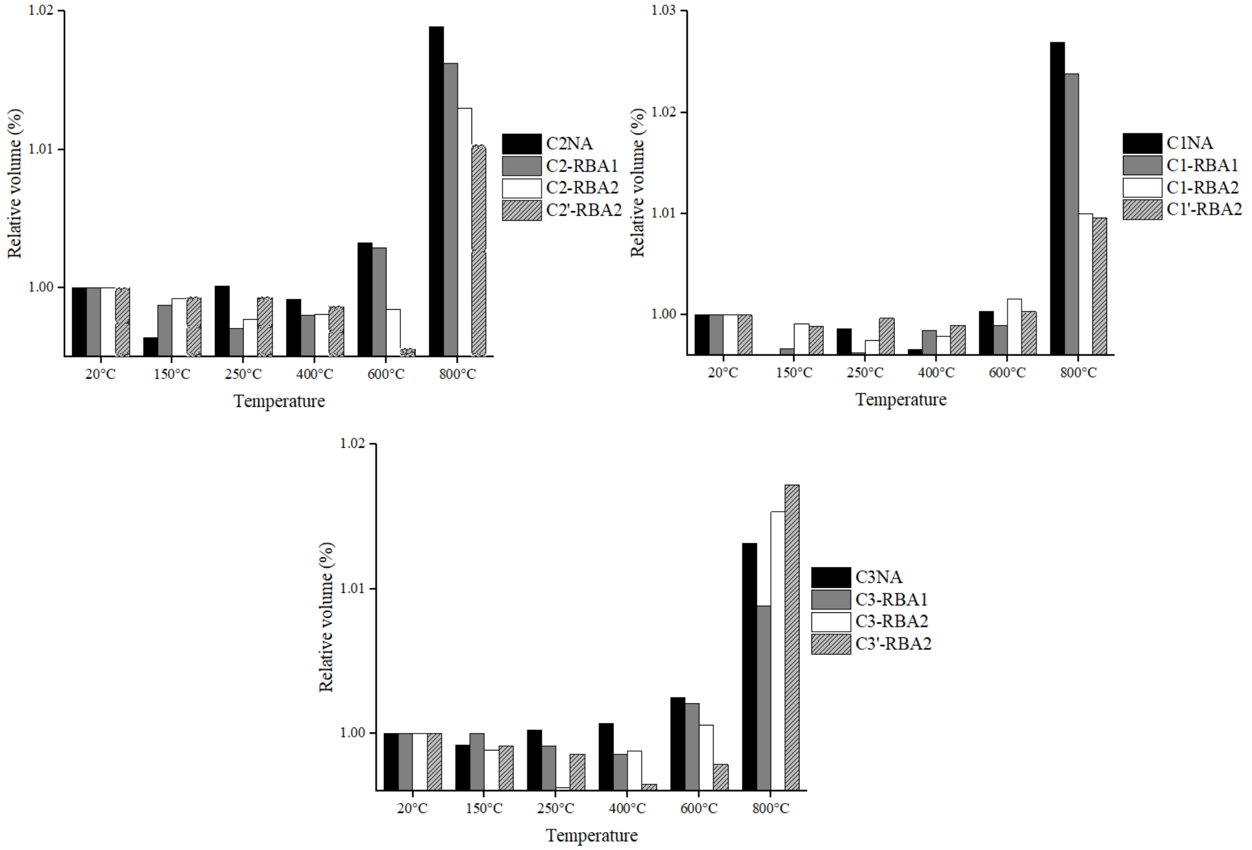


Figure IV.4. Relative volume of various specimens as a function of elevated temperatures.

Beyond 400°C, an important increase in specimen's volume was observed for all the mixes, especially after heating to 800°C. The increase of volume of concrete made with 20% of coarse RBA was lower than that of conventional concrete in the range temperature of 600°C–800°C. This can be explained by the increase in the volume of limestone aggregate due to the decarbonation of $CaCO_3$ which is then transformed into CaO. After cooling of concrete specimens, the CaO reacts with the ambient humidity to form $Ca(OH)_2$ with a significant volume expansion [Hager et al., 2013], [Xing., 2011]. However, concretes prepared by 20% of coarse RBA-2 exhibit a smaller increase in volume than that of concretes prepared by 20% of coarse RBA-1, this shows that coarse RBA-2 has better thermal stability than coarse RBA-1, on the one hand. On the other hand, the type of coarse RBA used in this study (as mentioned in paragraph **II.2.1.4)** can influence the variation of concrete volume. Coarse RBA-2 (used bricks) was obtained from a cement furnace which has been previously exposed to a temperature of 1400°C for one year; that gives a high thermal stability to RBA-2. However, RBA-1 (un-unused bricks) was obtained from new refractory brick which has not undergone any heat treatment.

Furthermore, it also found that the concretes made with 20% of coarse and fine RBA-2 have the lowest increase of volume than that of conventional concrete and concretes made with 20% of coarse RBA-2. This can be explained by the fact that conventional concrete and concretes made with 20% of coarse RBA-2 have contained a large amount of fine aggregates (siliceous sand containing quartz). Therefore, siliceous aggregates can cause cracking at the paste-aggregate interface at about 575°C, due to the transformation of quartz α to quartz β which is associated with a sudden volume expansion of the order of 1–5.7% [Khoury et al., 2007]. The results obtained in this test indicate that C1'-RBA2 and C2'-RBA2 concrete mixture expand less compared to other mixtures after exposure to 800°C. Coarse and fine RBA-2 are characterized by an excellent fire performance. However, the concrete mixture C3'-RBA2 represents the worst increase in volume.

IV.2.5 UPV

Figure IV.5 presents the evolution of the UPV of the different concretes studied as a function of the temperature. The results indicate that, whatever the type of aggregates, UPV decreases with the increase of temperature.

Conventional concretes (C1NA, C2NA and C3NA) present an excellent quality at room temperature (UPV > 4.5 km/s), a good quality when heating from 150°C to 250°C and doubtful quality after heating to 400°C. The quality of concrete made with 20% of coarse RBA-1 (C1-RBA1, C2-RBA1 and C3-RBA1) can be classified as good quality for the range temperature of 20°C - 250°C and doubtful after heating to 400°C. Concretes made with 20% of coarse RBA-2 (C1-RBA2, C2-RBA2 and C3-RBA2) and concretes made with 20% of coarse and fine RBA-2 (C1'-RBA2, C2'-RBA2 and C3'-RBA2) exhibit a good quality at room temperature and even after heating to 400°C (3.5 km/s < UPV < 4.5 km/s).

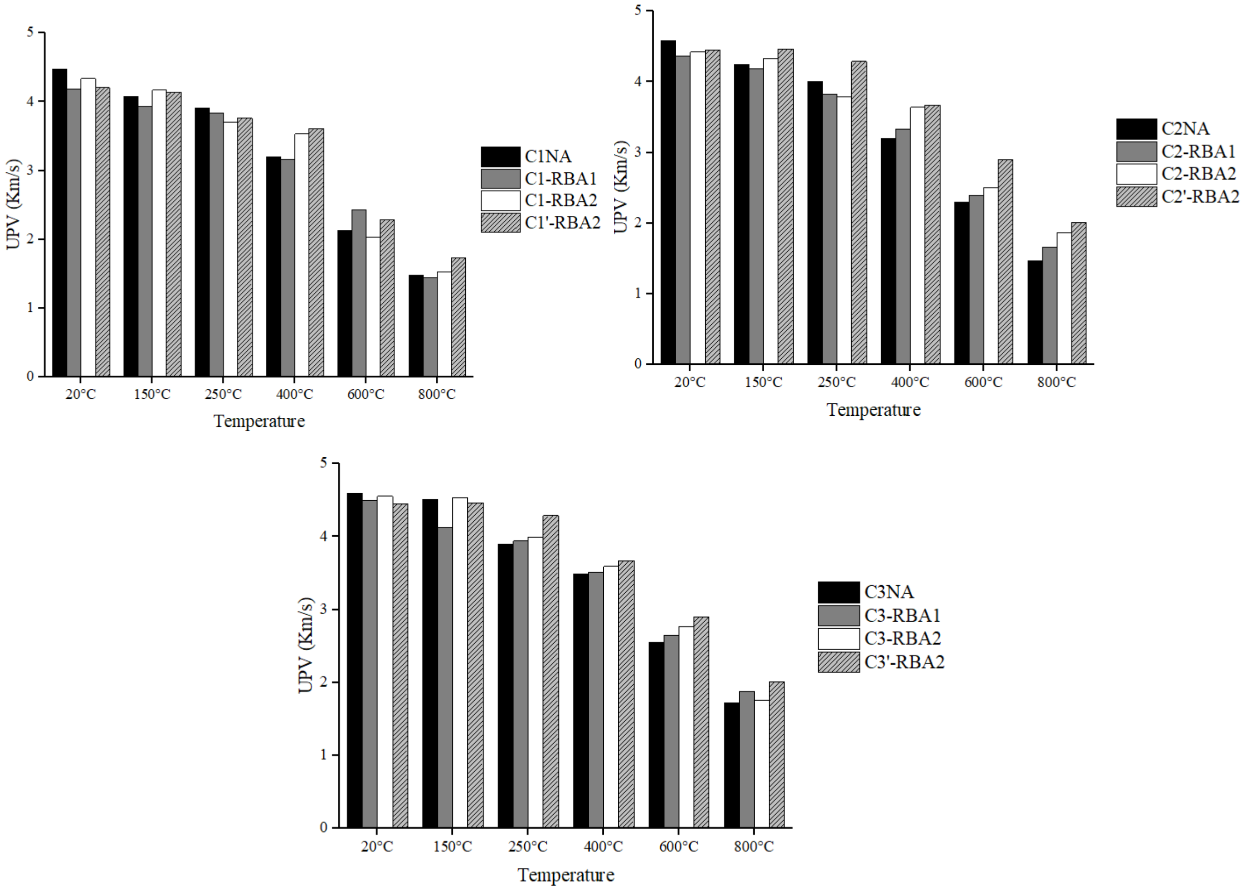


Figure IV.5. Evolution of UPV for the different concretes according to the temperature.

After heating to 600°C, all concrete mixtures present similar quality (poor quality) (2.0 km/s < UPV < 3.0 km/s). Furthermore, the quality of all concrete mixtures was classified as very poor after heating to 800°C (UPV < 2.0 km/s). It can be observed, from figure IV.5, that the decrease of UPV with the increase of temperature was much moderated for concretes made with RBA (coarse or both coarse and fine) when compared to conventional concrete made with NA.

By comparison with the UPV results between conventional concrete and concretes specimens made with coarse 20% of coarse RBA, it can be concluded that concretes made with 20% of coarse RBA present the highest values of UPV than that of conventional concrete. Another explanation relates that concrete prepared with coarse RBA shows an increase of volume lower than that concrete prepared with natural aggregates, which results in decreased time travel in pulse velocity testing.

On the other hand, the results obtained show that the reduction in UPV for the specimens containing 20% of coarse and fine RBA-2, in the range temperature of 150°C–800°C was lower than that of other specimens (conventional concrete and concrete made with 20% of coarse RBA-2). Conventional concrete, prepared with 100% of NA, shows significant deteriorations than concrete made with 20% of coarse and fine RBA-2 (which contains 80% of coarse and fine NA). Coarse NA are limestone aggregates and exhibit a thermal instability around 700°C, which has been observed by other researchers [Novak and Kohoutkov, 2018]. A similar distress can begin about 573°C in the case of siliceous sand (fine NA), where quartz α starts to transform into quartz β [Biró et al., 2019]. The percentage of replacement of coarse and fine NA by coarse and fine RBA-2 (20%) reduced the degradation of concrete and cracks propagation, on the one hand. On the other hand, the porosity of coarse RBA-2 generates better adhesion with cement paste, which reduces the damage of transition zone.

