

**CONCRETE INCORPORATING CONSTRUCTION & DEMOLITION
AND PLASTIC WASTES: DURABILITY AND APPLICATIONS IN
CONCRETE FILLED STEEL TUBES**

BY

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In

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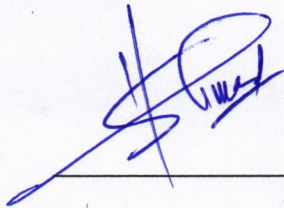
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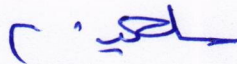
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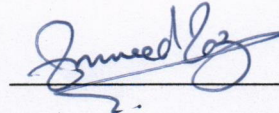


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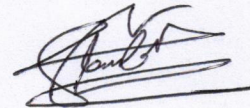


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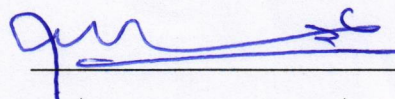
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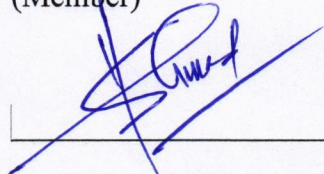
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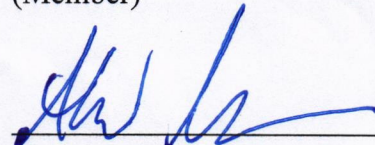
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2024

[Dedicated to my parents and beloved sisters]

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LIST OF ABBREVIATIONS

HDPE	:	High-Density Polyethylene
C&D	:	Construction and Demolition
NA	:	Natural Aggregate
FA	:	Fine Aggregate
SF	:	Silica fume
CA	:	Coarse Aggregate
RCA	:	Recycled Concrete Aggregate
RAC	:	Recycled Aggregate Concrete
PW	:	Plastic Waste
SCM	:	Supplementary Cementitious Material
SEM	:	Scanning Electron Microscopy
CFSTs	:	Concrete-Filled Steel Tubes
PWC	:	Plastic Wastes Concrete
HRAC	:	Hybrid Recycled Aggregate Concrete
RA	:	Recycled Aggregate
NAC	:	Natural Aggregate Concrete
SI	:	Strength Index
UPV	:	Ultrasonic pulse velocity

]

ABSTRACT

Full Name : [ALMOTASEEMBILLAH ALTIGANI ABDULHAFIZ AHMED]

Thesis Title : [CONCRETE INCORPORATING CONSTRUCTION & DEMOLITION AND PLASTIC WASTES: DURABILITY AND APPLICATIONS IN CONCRETE FILLED STEEL TUBES]

Major Field : [CIVIL ENGINEERING]

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[This study investigates the development of hybrid recycled aggregate concrete (HRAC) incorporating recycled concrete aggregates (RCA) and plastic waste (PW), specifically high-density polyethylene (HDPE), as fine aggregates. The HRAC was produced by replacing 60% of natural coarse aggregates volume with RCA and part of the fine aggregates volume was replaced by HDPE granules at replacement ratios of 0%, 15%, 30%, and 45%. In addition, (SF) was used as a supplementary cementitious material, replacing 5% of the cement weight in half of mixes. To evaluate the performance, eight concrete mixes were prepared and tested for their fresh, mechanical, and durability properties. The fresh and mechanical properties were investigated to serve as a reference for comparison and discussion of the durability and structural applications. The durability properties were assessed by measuring density, water absorption, porosity, thermal conductivity, chloride migration, electrical conductivity, electrical resistivity, sound absorption, drying shrinkage, and fire resistance. The results of the mechanical properties showed a decreased in the mechanical properties with the increase of PW content with minor role for the SF. However, the developed mixes have compressive strength of more than 27.0 MPa, an acceptable structural concrete. The durability study indicates varying impacts on the properties, particularly highlighting the behavior of SF in this combination. For examples, using PW granules as fine aggregates could not be feasible at 30% replacement or more as more plastic content tend to melt at temperature of 250°C and more.

To further explore potential structural applications, the concrete mixes were utilized in concrete-filled steel tubes (CFSTs), and their axial load capacity and failure modes were analyzed. The analysis showed that the axial load capacity decreased as the PW content increased and the deformed shape for case of high PW content and no SF has been severely impacted. Furthermore, the study assessed sustainability aspects, such as carbon footprint, embodied energy, and cost. Despite some performance limitations, incorporating RCA and PW in concrete offers a sustainable approach to waste management, contributing to circular economy goals. Nevertheless, challenges such as fresh consistency issues, low elastic modulus, and thermal degradation of PW concrete were identified. To address these challenges, the study proposed potential solutions and recommendations for enhancing the practical application of HRAC in construction.]

ملخص الرسالة

الاسم الكامل: المعتمد بالله التجاني عبدالحافظ احمد

عنوان الرسالة: الخرسانة المحتوية على نفايات البناء والهدم والبلاستيك: استكشاف خصائص الديمومة والتعرف على إمكانية استعمال الخرسانة في الأنابيب الفولاذية المملوءة بها.

التخصص: الهندسة المدنية

تاريخ الدرجة العلمية: أكتوبر-2024

تهدف هذه الدراسة إلى تطوير خرسانة هجينة من الركام المعاد تدويره (HRAC) التي تحتوي على ركام الخرسانة المعاد تدويره (RCA) والنفايات البلاستيكية (PW)، تحديداً البولي إيثيلين عالي الكثافة (HDPE)، كركام ناعم. تم إنتاج HRAC عن طريق استبدال 60% من الركام الخشن الطبيعي بـ RCA مع تغيير نسب استبدال HDPE للركام الناعم إلى 0% و 15% و 30% و 45%. بالإضافة إلى ذلك، تم استخدام غبار السيليكا (SF) كمادة إسمنتية مكملة، لتحل محل 5% من وزن الأسمنت في نصف الخلطات. لتقييم الأداء، تم إعداد واختبار ثمانية خلطات خرسانية من حيث الخصائص الطازجة، الميكانيكية، والمتانة. في البداية، تم تقييم قابلية تشغيل الخرسانة الطازجة باستخدام اختبار الهبوط. ثم تم تقييم الخصائص الميكانيكية من خلال اختبارات مقاومة الضغط، مقاومة الشد المنقسمة، مقاومة الانحناء، معامل المرونة، معامل المرونة الديناميكي، وسرعة النبضات فوق الصوتية (UPV) بالنسبة للمتانة، تم تقييم الخرسانة من خلال قياس الكثافة، امتصاص الماء، المسامية، الموصلية الحرارية، هجرة الكلوريد، التوصيلية الكهربائية، المقاومة الكهربائية، امتصاص الصوت، الانكماش الجاف، ومقاومة الحريق. أظهرت نتائج الخصائص الميكانيكية أن مقاومة الضغط، مقاومة الشد المنقسمة، مقاومة الانحناء، و UPV تظهر اتجاهات متشابهة نحو قيم أقل مع زيادة محتوى HDPE. كما أنها تشترك في اتجاه التحسن مع إضافة غبار السيليكا حتى نسبة استبدال البلاستيك 15%. ومع ذلك، يُلاحظ اتجاه معاكس عند نسب استبدال البلاستيك 30% و 45%، حيث يؤدي إضافة غبار السيليكا إلى انخفاض في الأداء. بشكل عام، تظهر الخرسانة المطورة لجميع الخلطات مقاومة ضغط تفوق 27 ميجا باسكال، مما يعتبر مقبولاً لخرسانة إنشائية. تشير نتائج دراسة المتانة إلى تأثيرات متباينة على الخصائص، مع تسليط الضوء بشكل خاص على سلوك غبار السيليكا في هذه التركيبة. على سبيل المثال، قد لا يكون استخدام الحبيبات البلاستيكية كركام ناعم ممكناً عند استبدالها بنسبة 30% أو أكثر حيث تميل إلى الذوبان عند درجات حرارة 250 درجة مئوية وأكثر. لاستكشاف المزيد من التطبيقات الهيكلية المحتملة، تم استخدام الخلطات الخرسانية في أنابيب فولاذية مملوءة بالخرسانة (CFSTs)، وتم تحليل قدرتها على التحمل المحوري وأنماط الفشل. أظهر التحليل أن قدرة التحمل المحوري تقل مع زيادة محتوى HDPE، وأن الشكل المتشوه في حالة المحتوى العالي من النفايات البلاستيكية وعدم وجود SF قد تأثر بشكل كبير. بالإضافة إلى ذلك، قيمت الدراسة جوانب الاستدامة مثل البصمة الكربونية، والطاقة المتجددة، والتكلفة، مما أبرز الفوائد البيئية والتحديات المرتبطة باستخدام HRAC. على الرغم من بعض القيود في الأداء، يمثل دمج RCA و PW في الخرسانة نهجاً مستداماً لإدارة النفايات، مما يساهم في تحقيق أهداف الاقتصاد الدائري. ومع ذلك، تم تحديد تحديات مثل مشكلات تماسك الخلطات الطازجة، انخفاض معامل المرونة، والتحلل الحراري للخرسانة البلاستيكية. وللتغلب على هذه التحديات، اقترحت الدراسة حلولاً محتملة وتوصيات لتحسين التطبيق العملي لـ HRAC في مجال البناء.

CHAPTER 1

INTRODUCTION

1.1 Background

The accumulation of plastic waste gives rise to concerns about environmental and health implications. In 2010, a total of 192 coastal countries produced 275 million metric tons of PW, with a percentage ranging from 1.7% to 4.6% being deposited into the ocean. This is a potential adverse impact on both the ecological ecosystem and human well-being [1]. A major proportion of PW s is either burned, disposed of improperly, or put in landfills, posing significant environmental concerns due to its slow degradation. This can lead to various problems such as blockages in sewer lines, drainages, waterways, filling up landfills, and health issues. To address the issue of PW accumulation, it is important to increase the amount of PW that is reused in various product streams including concrete, the second mostly consumed material worldwide. The incorporation of PWs within the construction sector is widely perceived as a trend towards sustainable development [2]. The majority of plastics exhibit notable non-degradable properties and are effective in thermal and electrical insulation. PWs can therefore be recycled and effectively used to produce concrete of an acceptable quality [3–8] and thus contributing to the creation of a circular economy [9].

On the other hand, (C&D) waste are extensively recycled and employed in the production of RAC [10,11]. These wastes are primarily used as CAs with evident of using them as FAs as well. The produced concrete in this case is generally found to be inferior to that of natural aggregates concrete (NAC) with some improvement after adding some mineral admixtures.

The aim of this study is to conduct a comprehensive investigation of both durability and mechanical properties and their application in (CFSTs)

1.2 Need for the research

There are different research studies on the literature on adopting either plastic or C&D wastes to produce green concrete. However, combination of both plastic and C&D wastes in the same concrete has yet not been identified specially its mechanical properties, durability properties and structural applications. The impact of their combination and possible enhancements of the produced concrete will be investigated. Success in producing a concrete of an acceptable quality and identifying some revenues of its application could lead to more wastes utilization in the construction sector as it contains two types of wastes.

1.3 Objectives

The available literature proved that there is no study on the durability properties of concrete containing both recycled concrete as CAs and proportion of PWs as FAs. Thus, the primary objective of this study is to explore the durability and structural application of a (HRAC) containing RCA as CAs and PW s as FAs. The specific objectives are to:

- 1- Develop concrete incorporating a selected percentage of recycled concrete as CAs and different percentages of PW s as FAs. This will be prepared with and without selected percentage of SF as a replacement of cement.
- 2- Investigate fresh and mechanical properties of concrete, including compressive strength, flexural strength (modulus of rupture), split tensile strength, elastic modulus, and ultrasonic pulse velocity (UPV) to form the basis for discussing the durability and structural behavior.
- 3- Investigate different durability properties of the developed concrete through comprehensive testing program. The tests conducted in this research includes: compressive strength under elevated temperature, density, water absorption, porosity, drying shrinkage, chloride migration test, electrical conductivity, electrical resistivity, thermal conductivity and sound absorption.
- 4- Investigate the applicability of the developed concrete in (CFST) and the results will be compared with control specimens.
- 5- Highlight the challenges and potential solutions for incorporating PW into concrete.
- 6- Provide recommendations for future utilization of the developed concrete as well as future studies needed to adopt the produced concrete in construction sector.

1.4 Thesis organization

This thesis is organized into seven chapters, as outlined below:

Chapter 1: Provides the background, the need for this research, and outlines the objectives of the study.

Chapter 2: Presents a comprehensive literature review, summarizing previous studies relevant to the subject of this research.

Chapter 3: Examines the workability and strength characteristics of HRAC, which includes RCA and PWs

Chapter 4: Explores the durability properties of concrete that incorporates RCA and PWs

Chapter 5: Examines the use of PWs and RCA in CFSTs.

Chapter 6: Investigates the challenges and potential solutions for concrete incorporating PW.

Chapter 7: Concludes the research with key findings, conclusions, and recommendations.

]

[CHAPTER 2]

[LITERATURE REVIEW

2.1 Background

Since the 1920s, plastic materials have become increasingly integral to daily life, from small electronic components to shopping bags, highlighting their role in driving continuous advancements in technological and industrial sectors [12,13]. Consequently, the amount of plastic waste has significantly increased, particularly over the past 50 years, due to its extensive use in producing medical devices, packaging, furniture, oil and water treatment systems, land soil conservation efforts, and electrical devices [14–17]. Reflecting this widespread use, global annual plastic production exceeded 400.3 million tons in 2022 [18]. Plastics, which belong to a group of materials known as polymers, are considered one of the most important materials in both industrial applications and everyday life [12,15,19–22]. However, as a non-biodegradable material, plastic poses a major concern for scientists, researchers, and environmentalists because it takes thousands of years to decompose, leading to significant environmental threats through its accumulation [3–8]. Plastics such as low-density polyethylene (LDPE), polypropylene (PP), polystyrene (PS), polyethylene terephthalate (PET), and (HDPE) contain hazardous and toxic substances that can pollute water, air, and soil [9,12,15,19,23].

The detrimental effects of plastic on the environment have led to harmful practices, such as disposing of PW in oceans, posing severe threats to aquatic life, biomes, and human

health [15,19]. Another common practice is burying plastic in landfills, which occupies large spaces and causes long-term pollution. Additionally, PW generates nanoparticles that enter the food chain, negatively impacting animal health [12]. burning plastic, while reducing waste volume, releases carbon dioxide and poisonous gases, further harming the environment [15,24]. Consequently, these methods for managing PW are neither environmentally sound nor easy to implement. In response, researchers are exploring effective ways to reduce the environmental impact of PW [19]. One approach involves waste-to-energy processes, such as burning plastic to generate fuel for heating limestone to produce clinker in cement manufacturing. Recycling is another viable solution, capable of mitigating the negative effects of PW. However, currently, only 6% of PW is recycled [12]. Challenges such as poor waste segregation systems, low waste collection rates, and inadequate waste management policies reduce recycling efficiency [25,26]. Nonetheless, researchers emphasize the necessity of increasing plastic recycling efforts. Plastics have two key advantages: longevity and endurance. These properties require less maintenance than other materials and allow for easy fabrication. As a result, recycled plastics are suitable for use in construction, providing an ideal solution to environmental problems [12,27]. Incorporating recycled plastics into concrete and utilizing (C&D) waste can prevent these components from ending up in landfills, thereby reducing their negative environmental impact [28].

2.2 Mechanical properties

The use of HDPE as a replacement for FA or CA in concrete has been explored, as well as its application in the form of fibers. Several studies have investigated the effects of HDPE on the mechanical properties of concrete such as Rahman *et al.* [29] used HDPE with a content of 10%, 15%, 20%, and 25% of the total volume of the sample. They reported reduced compressive strength and density by 58% to 80% and 68%, respectively. Abu-Saleem *et al.* [30] used HDPE with a content of 10%, 20%, and 30% of the CA volume and their results indicated that the compressive, flexural and split tensile strength are all reduced by a maximum of 40%, 30% and 36.2%, respectively. They also reported an increase of 5.2 to 10.6% in the water absorption. Jackson *et al.* [31] replaced the CAs with HPDE particles at replacement ratios of 2%, 4%, 6% and 8% (by weight) and a reduction of up to 52%, 50% and 35 were obtained for strength characteristics such as compression, bending, and tension splitting respectively. Maulana and Kurniawan [32] replaced 20%, 30% and 40% of the weight of CAs by HDPE particles and reported a reduction in compressive strength ranging from 40% to 52%. In a similar approach, the work of Jonbi *et al.* [33], Karthik *et al.* [34], Lopez *et al.* [35] and Rahim *et al.* [36] have all replaced the CAs by HDPE and reported a significant reduction in the mechanical properties. Thus, the use of PW to replace the CAs generally leads to a considerable reduction in the mechanical properties of the concrete. An improvement in the mechanical properties due to the use of PW has only been reported when using small content of HDPE aggregates, such as the work of Selvi *et al.* [37], in which they reported a slight increase in the compressive, flexural, and split tensile strength at 1% HDPE content. As a novel attempt to improve the

quality of concrete incorporating PW, Nasir et al. [38] adopted sand coated HDPE particles at a replacement ratio of 10%, 20% and 30% by volume of the CAs. However, the outcomes also show a reduction of up to 36%, 35% and 44% in the compressive strength, flexural strength, and split tensile strength, in that respective order. There are also studies that adopted HDPE to replace the FAs. Punitha et al. [39] replaced the FAs with HDPE at replacement ratio of 5%, 10%, 15%, 20%, 25% and 30% (by weight) and the mechanical properties are reduced by up to 44 %. Mustafa et al. [40] replaced the FAs volume by HDPE FAs at replacement ratios of 5%, 10%, and 20%. They found a reduction of up to 24%, 16% and 11.31% in the compressive strength, elastic modulus, and density, respectively. Aocharoen and Chotickai [41] also replaced the sand volume by 5 to 34 % HDPE particles and they obtained a reduction in compressive strength but, on the advantages side, they found that the drying shrinkage reduced by about 6.2 % to 14.8%. Studies related to the use of HDPE to replace both the fine and course aggregates in the same concrete can also be found in existing studies, including the works of Al-Osta et al. [42] and Santhi and Santhi [43]. Moreover, HDPE has also been used in the form of fiber to replace cement in small amounts, usually less than 10 %, such as the studies of Malagavelli and Rao [44], Nurdiana et al. [45] and Pešić et al. [46]. Unlike the replacement of the fine and CAs, the replacement of the cement by HDPE fiber has a positive impact on the properties of the concrete, but no significant role is expected in terms of reducing the PW accumulation as the volume of the cement in concrete is not significant as compared to the aggregates.

The use of (C&D) wastes in concrete, particularly as coarse or FAs, has gained significant attention from researchers as result of their role as the primary source of (RCA)[23,28]. It

is crucial to recognize that the performance of (RAC) can vary significantly based on the quality of the source concrete; RCA from high-quality concrete performs better than that from low-quality sources. For instance, Etxeberria and Álvarez [47] found that replacing coarse NA with RCA at ratios of 0%, 25%, 50%, and 100% did not significantly affect mechanical properties. Similarly, studies by Shah et al. [48] , Jain et al. [49], Sérifou et al. [50], Bui et al. [47], and Khartabil and Martini [51] investigated 100% replacement of CAs with RCA, reporting maximum reductions in compressive strength of approximately 16%, 19%, 29%, 36%, and 54%, respectively.

To mitigate the reduction in concrete properties caused by RCA or PW, researchers often turn to supplementary cementitious materials (SCMs), particularly SF. Numerous studies [52–55] have shown that replacing cement with SF in ratios of 10% to 30% can enhance concrete properties. For example, one study [56] reported a 20.68% increase in compressive strength with just 15% SF. However, other studies suggest that SCMs (SF), the properties related to mechanical aspects of concrete, which can sometimes be negatively influenced by SF[57–60]. Therefore, utilizing SF as a substitute for cement should be carefully considered and preceded by trial mixes to ensure its effectiveness before large-scale implementation.

2.3 Durability properties

Figure 1, which shows the percentage of the studies conducted each of the specified test. It can be clearly seen from the figure that the majority of the studies conducted compressive

strength and, to some extent, the other mechanical and physical properties, but obviously less concentration was given to the durability, thermal and sound insulation properties. However, it is important to highlight that many studies only assessed a limited number of durability tests. For example, Punitha et al. [39] and Alostta et al. [61] applied replacement ratios from 10% to 50%, discovering that the thermal properties decreased by 10% to 33% with HDPE as CA. Similarly, Santia et al. [43] examined HDPE replacement at 10%, 15%, and 20% ratios and found a significant increase in chloride diffusion. Abusaleem et al. [30] replaced the CA with PW at ratios of 10%, 20%, and 30% noting an increase in water absorption by 5.2% to 10.6%. Hannawi et al. [62] found that increasing the content of high-density polyethylene (HDPE) in concrete raises water absorption due to the formation of voids from evaporating water, with the maximum absorption noted at 30% HDPE content.

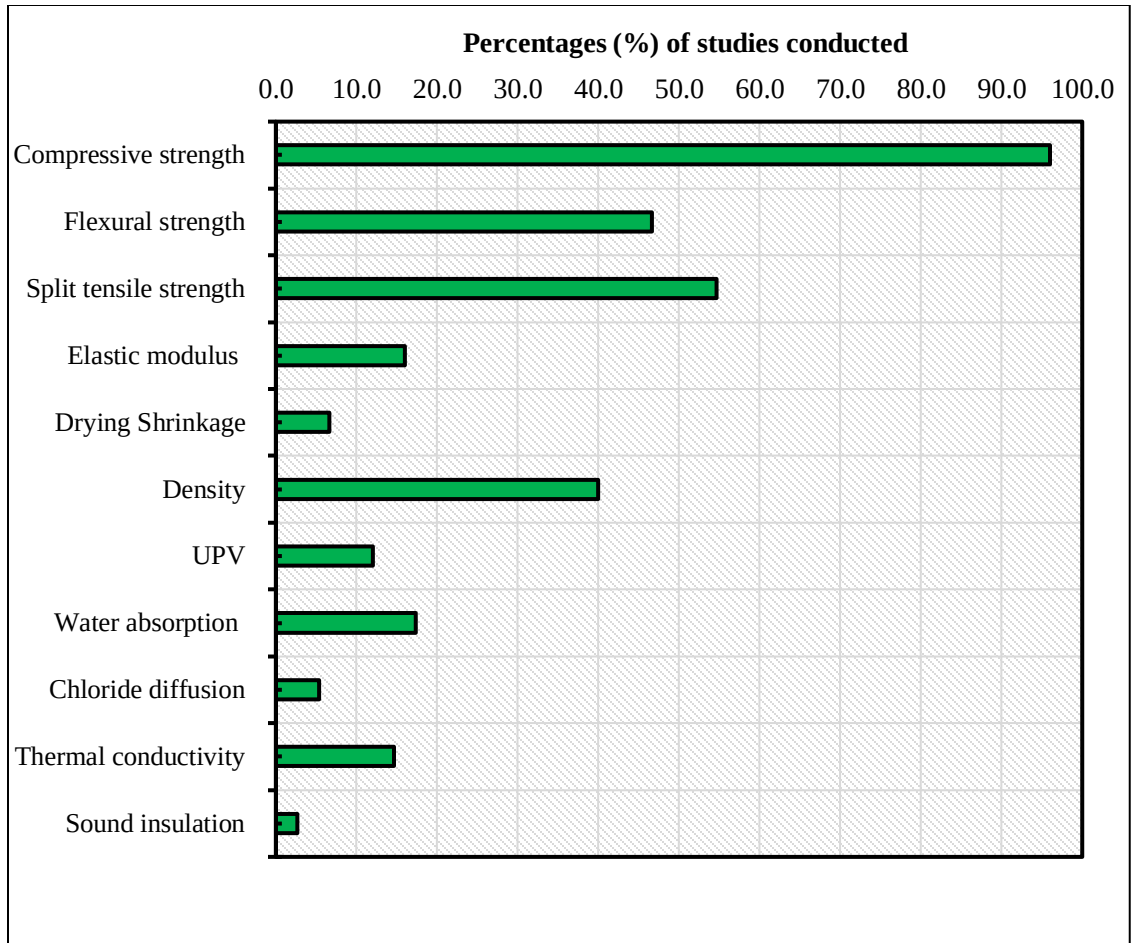


Figure 1: Percentages of studies conducted various tests

Durability properties of concrete incorporating other types of plastics can also be found in the literature. Kou et al. [63] reported a reduction in chloride migration in concrete containing granular coarse plastic aggregates. Ceran [64] demonstrated that incorporating PVC waste improved electrical resistivity, while Prasittisopin et al. [25] showed a notable reduction in noise, particularly at higher replacement ratios of PS. Similarly, Oancea et al. [65] observed a 28.5% increase in sound absorption with the use of (PET) granules, and Loong et al. [66] noted a 16% improvement with PET fibers, highlighting the importance of optimal material distribution. Manaf et al. [67] further emphasized that a balanced mix of HDPE and PS can enhance sound absorption up to 0.42 by improving air gaps and particle distribution. Choi

et al. [68] investigated the use of PET FAs as a replacement ranging from 25% to 75% and found an increase in drying shrinkage compared to (NAC). Ahmed et al. [69] explored the effects of (PP) as a FA substitution at levels of 8% to 40%, finding a reduction in thermal conductivity between 6.6% and 28.3%. Lerman et al. [70] explored using PP fibers as a partial replacement for cement (0.3% to 1%), which resulted in a decrease in water absorption from 4.7% to 3.87%. Hani et al. [71] found that replacing (PS) with 25% to 50% of FAs led to a 27% increase in water absorption. Jolaade et al. [72] reported that using PS to replace 5% to 30% of FA increased water absorption from 1.11% to 1.55%.

2.4 Concrete filled steel tubes

The growing focus on utilizing waste materials has drawn significant attention, particularly regarding the use of PW in construction to mitigate environmental issues like plastic accumulation. A promising solution involves incorporating PW into (CFST). In this composite system, the steel tube's high elastic modulus effectively offsets the negative impacts of waste in the concrete. CFST has become a widely adopted material in modern construction practices, where engineers aim to leverage the advantages of various materials[73]. Over the past few decades, composite steel-concrete structural systems like CFST have been increasingly employed, particularly in constructing high-rise buildings[74]. The concept of filled steel tubes is not new; its first practical application dates back to 1880, when it was used in the piers of the British Severn Railway Bridge[75]. The combination of concrete and steel in CFST significantly enhances the material properties, offering multiple benefits. For instance, CFST allows for a reduction in the

cross-section of reinforced concrete columns, minimizing the need for extensive formwork. Beyond column applications, CFST excels in axially loaded scenarios, providing superior fire resistance compared to steel tubes without concrete. Additionally, CFST's improved energy absorption capabilities contribute to its increased ductility[76]. Moreover, CFST is frequently employed in seismic and earthquake-resistant structures, Additionally, they are utilized in tall structures and bridge supports subjected to elevated strain rates due to traffic and train loads [77]. Its advantages extend to offering greater compressive strength and stiffness than conventional concrete, which helps to improve the steel tube's resistance to local buckling[78]. Additionally, CFST's excellent water and corrosion resistance enhances its durability and longevity. Its lightweight nature also simplifies transportation, installation, and construction, making CFST a multipurpose and practical choice for a wide range of construction applications[79].