It should be noticed from figure IV.5 that the effect of the cement dosage on the UPV of concrete is significant. The first series of concrete mixtures C1 with a cement dosage of 350 kg/m^3 had lower UPV value than that the second concrete mixtures (C2 with a cement dosage of 400 kg/m^3) and the third concrete mixtures (C3 with a cement dosage of 450 kg/m^3) before and after exposure to high temperature.

IV.2.6 Damage degree

Results of damage degree of each concrete specimen after exposure to high temperature are shown in figure IV.6. The damage degree of all concretes increases continuously with the increase of temperature, as shown in figure IV.6.

It is observed also, from figure IV.6, that the greatest damage degree was recorded by concretes containing coarse and fine NA (conventional concrete) in the temperature range of 150°C-800°C. On the other hand, the damage degree of concrete made with 20% of coarse RBA-2 is lower than that of concrete made with 20% of coarse RBA-1 between 150°C and 600°C. After heating to 800°C, concretes made with 20% of RBA-1 or RBA-2 present a similar damage degree.

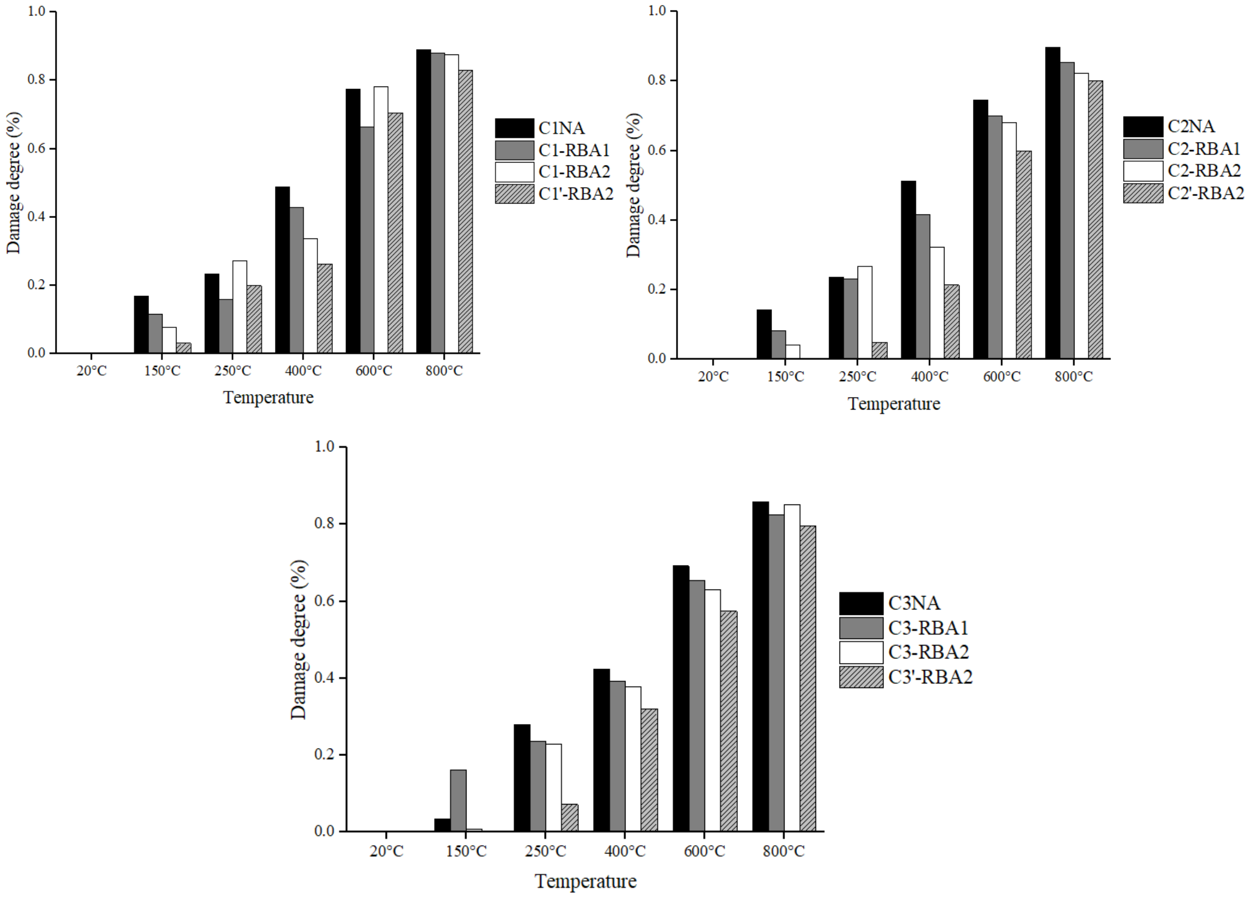


Figure IV.6. Evolution of damage degree for different concrete mixtures after heating.

For concrete containing 20% of coarse and fine RBA-2, the increase of damage degree, in the range temperature of 150°C to 800°C, was lower than that of other concrete (conventional concrete and concrete made with 20% of coarse RBA-2).

The results obtained in this test indicate that concrete mixtures C2 and C3 with a cement dosage of 400 kg/m^3 and 450 kg/m^3, respectively, exhibit a smaller increase in damage degree than that of concretes mixtures C1 with a cement dosage of 350 kg/m^3.

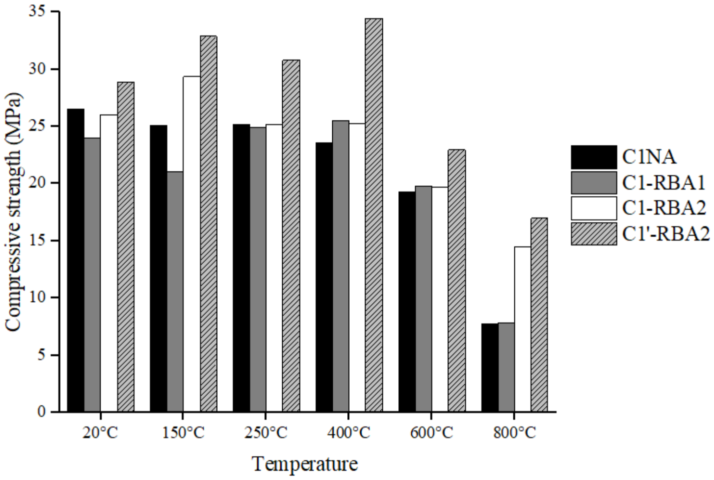
IV.2.7 Compressive strength

The average residual compressive strength of each concrete specimen before and after exposure to high temperature are shown in figure IV.7. The figure shows that the use of RBA as aggregates in concrete influences the evolution of compressive strength with temperature.

It is observed from figure IV.7, that compressive strength of conventional concrete (C1NA, C2NA and C3NA) decreases continuously with the increase of temperature. The results obtained show that, in the range of 150–400°C, there is an increase in residual compressive strength of concrete made with 20% of coarse RBA-1 (C1RBA1, C2RBA1 and C3-RBA1) than that the initial values (unheated specimens). An important increase in compressive strength was observed by C3-RBA1 at 150°C (about 38%) and for C2-RBA1 at 250°C (about 29%).

For concrete made with 20% of coarse RBA-2, the results show an increase in compressive strength of C1-RBA2 at 250°C and then decreased between 400°C and 800°C. The C2-RBA2 and C3-RBA2 have the best residual compressive strength, in the range of 150–600°C; its residual strength were higher than the initial strength (about 2% for C2-RBA2 and 5% for C3-RBA2 at 600°C). However, concretes made with 20% of coarse and fine RBA-2 show different evolution of compressive strength with temperature. Moreover, in the range of 20–800°C, the residual compressive strength of concrete made with 20 % of coarse and fine RBA-2 (C1'-RBA2, C2'-RBA2 and C3'-RBA2) is higher than that of conventional concrete (C1NA, C2NA and C3NA).

After heating to 800°C, conventional concretes lose a significant amount of compressive strength (about 71%). Whereas, concretes containing 20% of coarse RBA-1 lose less important amount of initial strength (about 54%). However, concretes prepared with 20% of coarse RBA-2 and with 20% of coarse and fine RBA-2 have the best compressive strength; they were able to retain more than 60% of initial compressive strength.



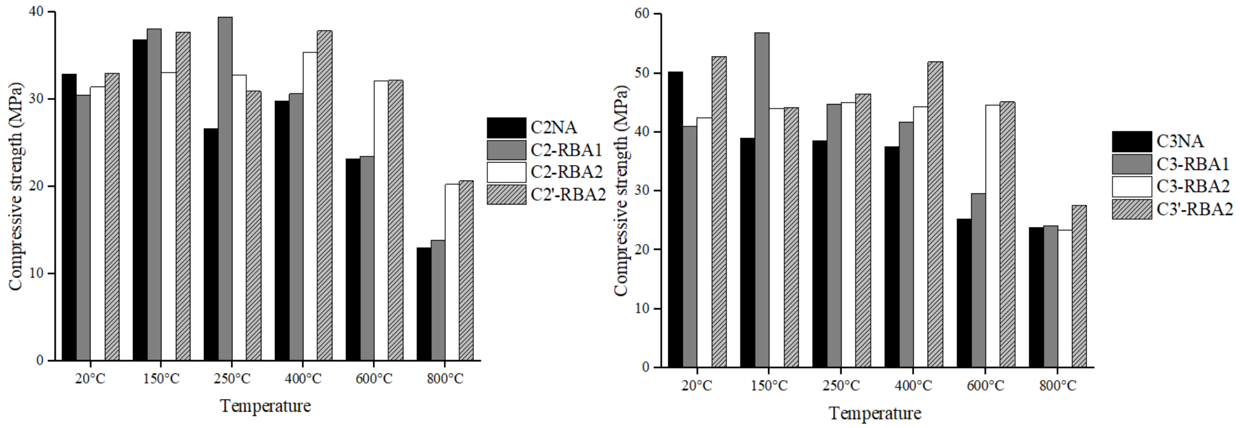


Figure IV.7. Evolution of compressive strength of elasticity for the different concretes after heating.

The comparison of the residual compressive strength in different concretes shows that the residual strength of concrete made with 20% of coarse RBA (RBA-1 or RBA-2) is higher than that of concrete made with natural aggregates (conventional concrete). However, the residual compressive strength of concrete made with 20 % of coarse RBA-2 is better than that of concrete made with 20% of coarse RBA-1. Moreover, the residual compressive strength of concrete made with 20 % of coarse and fine RBA-2 is similar or even better than that of concrete made with 20% of coarse RBA-2, especially in the 400°C to 800°C temperature range. It can be explained by the fact that fine RBA-2 is one of the best aggregates for concrete that may have to resist fire and it performs much better than similar concrete containing 100% fine NA (siliceous sand).

IV.2.8 Dynamic modulus of elasticity

The dynamic modulus of elasticity of all mixtures, obtained before and after the heating process, is shown in figure IV.8. A continuous decrease in dynamic modulus of elasticity was observed with the increase of temperature. The results obtained show that the evolution of the dynamic modulus of elasticity of all concrete mixtures was similar regardless of the type of aggregates used (natural or recycled).

It is observed from figure IV.8, that the largest loss of dynamic modulus of elasticity was recorded by concretes containing coarse and fine NA (conventional concrete) in the temperature range of 150°C to 800°C. On the other hand, the loss of dynamic modulus of elasticity of concrete made with 20% of coarse RBA-2 is lower than that of concrete made with 20% of coarse RBA-1 between 150°C and 800°C. Coarse RBA-2 has better absolute density, better abrasion resistance and lower porosity when compared to coarse RBA-1.

For concrete containing 20% of coarse and fine RBA-2, the decrease of dynamic modulus of elasticity, in the range temperature of 150°C to 800°C, was lower than that of other concrete (conventional concrete and concrete made with 20% of coarse RBA-2).

It should be noticed from figure IV.8 that the effect of the cement dosage on the dynamic modulus of elasticity of concrete is significant. The first series of concrete mixtures C1 with a cement dosage of 350 kg/m^3 had lower dynamic modulus of elasticity value than that the second concrete mixtures (C2 with a cement dosage of 400 kg/m^3) and the third concrete mixtures (C3 with a cement dosage of 450 kg/m^3) before and after exposure to high temperature.

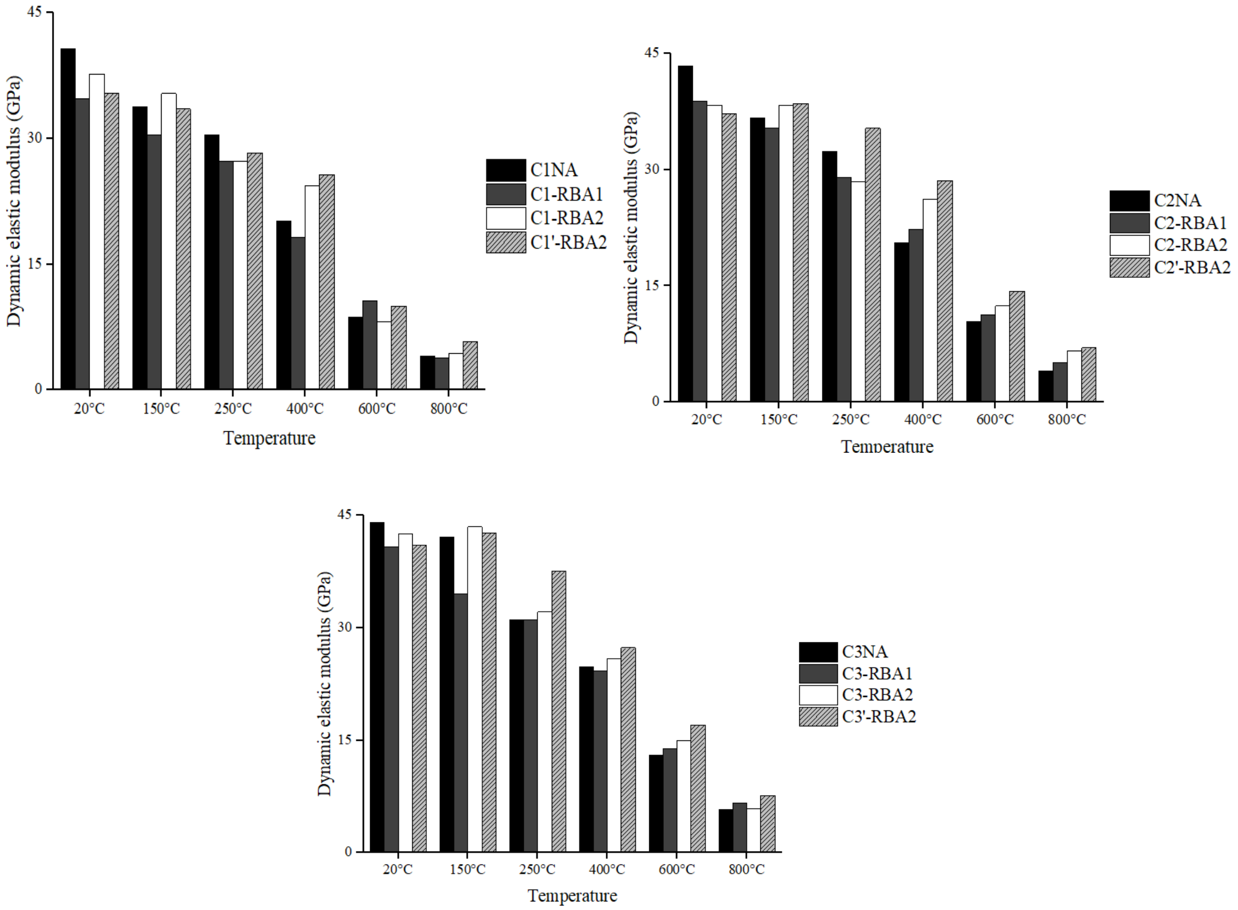


Figure IV.8. Evolution of dynamic modulus of elasticity for the different concretes after heating.

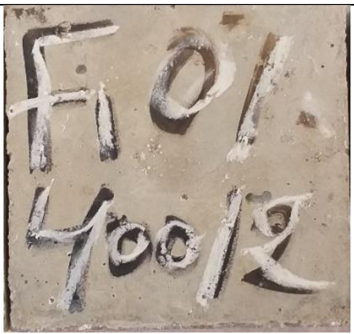
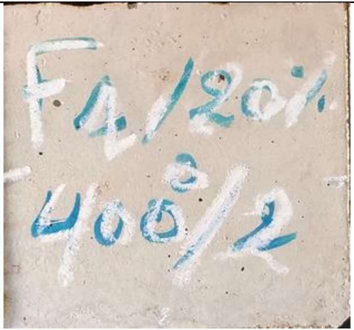
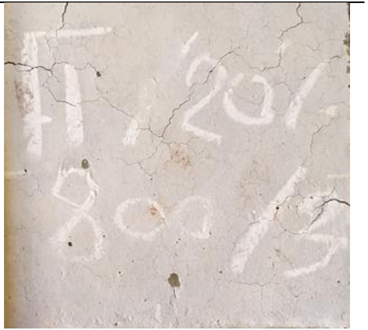
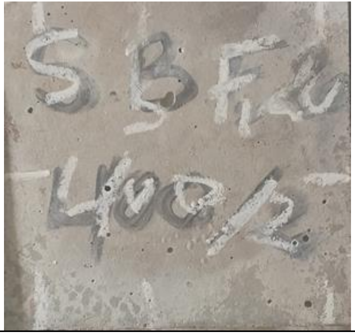
IV.3 SPALLING, COLOR CHANGE AND CRACKING OF SPECIMEN’S CONCRETE

Following exposure to elevated temperatures, neither of the concrete specimens exhibited any spalling or fragmentation with a heating rate of 3°C/min. Table IV.1 compares concrete color changes for a few specimens of some types of mixes (C1NA, C1RBA-1, C1RBA-2 and C1'RBA-2) at few selected temperatures. According to Rafi and Aziz [Rafi and Aziz,2019] and Zhou [Zhou et al.,2020], the change of color of heated concrete is explained by the gradual water departure from cement pste (free water, capillary water and physically bound water) and decomposition of CSH gel and CH.

It should be noted that the specimens remain intact up to 250°C. This means that, up to 250°C, the concretes maintained the color (gray-black) and their structural integrity. When the temperature reached 400°C, the surface color changed to red as shown in Table IV.1. The appearance of red color is an indication of the presence of hydrated iron oxides in the concrete [Hager, 2013]. At 600°C, the concrete samples turned into a gray-white color. Rafi and Aziz [Rafi and Aziz, 2019] indicated that the gray-white discoloration of heated concrete is a result of decomposition and calcination limestone aggregates containing $CaCO_3$. After exposure to a temperature of 800°C, the color of the samples changed to pink. This pink discoloration was more prominent in C1-RBA1, C1-RBA2 and C1'-RBA2 concrete samples. The color changes observed for the current specimens are overall consistent with literature reports [Wu et al., 2019], [Chen et al., 2018].

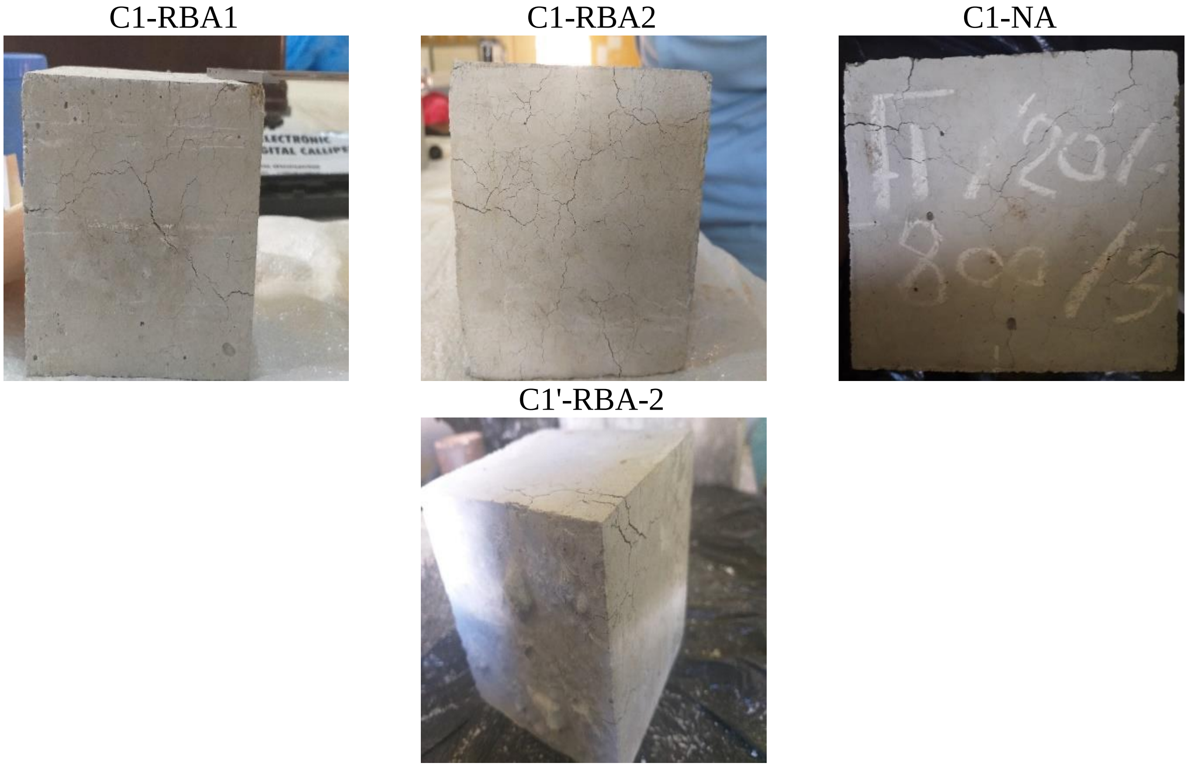
Apart from the aforementioned color changes of concrete, another noticeable distress was in the form of concrete cracking which was visible on the surface of concrete. The results indicate that whatever the mix, no crack was observed with the naked eye on the surface of the specimens in the range of 20°C–250°C. The first crackings observed on the surface of the specimens appear after the heating to 400°C. These cracks are scattered and short. When the temperature increased to 600°C, the number of cracks on the surface of all the specimens increased. These cracks was very pronounced and more prominent. The cracks further widened at 800°C. It can be seen from Table IV.1 that the cracks were sufficiently long to meet and form a connected network on exposure to 800°C.

Table IV.1. Apparent morphology of specimen subjected to elevated temperatures.

	400°C	600°C	800°C
C1NA			
C1-RBA1			
C1-RBA2			
C1-'RBA2			

The degradation of concrete structure can be coarsely identified by observing the concrete surface after being subjected to high temperatures, as shown in figure IV.9. The test results indicate that, after the heating-cooling cycle at 800°C, all specimens exhibited obvious damage in the structure.

Furthermore, the damage to concrete structure results principally from the deteriorations of the hydrate products of cement paste and also transformations occurring within aggregates after heating to 800°C. The deterioration of concrete structures at high temperatures is an indication of the changes in the physical and mechanical properties of concrete which reduces the ability of concrete to withstand the service loads.


C1-RBA1

C1-RBA2

C1-NA

C1'-RBA-2

Figure IV.9. The deterioration of concrete structure after the heating-cooling cycle to 800°C.

The interior morphologies of concrete after the heating-cooling cycle to 800°C are shown in figure IV.10. With exposure to higher temperature, the volume of aggregate increases, and at the same time the cement paste shrinks. As a result, cracks are developed along the aggregate – cement matrix interfaces. This might arise from the thermal incompatibility between aggregates and cement matrix. The above damage localization was more prominent between the coarse NA and cement paste, as shown in figure IV.10-a. Nevertheless, the cohesion between coarse RBA-1 and cement matrix was still sufficiently strong, as shown in figure IV.10-b. It can be observed from figure IV.10-c that some coarse RBA-1 particles have still adhered to the cement paste during the mechanical loading after fire exposure. This means that the bond between coarse RBA-1 particles and cement paste was very good. This can be explained by the rough surface of coarse RBA-1 which improves the cohesion with cement paste and thus creates a strong bond.