Many research studies have examined the performance of columns made from CFST under axial loads by altering characteristics like the diameter of the steel, thickness, or tube length. Despite this extensive research, relatively few studies have focused on the impact of using waste materials in concrete and their subsequent effect on CFST performance. Early work in this area, particularly by Konno et al.[80] introduced the concept of replacing NA with (RAC in CFST) to enhance the mechanical properties of (RAC). Building on this concept, Chen et al.[81] investigated the bond behavior between RAC and steel tubes. Their findings indicated that bond strength increased when recycled concrete aggregates were used, particularly at replacement ratios up to 50%. Other researchers, including Yang and Ma [82], Xiao et al.[83], Shi et al.[84], Yang and Han[85], and Wang et al.[86], further explored the effects of varying RAC replacement ratios in steel tubes. These studies

utilized specimens with different dimensions and shapes, such as rectangular, square, and circular tubes. The results consistently showed that the steel tube's confinement improved the performance of RAC up to a certain replacement ratio. However, when the replacement ratio reached 100%, the axial force decreased, although the failure mode remained unaffected. Further research by Wang et al. [87] on recycled CA concrete in filled steel tubes confirmed that using RAs reduced the axial compression capacity of the specimens. Similarly, Krishnamurthy et al. [88] found that replacing natural CAs with RAs led to a decrease in the ultimate axial load capacity. In another study, by Van Cao [73] compared steel tubes filled with RAs to those filled with NA, revealing that specimens with RAs exhibited greater ductility. However, these specimens also showed early local buckling during failure and demonstrated an ultimate axial load capacity that was 30% lower than that of tubes filled with NAs.

Other research has explored the application of rubber as a waste replacement in concrete. For example, Cholampour et al. [89] replaced 18% of the concrete volume with rubber and found that the axial compression strength of the rubberized concrete increased, especially when confinement was applied, compared to normal concrete. Similarly, research by Abende et al. [90] explored the effects of replacing 0% to 30% of FAs with crumb rubber in concrete, using increments of 10%. They examined the bond strength between this modified concrete and steel tubes, finding that circular-shaped specimens exhibited greater strength than square ones. However, increasing the rubber content did not lead to any significant improvement in ductility. Another significant study by Farahn et al. [79] focused on employing PW to substitute for CA in CFSTs. The replacement ratios ranged from 2.5% to 5%. The study found that the ultimate axial load decreased as the plastic

content increased, and ductility was also reduced. Additionally, in circular columns, local buckling was observed near the mid-height at failure.

Based on the existing literature, the use of (RAC) and PW individually in (CFST) has been explored. However, the combined use of PW, (RCA), and (SF) in CFST has not been extensively studied.

CHAPTER 3

Fresh and mechanical properties of hybrid recycled

aggregates concrete incorporating recycled concrete

aggregates and plastic wastes

3.1 Background

This chapter is, oriented to explore the features of (HRAC) incorporating RCA to replace the NCA and PW in the form of HPDE particles to replace the FA. The RCA content is maintained at 60% by the natural CAs volume, and the HDPE content is selected to be 0%, 15%, 30% and 45% of the sand volume. This has been considered with and without (SF) as a partial replacement of cement at a 5% replacement ratio (by weight). The resulting eight mixes have been subjected to various tests, including slump, compressive, flexural and split tensile strength, elastic modulus, density, and ultrasonic pulse velocity UPV. The findings show that the slump rises as the PW content increases, which could be attributed to the hydrophobic property of plastic. The results showed that the compressive strength, split tensile strength, flexural strength, and UPV exhibit a similar trend in lower values with increased HDPE content. They also share the improvement trend of adding SFs up to a 15% plastic replacement. However, an adverse trend is observed at 30% and 45% plastic replacement, where the addition of SF leads to a decrease. Overall, the developed concrete of all mixes results in a compressive strength of more than 27 MPa, an acceptable structural

concrete. The developed HRAC allows more waste utilization in the construction sector with an affordable compromise on the mechanical properties.

3.2 Experimental investigation

The experimental program of this research involved a number of (8) mixes, with a constant replacement ratio of CA by RCA (60%). Half of the mixes from (M1 to M4) had no SF, and vice versa from (M5 to M8) had a fixed percentage of substitute cement by SF (5%). Each group (with and without SF) has varied amounts of PW to replace the natural FAs at volumetric replacement ratios of 0%,15%,30% and 45%.

A number of twelve (12) specimens were prepared from each mix. Three cubes (100 mm x 100 mm x 100 mm) were used for evaluation of the compressive strength, three cylinders (100 mm diameter, 200 mm length) were used to evaluate the Ultrasonic Pulse Velocity (UPV) and then split tensile strength, another three cylinders, of the same size, were used for testing the elastic modulus, and three of prisms (75 mm x 75mm x 285 mm) were used for testing the flexural strength. This results in a total of 96 specimens throughout this study

3.3 Materials

3.3.1 Cementitious material

All the concrete mixes were prepared using ordinary Portland cement (Type I), following ASTM C-150 standards [91], and sourced from the local market in the Eastern Saudi Arabia.

The SF was obtained from Al Rashed Cement Company. Table 1 displays the physical properties of all the materials

Table 1: Materials and their physical properties utilized in this research

Material	Specific gravity	Water absorption	Moisture content
Cement	3.15	-	-
SF	2.3	-	-
Superplasticizer	1.095	-	65
NCA	2.49	3.199	0.21
FA	2.626	0.5	0.14
RCA	2.33	5.31	0.86
HPDE	0.95	0	0

3.3.2 Fine and natural coarse aggregates

The natural FA and CA were sourced locally from Eastern Saudi Arabia. Multiple experiments, including sieve analysis, specific gravity, water absorption, and moisture content, were conducted in accordance with the respective ASTM standards [92–94]. The physical properties of both fine and CAs are presented in Table 1. This study utilized two scales of CA (0.5 inch & 3/8 inch), while the sand was sieved using a 4.75 mm sieve. RCA were obtained from the Saudi Investment Recycling Company (SIRC) [95] based in Riyadh, Saudi Arabia. RCA was used to replace natural CA at a 60% replacement ratio. The physical characteristics of the RCA employed in this study are outlined in Table 1

, and the sieve analysis results are shown in Figure 2Error! Reference source not found..

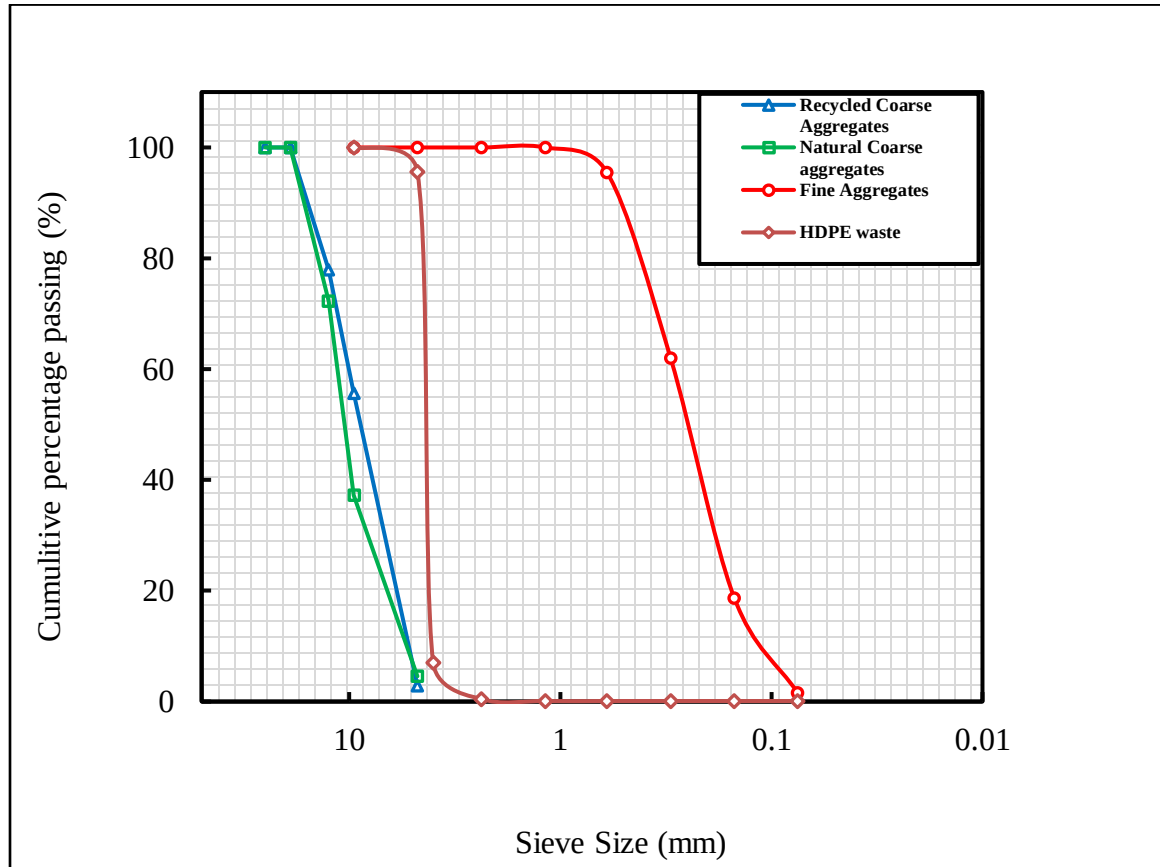


Figure 2: Distribution of particle sizes of FA, CA and (RCA)

3.3.3 Plastic waste (PW) aggregates

The HDPE used in this research, as shown in Figure 3, was obtained from a plastic recycling factory named Al Izdihar Al Awwal [96] in Dammam, Saudi Arabia. using recycled plastic was more costly than using non-recycled plastic during this research, as only this type was available at the time. Additionally, the lack of access to a shredding machine further limited the options for processing plastic waste, making it necessary to use the available recycled material despite the higher costs. The tiny plastic granules were not initially provided in the form shown in Figure 3; they underwent several preparation stages, including melting, casting, and cutting. The particle size distribution of the HDPE is illustrated in Figure 2, along with the FAs. The sieve analysis curve indicates that all the HDPE particles are within the upper limit of FAs, typically ranging from 4 to 4.5 mm.



Figure 3: Plastic waste (HDPE)

3.3.4 Water

Potable water was used for both mixing and curing in this study, in accordance with ASTM standard [97].

3.3.5 Superplasticizer

A superplasticizer was used to improve the consistency of the mix by enhancing its workability and overall strength by minimizing the w/c ratio. It was sourced from the local market and has a specific gravity of 1.095, as detailed in Table 1.

3.4 Mix Proportions

The concrete mixes were proportioned using the absolute volume method as outlined in the ACI 211 [98]. In this study, the control mix is defined as the mix without SF or HDPE FAs, to facilitate the comparison of results. Table 2 presents the mix proportions, considering four replacement ratios of HDPE content (0%, 15%, 30%, and 45%) and two replacement ratios of SF (0% and 5%) , In previous research, using the same plastic waste by Ahmed et al. [19] the authors used 10% silica fume to address the issue of plastic granules floating to the surface during compaction, which led to an uneven distribution in the concrete. This uneven distribution risked turning the concrete into a functionally graded material, complicating its structural behavior and analysis. However, while 10% silica fume improved uniformity, it also significantly reduced the workability of the mix. Therefore, in this study, the authors opted to reduce the silica fume content to 5%, aiming to maintain the benefits of improved plastic distribution while achieving better workability and ensuring sufficient durability. The RCA replacement ratio is consistently maintained at 60% of the natural CA volume in all mixes. The superplasticizer dosage is set at 0.4% of the cement content, and the water-to-cement ratio (w/c) is fixed at 0.45 for all mixes. For clarity, two naming conventions for these

mixes are shown in the table. The first column uses identifiers (M1 to M8), while the second column provides an alternative detailed name (M-PW%-SF%) that reflects the varying parameters. The letter M is followed by the replacement percentages of PW and SF, respectively, separated by dash signs within parentheses.

Table 2: Mix numbering and corresponding replacement ratios and quantities for each Mix

Key mix factors					Required quantities to produce ONE cubic meter of concrete							
Mix	M-PW %-SF%-	SF (%) of cement	PW (%) of FA	RCA (%) of NCA	Cement (kg)	SF (kg)	SP (kg)	NFA (kg)	PW (kg)	NCA (kg)	RCA (kg)	Water (Ltrs)
M1	M-00-00	0	0.0	60.0	390	0	1.56	881.4	0.0	352.8	498.4	222.1
M2	M-15-00	0	15.0	60.0	390	0	1.56	749.2	47.8	352.8	498.4	221.6
M3	M-30-00	0	30.0	60.0	390	0	1.56	617.0	95.5	352.8	498.4	221.1
M4	M-45-00	0	45.0	60.0	390	0	1.56	484.7	143.3	352.8	498.4	220.7
M5	M-00-05	5	0.0	60.0	370.5	19.5	1.56	881.4	0.0	352.8	498.4	222.1
M6	M-15-05	5	15.0	60.0	370.5	19.5	1.56	749.2	47.8	352.8	498.4	221.6
M7	M-30-05	5	30.0	60.0	370.5	19.5	1.56	617.0	95.5	352.8	498.4	221.1
M8	M-45-05	5	45.0	60.0	370.5	19.5	1.56	484.7	143.3	352.8	498.4	220.7

3.5 Methodology

To explore the slump and mechanical properties of (HRAC), the developed eight (8) mixes were subjected to the following test shown in Table 3, along with a quantity of needed specimens.

Table 3: List of tests in this study

#	Test	Standards	Shape of the molds	Dimensions of specimen	Quantity
1	Slump	ASTM C143[99]	Cone	-	Not applicable
2	Compressive strength	ASTM C39[100]	Cube	100 mm side length	3
3	Split tensile strength	ASTM C496[101]	Cylinder	100 mm Dia. x 200 mm length	3
4	Modulus of rupture	ASTM C78[102]	Prism	75 mm x 75 mm x 285 mm	3
5	Elastic modulus	ASTM C469[103]	Cylinder	100 mm Dia. x 200 mm length	3
6	UPV	ASTM C597[104]	Cylinder	Same as test no.3.	Same as test No.3.

3.5.1 Casting and curing

The experimental procedure started with preparing the casting place, cleaning molds and wiping them by a layer of oil to facilitate demolding after 24 hours. Standardization of

temperature and humidity across all mixtures was ensured. Then, the materials required for each mixture were weighed, and the components were placed in separate containers. The uniform mixing process involved operating the mixer, initially incorporating all aggregates and allowing them to mix for one minute. Specific attention was given to cementitious materials when SF was present, ensuring pre-mixing with cement in a dry state before dumping them into the mixer to mix with other solid concrete constituents. The next steps involved gradual water addition, allowing the mixture to operate for 4 minutes, followed by a pause for 1 minute. Finally, the superplasticizer was added and the mixer continued working for about one minute to yield the required concrete. After mixing, and before mixing the mix into molds, a slump test was conducted for each mix, and then samples for hardened properties of the concrete were cast. After 24 hours, the demolding process was performed and all samples were moved to the curing tank. Specific tests outlined in Table 3 will be discussed next.

3.5.2 Fresh concrete properties (Workability)

The concrete workability for each mix was assessed using the standard slump test in accordance ASTM C 143[99]. This test used slump values to assess mix's workability. The slump values (Figure 4) were used to assess the concrete's consistency for stirring, transportation, pouring, and compaction without material segregation, usually superplasticizer used to enhance the workability and compaction.



Figure 4: Slump test

3.5.3 Hardened concrete properties – mechanical properties

A.1 Compressive strength

Twenty-four (24) cubic samples (100 mm x 100 mm x 100 mm) were prepared to evaluate compressive strength for eight mixes, three cubes for each one. This test was conducted after 28 days of curing period and according to ASTM C39[100] by using a digital compression testing machine (Figure 5) to examine cubes with a constant loading rate of 2 kN/Sec.



Figure 5: Strength test (Compressive)

A.2 Split tensile strength

An assessment of split tensile strength was carried out in this research for 24 cylindrical samples (200 mm height x 100 mm diameter) for all mixes, three cylinders for each mix, according to ASTM C496 [101]. This was performed using the same machine of compressive strength (Figure 6) at a differing loading rate equal to 0.017 MPa/sec.

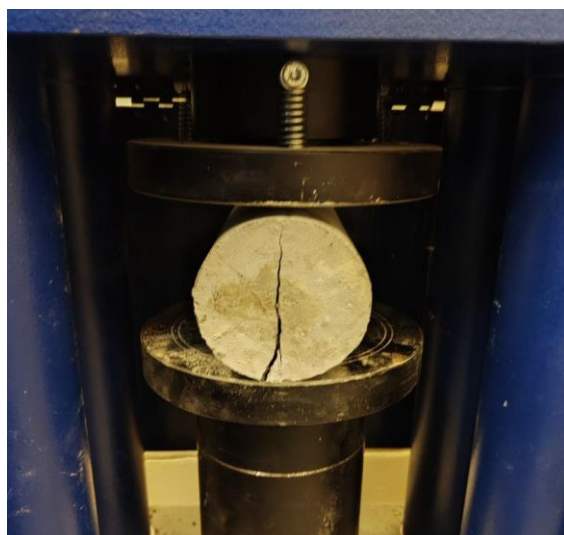


Figure 6: Split tensile test

A.3 Flexural strength (Modulus of rupture)

The test to assess flexural behavior for all mixes of the modulus of rupture was conducted according to ASTM C78. A setup with two points of loading, shown in Figure 7, was used to test (75 mm x 75 mm x 285 mm) prisms. The total number of prisms was 24 (3 for each mix). This test was performed using a compression machine with a fixed loading rate of 0.031 kN/Sec.



Figure 7: Strength test (Flexural)

A.4 Elastic modulus (Modulus of elasticity)

As per the ASTM C469 [102] requirements, the elastic modulus is defined as the slope between two points on the stress-strain curve. The first point is 50 micro strains, and the second is 40 % of the failure stress. For each mix, three cylinders (200 mm height x 100 diameter mm) and tested samples were 24. The experimental setup shown in Figure 8 involves three Linear Variable Differential Transformers (LVDTs), each connected with three channels. The specimen was centrally positioned, and the channels were connected

with a compression machine to accurately measure both strain and loading digitally at a steady load application rate of 0.2 MPa per second.



Figure 8: Elastic modulus test

A.5 UPV

In accordance with ASTM C597[104] , the ultrasonic pulse velocity (UPV) test was conducted on the samples of the split tensile strength. The UPV is indirectly related to the porosity of concrete and, during this test, the velocity of the transmitted pulse from one side of the sample to the other. For each mix, three samples were examined.

3.6 Discussion and Results

3.6.1 Workability

Results from workability (slump test), shown in Table 4, highlight a significant difference between (M-00-00) and (M-00-05), with (M-00-05) exhibiting a notably lower value—approximately 87.5% less. Conversely, values for (M-15-00) and (M-15-05) are equal (215 mm), showing no differences. Mixes containing 30% and 45% plastic, particularly those incorporating SF, demonstrate higher slump values. In conclusion, the addition of SF

substantially increases workability, especially at higher plastic replacement percentages. This increment may be attributed to the hydrophobic nature of plastic granules, prevent water attachment to the surface and influencing the consistency of concrete mixes (HDPE water absorption is almost 0%). The hydrophobic nature of plastics providing free water and the pellet shape of fine HDPE aggregates mitigating intergranular friction in mixtures with a relatively low W/C ratio explain the differences.

Table 4: Slump test results

Mixes		Slump
mix	Detailed name	(mm)
M1	M-00-00	200
M2	M-15-00	215
M3	M-30-00	220
M4	M-45-00	185
M5	M-00-05	25
M6	M-15-05	215
M7	M-30-05	240
M8	M-45-05	225

3.6.2 Compressive strength

Compressive strength results are presented in Figure 9 , it is noticeable that the general pattern is that the increase in HDPE content yields a decrease in compressive strength. However, the lowest compressive strength observed (27 MPa) exceeds the ACI 318 limit of structural concrete with a good margin, indicating good quality concrete with 45% HDPE replacing FA and 60% RCA replacing the CAs. The effect of SF is positive in cases of low HDPE content and negative in the higher two HDPE content. The difference, however, is not that appreciable and one can conclude that there is no appreciable trend to draw from the impact of 5% SF on the developed concrete.

The findings of the present study align with previous studies [105–108] confirming that the use of HDPE as FAs consistently decreases compressive strength with an increasing replacement ratio. For instance, Ref. [107] employed a replacement ratio of 0-25%, attributing the reduction in compressive strength to weaker bonding between granules. Ref. [108] also utilized HDPE with up to a 50% replacement ratio, explaining the decrease in compress strength to the weakened adhesion between cement paste and HDPE particles due to their smooth surface and lack of covalent coupling with the binder. The difference in compressive strength between M3 and M7, despite M7 having 5% silica fume, can be attributed to two potential factors.

First, the distribution of plastic within M3 might not have been uniform during the mixing process. This non-uniformity likely caused plastic particles to accumulate in certain regions, particularly toward the top of the mix, leading to segregation. As a result, the denser components, such as cement and aggregates, settled toward the bottom, contributing to a higher compressive strength in M3. Segregation during the casting process, which can often occur during lab work, may have inadvertently led to this strength increase in M3.

The second factor involves potential errors during the mixing process of M3. Although silica fume is typically added to improve mechanical properties like compressive strength, the inclusion of plastic waste may have led to improper mixing. Any issues with blending or mixing consistency could have negatively impacted the cohesion between the cementitious materials and the plastic waste, causing a reduction in strength. Despite this potential error in M3's mix, the general trend across all samples remains clear: as the percentage of plastic waste increases, compressive strength decreases. This trend reflects the inherent challenges of using plastic waste as a partial replacement for traditional

aggregates, where its low bonding strength and compatibility reduce the overall structural performance of the concrete.

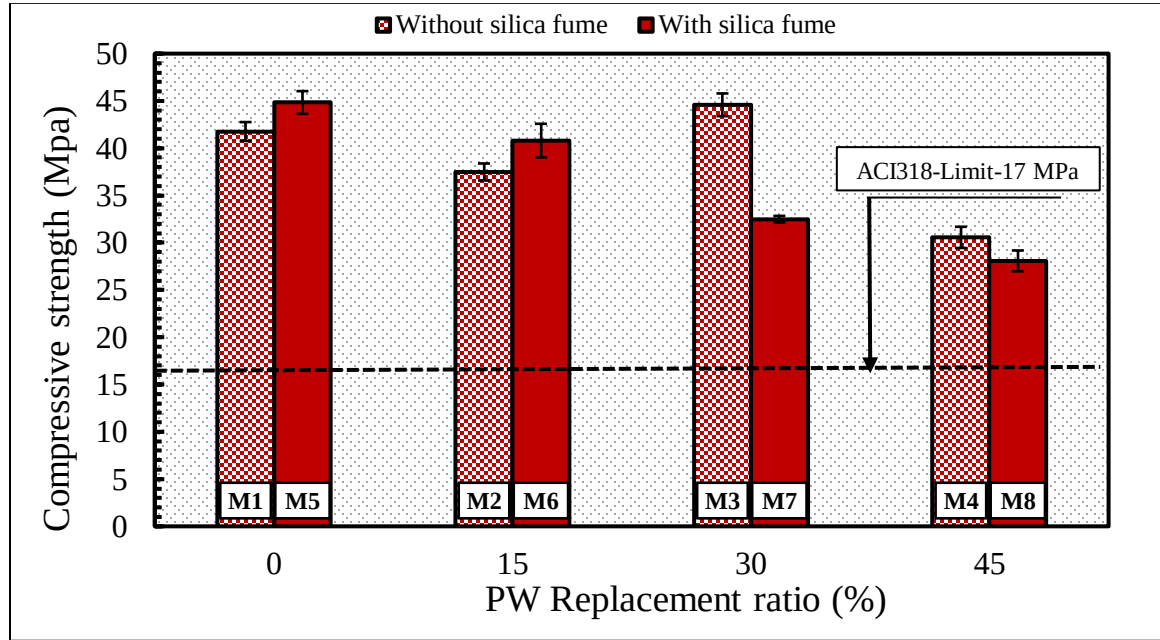


Figure 9: Compressive strength of Concrete mixes

3.6.3 Split tensile strength

Figure 10 illustrates the variation in tensile strength with different percentages of PW incorporated into concrete after 28 days of curing. The behavior of the split tensile strength follows that of compressive strength with a reduction of the split tensile strength with the increase of HDPE content and the SF is higher for cases of low HDPE content (up to 15%).

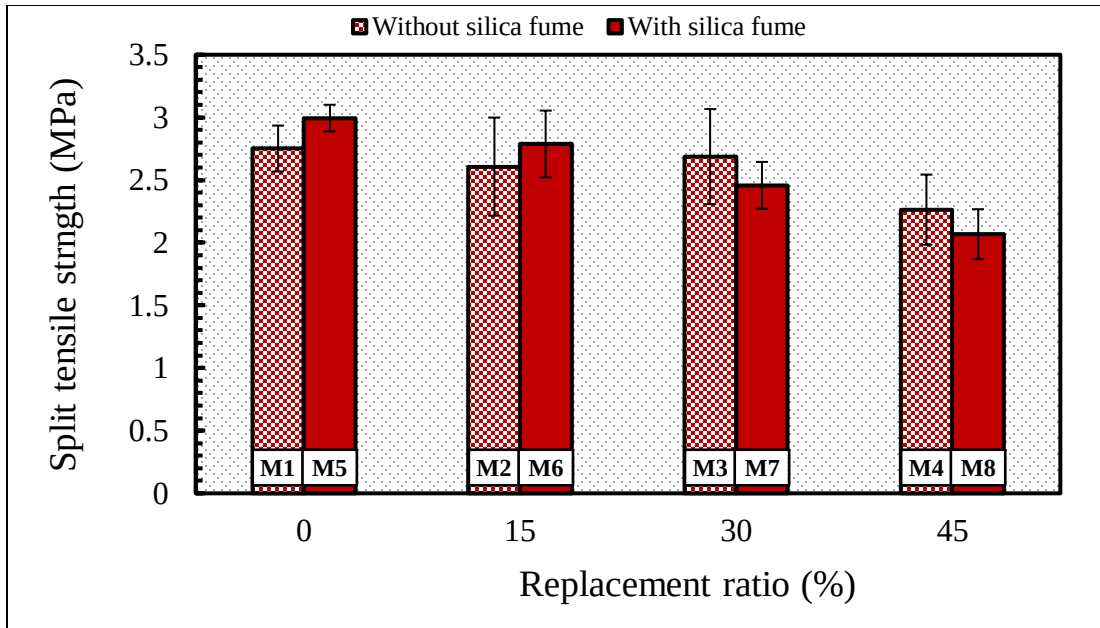


Figure 10: Split tensile strength

3.6.4 Flexural strength

As in the case of flexural strength, all the mixes fluctuate around 4.5 MPa with no distinguished trend observed, as shown in Figure 11.