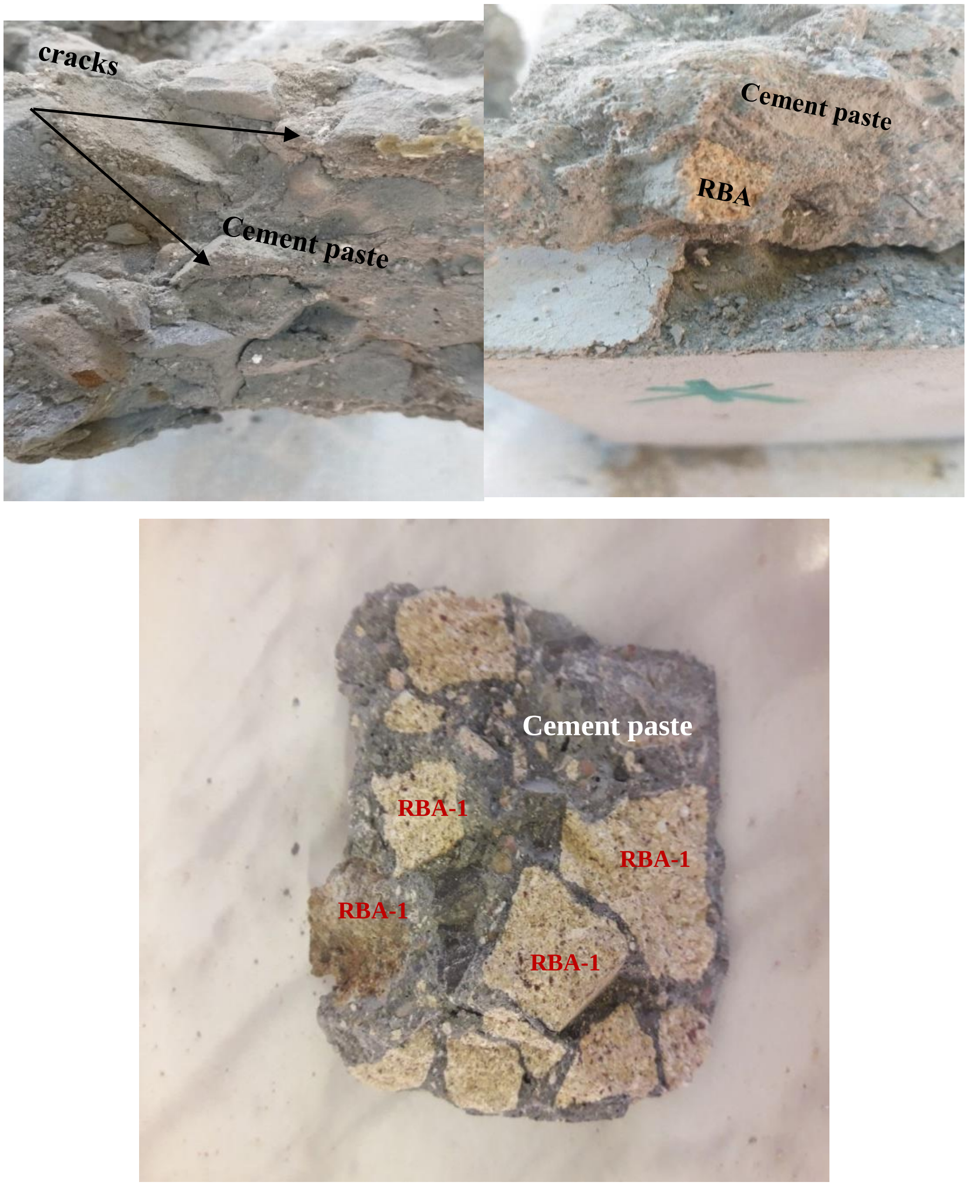


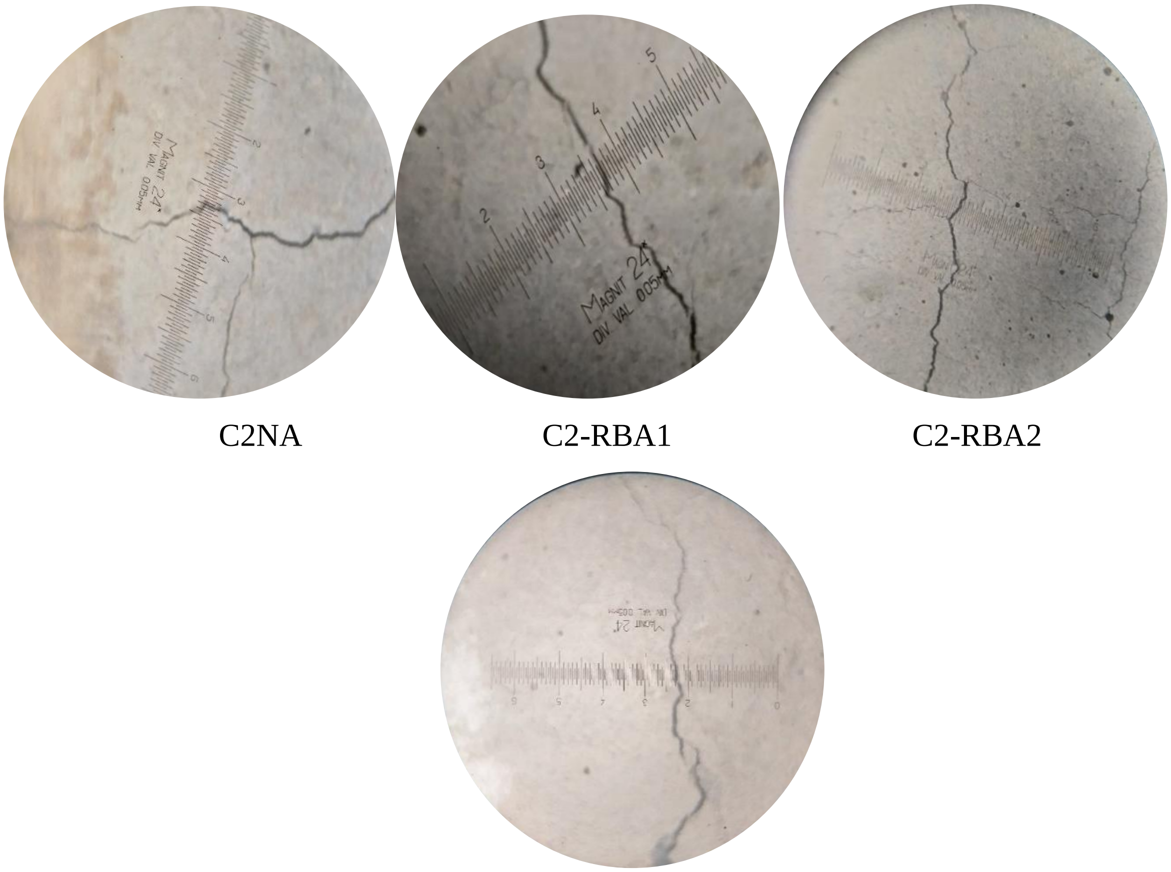
Figure IV.10. Interior damages of specimens after the heating-cooling cycle at 800°C.

IV.4 Microscope analysis

It was not possible to measure the development of cracks widths by the naked eye after the heating-cooling cycle. Therefore, the widths of the cracks in the concrete caused during heating and cooling processes were measured using a microscope.

IV.4.1 Cracks in concrete specimens after heating to 400°C

As mentioned before, all specimens started to show some cracks when the temperature increased to 400°C. As shown in figure IV.11, a few microcracks after the heating-cooling cycle at 400 °C was observed. According to an earlier study [Chen et al.,2018], this can be attributed to: (a) the damage of cement structure ; (b) the thermal properties of aggregates and cement paste which are inconsistent ; (c) the micro-deformation caused by thermal expansion, which causes the generation of cracks. In general, the cracks width of all specimens are at the same level at 400°C (about 0.1 mm).



C2NA C2-RBA1 C2-RBA2

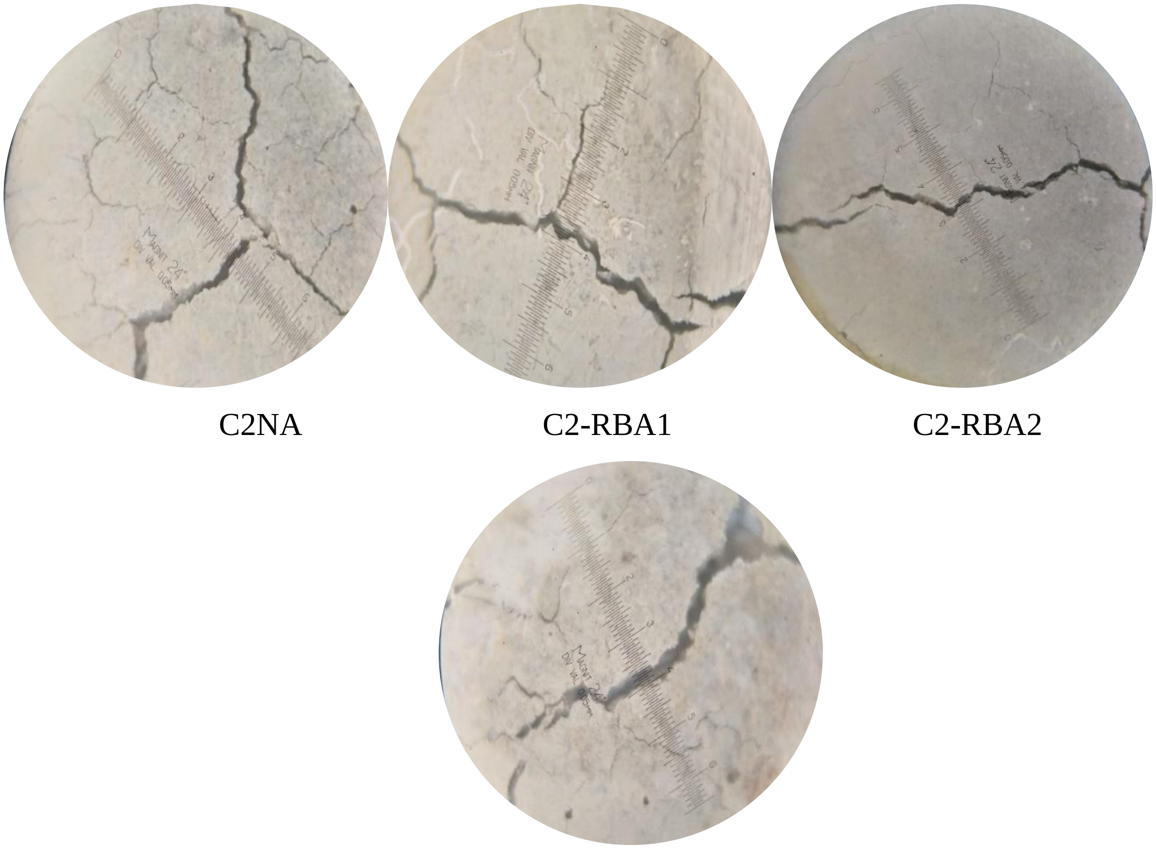
C2'-RBA2

Figure IV.11. Crack width of concrete specimens after heating to 400°C.

IV.4.2 Cracks in concrete specimens after heating to 600°C

With the increase of the elevated temperature, the crack width of specimens is found to increase. As can be seen in the photographs in figure IV.12, the development of the cracks width of all specimens becomes more significant when the temperature rises from 400 to 600°C.

The test results indicate that the maximum width of cracks was about 0.15 mm for C2-NA, C2-RBA1 and C2-RBA2 and about 0.3 mm for C2'-RBA2. With a longer crack width, the bonding between the aggregates and the cement paste reduces, and subsequently the physical and mechanical properties of concrete decrease. Furthermore, the microstructure of concrete was also significantly modified.