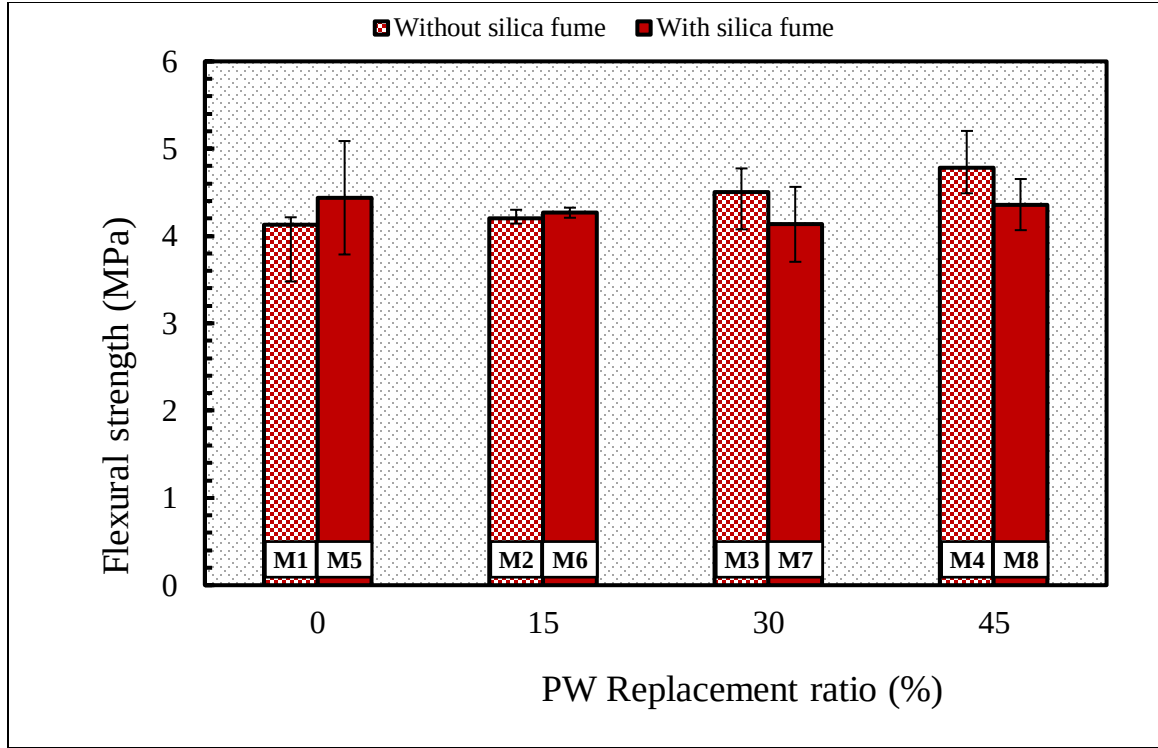


Figure 11: Flexural strength

3.6.5 UPV

Figure 12 visually presents the variation in ultrasonic pulse velocity (UPV) with varying percentages of PW in the concrete mixtures. The UPV exhibits a consistent decrease with the escalating content of plastic. These findings imply a reduction in material rigidity as the plastic content increases. Similarly, the UPV follows the same pattern to the strength behavior (compressive) with the introduction of SF. At 0 % and 15% plastic replacement, the UPV increases by 1% and 3.5%, respectively, showcasing a positive effect. Conversely, at 30% and 45% plastic replacement, the UPV decreases by 3.7% and 2.4%, respectively, indicating an adverse impact. A similar report is observed in the literature as in Refs. [109] and [110].

The observed reduction in UPV aligns with the increase in amount of plastic aggregate across all mixes. The addition of plastic aggregates leads to an increase in concrete porosity. Additionally, the voids between aggregates contribute to the reduction in UPV due to the obstructive effect of air. The incoming wave traveling through different materials (concrete, plastic, and voids) undergoes partial reflection and transmission, decreasing velocity.

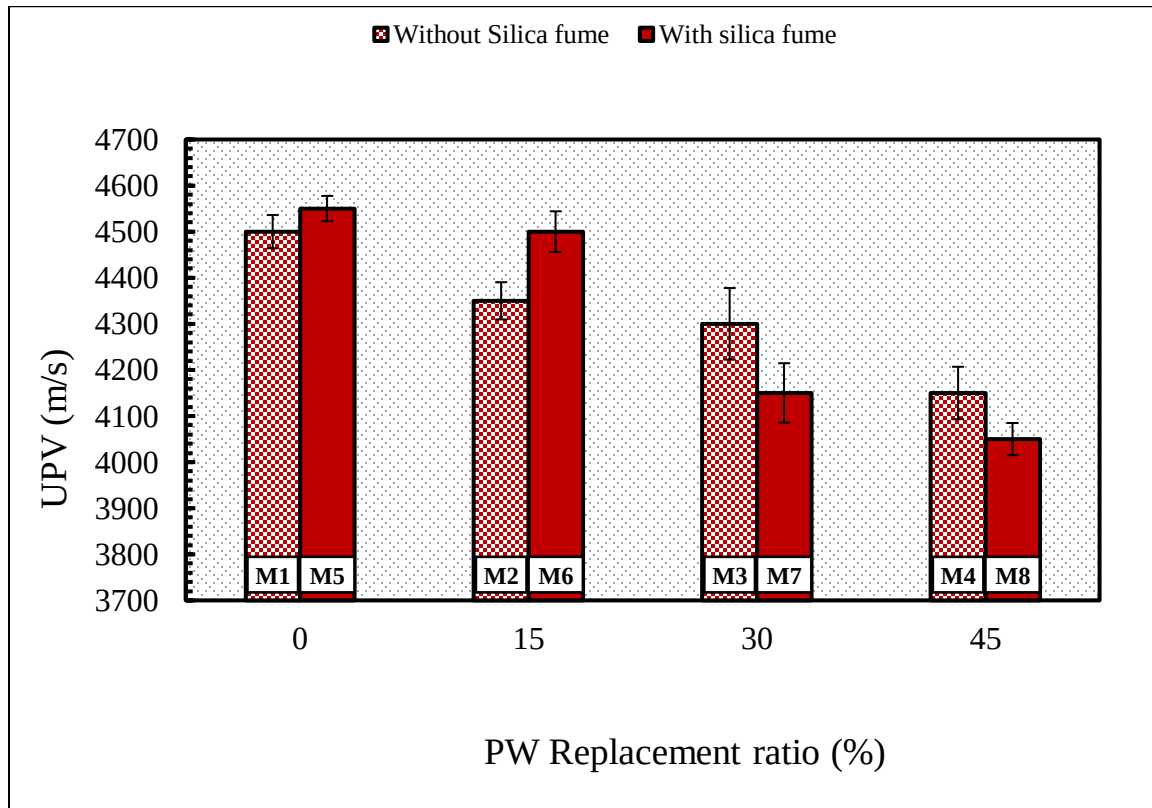


Figure 12: Ultrasonic pulse velocity

3.6.6 Static Modulus of Elasticity

Figure 13 illustrates the results of the static elastic modulus excluding (M-0-15) and (M-0-30). The results indicate that the values behave similarly to compressive strength as a

general trend. However, it cannot be confirmed with 100% certainty due to two missing values. These missed values arose from the testing machine, where the measured values were too high to be considered (more than 100 GPa). This problem suggests a potential error in the measurement process. Two Linear Variable Differential Transformers (LVDTs) out of three provided out-of-range measurements of strain. Also, strain should be positive; however, one of the LVDTs gave us negative values, while the other provided smaller values than the correct one, leading to inaccurate readings. To address this issue, it is decided to estimate the dynamic modulus of elasticity using density and UPV values. The dynamic method is expected to provide more reliable data to understand the material's behavior comprehensively.

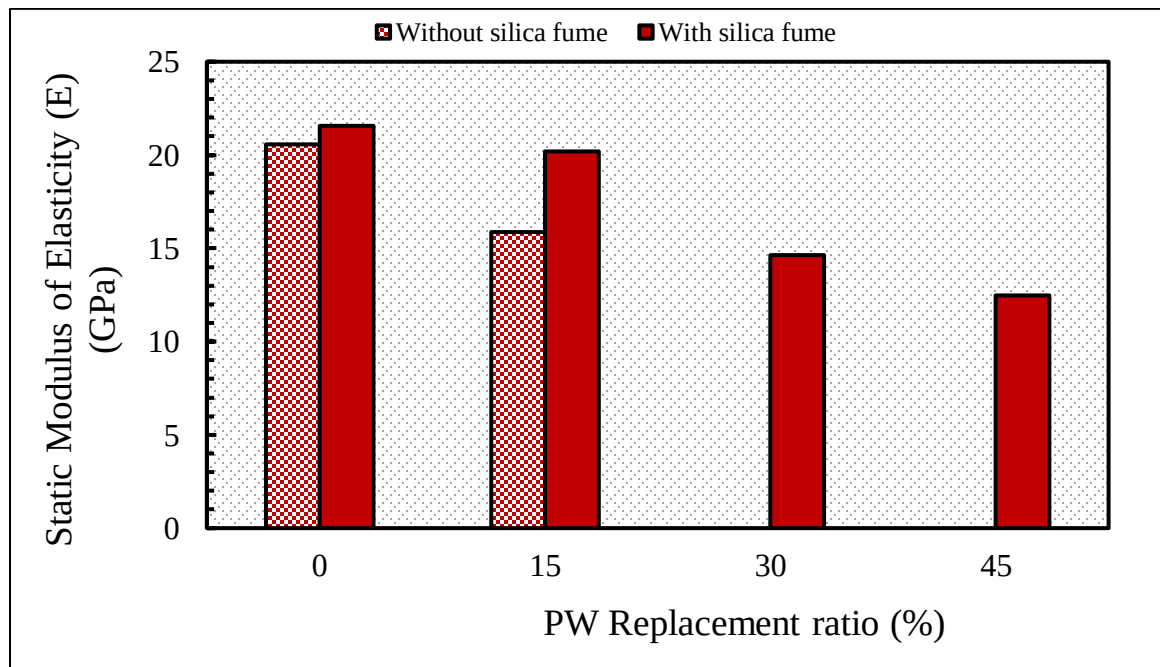


Figure 13: Static modulus of elasticity (E)

3.6.7 Dynamic Elastic modulus

The dynamic modulus of elasticity (E_d) was calculated for the eight mixes using the following equation:

$$E_d = (V^2 \cdot \rho / g) \cdot 10^{-2} \quad (1)$$

where E_d represents the dynamic elastic modulus (in GPa), V is (UPV) in km/s, ρ is the actual density in kg/m³, and g is the gravitational acceleration (9.81 m/s²).

Figure 14 demonstrates a notable trend where the dynamic elastic modulus decreases with the increase in plastic content for both cases of with and without SF. This reduction can be attributed to: (a) the relatively low elastic modulus of PW s when compared to those of (NAs) and (b) the bond of the polymer to the cementitious matrix is poor, resulting from the difference in elastic modulus between the polymer particles and the surrounding cement paste.

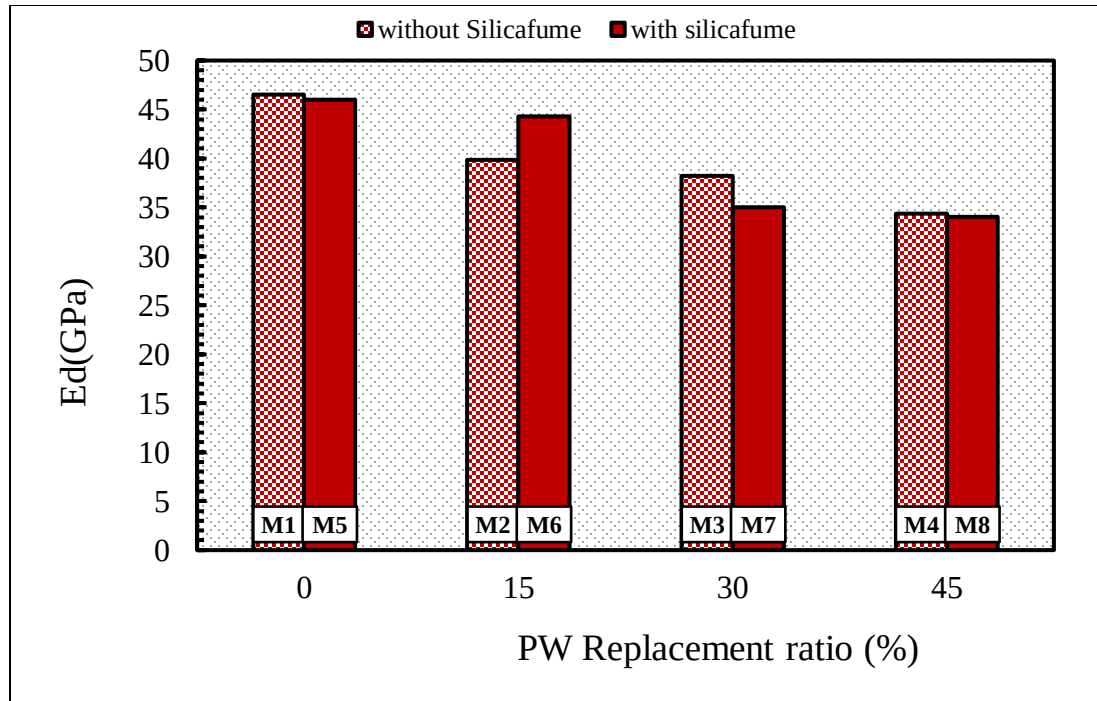


Figure 14: Dynamic elastic modulus

3.7 Conclusions

In the developed HRAC, recycled concrete aggregates partially replace natural CAs (RCA) with a volumetric constant ratio of 60%. The natural FAs (sand) undergo partial replacement by PW at volumetric ratios of 0%, 15%, 30%, and 45%, while the cement is substituted in part by SF at 0% and 5% replacement ratios (by weight). The concrete underwent various tests, including slump, density, compressive strength, split tensile strength, flexural strength, ultrasonic pulse velocity (UPV), and dynamic elastic modulus.

From the test results, the following conclusions can be drawn:

- 1) There is a general observation regarding the workability of fresh concrete, indicating a rise in workability (value of slump) as the PW content increased. The addition of SF significantly increases workability, especially at higher plastic replacement percentages. However, it is noteworthy that two values at M4 and M5 deviate from this trend.
- 2) Density decreases with an increase in PW for both cases of with and without SF. This results in lower weight and density due to the inclusion of plastic.
- 3) Compressive strength, split tensile strength, flexural strength, and UPV exhibit a similar trend in lower values with the increase of HDPE content. They also share the improvement trend of adding SFs up to a 15% plastic replacement. However, an adverse trend is observed at 30% and 45% plastic replacement, where the addition of SF leads to a decrease in these properties.
- 4) The dynamic elastic modulus values reduce as the amount of PW increases. This is attributed to the weak adhesion between the polymer and cement matrix, resulting from the difference in elastic modulus between the polymer particles and the surrounding cement paste.

In conclusion, the developed concrete offers the advantage of utilizing a high volume of recycled materials. For example, concrete containing 30% PW and 60% RCA achieves a 45% recycled material content. Despite this high percentage, the 28-day compressive strength is 28.087 MPa, which is acceptable structurally. Thus, this concrete can potentially be a sustainable material of acceptable quality.

CHAPTER 4

Durability properties of concrete incorporating recycled concrete aggregates and plastic wastes

4.1 Background

While RCA or PW generally impact concrete properties negatively, their combined effect in producing concrete having both RCA and PW remains largely unexplored. This chapter aims to fill this gap by investigating the durability properties of (HRAC). The concrete mixtures used for durability investigations in this chapter are consistent with those used in Chapter 3 for mechanical properties. Eight different mixes were tested for various durability properties, including density, water absorption, porosity, thermal conductivity, chloride migration, electrical conductivity, electrical resistivity, sound absorption, drying linear shrinkage, fire resistance and microstructure analysis by using photos of scanning electron microscope (SEM). Furthermore, the research encompasses a cost analysis and an assessment of the environmental impact, focusing on energy consumption and CO₂ emissions.

4.2 Experimental investigation

The experimental program in this study included eight different mixes as discussed in Chapter 3, all with a consistent replacement ratio of 60% (RCA) for CA. The mixes were divided into two groups: M1 to M4 contained no SF, while M5 to M8 incorporated a fixed

5% of SF as a cement replacement. Within each group, the natural FAs were replaced with PW at volumetric ratios of 0%, 15%, 30%, and 45%. Additionally, the materials and mix proportions used in designing the concrete mix for this study align with those described in Chapter 3.

4.3 Methodology

4.3.1 Preparation of the Specimens

The specimens needed to conduct the durability properties of the developed HRAC were cast and cured following the procedure described in Section 3.5.1. Figure 15 shows some of casted and demolded samples used in this chapter. The considered results are illustrated in Table 5.



Figure 15: Selected samples of concrete with PW

Table 5: Tests conducted in this research

#	Test	Standard
1	Compressive strength under elevated temperature	ASTM C39 [100]
2	Density	ASTM C642 [111]
3	Water absorption	ASTM C642 [111]
4	porosity	ASTM C642 [111]
5	Drying shrinkage	ASTM C157 [112]
6	Chloride migration test	NT BUILT 492 [113]
7	Electrical conductivity	ASTM C1760 [114]
8	Electrical resistivity	ASTM C1870 [115]
9	Thermal conductivity	ASTM D5334 [116]
10	Sound absorption	ASTM E1050[117]

4.3.2 Compressive strength under elevated temperature (Fire resistance)

PW is highly sensitive to elevated temperatures compared to other constituents of the considered concrete of the present study, primarily due to its low heat resistance. Therefore, understanding how PW behaves under high temperatures is crucial, especially when used in developing concrete. To assess this, nine cubic samples (100x100x100 mm) were prepared for each mix from M1 to M8. These samples to be used to evaluate the compressive strength under elevated temperatures (room temperature (27°C), 250°C, 500°C), three specimens for each elevated temperature. The samples were gradually heated

in a furnace capable of reaching a maximum temperature of 3000°C. The heating process took 2 hours to reach the target temperature, which was then maintained for an additional 2 hours. Afterward, the furnace was switched off, and the specimens were allowed to cool slowly over a span of 24 hours. Once cooled and becomes at room temperature, the samples were evaluated for cracks, compressive strength, and weight loss. Figure 16 shows the furnace that was used for heating the samples.



Figure 16: The furnace used

4.3.3 Density

The densities were calculated for eight mixes according to ASTM C642 [100], with each mix containing three cubic samples (100x100x100 mm). The process began by recording the initial weight of each sample. The process begins by measuring the oven-dried weight of the samples, which involves drying the samples at 100°C until they reach a constant weight. This constant weight is recorded. Following this, the mass of the surface-dried samples is measured in air after they have undergone immersion and boiling. Finally, the weight of the samples is recorded in water after the same immersion and boiling process.

By using these recorded weights and adhering to ASTM C 642 standards, the bulk density of the samples can be calculated. Visual illustration shown in the following Figure 17.



Figure 17: Concrete samples used for density, water absorption and porosity

4.3.4 Water absorption

Water absorption is a critical factor in determining the durability of concrete because it indicates the water permeability within the samples. To assess this, the same samples used for calculating density showed in Figure 17, as per ASTM C642 [100], were also utilized for measuring water absorption. Measurements were taken both after immersing the samples in water and again after immersion and boiling. The incorporation of HDPE into the mix affects the overall water absorption characteristics of the concrete due to HDPE's natural resistance to absorption. This resistance plays a significant role in enhancing the durability of the concrete.

4.3.5 Porosity

Porosity measures the volume of voids within concrete, particularly those interconnected voids through which moisture can move. According to ASTM C642 [100] standards, the weights used for calculating density and water absorption, along with the mass of surface-dry samples measured in water after immersion and boiling, can also be employed to determine porosity. This measure indicates the presence of empty voids inside the concrete. By utilizing the same three cubic samples used for density calculations, it is straightforward to evaluate porosity and understand the effects of PW and SF on it. This approach allows for a comprehensive analysis of how these additives impact the internal structure of the concrete, thereby influencing its overall durability.

4.3.6 Drying shrinkage

Shrinkage in concrete, caused by moisture transfer, is a complex, time-dependent process that occurs in cementitious materials, leading to volumetric contraction and potentially causing crack formation. The difference in stiffness between fine and CAs significantly influences the amount of shrinkage. Aggregates affect concrete's deformation properties through their interaction with the cement paste, impacting water absorption and porosity. To estimate linear drying shrinkage, ASTM C157 [112] was employed to evaluate the length changes in samples due to temperature or moisture variations. This test determines the potential volumetric contraction or expansion of concrete. For this purpose, three prismatic samples (75x75x285 mm) for each mix were prepared and measured for length changes. Initial measurements were taken on the first day, followed by subsequent readings at 7, 14, and 28 days while the samples were stored in water. Afterward, the samples were stored in air, with further readings at 8, 16, and 32 weeks (49, 98, 224 days, respectively),

in accordance with ASTM C157 [112]. The setup, including the prisms and measuring device, is illustrated in the Figure 18.



Figure 18: linear drying shrinkage device and concrete sample

4.3.7 Chloride migration test

The primary purpose of conducting the chloride migration test on concrete is to assess the potential for corrosion in reinforced concrete structures. The NT-BUILD 492 standard [113] is used to calculate the chloride migration coefficient for each previously mentioned concrete mix in Table 2. This method involves using cylindrical specimens with a diameter of 100 mm and a height of 50 mm, sliced from cast cylinders. The test setup includes a device designed for the NT-BUILD method for chloride migration testing. The specimens are placed inside the cell, with sodium chloride (NaCl) in the catholyte solution part and sodium hydroxide (NaOH) in the anolyte solution part. An external electrical potential is applied axially across the specimen, forcing the chloride ions to migrate into the specimen.

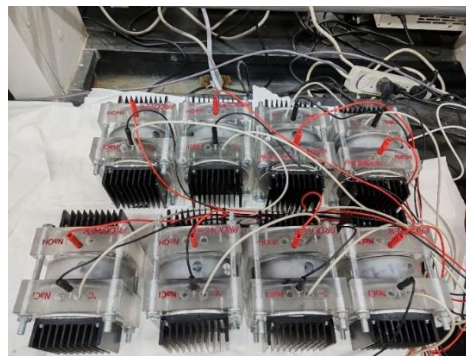
The depth of chloride penetration is measured based on the visible white silver chloride precipitation. From this penetration depth, the chloride migration coefficient was calculated. All parts of chloride migration test are shown in Figure 19 .



A. Samples



A. Chloride migration test device



A. Cells (Concrete samples inside cells)

Figure 19: Chloride migration test parts

4.3.8 Electrical conductivity

According to ASTM 1760 [114], this method is used to determine the electrical conductivity of hardened concrete, providing a rapid indication of the concrete's resistance to chloride ion diffusion. The process involves measuring the current passing through a specimen under a specific voltage difference. The samples used are standardized cylindrical molds with a diameter of 100 mm and a length of 200 mm. The same device used for the chloride migration test showed in Figure 19 was adopted for electrical conductivity measurements. As illustrated in Figure 20, the setup involves adding sodium chloride to both parts of the device and adjusting the voltage to pass current for one minute. This test is particularly useful for evaluating the electrical conductivity of concrete in structures designed with cathodic protection systems.



Figure 20: Electrical conductivity cell

4.3.9 Electrical resistivity

According to ASTM 1876 [115], this method measures the electrical resistivity (ER) of hardened concrete to quickly assess its durability against fluid infiltration and aggressive ions, which can contribute to corrosion, especially in reinforced concrete. Using the Proceq

Resipod model device, equipped with four sensors, surface resistivity is measured, as shown in Figure 21. This method uses cylindrical specimens of size 100×200 mm. The device quantifies how strongly the material opposes the flow of electrical charges when an electric field is applied using electrodes placed on opposite faces of the test specimen, with results measured in units of ohm-meter.



Figure 21: Proceq device and electrical resistivity test on concrete sample

4.3.10 Thermal conductivity

Thermal conductivity, represented by the symbol (K), describes a material's ability to transfer heat from the higher temperature side to the lower temperature side. This property varies depending on factors such as the material's composition, temperature, humidity, surface area, type of aggregates, density, curing methods, mineral admixtures, and thickness. Evaluating thermal conductivity is crucial for understanding the heat transfer rate in concrete. To assess the thermal conductivity of the new concrete mix, the ASTM

D5334 [116] standard procedure was employed. This method utilizes a thermal needle for testing soil, rock, or any hardened specimens. Cylindrical specimens sized 50×150 mm were prepared, each containing a 5 mm hole to facilitate the use of the device needle. Figure 22 shows the setup of the test. The needle operates by applying voltage and current to generate heat over a specific period, followed by a cooling cycle, then automatically analyzes the data to determine the thermal conductivity coefficient.



Figure 22: Thermost device for estimating the thermal conductivity and concrete Sample

4.3.11 Sound absorption

Sound absorption is a property used to evaluate the acoustic noise prevention capabilities of various materials such as glass, wood, and concrete. This property is crucial for reducing noise in civil and architectural applications. To measure the sound absorption in this study, a Holmarc ITA219 device was utilized. This device is a comprehensive impedance tube system comprising microphones, a measurement unit, sample holders, and hollow tubes. Cylindrical samples with a diameter of 100 mm and a height of 50 mm were placed inside the tube. The power amplifier was then activated to generate sound through the microphones, with the sample positioned in the middle of the impedance tube, as shown in Figure 23. The sound absorption coefficient was estimated using software associated with the impedance tube, adhering to ASTM E1050 [117] and ISO 10534-2 standards[119].



Figure 23: Holmarc impedance tube for estimating the sound absorption coefficient

4.4 Results and Discussion

4.4.1 Fire resistance

This test aimed to assess the behavior of concrete under elevated temperatures. It was initially planned to be conducted on the eight considered mixes (M1 to M8) in the present study at temperatures of 27°C, 250°C and 500°C. However, during testing of M3 at 250°C, it became deformed and unsuitable for testing due to melting of the segregated plastic as shown in Figure 24. Consequently, it was decided that samples containing more than 15% plastic content (M3, M4, M7, and M8) should be eliminated from the testing program of the compressive strength at elevated temperature. This initial observation indicates that using plastic granules as FAs could not be feasible at 30% replacement or more.

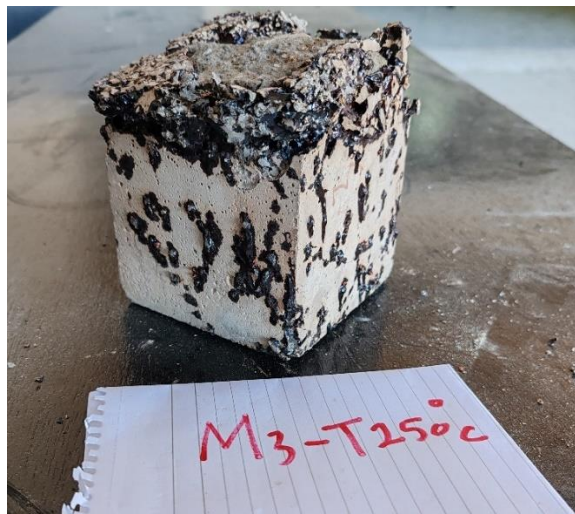


Figure 24: Heated Samples with 30% PW

The results as illustrated in the Figure 25 indicate a general decrease in compressive strength with the increase in PW replacement and elevated temperature with some cases

deviate from the trend with an insignificant difference. For example, the compressive strength of M2 and M6 at 250°C decreased by 26.6% and 25.3%, respectively. Cracks were observed, with M2 showing a crack width of 0.1 mm and M6 a width of 0.2 mm at 250°C. The plastic in these samples turned black and expanded outside the samples due to partial melting, causing the cracks as shown in Figure 26. The compressive strength of M1 and M5, which are without plastics, show relatively different trend at 250°C while they follow the trend of reduction in the compressive strength with the increase of elevated temperature to 500°C. M1 at 250°C increased by 1.6% compared to room temperature, and M5 at 250°C increased by 9.5%. According to Roa and Reddy[120], normal concrete subjected to elevated temperatures ranging from room temperature (27°C) to 300°C, in 50°C increments, exhibited an increase in strength up to 250°C, followed by a decrease. This strength increase is attributed to the rapid loss of humidity, which breaks further hydration and consequently enhances strength. Furthermore, the rapid temperature changes lead to rapid alterations in the stress-strain relationship, which also affects the strength of the concrete.

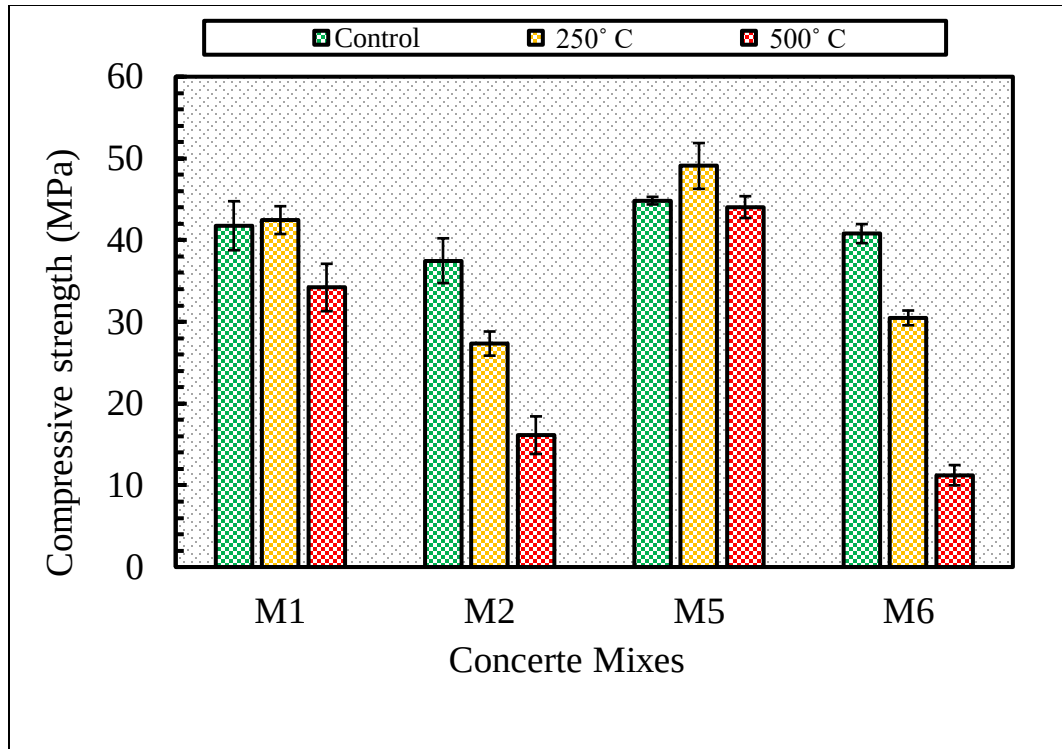


Figure 25: Compressive strength under elevated temperature (MPa)



A. M1 at 250° C and 500° C



B. M2 at 250° C and 500° C



C. M5 at 250° C and 500° C



D. M6 at 250° C and 500° C

Figure 26: Samples of concrete Under elevated temperature

At 500°C, all mixes (M1, M2, M5, M6) showed a significant decrease in compressive strength compared to room temperature, with reductions of 18.1%, 57%, 2%, and 72.5%, respectively. The plastic completely melted inside the samples at this temperature, resulting in visible cracks, with crack widths of approximately 0.2 mm in samples containing 15% PW as shown Figure 26 . The samples without plastics show a better heat resistance with reduction limited to 18.1 % for M1 (without SF) and 2% for M2 (with 5% SF).

Weight loss was also evaluated, with the maximum occurring at 500°C for M2 and M6 as illustrated in Figure 27, which shows reductions of 9.2% and 9.8%, respectively, compared to their weight at room temperature. The general trend suggests that there is an increase in weight loss with the increase of temperatures due to the emptying of voids and the melting

of plastic. These results suggest that concrete incorporating PW aggregates are not resistant to high temperatures. This limits its application to locations where the possibility of fire accident is minimal as will be discussed later in the recommendations by the end of this study.

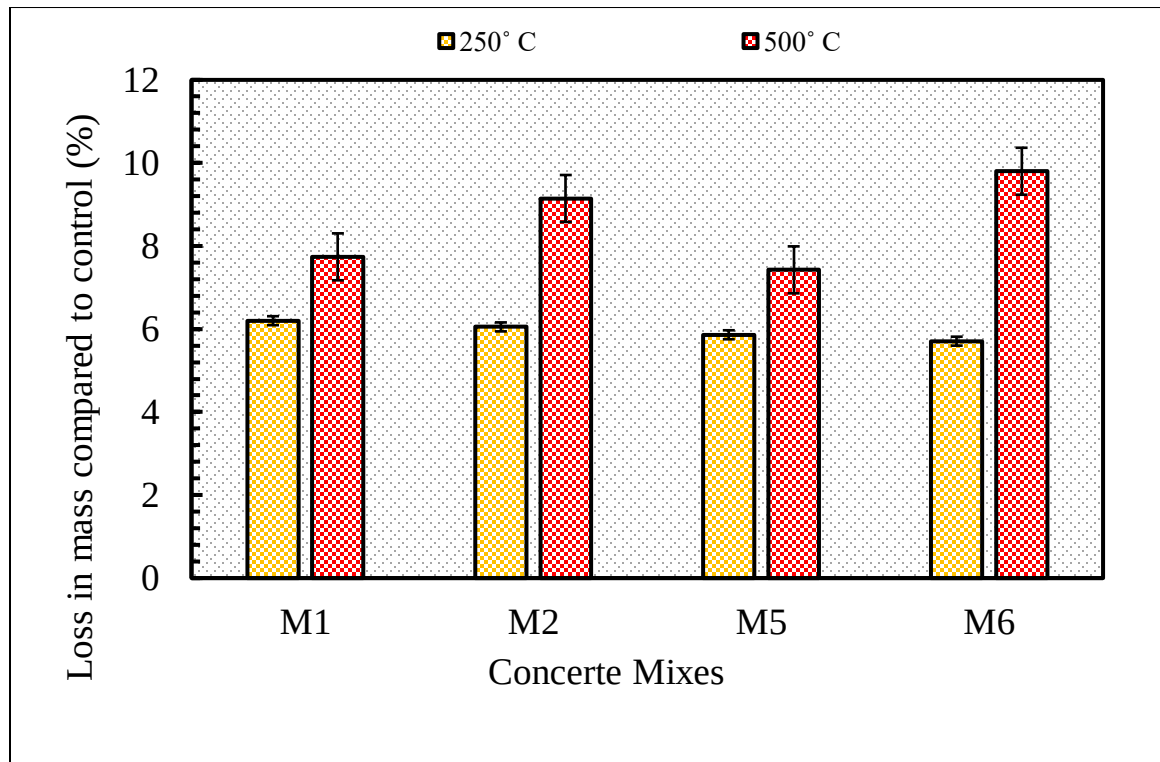


Figure 27: Weight loss in concrete samples due to elevated temperature (%)

Elevated temperatures contribute to the formation of cracks, particularly at a 15% plastic waste (PW) replacement ratio. As shown in the **Error! Reference source not found.**, when the M6 samples were heated to 250°C, it became evident that the outer layer of the plastic on the surface began to melt. This melting created gaps between the softened plastic and the surrounding cement paste, which negatively affected the concrete's strength. At 500°C, the plastic completely degraded and disappeared, resulting in significant breakdown of the samples. The areas previously occupied by plastic turned into voids,

leading to large cracks, as illustrated in the **Error! Reference source not found..** T herefore, it can be concluded that plastic waste concrete is not suitable for high-temperature environments due to deformation and cracking.



M6 at 250 °C



M6 at 500 °C

Figure 28: Cracks being in M6 samples

4.4.2 Density

The results of the density measurements taken after 28 days of curing are shown graphically in Figure 29, which indicates a general decrease in the density with the increase of the replacement ratio of PW. This decrease is attributed to the light weight and low specific gravity of plastic, i.e.: the density of HDPE is approximately 40% less than that of the sand. When comparing mixes with and without SF, it is evident that M5 has a higher density than M1, and similarly, M6 has a higher density than M2. Mixes M3 and M7 showed nearly similar densities with minimal differences, while M4 and M8 behaved conversely, with M4 having a higher density than M8 with a practically negligible difference. Therefore, it can generally be concluded that SF increases the density when it replaces 5% of the cement.

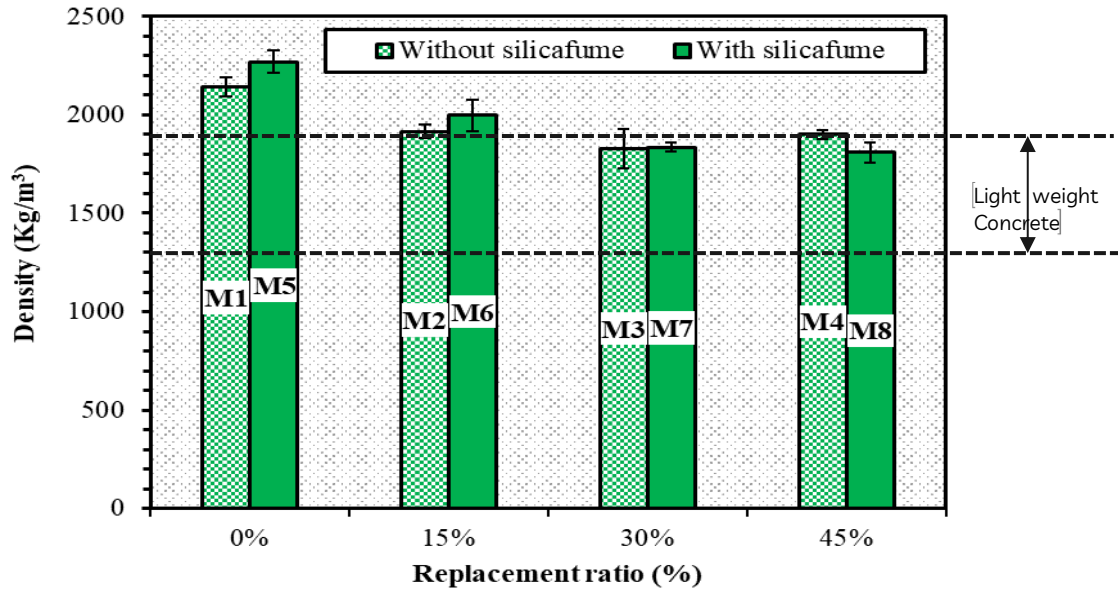


Figure 29: Density of concrete mixes with PW (Kg/m³)

4.4.3 Water absorption

As shown in Figure 30 and Figure 31, the general trend indicates that water absorption, both after immersion and after immersion followed by boiling, increases with a higher replacement ratio of PW. It is also evident that mixes containing SF generally exhibit lower water absorption compared to those without SF. However, beyond a 15% replacement ratio, mixes with SF absorb more water after immersion and boiling.

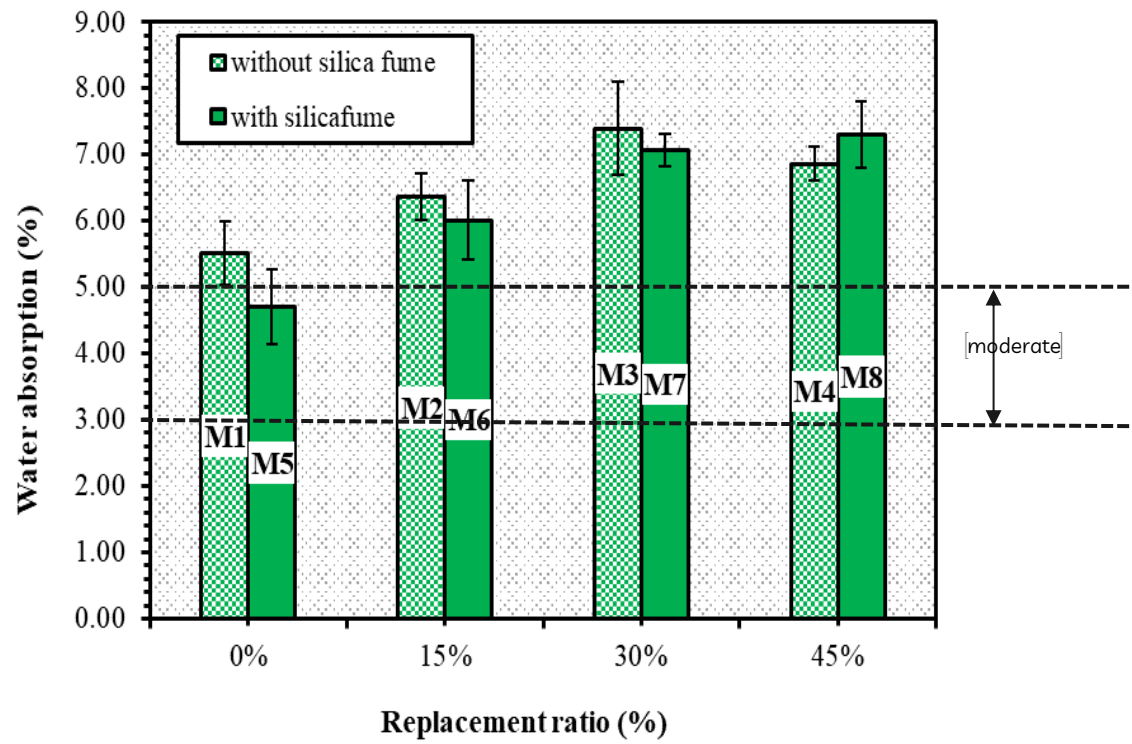


Figure 30: Water absorption after Immersion (%)

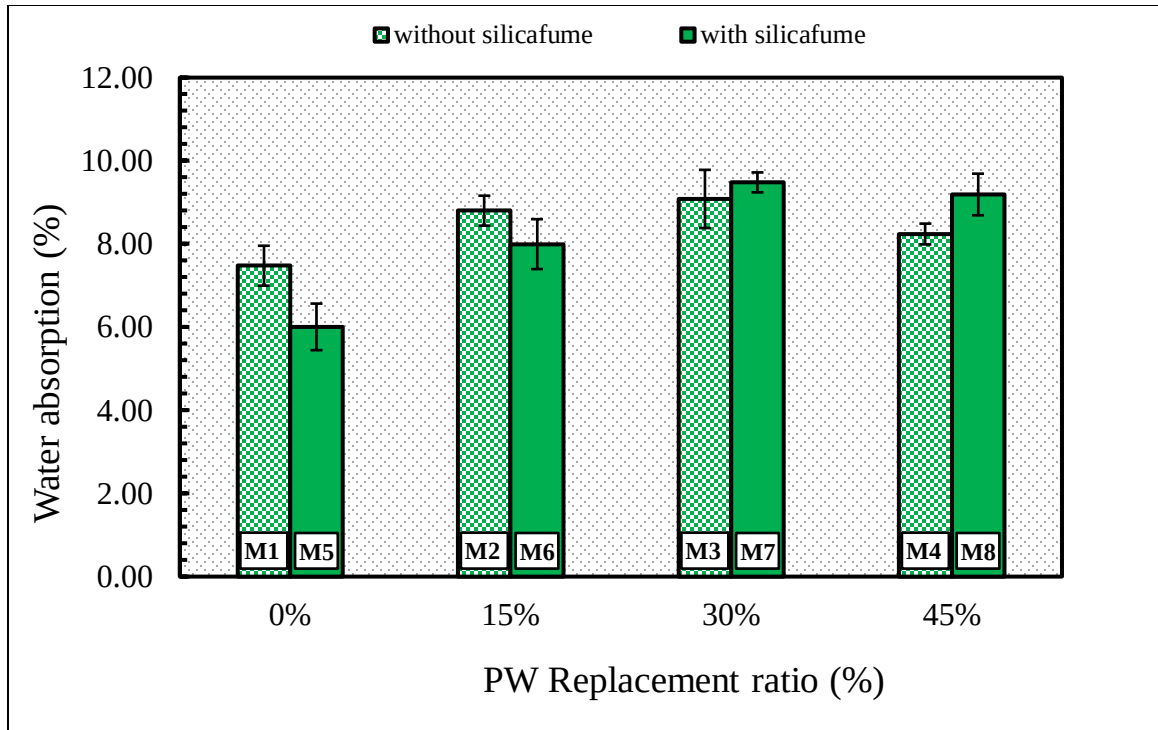


Figure 31: Water absorption after Immersion and boiling (%)

Overall, it can be concluded that adding 5% SF reduces water absorption. In addition, increasing the replacement ratio of PW leads to higher water absorption, with the maximum absorption occurring at 30% and 45% replacement ratios for both mixes with and without SF, which due to the segregation of plastic granules as suggested in these studies[23,121]. The rise in water absorption for concrete containing HDPE as FA, with higher HDPE percentages is primarily because of insufficient bonding at the contact point between HDPE and the surrounding aggregates [122]. The increased HDPE content within the concrete occupies space, resulting in water evaporation and void formation, which increases the absorption value as these voids get filled with water. This trend is consistent with the findings of Hannawi et al.[62], who reported that the highest increase in water

absorption was achieved by HDPE content of 30%. The findings demonstrate that Increasing water absorption leads to greater water penetration, which additionally raises the risk of corrosion and results in low resistance to abrasion.

4.4.4 Porosity

The results of porosity are shown in Figure 32, which indicates that the porosity percentage increases with a higher replacement ratio of PW similar to the water absorption. This increase in porosity is attributed to the hydrophobic nature of HDPE, its weak bond with the cement matrix, and the smooth, circular shape of HDPE granules, which contribute to the formation of pores in the concrete [123,124]. Additionally, mixes containing SF exhibit lower porosity in contrast to those lacking SF. For instance, comparing reference mixes without SF (M1-M4) to those with SF (M5-M8), the reference mixes show higher porosity by 8.5%, 10.15%, 10.65%, and 1.2%, respectively. This trend agrees with the findings of various researchers on the impact of SF on the microstructure properties of concrete. To understand the effect of SF on concrete, Poon et al. [125] found that using 5% SF reduced porosity. This finding is consistent with Khan et al. [126] research, which showed a more significant reduction in porosity with SF usage between 0-15%. Additionally, Song et al. [127] research explained the mechanisms behind this reduction in porosity, attributing it to pore size refinement, matrix densification, reduced Ca(OH)_2 content, and improved cement paste-aggregate interfacial refinement. During hydration, the pozzolanic reaction between SF and calcium hydroxide further densifies the transition interfacial zone. This suggests that SF reduces the porosity of concrete with PW, which can reduce affect the penetration of ions that cause corrosion, particularly in reinforced concrete.

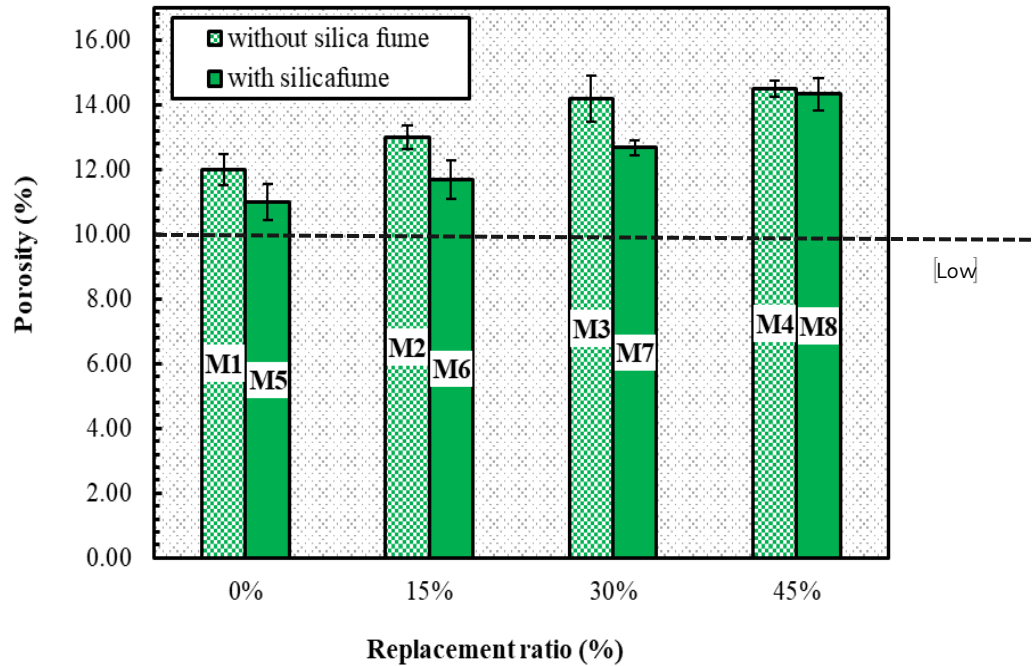


Figure 32: Porosity (%)

4.4.5 Drying shrinkage

Figure 33 presents the results of the drying shrinkage of this newly developed concrete. The general trend indicates that drying shrinkage increases with the increase of replacement ratio of PW for each set of mixes, both with and without SF. This increase occurs because plastic increases the porosity, directly affecting shrinkage. The weak bond between the smooth surfaces of cement and plastic further contributes to the increase in shrinkage. A deeper comparison between mixes with and without SF reveals that mixes with SF exhibit more shrinkage than those without SF, which may be due to the fact that SF increases water consumption in the mix, thus negatively impacting drying shrinkage. This agrees with the findings of Peng Zhang and Qing-Fu Li [128], who examined how SF influences the drying shrinkage of concrete. They found that drying shrinkage increases

with higher SF ratios (3%, 6%, 9%, and 12%) due to the negative impact on the concrete's resistance to drying shrinkage cracking, which becomes more pronounced with higher SF content. The data show that An increase in drying shrinkage leads to more cracks when this concrete is used in structural elements, which can cause damage and increase maintenance costs.

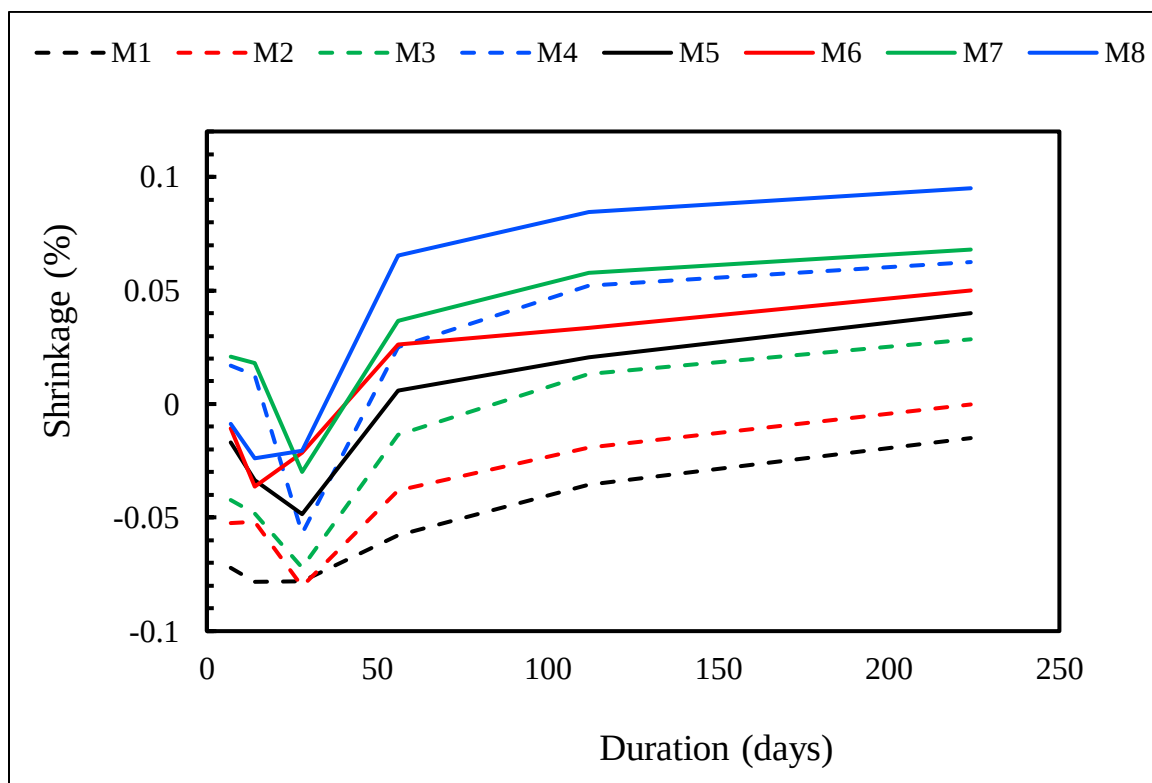


Figure 33: Linear drying shrinkage (%)

4.4.6 Chloride migration test

The chloride migration coefficient is shown in Figure 34, which indicates that the migration coefficient decreases with the increase of replacement ratio of HDPE. This implies that chloride penetration is reduced as the HDPE ratio increases. The reduction in the chloride migration coefficient can be attributed to the low ion conductivity of the plastic, as well as its shape and physical properties. This behavior is consistent with findings of Algahtani et al. (2018) [129], who used HDPE plastic aggregates as a replacement for FAs. It also agrees with the results of Kou et al. [63] in his investigation of the chloride migration coefficient in concrete incorporating granular shapes CAs PW. Additionally, mixes with SF (M5-M8) show lower migration coefficients compared to mixes without SF (M1-M4), indicating that the inclusion of 5% SF effectively reduces chloride ion penetration. For example, comparing M1, the control sample without plastic and SF, with M5, the mix with 5% SF, reveals that M5 has a migration coefficient that is 10% lower. Similarly, M3 compared to M7 and M4 compared to M8 show reductions of 4% and 10.5%, respectively. This could be attributed to the low porosity as discussed in Section 4.4.4. Due to the low ionic properties of plastic waste, the reduction in the chloride migration coefficient means that fewer chloride ions penetrate the concrete. This results in less chemical attack, making the concrete more suitable for use in marine environments.

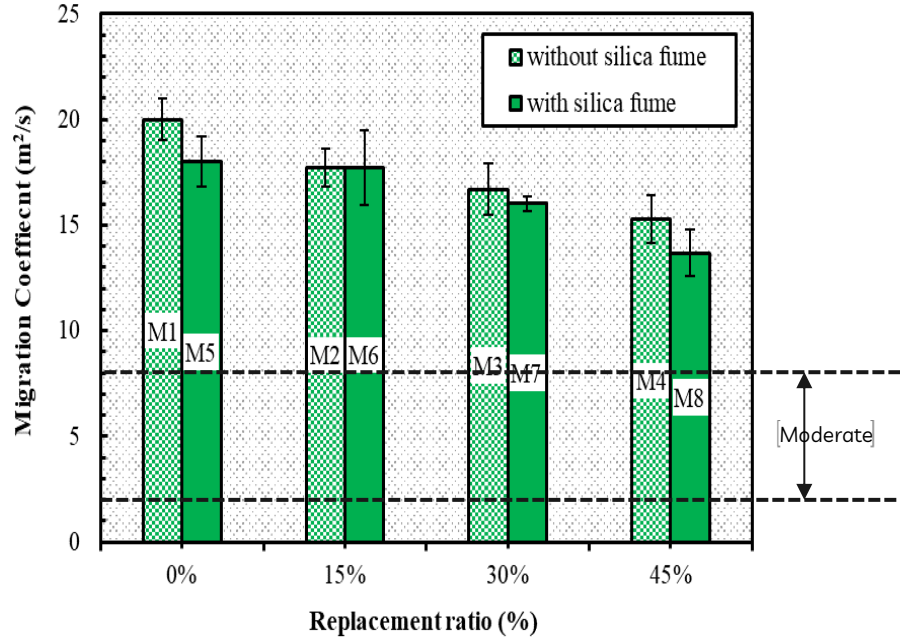


Figure 34: Chloride migration coefficient (m²/s)

4.4.7 Electrical conductivity

The results of the electrical conductivity, measured in millisiemens per meter (mS/m), are shown in Figure 35. The figure clearly indicates that there is a considerable reduction in the electrical conductivity with the increase of the PW content. The reduction constantly increases to reach about 34 % at the highest replacement ratio of 45%. This could be due to the low electrical conductivity of plastic as compared to another concrete constituent. On the other hand, it can also be seen from the figure that all the samples with SF reported less electrical conductivity as compared to those without SF. For instance, M5 shows a 26.3% reduction in electrical conductivity as compared to M1. Similarly, the comparisons between M2 and M6, M3 and M7, and M4 and M8 reveal reductions of 37.72%, 41.2%, and 26.8%, respectively. This indicates that adding SF significantly reduces electrical

conductivity. When the electrical conductivity of concrete is low, it indicates better chloride resistance and a reduced risk of corrosion.