C2NA C2-RBA1 C2-RBA2

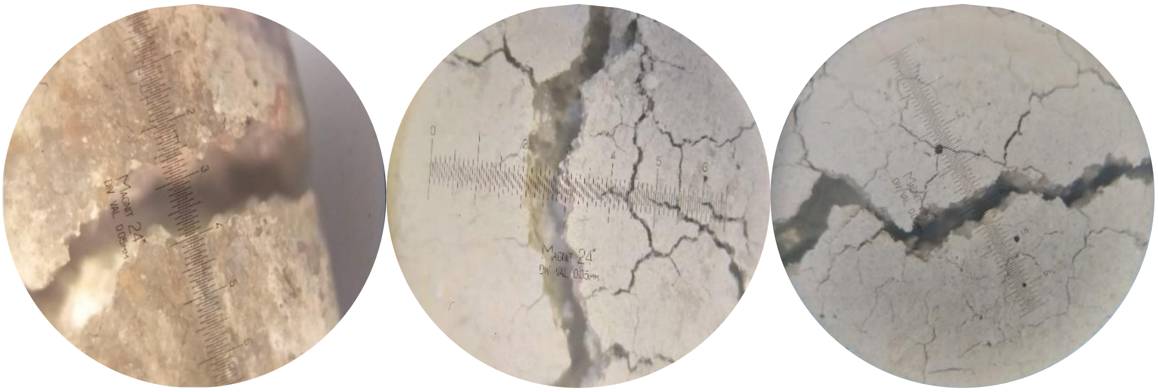
C2'-RBA2

Figure IV.12. Cracks width of concrete specimens after heating to 600°C.

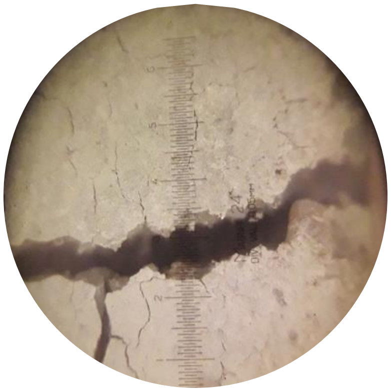
IV.4.3 Cracks in concrete specimens after heating to 800°C

As illustrated from the microscope photographs at the temperature of 800°C in figure IV.13, the cracking width of concrete was developed sharply. Moreover, wider and deeper cracks were observed continuously at that temperature level. The maximum width of cracks after heating at

800°C was about 0.8 mm for C2NA, 0.55 mm for C2-RBA1, 0.7 mm for C2-RBA2 and 0.65 mm for C2'RBA-2. This increase can be explained by the destruction of CSH, decomposition of CH and desiccation of pore structure [Liu et al., 2018], [Zhou et al., 2020].



C2NA C2-RBA1 C2-RBA2



C2'-RBA2

FigureIV.13. Cracks width of concrete specimens after heating to 800°C.

By comparing figure IV.11, IV.12 and IV.13, it was found that the development of the cracks width with temperature is different for each type of specimens. C2NA specimens experienced the higher crack width than those of C2-RBA1, C2-RBA2 and C2'RBA-2 specimens, in particular after the heating–cooling cycle of 800°C. This can explain the observation of the increase of volume of C2NA which was relatively more important than that of C2-RBA1, C2-RBA2 and C2'RBA-2 specimens after heating to 800°C, which seems to indicate that the development of the crack width of concrete is affected by the nature of the aggregates used.

IV.5 RELATIONSHIP BETWEEN MECHANICAL AND PHYSICAL PROPERTIES OF DIFFERENT CONCRETES

IV.5.1 Relationship between residual compressive strength and UPV

The relationship between compressive strength and UPV has been plotted in figure IV.14. From this figure, it is observed a good relationship between compressive strength and UPV. It can be seen that the reduction in the UPV leads to a decrease in compressive strength. That loss in compressive strength is due to the deterioration of cement hydrates, for instance the CSH and CH. The process caused emptiness into the capillary pores of the cement matrix, and also micro cracking occurred. Those physico-chemical transformations could be responsible of this situation. However, that loss of compressive strength depends on the type of aggregates (natural or recycled).

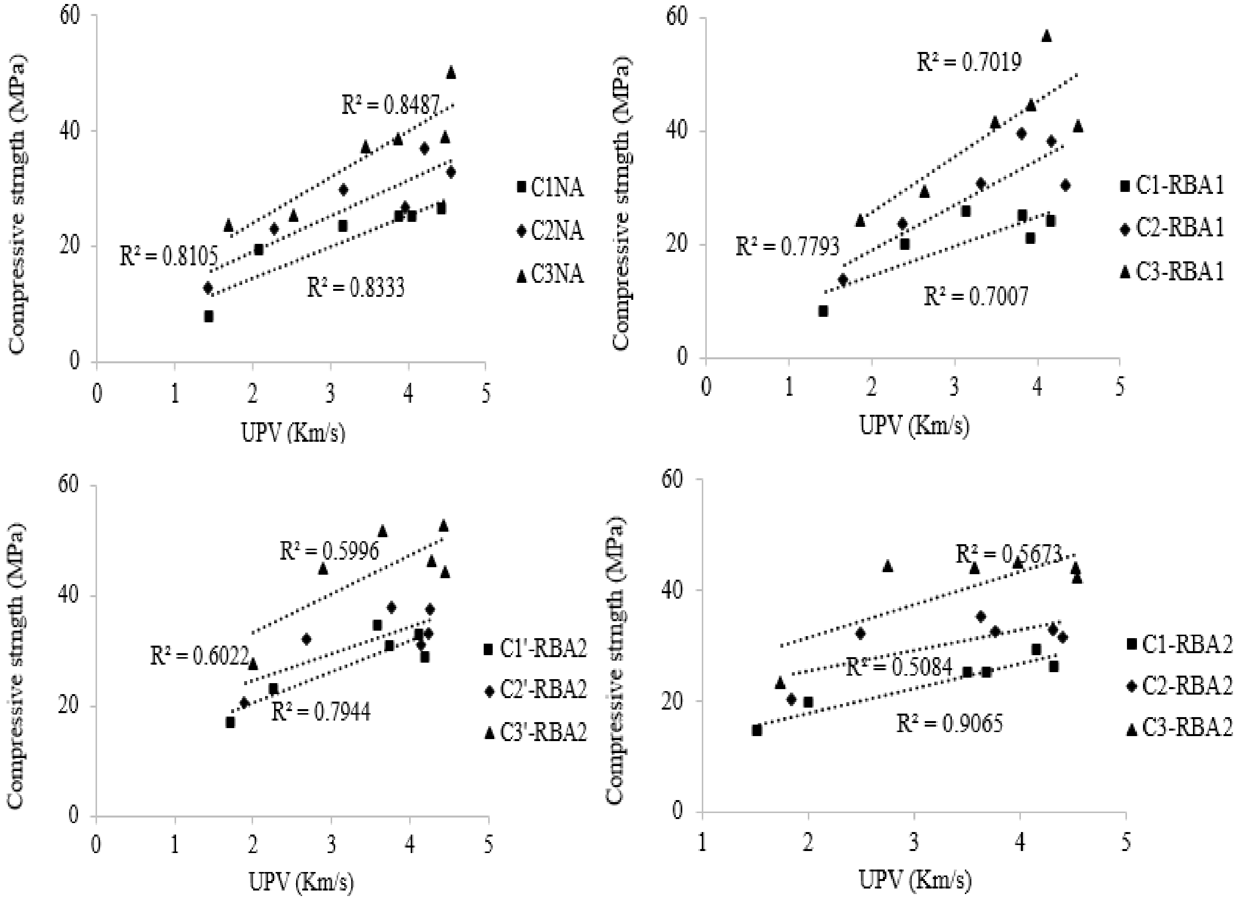


Figure IV.14. Evolution of compressive strength strength as function of UPV.

IV.5.2 Relationship between residual compressive strength and damage degree

The relationship between compressive strength and damage degree has also been plotted in figure IV.15. From the figure IV.15, it is noted that the augmentation of the damage degree goes with a big loss of compressive strength. The aspect of those curves is denoted by many correlations. This result seems to find its origins in the type of aggregates (natural or recycled). In fact, the loss of compressive strength can be correlated with the deterioration of aggregates, for instance, the limestone, $CaCO_3$. Limestone aggregates undergo visual degradation following the decarbonation of limestone, $CaCO_3$ transformed into CaO following the departure of CO_2. These thermal instabilities promote further degradation of concrete**.**

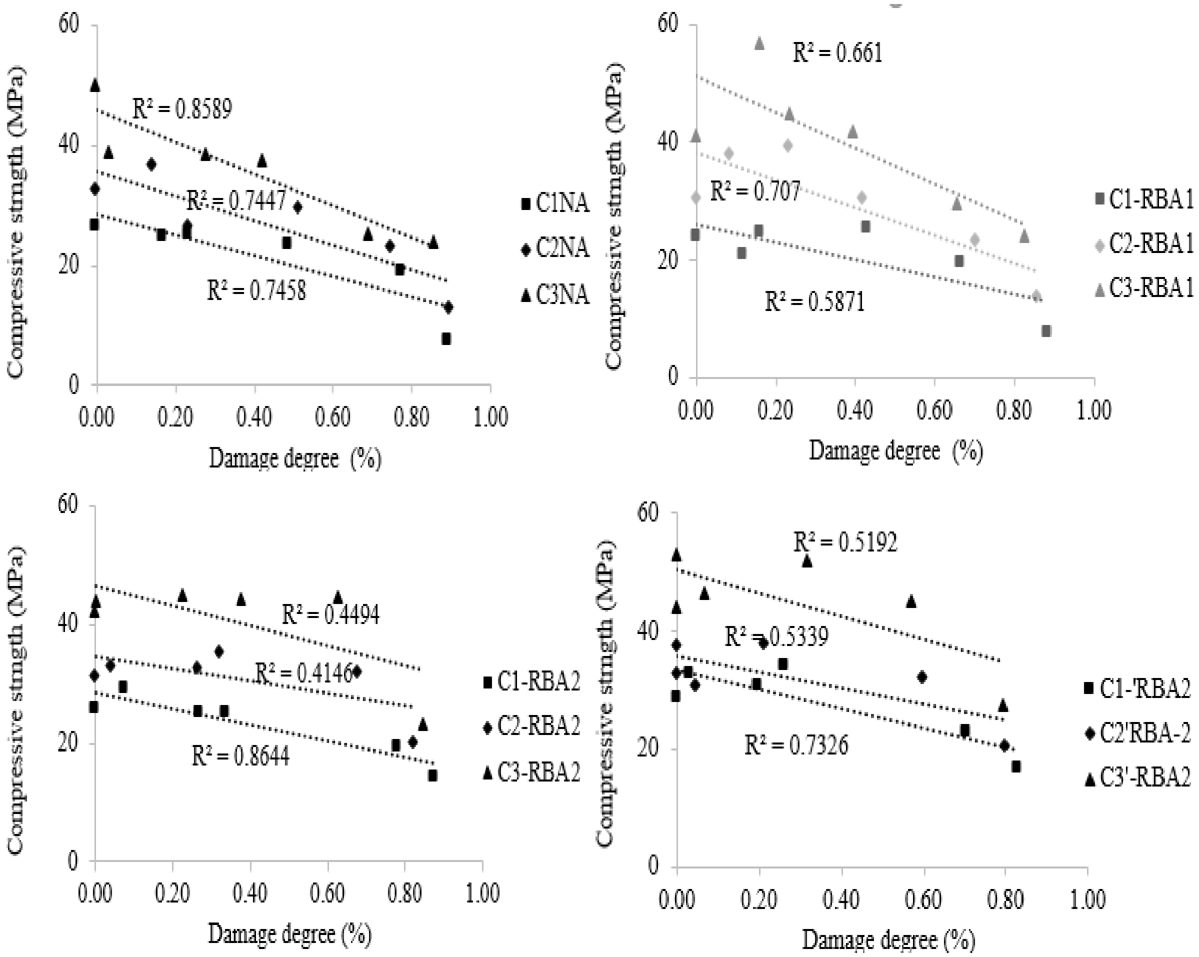


Figure IV.15. Evolution of compressive strength strength as function of damage degree.