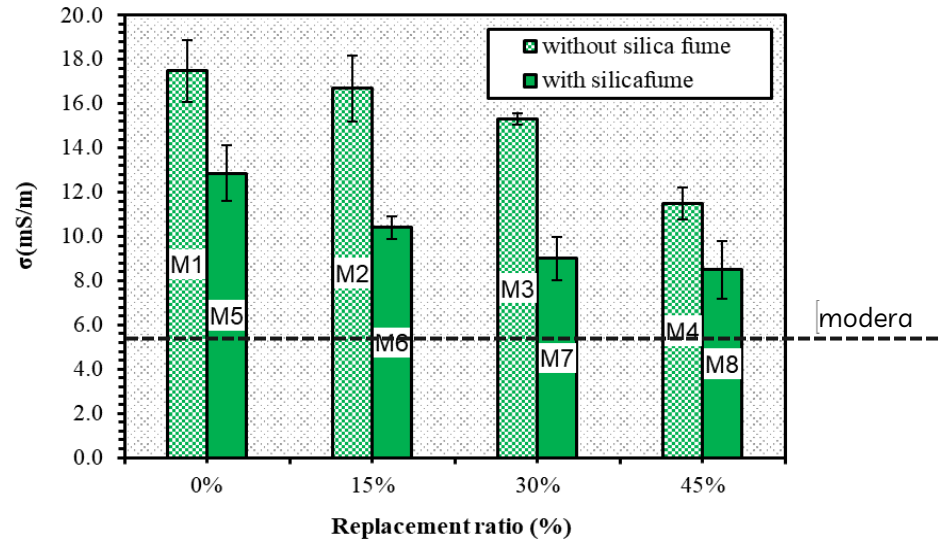


Figure 35: Electrical conductivity (mS/m)

4.4.8 Electrical resistivity

The results of the electrical resistivity, measured using the Proceq device and expressed in kilo-ohm centimeters ($k\Omega \cdot cm$), are presented in Figure 36, which shows a clear trend opposite to that of the electrical conductivity. i.e.: there is an increase in the electrical resistivity with the increase of plastic content. This is because the plastic is more electrical resistant material than other concrete constituents. In addition, the use of SF also increases the electrical resistivity. For instance, comparing M1 and M5 reveals that the mix with SF (M5) has higher resistivity than the one without (M1), M5 exhibits 2.5% higher electrical resistivity than M1. According to Ghoddusi and Saadabadi[130] , adding SF to self-compacting concrete significantly increases its electrical resistivity compared to other mixtures. This improvement is due to the accelerated pozzolanic reaction rate induced by the SF, which modifies the microstructure and reduces porosity. Similarly, Ceran [64]

research found that incorporating PVC PW into concrete also enhances its electrical resistivity. The trend of the resistivity is known to be opposite to that of resistivity and obtained results of resistivity in the present study follows the norm as expected.

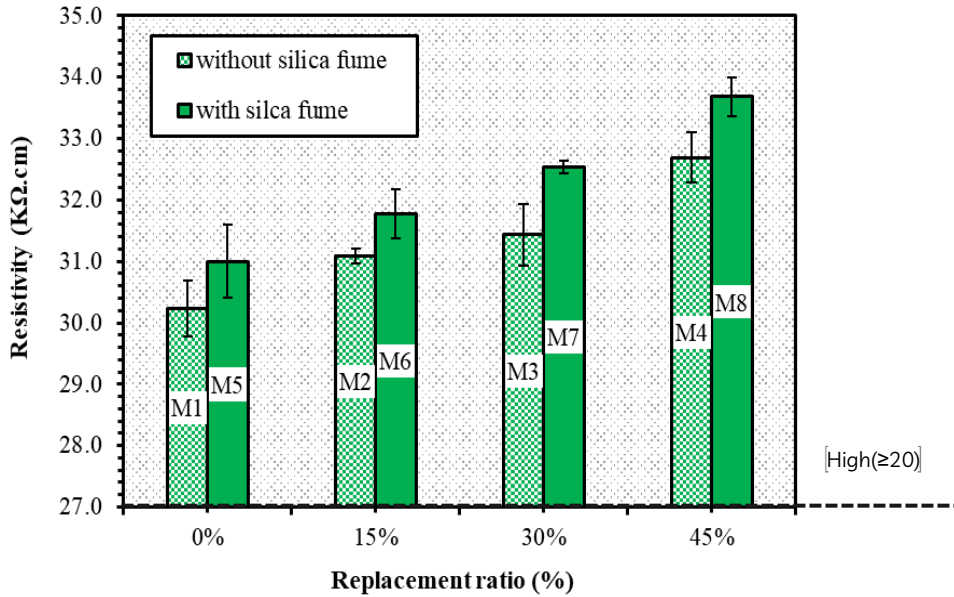


Figure 36: Electrical resistivity (KΩ.cm)

4.4.9 Thermal conductivity

Figure 37 illustrates the thermal conductivities of concrete mixed with recycled HDPE. The data reveals that the thermal conductivity values decrease as the volume fraction of plastic granules increases. This is because polymers like plastic inherently have lower thermal conductivity compared to natural sand, which contains a high percentage of conductive silica. These findings are consistent with results of Basha et al.[15,19] in their investigation on the use of PW to replace the natural CA. They are also in agreement with the findings of Senhadji et al[131], Jassim [132] and Uthaichotirat et al. [133]. In addition, the figure indicates that the use of SF results in a reduction of thermal conductivity. For example, when comparing mixes with and without SF, it is evident that SF further reduces

the thermal conductivity coefficient. Specifically, mixes with SF (M5-M8) exhibit reductions of 8.7%, 5.7%, 1.06%, and 5.3%, respectively, compared to mixes without SF (M1-M4). This is because the SF densifies the microstructure resulting in a reduction in less pores and consequently less thermal conductivity. A reduction in thermal conductivity means that the material is a good insulator, which helps reduce thermal stress and decreases heat transfer.

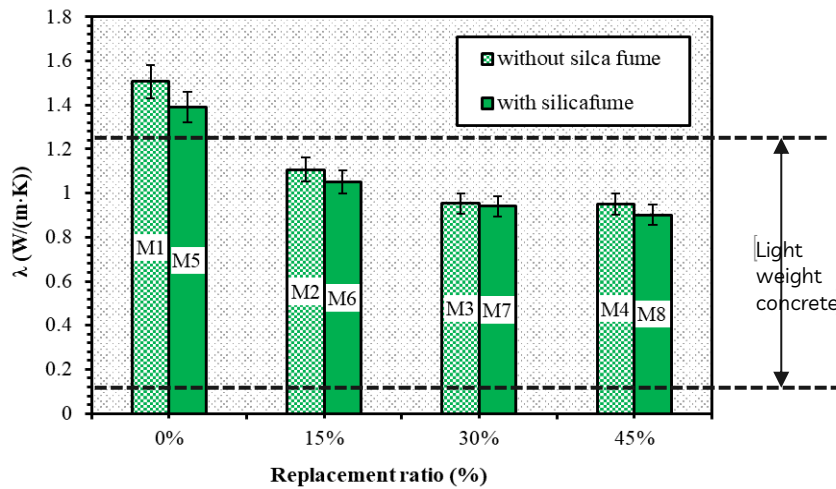


Figure 37: Thermal conductivity (W/(m.K))

4.4.10 Sound absorption

Figure 38 shows how SF affects the sound absorption of concrete at different replacement levels of PW granulate (HDPE), specifically at 0%, 15%, 30%, and 45%. The results indicate that adding SF consistently improves sound absorption across all replacement levels compared to mixes without it. For example, at a 0% replacement level, adding SF increases the sound absorption by about 13%. This effect is even stronger at a 15% replacement level, with a 20% increase. At 30%, the improvement is still notable, with a

14% increase. However, at a 45% replacement level, the mix without SF (M4) has a sound absorption that is about 10% higher than the mix with SF (M8). This change at the 45% replacement level suggests that too much plastic causes the materials to separate, with HDPE particles gathering at the top of the concrete samples. This uneven distribution negatively impacts the evenness of the concrete mixture, which is key for optimal sound absorption. When materials separate at higher replacement levels, it reduces the expected benefits of increased porosity and weakens the bond between particles, ultimately resulting in a lower sound absorption than expected. Similar findings have been reported in other studies. Oancea et al.[65] observed a maximum increase of 28.5% in sound absorption when replacing aggregates with PET granules due to increased air gaps and reduced density, which helps dissipate sound. Loong et al.[66] also reported a 16% improvement in sound absorption with a 0.5% replacement of PET fibers, highlighting the role of optimal material distribution. Likewise, Manaf et al.[67] found that the sound absorption increased to 0.42 with a well-balanced blend of HDPE and polystyrene, attributing the improvement to better air gaps and particle distribution. A higher sound absorption coefficient means that more sound energy is absorbed, which is beneficial for noise reduction

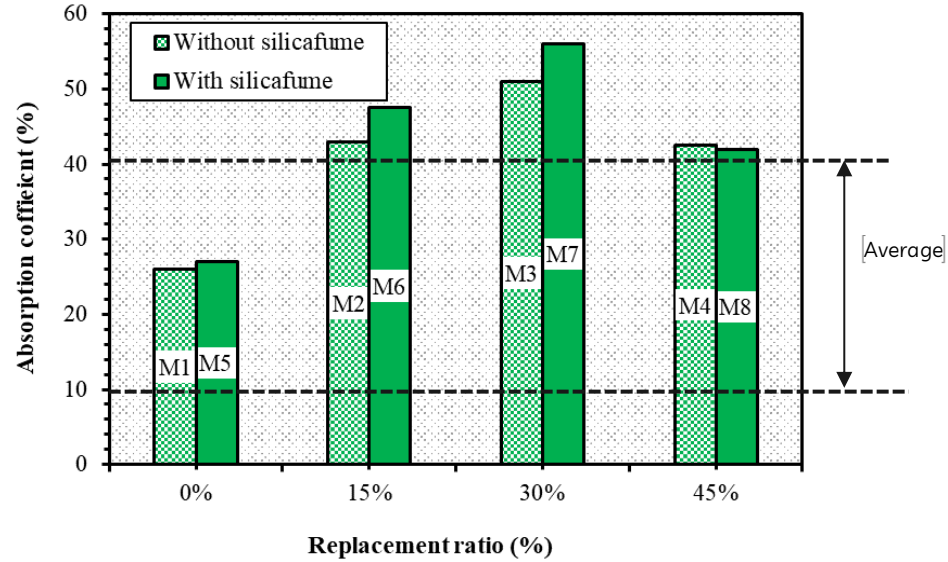


Figure 38: Sound absorption coefficient (%)

4.5 Sustainability and cost analysis

4.5.1 Embodied carbon dioxide (Carbon footprint estimation)

Embodied carbon refers to the carbon dioxide emissions associated with the construction process and the materials used throughout the entire lifecycle of infrastructure or buildings. This includes CO₂ generated during the extraction, transportation, and manufacturing of materials. By 2060, it is expected that building construction will double, benefiting concrete producers but significantly increasing CO₂ emissions. Concrete, the most widely produced human-made material, is particularly impactful, with cement alone accounting for 7% of global CO₂ emissions, making it the largest contributor to embodied carbon in the built environment. Table 6 shows the carbon emissions of concrete mix constituents. As illustrated in Figure 39, the general trend of embodied carbon dioxide for all concrete mixes slightly increases with the increase of HDPE replacement ratio. This is because plastic, even recycled, has a high embodied carbon value of 0.24 kg CO₂ per kg. However,

the increase in the embodied carbon is insignificant and found to be less than 10 % for case of the highest HDPE replacement ratio considered. Despite the increase in embodied carbon, using recycled plastic in concrete mixes is considered an effective way to reduce pollution due to accumulation of PW, and it could eliminate harmful waste disposal [89,90].

Table 6: Embodied CO₂ of concrete mix components

Materials	Cement	SF	Superplasticizer	FA	PW	CA	RA	Water (Ltrs)
Embodied CO₂ for 1 kg (Kg.CO₂)	0.912 [138]	0.024 [57]	1.88 [138]	0.00493 [138]	0.24 [139]	0.004 [138]	0.00747 [138]	0.0008 [138]

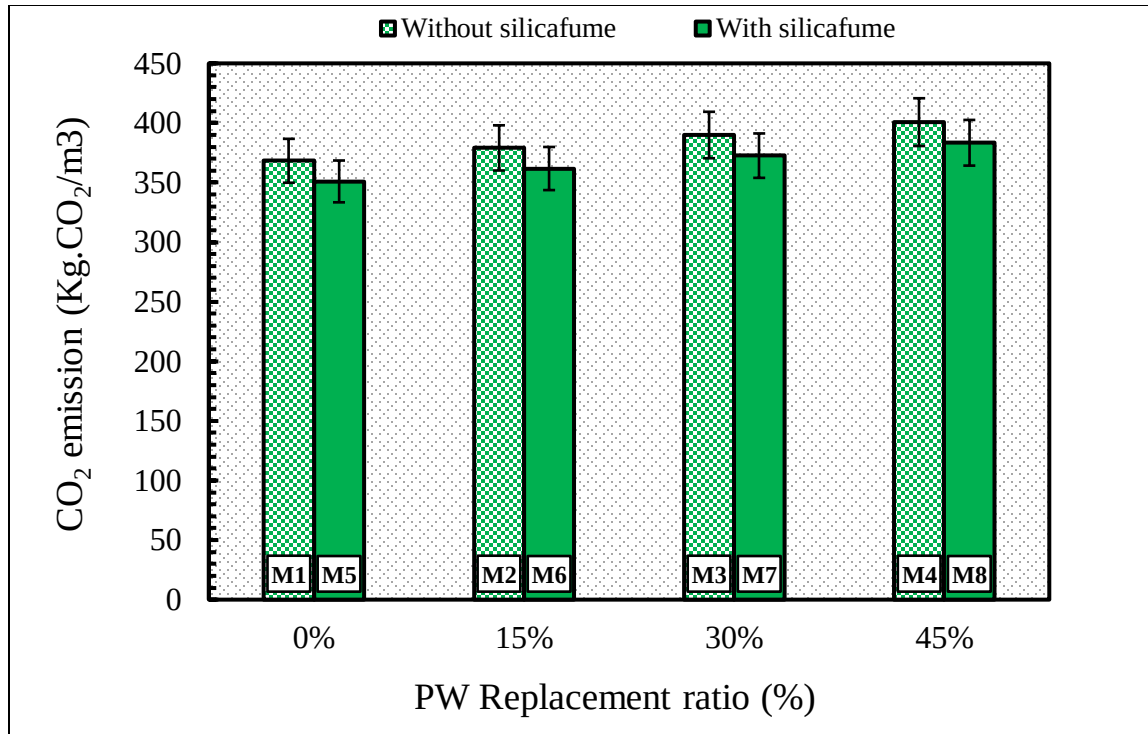


Figure 39: Concrete mixes CO₂ emissions (Kg.CO₂/m³)

Comparing mixes without SF to those with SF reveals that the former has higher carbon dioxide emissions. Replacing 5% of cement, which has a high carbon footprint of 0.912 kg CO₂, with SF, which has a carbon footprint of 0.024 kg CO₂, significantly reduces emissions. Mixes M5 to M8 show lower CO₂ emissions than mixes M1 to M4 by 4.68%, 4.57%, 4.4%, and 4.32%, respectively. NAs have a lower embodied CO₂ compared to RAs, which have an estimated CO₂ emission of 0.00747 kg CO₂. Cement and plastic are the primary materials contributing to CO₂ emissions, meaning that increasing their use leads to higher CO₂ emissions, negatively impacting the environment.

4.5.2 Embodied energy

Embodied energy refers to the total energy required to produce a product or service. This concept is crucial for determining the effectiveness of energy-saving or energy-producing measures, as it impacts the cost of building due to energy inputs. It also helps in assessing whether a product contributes to reducing global warming and gas emissions. The primary purpose of measuring embodied energy is to compare the energy produced or saved by a product to the energy consumed in producing it. In the building sector, this includes the energy used in material production, transportation, and manufacturing [140]. Table 7 displays the energy required for each component in the concrete mix. As shown in Figure 40, the general trend indicates that embodied energy increases with a higher replacement ratio of HDPE. This suggests that plastic consumes more energy to be produced and recycled, with an estimated value of 18.6 MJ/kg. When comparing mixes without SF to those with SF, the latter shows lower embodied energy. SF has very low energy consumption, estimated at 0.1 MJ/kg. Consequently, mixes M5 to M8 consume 4.6%, 3.13%, 2.4%, and 1.9% less energy, respectively, than mixes M1 to M4.

Table 7: Embodied energy of concrete mix components

Materials	Cement	SF	Superplasticizer	FA	PW	CA	RA	Water (Ltrs)
Embodied Energy for 1 kg (MJ/kg)	4.53 [138]	0.1 [57]	9 [138]	0.02 [138]	18.6 [138]	0.1 [138]	0.08 [140]	-

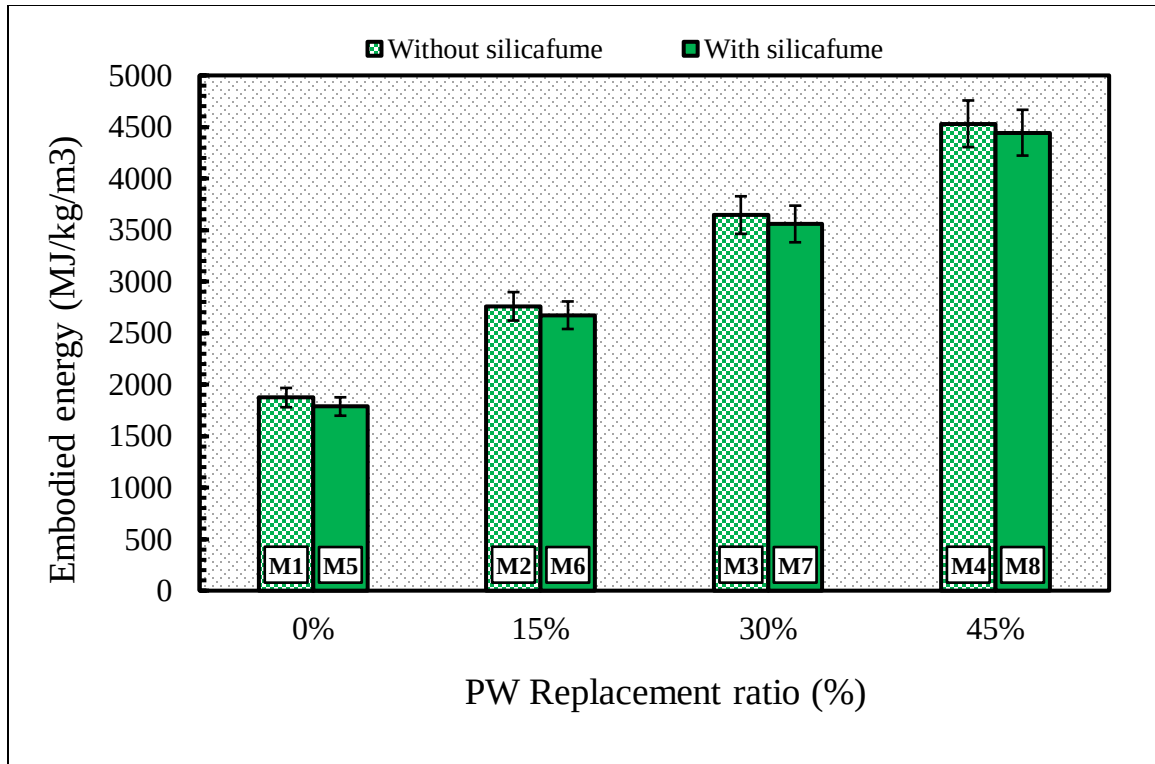


Figure 40: Concrete mixes Embodied energy (MJ/Kg/m3)

Materials with low embodied energy include NAs, which have lower values than RAs, estimated at 0.08 MJ/kg. Conversely, cement and plastic are the primary consumers of energy in the concrete mix. Increasing the content of these materials necessitates more energy, thereby negatively impacting costs and contributing to higher overall expenses.

4.5.3 Cost analysis

After finding concrete prices in Saudi Arabia, Table 8 below outlines the costs of each component in the mix. Among the replacement materials, plastic has a considerably high price at 2.50 SAR/kg, followed by SF at 3.00 SAR/kg. Superplasticizer and cement also have high prices, listed at 5.00 and 3.00 SAR/kg, respectively. Figure 41 illustrates that the overall cost the value increases with the addition of more PW, and this is observed across the eight mixture. When comparing mixes without SF to those with SF, the latter incurs a

minimal additional cost due to the 5% replacement ratio of SF, with an estimated maximum increase of up to 2%. The least expensive mix is the reference mix without plastic. However, the primary goal is not to reduce costs but to mitigate environmental impacts by using RAs and PW. Using plastic aggregates addresses environmental concerns associated with NAs, making concrete more sustainable. For instance, Attanasio et al. (2015) [141] found that replacing 10% of sand with recycled plastic could save 820 million tons of sand annually. In addition, there are other factors directly impacting the cost such as the use of plastics to replace the NAs reduces the overall weight of the building [142], leading to smaller foundation sizes and eventually reduce the cost [95].

Table 8: Cost of concrete mix components

Materials	Cement	SF	Superplasticizer	FA	PW	CA	RA	Water (Ltrs)
Cost for 1 kg (SAR)	2.78[143]	3	5	0.007	2.5[61]	0.022[143,144]	0.02[144]	0.005[61]

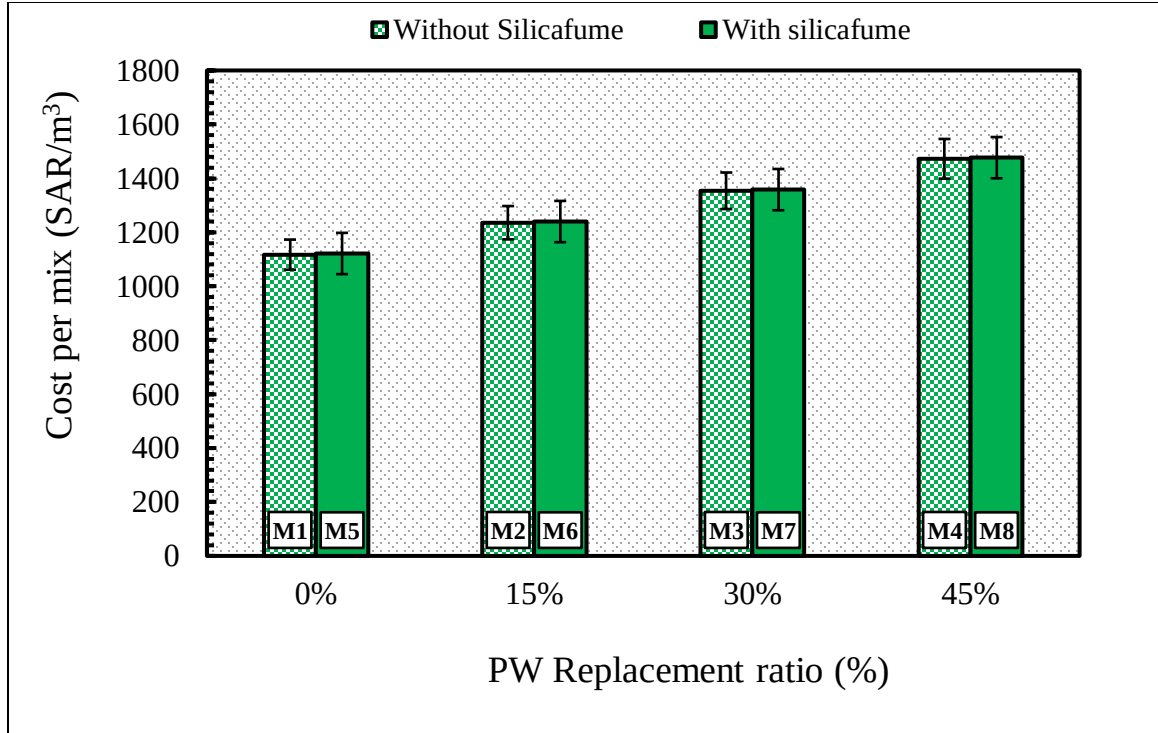


Figure 41: Concrete mixes cost (Saudi riyals /m³)

Despite the slight increase in the cost by adoption of plastic, there still a room for improvement and possibility for reducing the unit cost of concrete incorporating PWs. Changing the forms of plastics such as fibers, flakes, and granules could be one of the solutions because the cost of plastics varies depending on the manufacturing or recycling process. For instance, Siddique et al.[145] reported a cost reduction of 37% when using plastic bottle particles without recycling as CAs compared to gravel.

4.6 Conclusions

This chapter investigated the durability properties of a hybrid RAs concrete incorporating PWs and construction & demolition wastes. The test results reveal the following behaviors:

- Using plastic granules as FAs could not be feasible at 30% replacement or more as more plastic content tend to melt at temperature of 250°C and more.
- At 500°C, mixes with 15% HDPE reported 57 and 72.5% reduction in compressive strength for cases of without and with SF, respectively.
- There is a reduction in the density with the increase of HDPE content and the use of SF increases the density.
- The water absorption, both after immersion and after immersion followed by boiling, increases with the increase of HDPE content, and the SF reduces the water absorption.
- The porosity percentage increases with a higher replacement ratio of PW similar to the water absorption, and the use of SF reduces the porosity leading to a dense microstructure.
- The drying shrinkage increases with the increase of replacement ratio of PW for cases of both with and without SF.
- The migration coefficient decreases with the increase of replacement ratio of HDPE. This implies that chloride penetration is reduced as the HDPE ratio increases. Additionally, mixes with SF show lower migration coefficients compared to mixes without SF indicating that the inclusion of 5% SF effectively reduces chloride ion penetration.
- There is a considerable reduction in the electrical conductivity with the increase of the HDPE content and the mixes with SF show constantly lower electrical conductivity as compared to those without SF.

- The electrical resistivity is found to rise as the HDPE amount increases, and the mixes with SF show constantly higher electrical resistivity as compared to those without SF.
- The thermal conductivity values decrease as the volume fraction of plastic granules increases, mixes with SF show lower thermal conductivity values as compared to those without SF.
- Adding SF to concrete improves sound absorption up to a 30% replacement level of HDPE PW, with the most significant improvement (20%) at 15%. However, at 45% replacement, sound absorption decreases due to segregation of HDPE particles, which disrupts the mixture's uniformity and reduces its effectiveness.
- The sustainability and cost analysis indicate that the embodied carbon dioxide and cost per cubic meter slightly increase with the increase of HDPE content. This could be attributed to the adopted form of plastic (granules), which require relatively costlier and energy consuming operations as compared to flakes and other forms. However, there are still rooms for improvement and creating sustainable and cost-effective concrete.

Finally, while incorporating HDPE PW and SF in concrete offers both benefits and challenges, careful consideration of the proportion of these materials is crucial to optimizing the durability and sustainability of the concrete.

CHAPTER 5

Plastic wastes and recycled aggregates concrete-filled steel tubes

5.1 Background

This chapter investigates the use of (HRAC) incorporating PW and RCA in CFSTs. The same eight concrete mixes discussed in Chapter 3 were used to investigate the behavior of CFSTs in this chapter. A total of 19 steel tubes of similar dimensions were prepared. Each concrete mix was used to filled two steel tubes, resulting in a number of sixteen (16) CFSTs. The remaining three steel tubes are made without concrete to serve as a reference. The failure modes and load-axial shortening curves were investigated and the results are discussed. Furthermore, the experimental results are compared with design codes estimations to determine which design code provides the most accurate predictions.