IV.5.3 Relationship between modulus of elasticity and porosity

The correlations between modulus of elasticity versus porosity for all concretes are plotted in figure IV.16. There is a very good power relationship between modulus of elasticity versus porosity, the R^2 value is higher than 0.8. We conclude that the more the concrete is less porous the more it is the modulus of elasticity. That augmentation of porosity causes the disintegration of the microstructure of aggregates and cement paste resulting from deterioration of the physical and chemical properties of concrete subjected to high temperature.

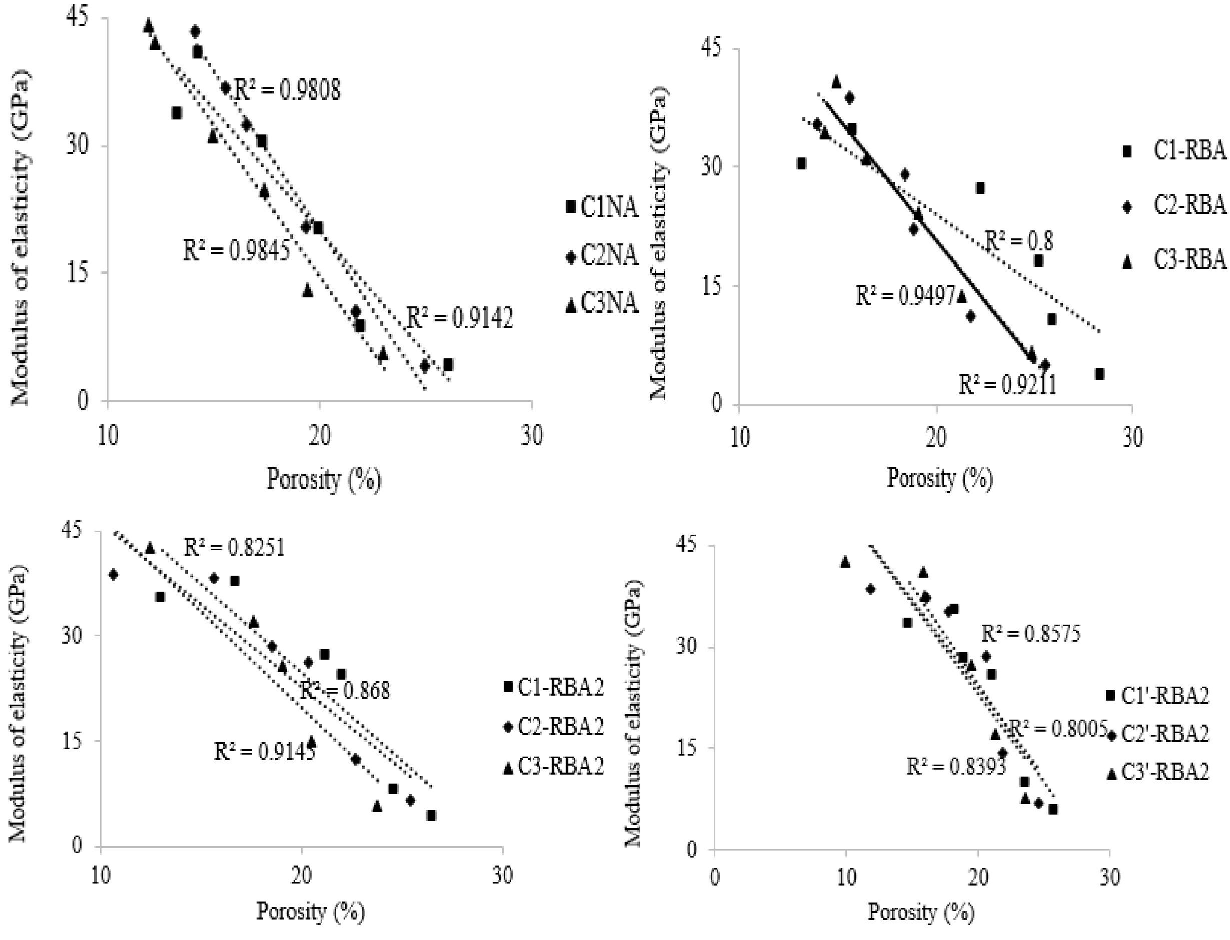


Figure IV.16. Evolution of modulus of elasticity as function of porosity.

IV.5.4 Relationship between modulus of elasticity and density

Figure IV.17 shows the relationship between modulus of elasticity and density of heated and cooled concretes. As it is observed in figure IV.17, the correlation coefficient (R^2), modulus of elasticity, and density of the various mixed designs range between 0.48 and 0.93 at temperatures from 20°C to 800°C. From the figure IV.17, it is noted that the diminution of density goes with the loss of modulus of elasticity. The correlation between those two parameters seems to evaluate by inverse way.

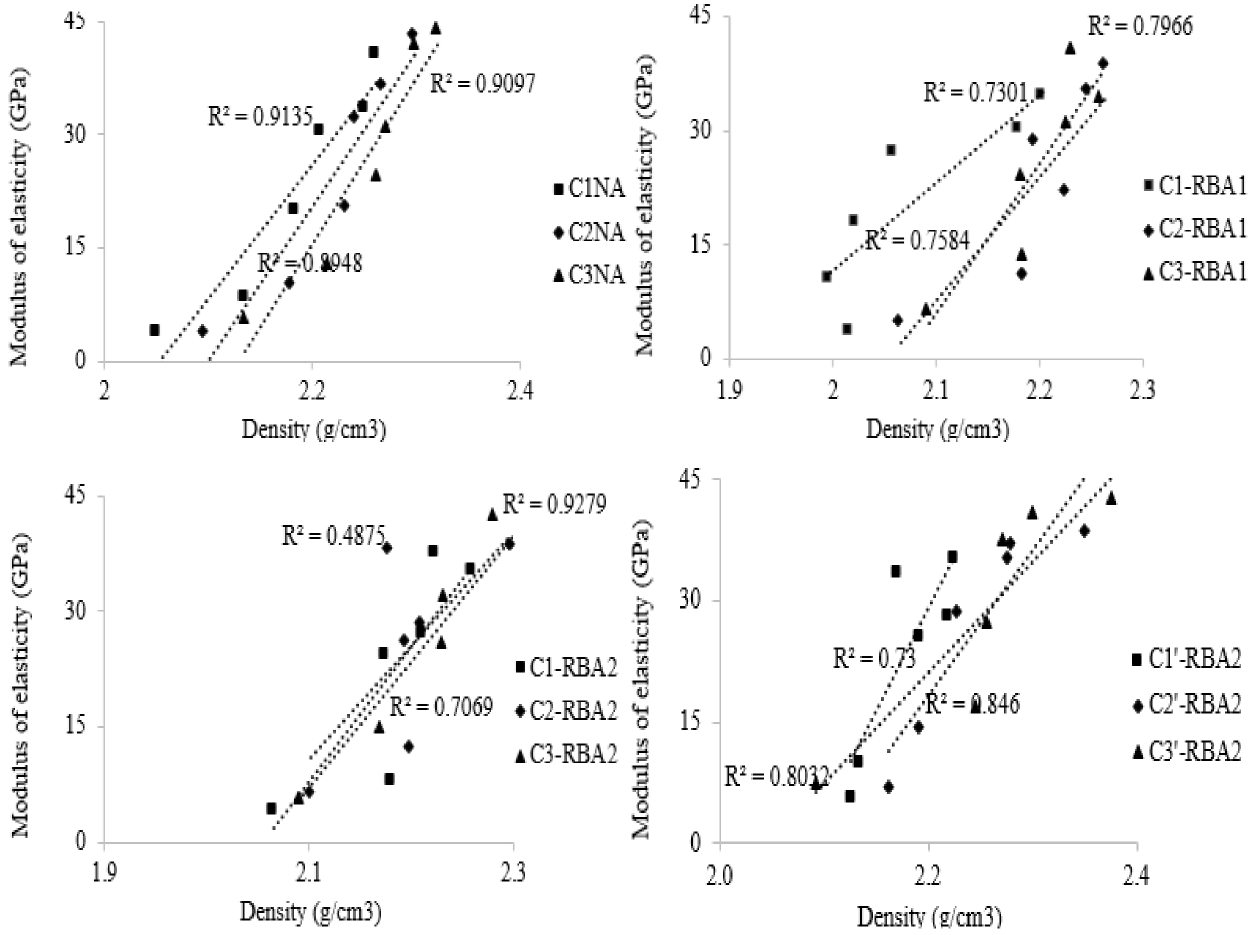


Figure IV.17. Evolution of modulus of elasticity as function of density.

IV.5.5 Relation between damage degree and porosity

The relationship and correlation between the damage degree and porosity were investigated at various temperatures (figure IV.18). It can be observed in figure IV.18 that the temperature increase causes an R^2 increase in hence a more appropriate correlation between the damage degree and samples’ porosities. Based on figure IV.18, it can be concluded that porosity is amongst the factors influencing the damage degree, especially at higher temperatures. Indicating that the augmentation of porosity generates a crack in the concrete and increases with temperature due to the decomposition of some hydration products.

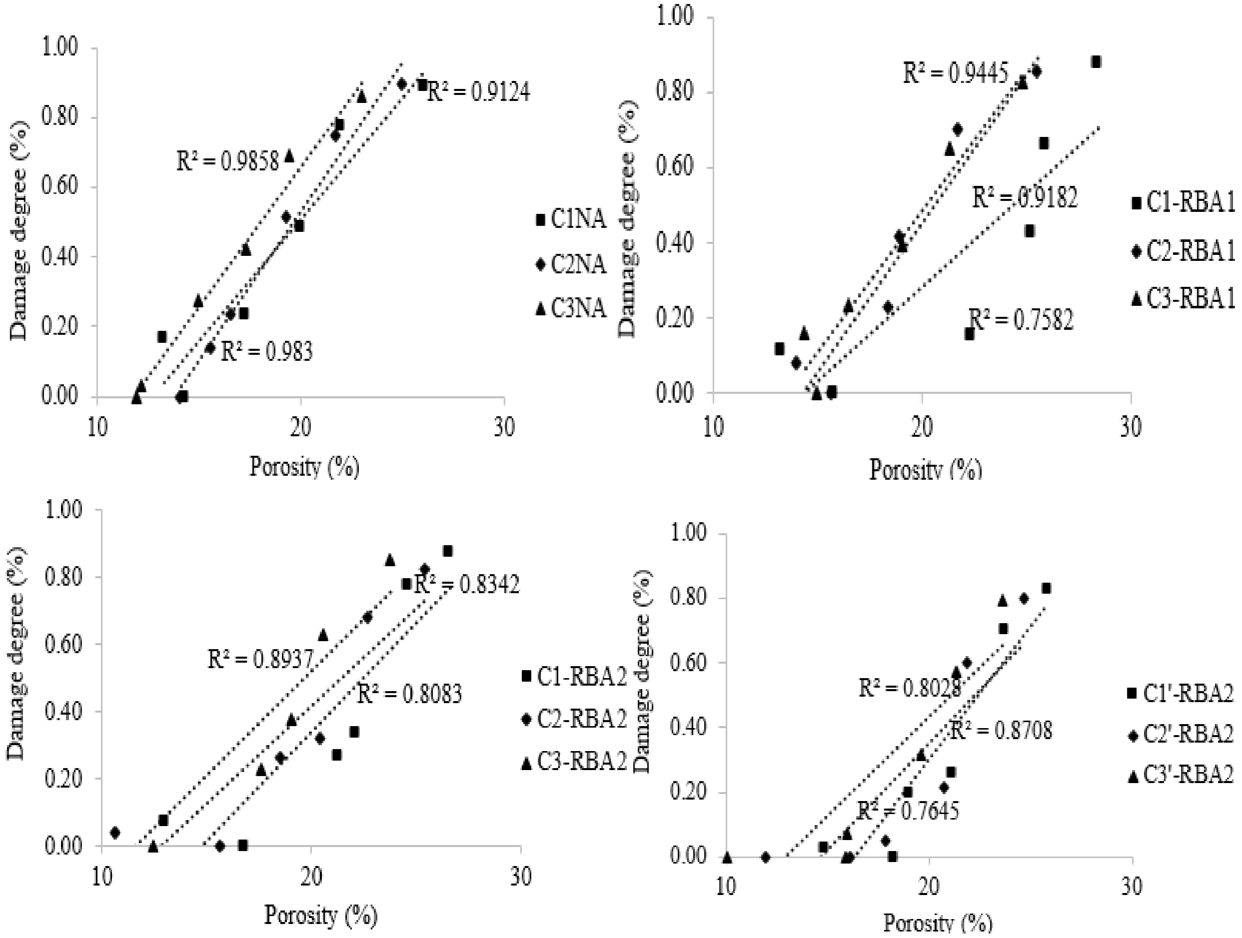


Figure IV.18. Evolution of damage degree as function of porosity.