5.2 Experimental investigation

The experimental program in this study began by selecting eight different concrete mixes. Each mix included a 60% replacement of NAs by RCA. The mixes were divided into two groups: M1 to M4, which did not contain SF, and M5 to M8, which included a fixed 5% SF as a replacement for cement (by weight). Within each group, the natural FAs were replaced by PW in the form of HDPE granules at volumetric ratios of 0%, 15%, 30%, and

45%. From each mix, two steel tubes were prepared resulting in a number of sixteen CFSTs. In addition, three tubes were left empty to serve as reference specimens.

The steel tubes had dimensions of 127 mm in diameter and 470 mm in height, with a thickness of 3 mm. The length-to-diameter ratio (L/D) was 3.78, and the diameter-to-thickness ratio (D/t) was 42.3, both of which remained constant across all the specimens due to the uniform dimensions of the circular steel tubes, as illustrated in Figure 42. Additionally, three coupon specimens were tested to obtain the stress-strain curve and determine the elastic modulus of the steel tubes, as shown in Figure 43.

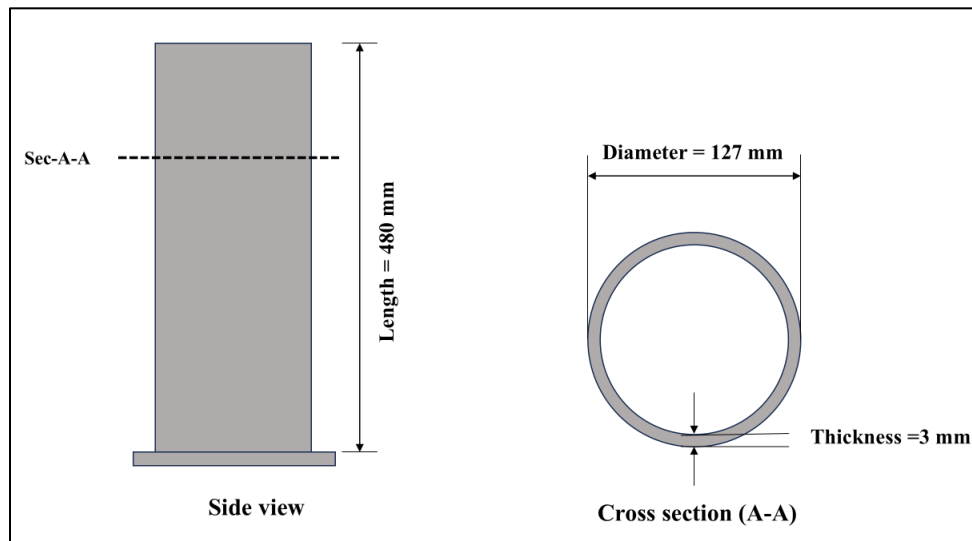


Figure 42: Steel tube dimensions (side view and cross section)

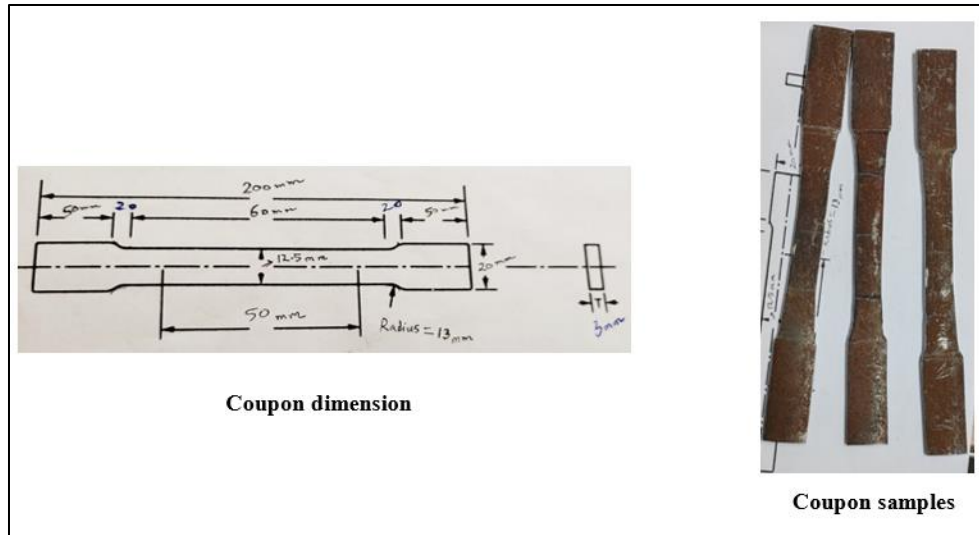


Figure 43: Steel coupon (dimensions and shape)

The mixes prepared and shown in Table 2 were used to fill sixteen (16) steel tubes. The CFST are designated by the mix designation followed by 1 or 2 for the first and second specimen, respectively. For example, M1-1 and M3-2 indicates the first steel tube made of concrete mix M1 and the second CFST filled with concrete M3, respectively. The designations of the CFST can be seen in Table 9. Additionally, all materials used in these mixes are discussed in detail in Chapter 3.

Table 9: CFST specimen identification and corresponding concrete mix designations

CFST ID	Adopted Concrete Mix	CFST ID	Adopted Concrete Mix
M1-1	M1	M5-1	M5
M1-2		M5-2	
M2-1	M2	M6-1	M6
M2-2		M6-2	
M3-1	M3	M7-1	M7
M3-2		M7-2	
M4-1	M4	M8-1	M8

5.3 Methodology

5.3.1 Preparation of mixes and design of specimen

The preparation process began by selecting steel tubes with the specified diameter. These tubes were initially longer (6.0 m) and required cutting to a length of 480 mm. Following the cutting, square steel bases measuring 200 x 200 mm and 8 mm thick were welded to one end of each tube. This procedure was repeated until 19 steel tubes were ready for filling with the newly developed concrete mixes. Prior to testing, three samples were set aside for reference. Next, all the materials mentioned earlier were carefully weighed and placed in separate containers. The mixing process began with the aggregates, which were mixed for one minute. The cement and SF, which had been pre-mixed in their dry state, were then

added to the mixer along with the aggregates. Water was gradually introduced while the mixer continued to operate. After a brief pause, the superplasticizer was added, and mixing resumed until the mixture was thoroughly homogenized. After the mixing was completed, the concrete was poured into the prepared steel tubes. These tubes were then transferred to the curing room, where the curing process involved wrapping the vertical surfaces of the tubes with wet burlap to keep them moist throughout the entire curing period. Samples of the CFSTs after casting are shown in Figure 44.



Figure 44: Concrete steel tubes samples after casting

5.3.2 Testing of Materials:

The materials used in this investigation are concrete and steel. To evaluate the compressive strength of the eight concrete mixes, three cubic samples (100 mm x 100 mm x 100 mm) were prepared for each mix, resulting in a total of 24 cubes. After 28 days of curing, the

compressive strength tests were conducted in accordance with ASTM C39 [100]. A steel coupon was extracted from the main steel tubes and then fabricated into an I-section or longitudinal flat tension specimen in accordance with ASTM A370 standard [146]. This preparation involved attaching a strain gauge at the center of each sample, with the gauge placement area smoothed using an iron softening sheet. Once prepared, the I-section samples were securely positioned in the Universal Testing Machine (UTM), where tension was applied until the samples fractured into two parts. Throughout this process, the loading and strain data were continuously recorded by a data logger. This data was then used to generate and plot the stress-strain curve, allowing for the determination of ultimate and yield strengths as well as the elastic modulus, Figure 45 shows the process of the tensile strength test of the coupon specimens.

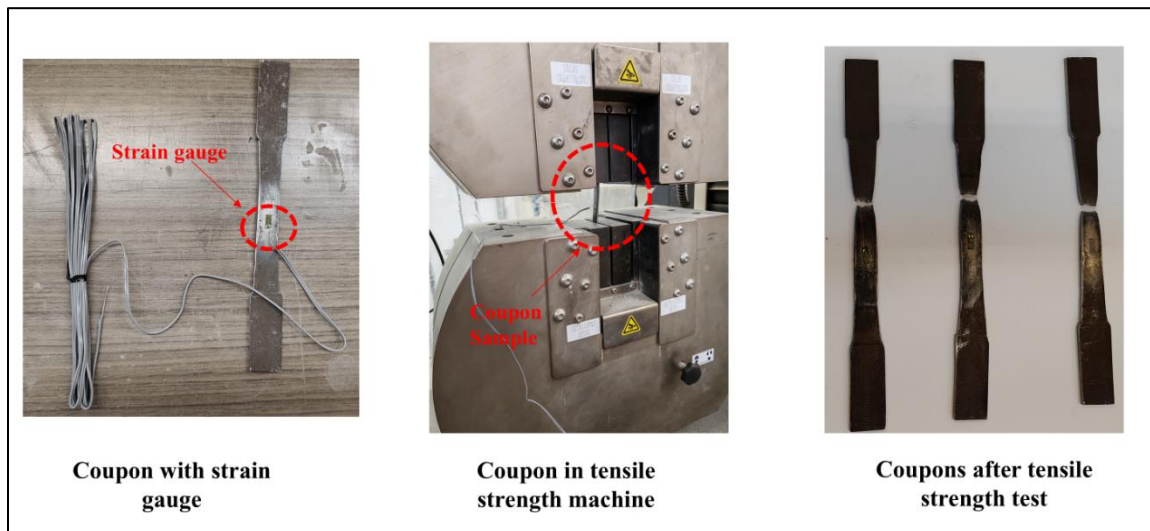


Figure 45: Steps for testing Coupons in tensile strength machine

5.3.3 Testing of Steel Tubes and CFSTs:

After the 28-days curing of the CFST, the empty steel tubes and the CFSTs were transferred to the compression testing room, which is equipped with a 5000 kN capacity compression machine. Testing began with the reference steel tubes, which were evaluated without any concrete filling. Two Linear Variable Differential Transformers (LVDTs) were attached one at the top to measure the end shortening of the tubes or CFSTs. The samples were then subjected to compression load until failure, with the load and shortening data recorded by the data logger. This recorded data was subsequently used to generate load-shortening curves for all the specimens. Figure 46 and Figure 47 illustrate the compression room, the testing process following ASTM standards, and examples of CFSTs during testing.

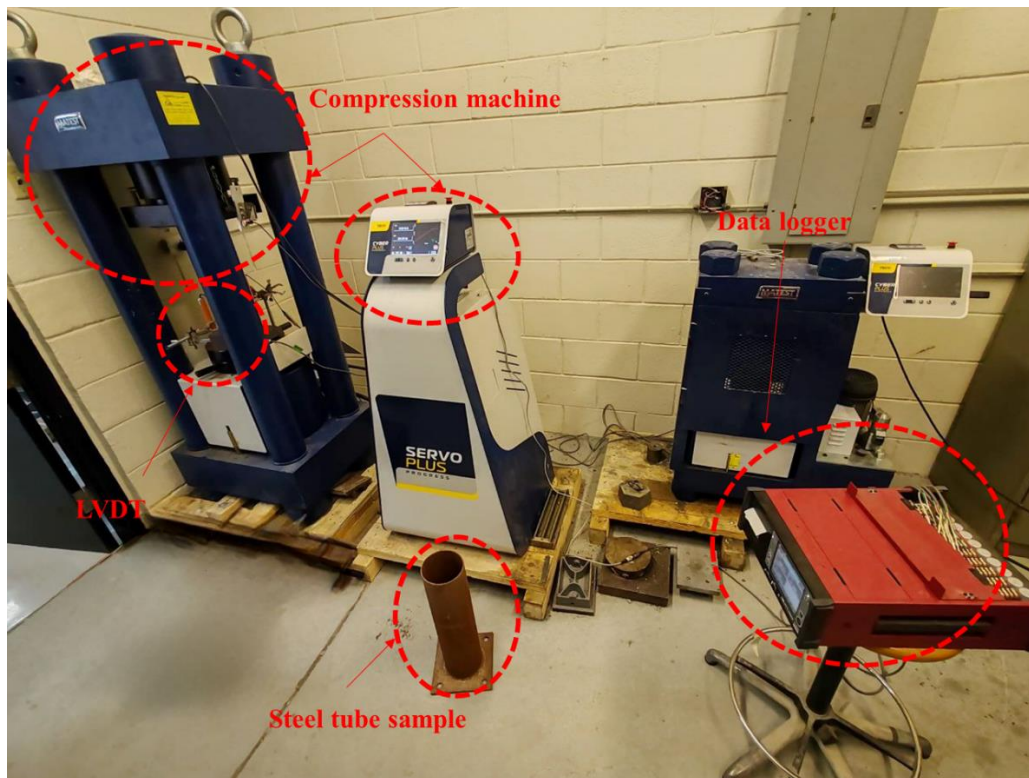


Figure 46: Compression testing room

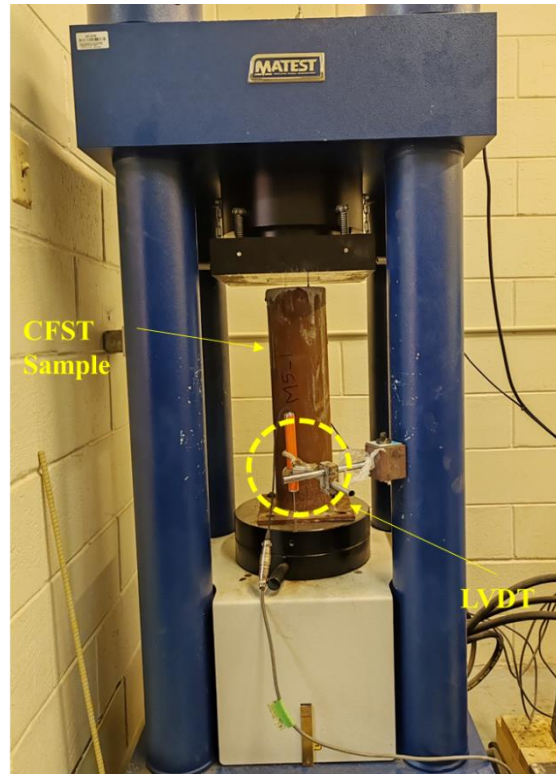


Figure 47: CFST sample under compression testing

5.4 Result and discussion

5.4.1 Mechanical Properties of Steel

This test aimed to determine the essential mechanical properties of the steel, including elastic modulus, ultimate strength, and yield strength. The testing process began with the placement of the coupon specimen in the Universal Testing Machine (UTM). While the outer 50 mm of the specimen functioned as a holder, the actual testing was conducted on the central 200 mm section. Figure 48 displays the results as graphs for the three tested coupons, which demonstrated consistent behavior in terms of yield strength, ultimate strength, and elastic modulus. The average values obtained were a yield strength of 358 MPa, an ultimate strength of 387 MPa, and an elastic modulus of 209 GPa , confirming

that the steel met the specified standards (EN 1993-1-1[147], ASTM A992[148], AISC 360 [149], ISO 630-1[150]).

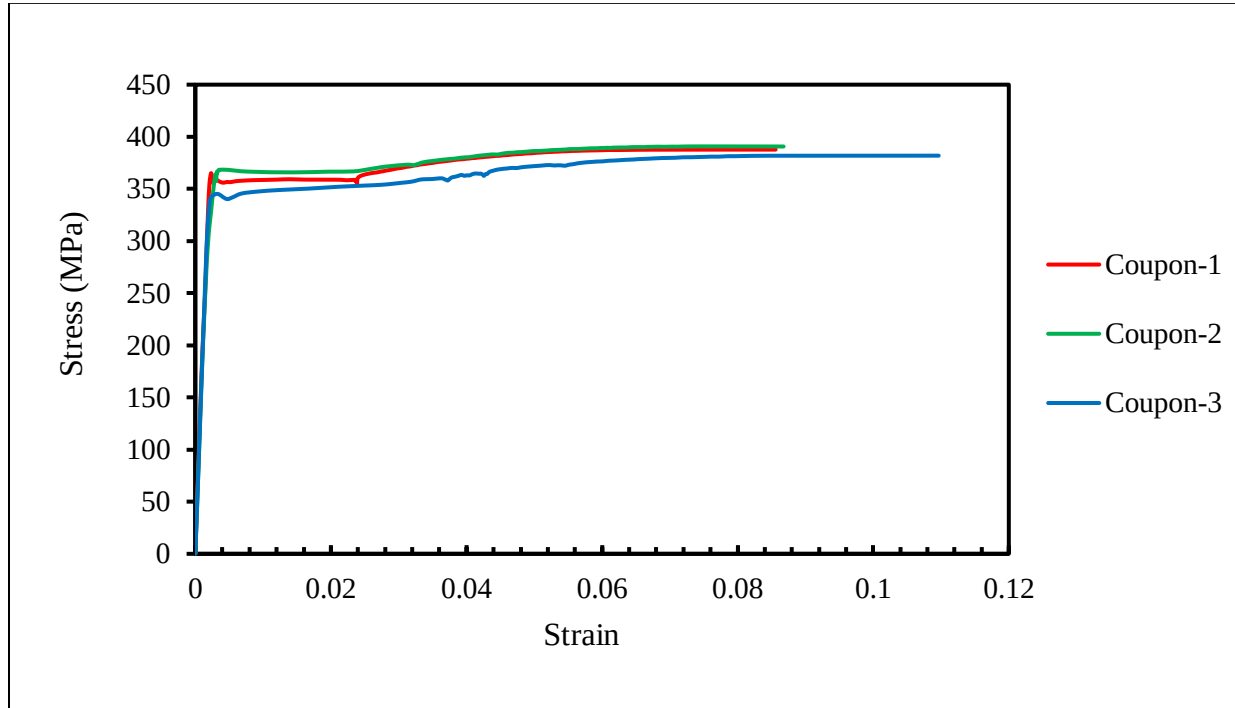


Figure 48: Stress strain curves for tested coupons

5.4.2 Failure modes

Based on the experimental observations made during testing of the cylindrical steel sections and CFST specimens of the present study, different types of failure mechanisms have been noticed. Starting with the reference specimens, the empty steel tubes undergo plastic buckling and forming what is known in the literature as elephant-foot buckling. This is because the empty tube behaves like a cylindrical shell and due to the small radius-to-thickness ratio, plastic buckling controls the behavior [151–153]. Figure 49 shows the deformed shape of the three empty steel tubes where the top of the specimens deformed laterally making an elephant-foot like shape. This deformed shape formed at the top because the compression testing machine used for testing has the upper platen plate fixed

and the lower one moves upward during testing. Similar elephant-foot buckling was noticed in the intact steel tubes specimens reported in the works of Ghazijahani et al. [154,155].



Figure 49: Failure modes of the empty steel tubes

The failure modes of the CFSTs specimens made with the concrete mixes from M1 to M4 (without SF) is shown in Figure 50. The deformed shape in these specimens is different than the localized plastic elephant-foot buckling observed for cases of the empty steel tubes (Figure 49). This is because the infilling concrete help in delaying the local buckling [156,157]. It can also be observed from Figure 50 that there is an increase in the deformation from with changing the mixes from M1 to M4. These concrete mixes have PW content from 0% to 45% in the order of 15%. Thus, the deformation becomes noticeable with the increase of PW content. For the first two specimens, for example, experienced outward local buckling in the middle in addition to a little deformed shape at

the top far less than for the case of empty steel tubes. The outward local buckling is very common in steel CFST as can be seen in the work of Jiang et al. [158] and Diao et al. [159]. Other cases like M2-1&2 and M3-1&2 follow similar buckling behavior to M1-1&2 with larger deformations. In the case of specimens M4-1&2, which contains large amount of PW, the distribution of the PW granules was not uniform along the height of the CFST due to the less specific gravity of PW making the PW granules float to the surface of the CFST. This consistency of the mix has been solved in the second group of mixes where SF was added to improve the consistency of the fresh concrete. The CFSTs of M4-1&2 followed a drum-like shape at the upper portion. This appears to be because the relatively weak concrete does not have sufficient stiffness to resist the vertical load and it has good capacity to prevent any inward local buckling.



Figure 50: Samples after testing for Mixes without SF

The deformed shapes for the CFSTs made of concrete mixes containing SF are shown in Figure 51. They followed nearly similar behavior to the mixes without SF with two exceptions. First, M6-2 experience premature failure characterized by the rupture of the weld. This could be attributed to the insufficient weld at the joint of steel tube. It is therefore reasonable to ignore the deformed shape of this specimen from comparisons of the deformed shapes. The second exemption is that the deformed shape of M8-1&2 is different than that of M4-1&2. This is because the SF has improved the consistency of the fresh concrete [23] and no drum-like shape is noticed for case of M8-1&2. Therefore, except M6-2, all the CFSTs in this category followed primarily outward local buckling near the middle of the CFST with little deformation near the top of the CFST.



Figure 51: Samples after testing for Mixes with SF M5 to M8

5.4.3 Load deformation curves

Figure 52 presents the results of the axial test conducted on a reference sample without concrete, indicating an average load-carrying capacity of 307 kN. The graphs in Figure 53 indicate that for mixes without SF, there is a general reduction in the axial load as the PW substitution ratio increases. However, an exception is observed in M3, which exhibits a higher axial load compared to M4 and M2. This could be attributed to the higher compressive strength of M3 as previously discussed in Section 4.1. Furthermore, the

deformation values at which the maximum loads were achieved are comparable for cases of M1, M2, and M3, while M4, which has 45% PW, shows no well-defined maximum load and the specimens undergo considerable deformation. This could be linked to the drum-like deformed shape of the specimen shown earlier in Figure 50. The PW granules concentrated more on the upper part of the tube making the upper portion weaker than the lower part. In comparison with the reference tube without concrete, the increase in the load carrying capacity is approximately three times greater for each of the four concrete mixtures.

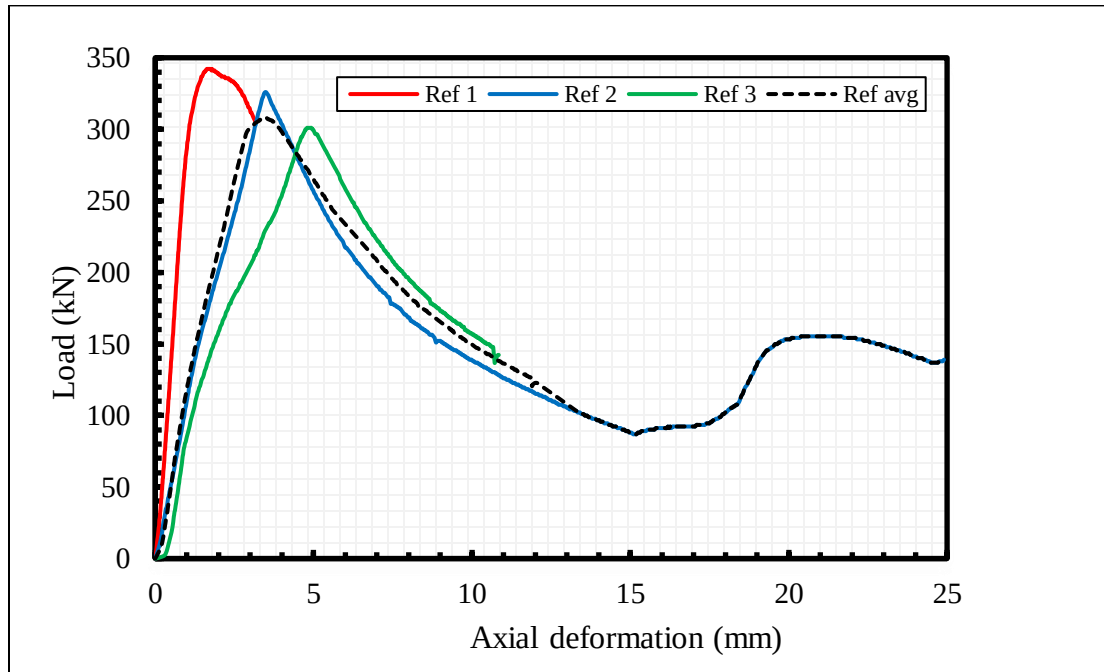


Figure 52. Load deformation curve for reference samples

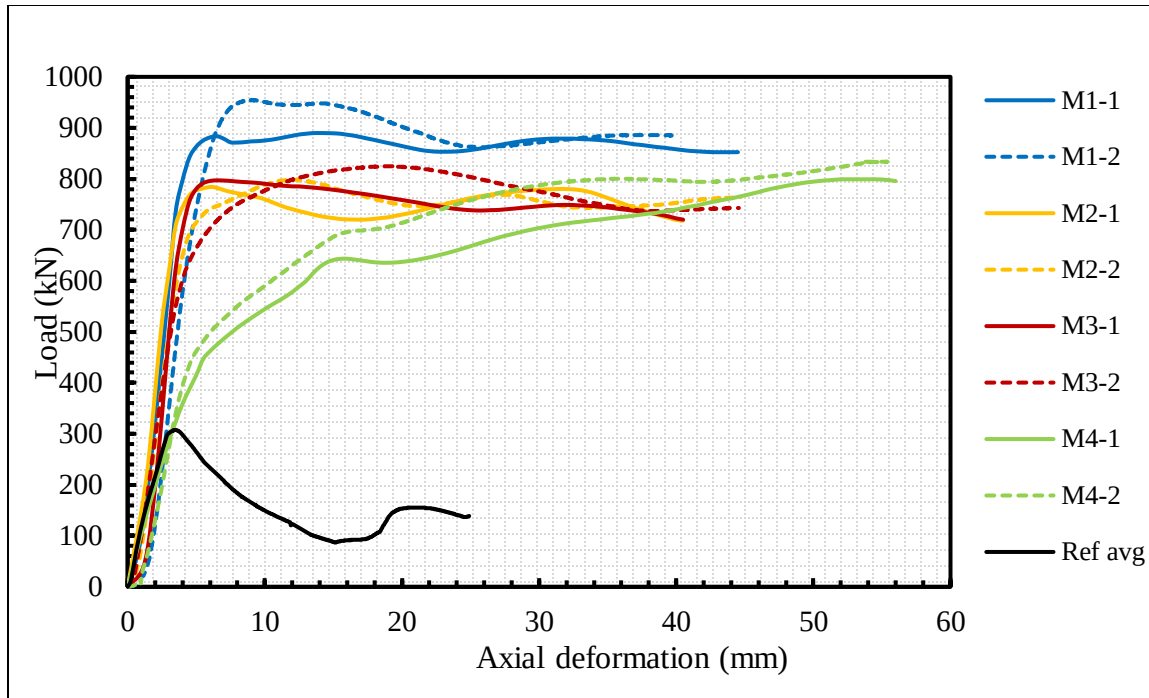


Figure 53. Load Deformation Curve for mixes without SF from M1 to M4

The results of the CFSTs specimens made of concrete mixes that contain SF are shown in Figure 54, which shows a similar trend to those without SF, where axial compressive force decreases with increasing of PW content. It is important to highlight that up to a 30 % replacement of PW, the primary trend remains a decrease in axial load as the PW content increases. This is true if M6-2 is omitted from the comparison as this specimen experienced rupture of the weld as shown by the its failure mode in Figure 51 and its load-shortening curve should be excluded. If the average of the M8-1&2 curves is considered, it will be found that it has slightly higher axial load than the average of M7-1&2, but the difference is small and can be tolerated. Notably, M8-1&2 despite having similar PW content to that of M4-1&2, the axial load-end shortening curve is comparable to their counterparts in the same mix (M5-1&2 to M7-1&2) and they show a well-defined maximum load. This could be attributed to the positive impact of the SF in improving the consistency of the mix

making the PW granules evenly distributed along the height of the tube. This role of the SF in improving the consistency of concrete incorporating PW can be seen in other works of the authors [23].

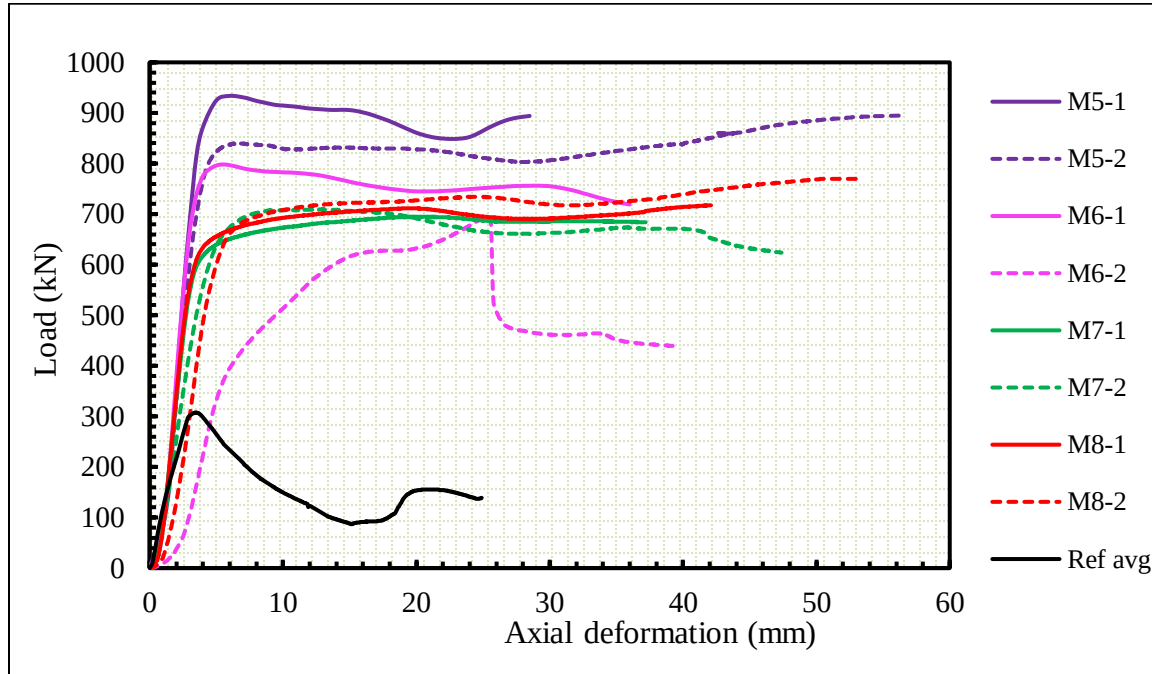


Figure 54. Load deformation curve for mixes with SF from M5 to M8

To further compare the role of the added 5% SF, each sub-figure shown in Figure 55 (a) to (d) compares two series of mixes, one with SF and one without. In each graph, two mixes with the same PW replacement ratio are contrasted, in terms of the maximum load the CFST can carry, there is no clear distinguish trend. For instance, the comparison between M1-1&2 and M5-1&2 as shown in Figure 55 (a) indicates that M1-1&2 perform slightly better than M5-1&2, which deviates from the expected trend in compressive strength. The noticeable role of the SF observed is that it helped in achieving the maximum load at a smaller deformation and the trend gets more noticeable with the increase of the PW content.

For instance, M8-1&2 achieved their maximum load at a shortening of less than 8 mm while their counter parts M4-1&2 did not reach their maximum load even at 40 mm shortening. Despite the maximum load for cases of M4-1&2 is higher than M8-1&2, no actual structure can be designed to tolerate such large deformation as shown by the drum-like deformed shape of M4-1&2 in Figure 50. The nominal resistance in such cases can be limited to the load at the maximum tolerable shortening [160].

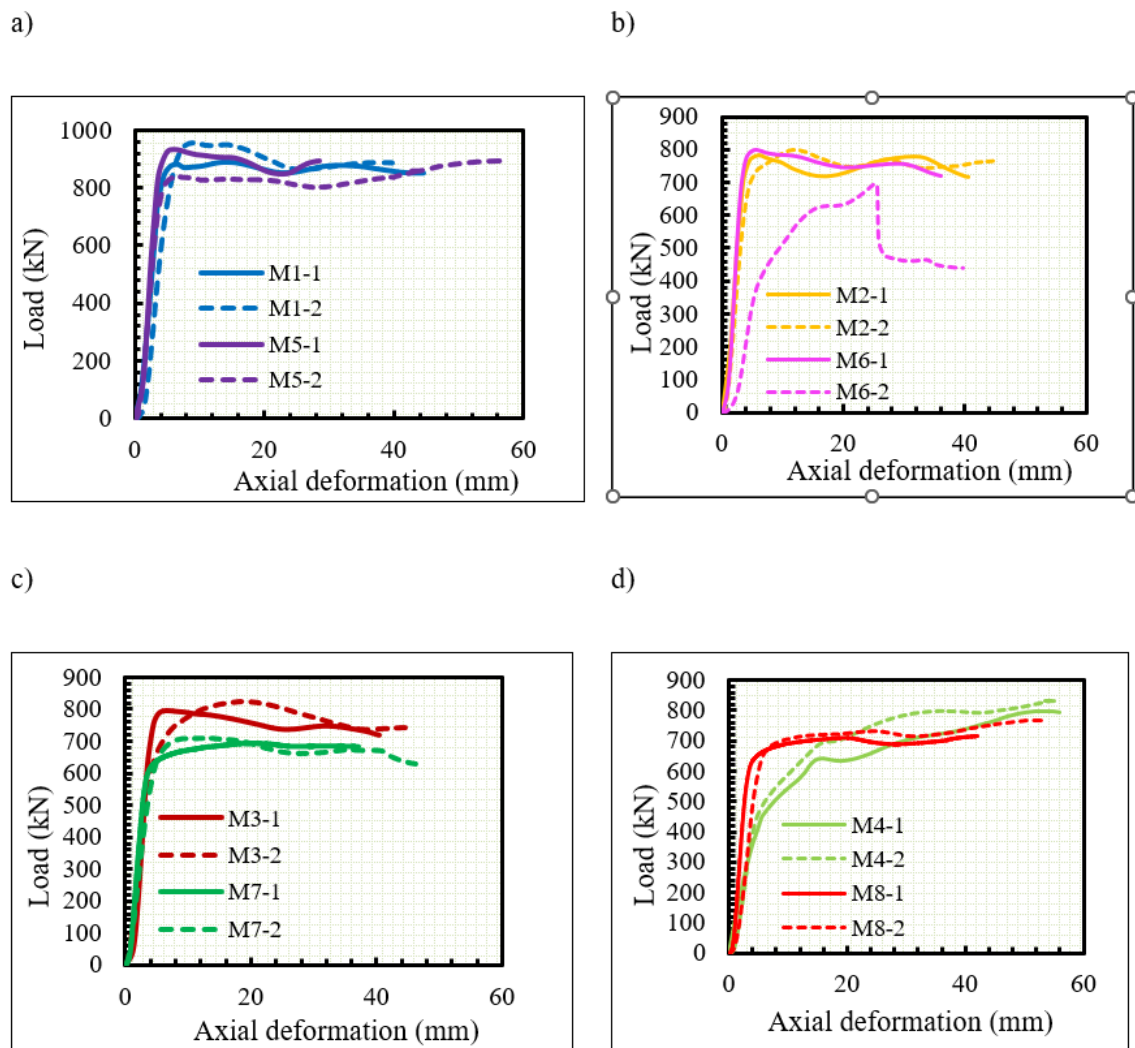


Figure 55. Comparative load-deformation curves for plastic-replaced mixes with and without SF

The reduction in the load-carrying capacity of CFSTs with the increase of the PW content can be attributed to several factors. First, the lower stiffness of PW and its weak bonding with the cement matrix, which lead to low compressive strength and low elastic modulus. The smaller the elastic modulus of the concrete the higher the load proportion taken by the steel, leading to yielding of the steel first before redistribution of the load to the concrete. Secondly, the low specific gravity of PW (less than that of water (1.0)) allows the plastic to segregate and move to the upper part of the steel tube making uneven distribution of PW across the height of the specimen. This makes the steel at the upper portion of the CFST carrying more loads initially, then undergo plastic buckling as if there no concrete. This behavior has been encountered for cases of high plastic content without SF (M4-1&2). The addition of 5% SF has made the mix more consistent and better distributed along the height, leading to a well-defined maximum load at considerably lower end shortening as compared to the cases without SF.

5.4.4 Axial load capacities of CFSTs

To better discuss the axial load capacity, the maximum load for each specimen should be identified. As shown in the previous section that M4-1&2 exhibit increase in the axial load with the increase of deformation without well-defined maximum load, and the maximum tolerable deformation should govern in this case. Since the maximum allowable shortening depends on the application and supporting conditions, it has been assumed in this study that the allowable shortening is 3.0% of the column height (14.0 mm). For each two CFSTs made of the same mix the average maximum axial load is taken and referred to as M_i^* where the letter (i) is the number of the mix (For example, $M1^*$ is the average of M1-1&2). For case of M6-1&2, however, $M6^*$ represents the maximum load of M6-1 only as M6-2

experienced unexpected premature rupture (Figure 51) of the weld and should be omitted from the comparison. Figure 56 represents the average axial load at a deformation of 3.0 % of the sample's height. The data indicates that the general trend in axial loading decreases as the PW content increases in the groups of mixes, with and without SF.

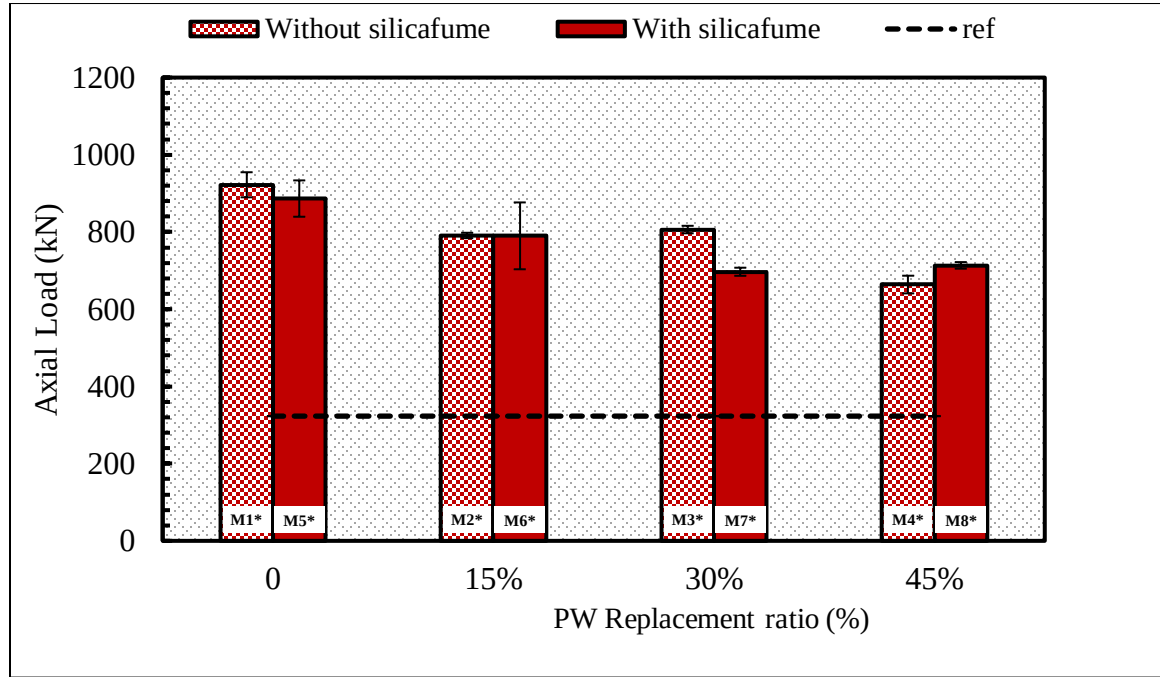


Figure 56: Average axial load vs replacement ratio of PW

One of the indicators of the effectiveness of CFST in load redistribution is the dimensionless metric, (SI), quantifies the increase in strength seen in CFST resulting from concrete confinement. The (SI) is calculated as the percentage between the axial capacity of CFST, derived from experimental data, and the combined capacities of its concrete and steel components. This index can be calculated using the following equation:

$$SI = \frac{P_e}{A_c f_{cu} + A_s f_y} \quad (1)$$

where P_e is the experimentally obtained load carrying capacity; A_c represents the cross-sectional area of the concrete core; f_{cu} refers to the compressive strength of the concrete cube; A_s and f_y denote the area and yield strength of the steel, respectively. Figure 57 illustrates the average (SI) for all CFST specimens. For the specimens made concrete mixes without CFST, the SI values range between 0.86 and 1.03, with only M1* exceeding 1, while the others fall below this threshold. In contrast, for the mixes containing SF, the SI ranges from 0.88 to 0.96, with M8 reaching 0.96, though all values remain below 1.

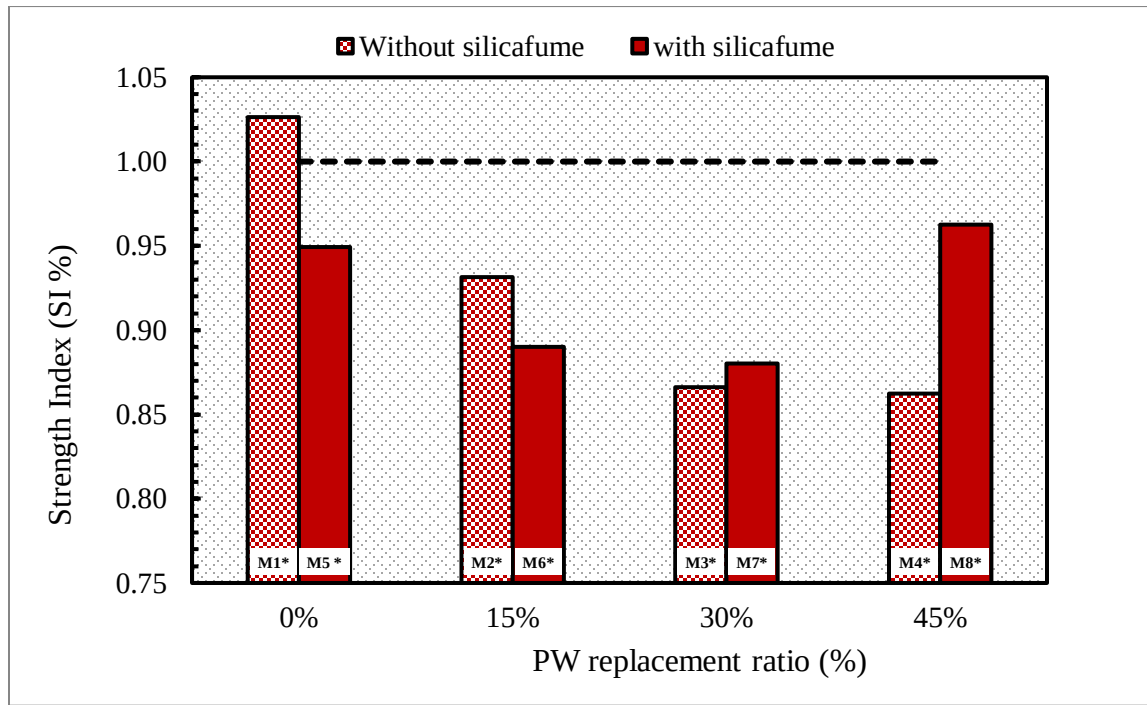


Figure 57: SI vs replacement ratio of PW

To provide further clarification, Table 10 shows the contributions of concrete (P_c) and steel (P_s) to the total load-carrying capacity of the CFST. By determining P_c and P_s , it becomes clear that an increase in PW content leads to a decrease in compressive strength. This decrease adversely affects P_c , suggesting that a higher PW content reduces the load-bearing capacity of the concrete core. Additionally, it was observed that at 30% and 45% PW, the

distribution is not uniform, which further negatively impacts the concrete, resulting in steel bearing a greater proportion of the load. Thus, the steel's contribution becomes more significant relative to the concrete.

Table 10: The roles of steel and concrete in axial load-bearing capacities

	Specimen	f_{cu} (Mpa)	P_e - (kN)	A_c (mm ²)	A_s - (mm ²)	P_s ($f_y^* A_s$) (kN)	P_s - (%)	P_c ($P_e - P_s$) (kN)	P_c - (%)	SI
0% SF	M1-1	41.76	889.85	11499.01	1168.67	418.38	47.02	471.47	52.98	0.99
	M1-2	41.76	954.80	11499.01	1168.67	418.38	43.82	536.42	56.18	1.06
	M2-1	37.48	784.05	11499.01	1168.67	418.38	53.36	365.67	46.64	0.92
	M2-2	37.48	798.00	11499.01	1168.67	418.38	52.43	379.62	47.57	0.94
	M3-1	44.59	796.70	11499.01	1168.67	418.38	52.51	378.32	47.49	0.86
	M3-2	44.59	816.00	11499.01	1168.67	418.38	51.27	397.62	48.73	0.88
	M4-1	30.57	641.00	11499.01	1168.67	418.38	65.27	222.62	34.73	0.83
	M4-2	30.57	686.80	11499.01	1168.67	418.38	60.92	268.42	39.08	0.89
5% SF	M5-1	44.83	933.63	11499.01	1168.67	418.38	44.81	515.25	55.19	1.00
	M5-2	44.83	839.50	11499.01	1168.67	418.38	49.84	421.12	50.16	0.90
	M6-1	40.80	790.00	11499.01	1168.67	418.38	52.96	371.62	47.04	0.89
	M6-2	40.80	616.80	11499.01	1168.67	418.38	67.83	198.42	32.17	0.69
	M7-1	32.49	686.50	11499.01	1168.67	418.38	60.94	268.12	39.06	0.87
	M7-2	32.49	707.50	11499.01	1168.67	418.38	59.14	289.12	40.86	0.89
	M8-1	28.08	705.00	11499.01	1168.67	418.38	59.35	286.62	40.65	0.95
	M8-2	28.08	722.00	11499.01	1168.67	418.38	57.95	303.62	42.05	0.97

5.5 Comparison of experimental results with design codes

5.5.1 Code provisions

B.1 Euro code (EC4):

The EC4 [161] code suggests that for circular CFSTs subjected to concentric loading, the axial capacity of the column is notably enhanced by the confinement effect, especially when the slenderness ratio is considered. The following equation, reported in EC4, describe the strength of CFSTs:

$$P_{EC4} = \eta_s f_y \frac{A_s}{\gamma_s} + \left(1 + \eta_c \frac{t f_y}{D f'_c} \right) f'_c \frac{A_c}{\gamma_c} \bar{\lambda} \quad (2)$$

Where:

$$\eta_s = 0.25(3 + 2\bar{\lambda}) \leq 1$$

$$n_c = 4.9 - 18.5\bar{\lambda} + 17\bar{\lambda}^2 \geq 0$$

$$\bar{\lambda} = \sqrt{\frac{N_{ns}}{N_{cr}}}$$

$$N_{ns} = f'_c A_c + f_y A_s$$

$$N_{cr} = \frac{\pi (EI)_{eff}}{L_e^2}$$

$$(EI)_{eff} = 0.6E_c I_c + E_s I_s$$

$$E_c = 22000 \left(\frac{f'_c + 8}{10} \right)^{0.3} MPa$$

In the EC4 prediction model, several key terms are used to describe the factors influencing the strength and stability of the composite CFST columns. Specifically, η_s and η_c represent the reduction factor for steel and the enhancement factor for concrete, respectively. The term $\bar{\lambda}$ indicates the relative slenderness of the column, while N_{ns} and N_{cr} correspond to the plastic resistance and the critical buckling load of the column. The effective length of the column is denoted by L_e , and $(EI)_{eff}$ refers to the effective flexural stiffness. Additionally, E_c and E_s are the modulus of elasticity for concrete and steel, f'_c is the compressive strength of concrete, and f_y is the yield strength of steel. Typically, the partial safety factors are designated as 1 for steel γ_s and 1.3 for concrete γ_c . Finally, the second moments of area for concrete and steel are represented by I_c and I_s , respectively.

B.2 American Concrete institute (ACI 318-14) and Australian code (AS-3600)

The axial capacity of a CFST using the ACI 318-14 [162] and AS-3600 [163] is determined using Eq. (3), which combines the contribution of steel with 85% of the concrete's contribution. The 15% reduction in the concrete's contribution accounts for uncertainties associated with concrete performance. It is important to note that the AS-3600 code uses the same equation as the ACI 318-14 code for this calculation. Thus, it will be referred to both of them as ACI/AS hereinafter.

$$P_{ACI/AS} = 0.85 f'_c A_c + f_y A_s \quad (3)$$

B.3 Architectural Institute of Japan (AIJ)

In the Japanese standard for calculating the maximum load capacity of CFST, the steel's contribution is increased by a factor related to confinement $(1 + \eta)$ by using following equation:

$$P_{AIJ} = 0.85 f_c A_c + (1 + \eta) f_y A_s \quad (4)$$

where this equation for $\frac{L_k}{D} \leq 4$, η represents a constant value of 0.27 for round cross sections, while L_k indicates the column's effective length and D is the diameter of the CFST.

B.4 Chinese code for CFST (CD)

The ultimate strength of a CFST, according to the Chinese code, can be calculated using the following equation:

$$P_u = f_{scy} A_{sc} \quad (5)$$

Where:

$$f_{scy} = (1.212 + B\theta + C\theta^2)f_{ck}$$

$$A_{sc} = A_c + A_s$$

$$B = 0.176 \frac{f_y}{235} + 0.974$$

$$C = -0.1038 \frac{f_{ck}}{20} + 0.0309$$

$$\theta = \frac{A_s f_y}{A_c f_{ck}}$$

where f_{scy} , A_{sc} , and θ represent section the specified yield strength, the cross-sectional area of the combined materials, and the factor representing confinement, respectively. B and C represent coefficients that correspond to the respective contributions of steel and concrete. The term f_{ck} denotes the characteristic compressive strength of concrete cylinder, which is equal to 0.67 times f_{cu} where f_{cu} is the concrete's compressive strength.

B.5 American Institute of Steel Construction (AISC)

To calculate the ultimate strength of compact circular CFST, the AISC 360-16 specification provides the Eq. (6). This equation outlines the process for determining the ultimate strength.

$$P_{AISC} = P_{no} \left(0.658^{\frac{P_{no}}{P_e}} \right) \text{ when } \frac{P_{no}}{P_e} \leq 2.25 \quad (6)$$

where $P_{no} = 0.95 f_c A_c + f_y A_s$

$$P_e = \frac{\pi^2 (EI)_e}{L_e^2}$$

$$(EI)_e = C_3 E_c I_c + E_s I_s$$

$$C_3 = 0.6 + 2 \left(\frac{A_s}{A_c + A_s} \right) \leq 0.9$$

where P_{no} , and P_e represent the nominal capacity and plastic resistance of the column cross-section, respectively, and $(EI)_e$ denotes the effective stiffness of the composite cross-section.

5.5.2 Design Codes Prediction

This section focuses on evaluating existing methods for predicting the ultimate strength of CFSTs considering the material properties of concrete (compressive strength) and steel (yield and ultimate strength) tested in the present study. To assess these predictions, various design codes are compared to the experimental findings. It is important to note that these design codes, typically developed for conventional concrete, may not be fully applicable to the developed HRAC that incorporates PW and RCA without modifications. The analysis examines different models for estimating axial capacity, as outlined in several codes, including the Chinese Code [164], the AISC, or American institute of steel construction[149], Eurocode (EC4)[161], the American Concrete Institute (ACI)[162], the Australian Standard (AS)[163], and the Architectural Institute of Japan (AIJ)[165]. All equations used in this analysis are determined according to the ultimate limit state, incorporating safety factors excluded from the calculations.

Each code provides one single value for each pair of mixes Mi-1&2 as the material and geometrical data are similar for the two specimens in each pair. The load-carrying capacities calculated using each one of the considered codes are tabulated in Table 11.

Table 11: Predicted values of axial load for CFST by codes

Mixes	P EC4	P ACI/AS	P CD	P AIJ	P AISC
M1-1 and M1-2	874.4	826.6	782.3	857.9	863.0
M2-1 and M2-2	836.9	784.7	739.3	824.4	816.7
M3-1 and M3-2	892.9	854.2	810.9	880.0	892.0
M4-1 and M4-2	779.7	717.1	670.2	770.3	742.7
M5-1 and M5-2	899.8	856.6	813.4	881.9	895.8
M6-1 and M6-2	866.5	817.2	772.6	850.4	852.7
M7-1 and M7-2	796.5	735.9	689.3	785.4	763.5
M8-1 and M8-2	760.2	692.8	645.7	750.9	716.2

Comparison of the findings from the experiments on all the CFST specimens with the estimates of the design codes is shown in Figure 58, which shows the experimental axial capacities and corresponding prediction values from various design codes. The general trend that can be seen in the figure is that there is a decrease in the axial load-carrying capacity with the increase in the PW content. The EC4 generally provides higher estimates than other design codes considered. In most cases, this is followed by the AIJ code.

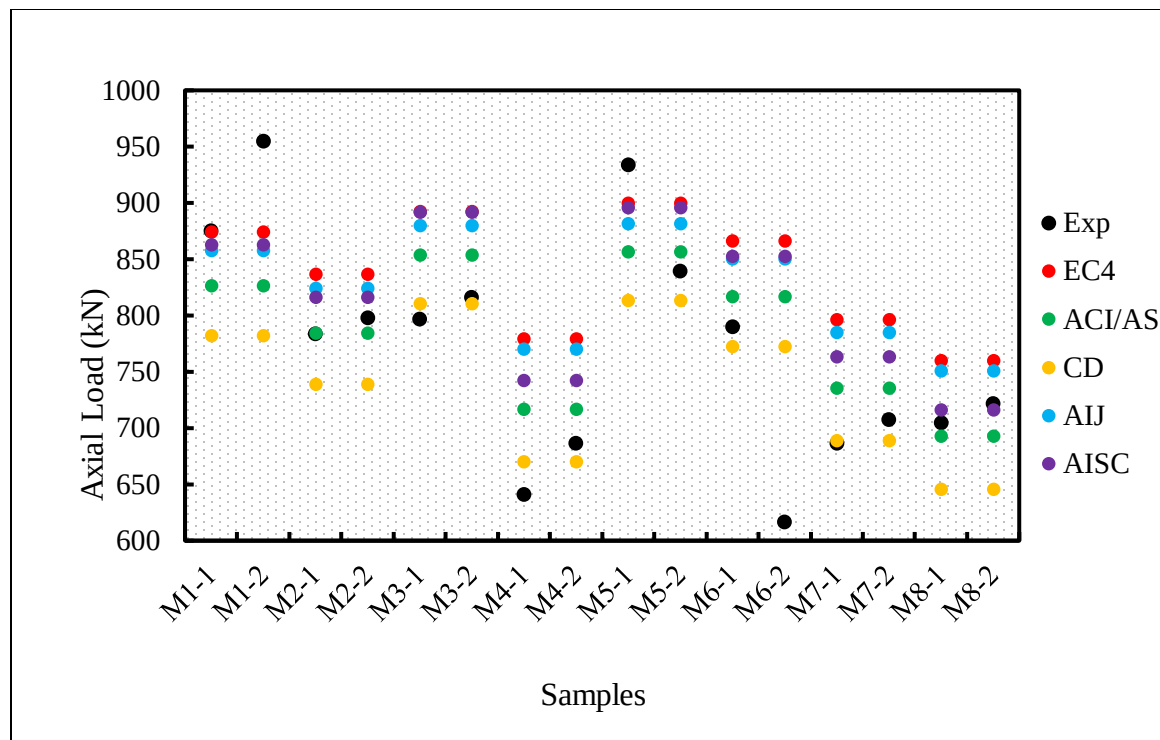


Figure 58: Experimental and predicted axial load capacities using different design codes

to deepen the comparison, the results from the experiments are compared with the estimates of the design codes in terms of strength ratio ($P\text{-Code}/P\text{-Exp}$), with calculations provided for the average, standard deviation, coefficient of variance, and standard error shown in Table 12. The strength ratio is also shown in Figure 59 for all the tested CFST specimens. It can be seen that the Chinese code (CD) provides conservative estimates in most of the cases, with an average strength ratio of 0.97 as shown in Table 12. The codes shown in Figure 59 are all in the non-conservative side with the ACI/AS being the closest to the experimental values than the EC4, which is the most non-conservative one, with an average strength ratio of 1.10.