IV.5.6 Relation between UPV and mass loss

Figure IV.19 shows the relationship between the UPV and mass loss of each concrete specimen. Based on the results, the correlation coefficient (R^2) of the various mixed designs range between 0.71 and 0.93 at temperatures from 20°C to 800°C. From the figure IV.19, it is noted that the UPV of different concretes continuously decreases with increasing mass loss. This loss of mass corresponds to the removal of both capillary and gel water from the dehydration of the CSH gel. In addition, the destruction of CSH gel and decomposition of CH cause the formation of air voids and microcracks. Those physico-chemical transformations could be responsible of this situation.

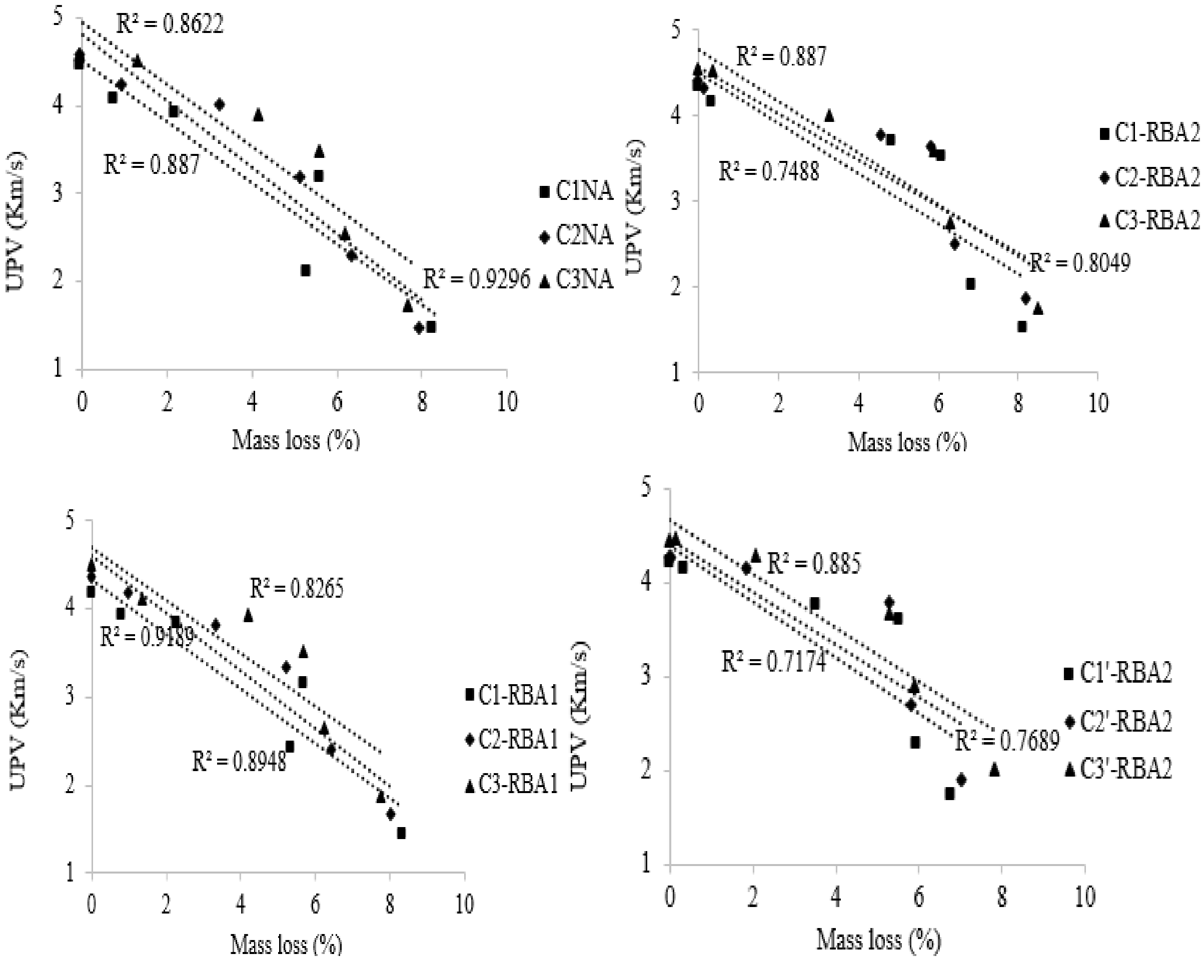


Figure IV.19. Evolution of UPV as function of mass loss.

IV.5.7 Relation between UPV and porosity

The evolution of the UPV as a function of the porosity is presented in figure IV.20. According to figure IV.20, the UPV of different concretes continuously decreases with increasing porosity. The porosity of concrete increases with the increase in temperature, this is the consequence of the damage to the concrete microstructure by the propagation of cracks and the decohesion between the cement paste and the aggregates. These factors reflect the decrease in the UPV.

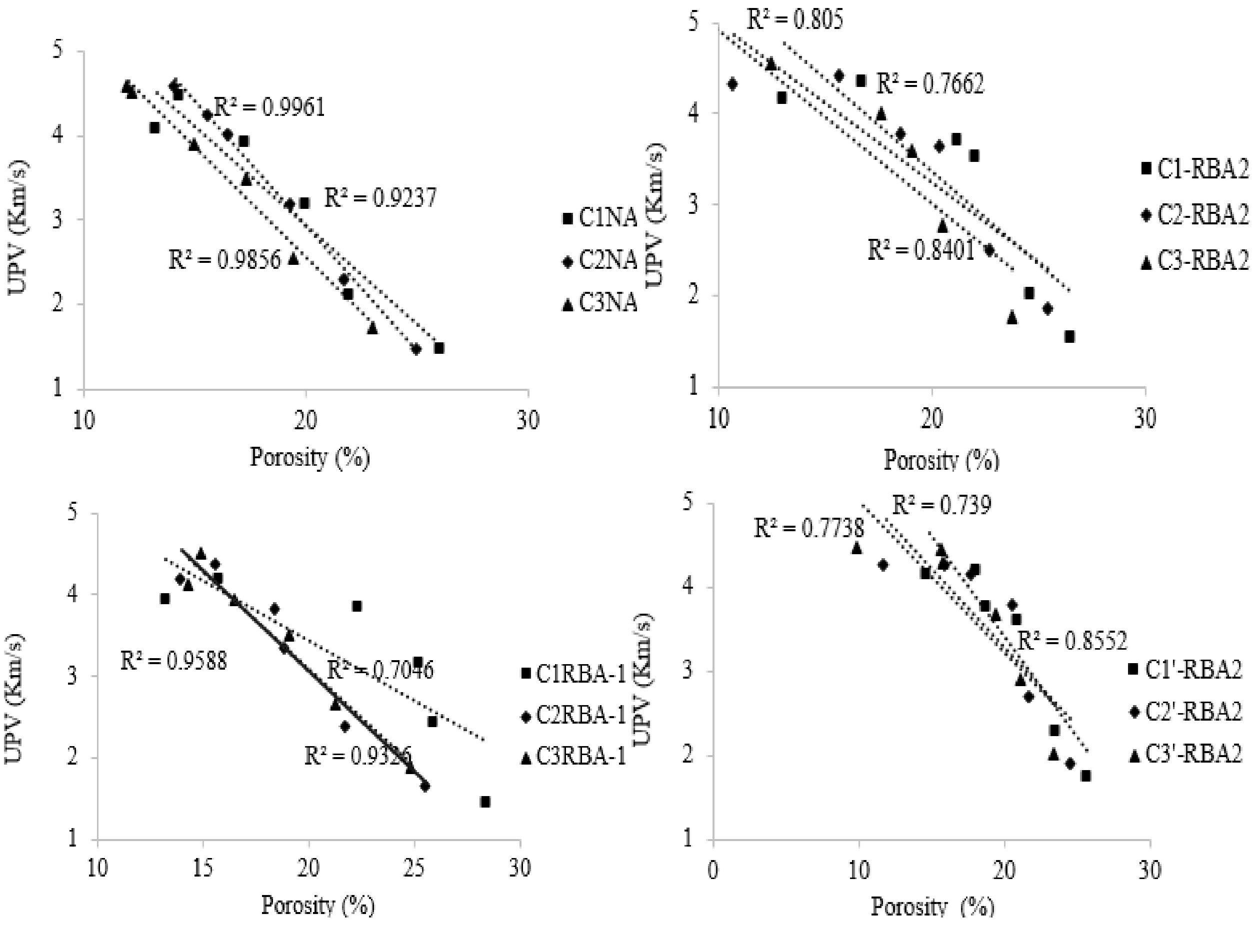


Figure IV.20. Evolution of UPV as function of mass loss.

IV.5.8 Relation between mass loss and porosity

Figure IV.21 demonstrates the relationship between the results of mass loss and porosity of the specimens under different temperatures. From figure IV.21, the results have a good consistency and parallel behavior. The increase in mass loss is accompanied by a great increase in porosity. This loss of mass corresponds to the departure of free water and chemically bound water. The dehydration of the CSH gel seems to leave a lot of capillary voids in the cement paste and causes the appearance of microcracks which determines the evolution of the porosity.

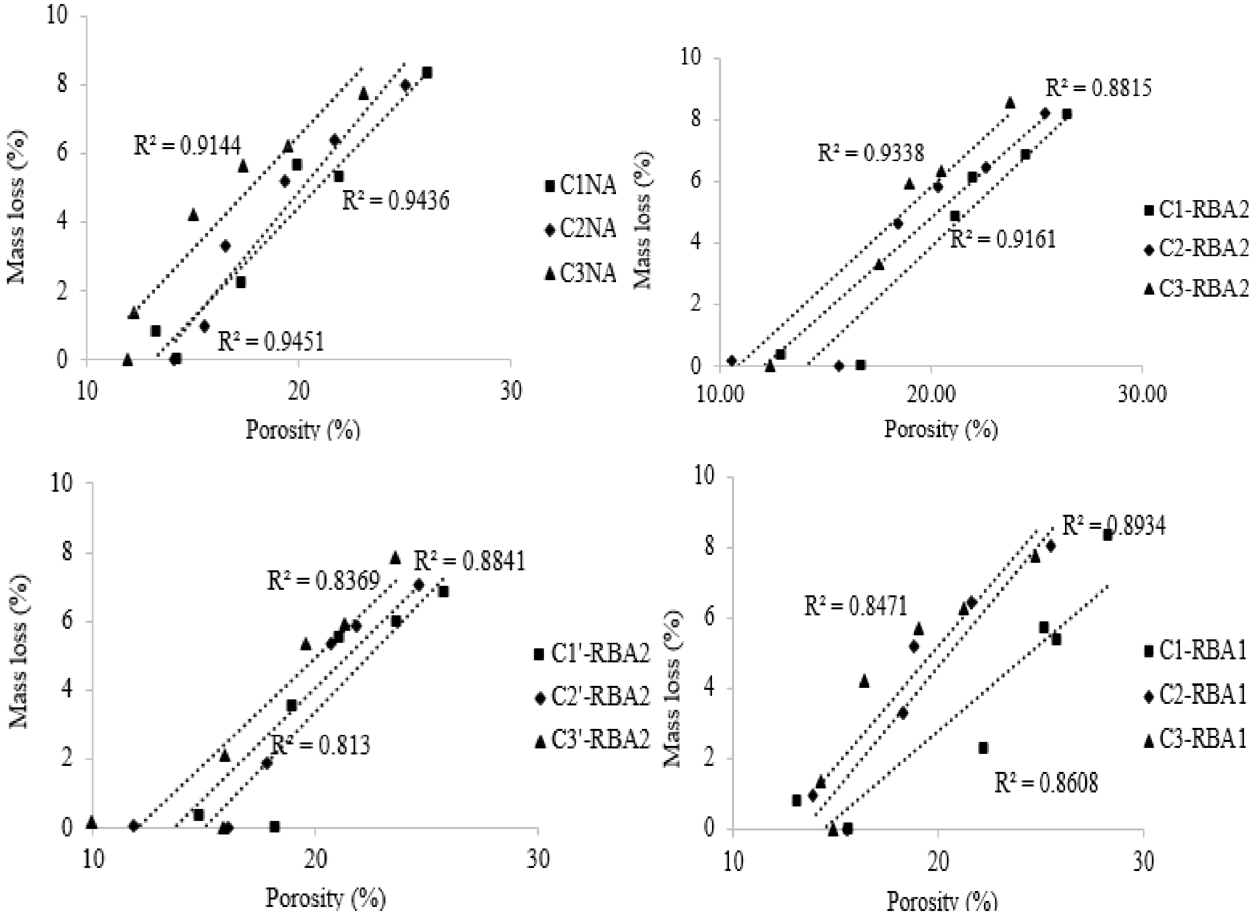


Figure IV.21. Evolution of mass loss as function of porosity.

IV.6 CONCLUSION

The first part of this chapter presents a comparative analysis of the conditions of concrete specimens of different concretes before and after heating. The color change of concrete specimens was observed after heating to 400°C. Furthermore, the crack widths increased with the increase in temperature it can be shown that the surface cracks concrete started to appear at around 400°C and continued to grow till the final rise in temperature up to 800°C. This is mainly due to the thermal expansion of aggregates and the shrinkage of the cement paste generate an internal deterioration in the concrete structure which facilitates the propagation of cracks.

The second part of this chapter aims to discuss the relationship between the physical and mechanical properties of the different concretes, presented in chapter IV. It makes it possible to highlight the experimental relationship between these properties. The results indicate a good relation between damage degree and porosity. The increase in porosity causes an increase in damage degree. It has also been observed that the augmentation of the porosity goes with a big loss of mass.

The results obtained in this test indicate that the density of concrete is influenced by the decrease in modulus of elasticity. Furthermore, the UPV of concrete are also adversely affected due to increase of porosity and mass loss.

GENERAL CONCLUSION

GENERAL CONCLUSION

The objective of this study was to deepen the knowledge of the behavior of concrete made with waste refractory brick as aggregates. Two types of recycled refractory brick (used and un-unused bricks) were considered. This experimental campaign also studied the effects of replacing 20% of NA by RBA for concrete subjected to elevated temperature. Based on the experimental results obtained in this work, the following conclusions can be drawn:

- Coarse RBA-1 and RBA-2 have a lower resistance to fragmentation, they present higher water absorption values, higher density and higher porosity compared to coarse NA. Furthermore, the fine RBA-2, by comparison with the fine NA, presents higher water absorption values, higher density. It is important to note that these characteristics are responsible for the results obtained of concrete containing this type of recycled aggregates.
- Coarse RBA should be saturated before use to avoid the reduction of mixing water.
- A change of surface color started to appears on fine NA after heating to 300°C. For coarse NA, their color changes at 400°C. Coarse RBA-1, coarse RBA-2 and fine RBA-2 remain intact up to 900°C.
- A different mineralogical nature of coarse NA, RBA-1 and RBA-2 lead to a different thermal behavior. When subjected to high temperature, coarse NA shows the highest values of porosity, mass loss and water absorption, while they show the lowest values of density. However, coarse RBA-1 and coarse RBA-2 have similar evolution of physical properties; they show no significant change in their properties after exposure to high temperatures.
- According to their mineralogical nature, fine NA and fine RBA-2 have a different behavior at high temperatures. Fine NA underwent a significant mass loss at 300°C; however, the mass loss for fine RBA-2 is negligible until heating temperature of 900°C.
- Macrocracks in coarse NA appeared at 700°C. However, no degradation is observed at macroscopic scale for coarse RBA-1 and coarse RBA-2.
- The porosity of concrete containing coarse RBA is the most affected property; it increased with the incorporation of RBA. The use of coarse RBA-1 had significant impact on porosity values,

their utilization resulted into a higher concrete porosity compared to concrete with coarse RBA-2.

- Concrete density decreased slowly with the increase of coarse RBA replacement ratio. Up to 50% of coarse RBA-1 substitution, the reduction of concrete density is below 10%, it achieves a maximum reduction of 18% for 100% substitution with coarse RBA-1. The use of coarse RBA-2 produces a less concrete density reduction (below 6% for 100% coarse RBA-2).
- The UPV does not seem to be affected by the coarse RBA replacement ratio, at least up to 40% replacement ratios; it presents an acceptable quality of concrete. The highest loss of UPV was observed when the coarse NA were completely replaced with coarse RBA.
- The increase of coarse RBA content has a negative effect on compressive strength. The increase of coarse RBA quantities means an increase of concrete porosity which leads to reduce the compressive strength. The decrease of compressive strength is more pronounced for concrete with coarse RBA-1 and substantially lower for concrete with coarse RBA-2.
- Increasing the replacement ratio of coarse RBA resulted in lower dynamic modulus of elasticity of concrete compared to conventional concrete. It can be noted that the effect of inclusion coarse RBA on the loss in modulus of elasticity is more obvious than that of compressive strength.
- Coarse RBA-1 impose the largest negative effect on the physical and mechanical properties of concrete, which might be attributed to the low density and high porosity of these aggregates.

It can be concluded that concrete made with coarse RBA shows a lower performance (when percentage above 50%) compared to conventional concrete. In general, concrete made with RBA-2 has better performance than concrete with made RBA-1. It would be acceptable to use RBA as coarse aggregates with a percentage of substitution of 20% to produce concrete with acceptable properties and good quality.

- The results indicate that the inclusion of 20% of coarse and fine RBA-2 in concrete gives it higher porosity than that of concrete prepared with 20% of coarse RBA-2.
- When using 20% of coarse and fine RBA-2, the decrease of density is negligible; this can be explained by the high density of fine RBA-2.
- In general, concretes made with 20% of coarse and fine RBA-2 has slightly lower UPV value than concretes made with 20% of coarse RBA-2.

- The dynamic modulus of elasticity of concrete is affected by using 20% of coarse and fine RBA-2 while concrete compressive strength does not. The compressive strength of concretes made 20% of coarse and fine RBA-2 is slightly higher than those of concretes made with natural aggregates and 20% of coarse RBA-2.
- It can be concluded that the inclusion of 20% coarse and fine RBA-2 in concrete exhibited lower or even similar performance compared to concrete prepared with coarse RBA-2.
- The use of coarse RBA has a significant impact on the porosity of the heated concrete. It is important to note that the high porosity of coarse RBA is the responsible for the results obtained of concrete after heating.
- The density of concrete slowly decreases with increasing temperature (less than 10% after heating to 800°C). The inclusion of coarse RBA-2 in concrete exhibited better density development compared to concretes prepared with coarse NA or coarse RBA-1.
- The pre-saturation of coarse RBA induces a slight increase of concrete mass loss. The pre-saturation of coarse RBA provides to concrete an additional quantity of water which escapes during the heat treatment.
- The coarse RBA has a positive effect on the variation of specimen’s volume. The use of coarse RBA in concrete decreases the concrete volume variations, especially at 800°C.
- Regardless of the type of aggregates used, UPV decreases with the increase of temperature. However, the decrease in UPV of concrete containing 20% of coarse RBA is the modest.
- Concrete specimens containing 20% of coarse RBA-2 exhibit good thermal stability compared to other concrete specimens, their damage degree was the lowest.
- For C2 and C3 concretes (low w/c ratio), the use 20% coarse of RBA reduces concrete cracking. The porosity of coarse RBA facilitates the evacuation of water when the temperature rises and consequently reduces the degradation of concrete.
- Coarse RBA improves the compressive strength and the dynamic modulus of elasticity of concrete after exposure to high temperatures. Concrete made with 20% of coarse RBA-2 and with 20% of coarse and fine RBA-2 have the best compressive strength, it was able to retain more than 60% of initial compressive strength after heating to 800°C.
- However, concretes prepared with 20% of coarse RBA-2 and with 20% of coarse and fine RBA-2 have the best compressive strength; they were able to retain more than 60% of initial compressive strength.

Mechanical and physical properties obtained in this research show that the inclusion of either coarse RBA-1 or coarse RBA-2 in concrete exhibited similar or even better performance compared to concrete prepared with coarse NA. In general, the use of coarse RBA-2 improves the behavior of concrete after exposure to high temperature.

- The porosity of concretes made with 20% of coarse and fine RBA-2 is higher than that of concretes with 20% of coarse and fine RBA-2 in the temperature range of 20–800°C. The use of fine RBA-2 lead to increased porosity of concrete.
- The concretes made with 20% coarse and fine RBA-2 exhibited higher density between 150°C and 800°C than their counterpart control concrete containing NA and 20% of coarse RBA-2.
- Concretes made with 20% coarse and fine RBA-2 have the lowest weight loss according to the results obtained on conventional concretes and concretes made with 20% of coarse RBA-2.
- A lowest increase in specimen's volume was recorded for concretes made with 20% of coarse and fine RBA-2, especially at 800°C, the fine RBA-2 has a positive effect on the variation of specimen's volume.
- By comparing with the existing data, it was found that concretes made with 20% of coarse and fine RBA-2 is good in terms of its UPV value compared with conventional concretes and concretes made with 20% of coarse RBA-2.
- The damage degree of concrete increases with increasing temperature. However, the increase in damage degree of concrete containing 20% of coarse and fine RBA-2 is the lowest compared to other concrete specimens (conventional concretes and concretes made with 20% of coarse RBA-2).
- Concretes made with 20% of coarse and fine RBA-2 had the highest compressive strength and dynamic elastic modulus at all test temperatures in comparison to conventional concretes.

Based on our experimental research, the inclusion of 20% of coarse and fine RBA-2 in concrete exhibited similar or even better performance compared to concrete prepared with 20% of coarse RBA-2.

- The type of aggregates (natural and recycled) has an effect on both the pristine values of mechanical and physical properties as well as on those tested after heating to successive temperature levels.

- Visual and microscopic assessments of CNA, CRBA-1, CRBA-2 and C'RBA-2 exhibit less physical deterioration and microstructural damage in CRBA-1, CRBA-2 and C'RBA-2 compared to CNA, especially towards higher temperatures.
- After thermal exposure, extensive cracking produced along the aggregate–cement paste interfaces dispersed in CAN specimens during the compressive loading, but the CRBA-1, CRBA-2 and C'RBA-2 themselves generally remained integrated.
- No spalling was found for all specimens (conventional concretes, concretes made with 20% of coarse RBA-1 and concretes made with 20% of coarse and fine RBA-2) during thermal exposure.
- For visual observations of concrete samples subjected to elevated temperatures, it was noticed that the surface cracks became visible when the temperature reached 400°C. The cracks were very pronounced at 600°C and increased extremely when the temperature increased to 800°C.
- The relationship and correlation between the different experiments' results by using linear regression indicate the good correlation and the acceptable value of the R^2 coefficient.

It can be concluded that the use of RBA as aggregate help to remain the concrete properties after heating.

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