Table 12: values of the experimental results with the design codes prediction as ratio.

Specimens	P_{Exp} (KN)	$P_{ACI/AS}/P_{Exp}$	P_{CD}/P_{Exp}	P_{EC4}/P_{Exp}	P_{AISC}/P_{Exp}	P_{AIJ}/P_{Exp}
M1-1	889.85	0.93	0.88	0.98	0.97	0.96
M1-2	954.80	0.87	0.82	0.92	0.90	0.90
M2-1	784.05	1.00	0.94	1.07	1.04	1.05
M2-2	798.00	0.98	0.93	1.05	1.02	1.03
M3-1	796.70	1.07	1.02	1.12	1.12	1.10
M3-2	816.00	1.05	0.99	1.09	1.09	1.08
M4-1	641.00	1.12	1.05	1.22	1.16	1.20
M4-2	686.80	1.04	0.98	1.14	1.08	1.12
M5-1	933.63	0.92	0.87	0.96	0.96	0.94
M5-2	839.50	1.02	0.97	1.07	1.07	1.05
M6-1	790.00	1.03	0.98	1.10	1.08	1.08
M6-2	616.80	1.32	1.25	1.40	1.38	1.38
M7-1	686.50	1.07	1.00	1.16	1.11	1.14
M7-2	707.50	1.04	0.97	1.13	1.08	1.11
M8-1	705.00	0.98	0.92	1.08	1.02	1.07
M8-2	722.00	0.96	0.89	1.05	0.99	1.04
Average	773.01	1.03	0.97	1.10	1.07	1.08
Standard deviation		0.103	0.097	0.111	0.107	0.110
Coefficient of variation		10.03	10.05	10.15	10.04	10.20
Standard error		2.57	2.43	2.78	2.68	2.75

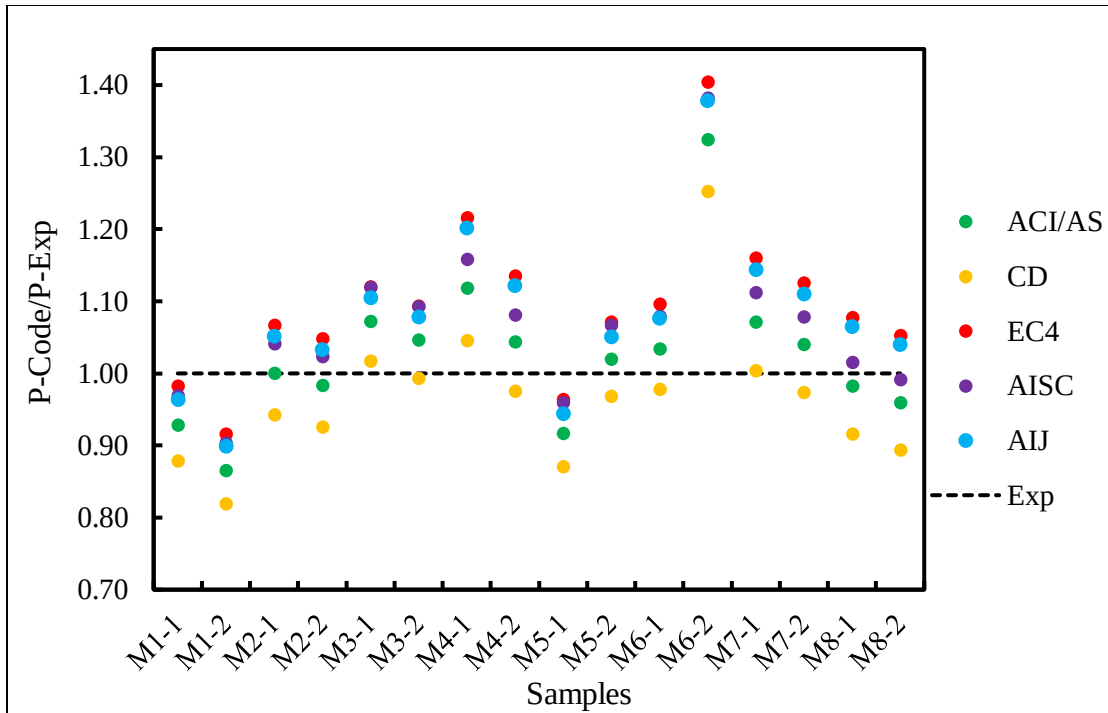


Figure 59. Comparison of the experimental results with the design codes predictions.

The findings suggest that most of the considered codes tend to overestimate the column capacity, except the Chinese code (CD). The ACI code and AS code specification slightly overestimates the capacity, with a ratio of 1.03, but remains close to the actual values. while the AISC, AIJ, EC4 codes significantly overestimate the capacities. The coefficient of variation and standard error are relatively consistent across all codes. The standard deviation, indicates that the Chinese and ACI/As codes have the lowest value, 0.097 and 0.103, respectively, followed by AISC by 0.107. The higher standard deviation in the other codes suggest that their predictions deviate more from the experimental results. In this analysis, the CFST is treated as a unified material, with empirical formulas derived from the regression of extensive experimental data. However, variations in material properties and the overlooked interaction between concrete and steel may lead to significant differences between predicted and experimental results.

5.5.3 Proposed Correction Factor for Design Codes

As it has been shown in the previous section, most of the design codes are overestimating the axial load capacity of the CFST made of concrete containing PW. This could be attributed to the fact that these design codes are derived for normal aggregate concrete and there is sufficient evident in the literature that they can also be applied to concrete incorporating (RCA) [80–86]. However, more effort is needed to apply these codes to concrete incorporating RCA and PW at the same time or concrete incorporating PW alone. From the results of this investigation, correction factors are derived to allow utilization of the considered codes for concrete incorporating PW.

To clearly visualize the variation of the strength ratio (P-Code/P-Exp.) with PW content, it is required to take the average strength ratio of each two pair of CFSTs, except M6-1&2 where M6-1 should be taken to present the average as M6-2 has experienced premature failure of the weld (Figure 51). The calculated average values of the strength ratios are plotted against the PW content as demonstrated in Figure 60, illustrates that at 0% PW nearly all the codes provide a strength ratio less than 1.0 for both with and without SF. Exempted from this trend is the EC4 and AISC codes for cases of concrete having SF, but the overestimation is less than 1.5 % which can be tolerated. This confirms the suitability of the considered codes for normal concrete and for concrete incorporating RCA as the natural CAs was replaced by 60% for all the concrete mixes used in this study (Table 2). With the increase of PW content, however, most of the codes moves to the non-conservative side of the curve (above the reference of 1.0). It can also be seen that at small

PW content cases with SF tend to overestimate the strength ratio while this reduces as the PW content increase to be less than those without SF.

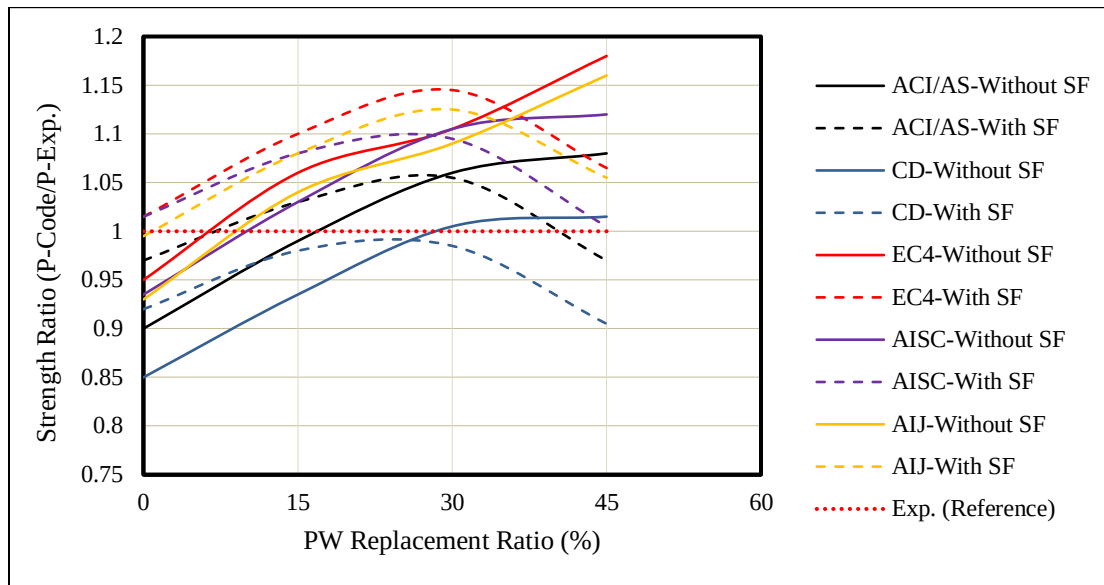


Figure 60: Variation of the strength ratio with PW replacement ratio

To develop simplified correction factor for each code, it is more convenient to merge each pair of curves (with and without SF) to the controlling one at each PW content. This is achieved by selecting the highest value of the of the strength ratio at each PW content for each code. The highest value is selected to develop a correction factor that prevent possible overestimation of the axial load capacity by the considered codes. The results of this merging process have been shown in Figure 61. Correction factors can therefore be derived from this figure.

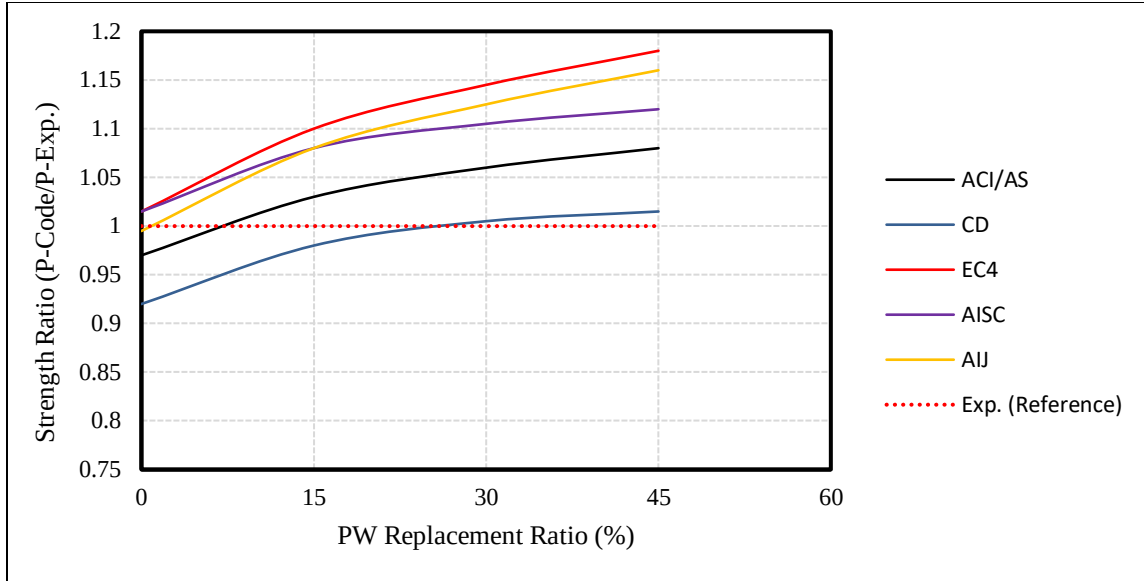


Figure 61: The variation of the controlling strength ration with the PW replacement ratio

To derive the correction factors, it is required to fit the distribution of the strength ratio with the PW content. This has been achieved using the “Trend Line” option available in Excel as shown in Figure 62. In this figure, the discrete data are the controlling strength ratios for each pair of specimens, the curves shown by dashed lines are plots of the formulas that corresponding to the best fit. Each formula and its R^2 are shown in rectangle having borders of the same line of its fitting curve. For all them, R^2 is more than 0.993 which indicates an excellent match between the data and fitting curves.

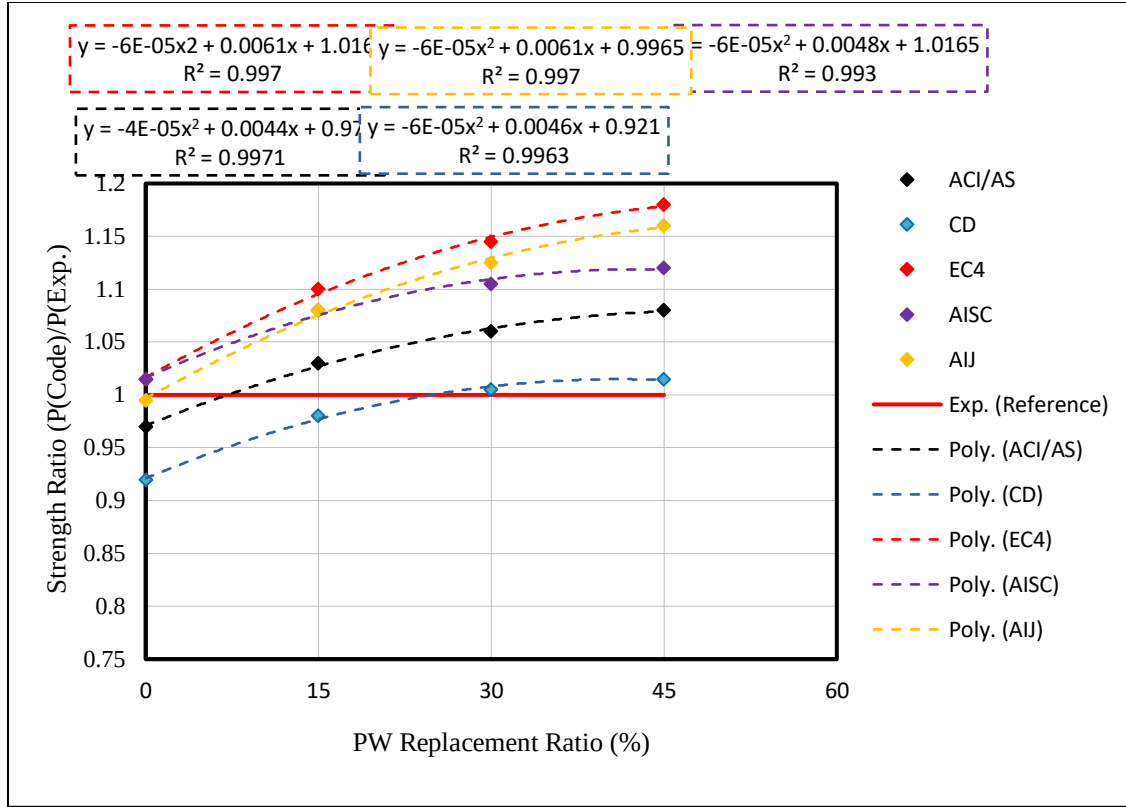


Figure 62: Fitting the strength ratio with the PW replacement ratio

If the considered design codes in this study are used to estimate the axial load-carrying capacity of CFST for cases the concrete involves PW to partially replace NAs, the codes estimates should be multiplied by correction factors as follows:

$$C.F = \frac{\varphi}{\alpha} \quad (7)$$

where φ should be taken as 0.95 to account for uncertainties encountered during derivation and α is a coefficient depending on the design code and PW content derived from the fitting formulas (Figure 62), and can be taken from Table 13. In the formulas shown in the table, PWC is the PW content in %. It should be noted that the value of α in the table has been conditioned to be more than 1.0. This is because it is particularly made to prevent the overestimation and ensure that the codes estimates are in the safer side. For cases where

the codes provide conservative estimates, the C.F should be taken as 1.0. For example, the Chinese code with PWC less than 20% the value of α is less than 1.0 (can also be seen in Figure 62) and the consequently Eq. (11) provides $\alpha = 1.0$ leading to a C.F of 1.0. This means the estimates of the Chinses codes should not be corrected as it provides a conservative estimate.

Table 13: The expression of α for various design codes

Code	α	Eq. #
EC4	$-6 \times 10^{-5}(PWC)^2 + 0.0061 PWC + 1.0165 \geq 1.0$	(8)
ACI/AS	$-4 \times 10^{-5}(PWC)^2 + 0.0044 PWC + 0.971 \geq 1.0$	(9)
AII	$-6 \times 10^{-5}(PWC)^2 + 0.0061 PWC + 0.9965 \geq 1.0$	(10)
CD	$-6 \times 10^{-5}(PWC)^2 + 0.0046 PWC + 0.921 \geq 1.0$	(11)
AISC	$-6 \times 10^{-5}(PWC)^2 + 0.0048 PWC + 1.0165 \geq 1.0$	(12)

Considering the correction factor proposed for the considered design codes (Eq. (7)), the corrected strength ratio (P-Codes/P-Exp) * is shown in Figure 63. The figure shows that after adopting the correction factors all the codes estimates has significantly improved and comes to the conservative side. Except M6-2 which encountered premature rupture of the weld.

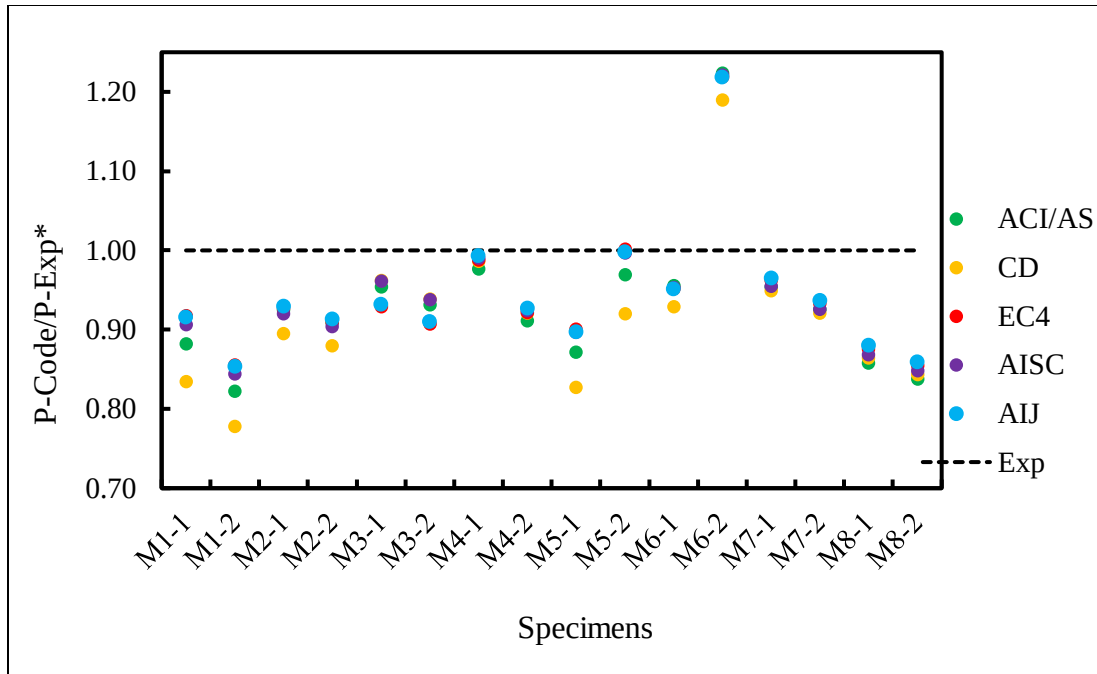


Figure 63: Evaluation of experimental outcomes against design code predictions after applying the correction factor (C.F)

5.6 Conclusions

The present chapter has investigated the use of concrete incorporating both (RCA) and (PW) in CFST. A total number of (19) steel tubes of similar dimensions were prepared, sixteen (16) were used to prepare CFSTs with two replicates from each concrete mix. The remaining three (3) steel tubes were tested without concrete to serve as references for comparison. The steel tubes and CFSTs were tested and the axial load-end shortening curves were generated. The data were then analyzed and the outcomes were compared to the selected design codes. According to this study the following can be concluded:

- There is a decrease in the axial load capacity of the CFST with the increase of PW content. This is partly because the concrete resistance to compression decreases as the PW content rises.
- At high replacement ratio of the PW, there is a considerable reduction in the load-carrying capacity especially for cases without SF where no well-defined maximum behavior was achieved. This could be attributed to the fact that the specific gravity of the PW is less than that of water, which allows more plastic float to the surface of the concrete especially for mixes having high slump values.
- The use of SF has improved the consistency of the concrete and allows the PW to be evenly distributed allowing the specimens to take more load compared to their counterparts without SF.
- Comparing the experimental data with the predictions from EC4, ACI/AS, AIJ, DC, and AISC design codes indicated that all the codes provide excellent estimates of the axial load capacity in the absence of plastics (0% PW). The strength ratio (P-Code/P-Exp.) was in the range of 0.850 to 1.015. This confirms the suitability of these design codes for concrete incorporating RCA.
- With the increase of the PW content, there is a trend of considerable overestimation by the EC4, ACI/AS, AIJ, and AISC. Only the DC remains mostly in the conservative side except at high PW content. This suggests that the existing design codes should not be used for concrete incorporating PW without corrections or modifications.
- Finally, the obtained results were utilized to derive correction factor to be multiplied by the estimate of the design codes for cases of concrete incorporating

PW. The derived correction factor will allow the utilization of the considered design codes for concrete incorporating PW.

CHAPTER 6

Concrete incorporating plastic wastes: challenges and possible solutions

6.1 Background

Due to huge (PW) accumulation, there have been considerable efforts in recycling and reuse of PW including its utilization in concrete. The developed PW concrete (PWC) has extensively been studied and its mechanical and durability properties are well-documented in the literature. This chapter highlights the challenges associated with utilization of PWC and proposes possible solutions and recommendations. The identified challenges will be discussed in this study are: (a) the consistency of the fresh PWC, (b) the impact of low elastic modulus on the application of PWC in floor slabs and beams, and (c) the degradation at high temperature environment. Potential solutions and recommendations to overcome these challenges are also discussed.

6.2 Scope and Limitations

This chapter is part of an on-going research projects in utilization of PW in concrete to produce PWC. It is oriented to discuss the challenges associated with practical applications of PWC. Potential solutions and recommendations are within the scope of the present article. To enhance the discussion and make valid comparisons, two concrete mixes, without and with PW, will be used throughout the present study. Sample preparation and

the detailed experimental program are not within the scope of the present article and their results will be presented wherever appropriate.

The adopted two mixes throughout the present study are shown in Table 14. The two mixes are identical in their constituents except that the (NFA) in the control mix without PW (M-Ref.) are partially replaced by 30% PW (by volume) to produce M-PW. The adopted PW is in the form of HDPE particles as shown in Figure 64. Further details about the experimental program can be seen in study by Ahmed et al [23].

Table 14: The constituents of the adopted mixes

Key mix factors		Required quantities to produce ONE cubic meter of concrete							
Mix	PW (%) of FA	Cement (kg)	SF (kg)	SP (kg)	NFA (kg)	PW (kg)	NCA (kg)	RCA (kg)	Water (Ltrs)
M-Ref.	0.0	370.5	19.5	1.56	881.4	0.0	352.8	498.4	222.1
M-PW	30.0	370.5	19.5	1.56	617.0	95.5	352.8	498.4	221.1



Figure 64: The adopted PW in the form of HDPE particles

6.3 Challenges of PWC

6.3.1 The Consistency of The Fresh PWC

It has been noticed that PW particles segregate and float to the surface of the concrete. This occurs during vibration and it is higher for cases of samples with high slump values. For example, Figure 65 (a) presents a fresh PWC placed in a transparent plastic cylinder before vibration. It can clearly be seen that the PW particles (in orange) are evenly distributed along the height of the concrete sample. Immediately after vibration, however, the PW particles float to up to the surface of the concrete as shown in Figure 65(b). This phenomenon could be linked to the low relative density of PW, which is less compared to water (<1.0). The slump of M-PW was found to be 215 mm as compared to 200 mm of M-Ref.

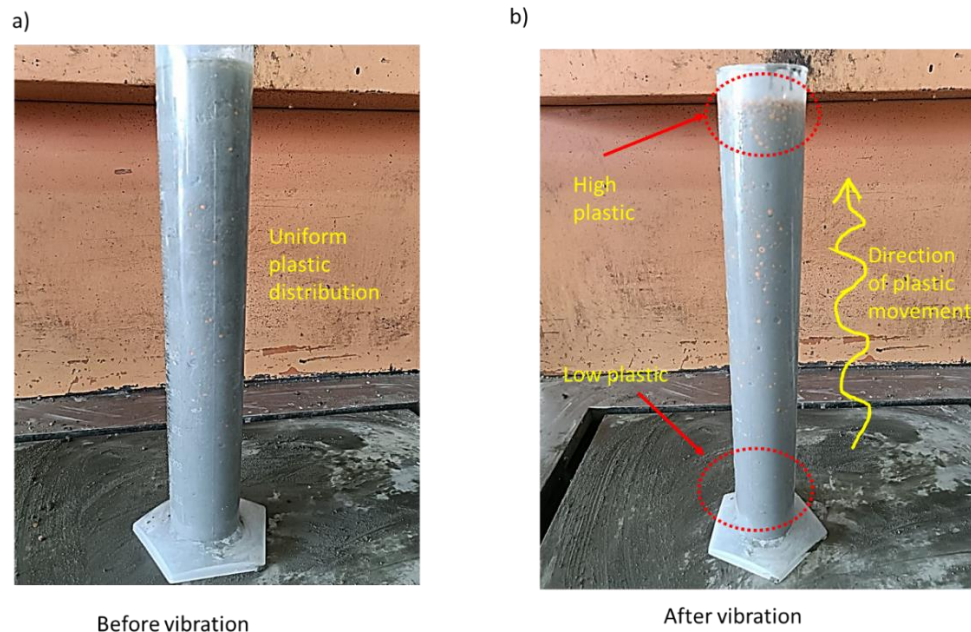


Figure 65: The distribution of plastic granules: (a) before vibration and (b) after vibration

6.3.2 The Impact of Low Elastic Modulus

In many instances the designers find that the floor slab being designed is controlled by the long-term deflection. One of key concrete parameters related to the deflection of slabs is the elastic modulus of the concrete E . To identify the impact of the concrete elastic modulus of PWC on the long-term deflection of slabs, let us consider a numerical example of a simply supported floor slab with plan dimension of 5.0 m by 5.0 m and a thickness of 150 mm. In addition to its own weight, the slab is considered to support a super dead load (SD) that includes finishes and attachments, and live load (LL). The slab is modeled in CSI SAFE software to obtain the long-term deflection considering the two mixes (M-Ref and M-PW) separately. The numerical data used in the model are shown in Table 15 and the meshed slab model is shown in Figure 66.

Table 15: Numerical data used in the SAFE models

Material data (based on test results unless otherwise specified)		
Property	M-Ref.	M-PW
Compressive strength (f_{cu}) in MPa	44.83	32.48
Cylindrical compressive strength ($f'c$) in MPa taken as $0.8 f_{cu}$ [16]	35.86	25.98
Elastic modulus (E) in GPa	21.56	14.63
Unit weight in kN/m ³	22.3	18.0
Load Patterns and Combination		
Own weight (OW)	Calculated by the software	
Live load (LL) in kN/ m ²	4.0	
Super dead load (SD) in kN/ m ²	5.0	
Design load combination	1.2 (OW+SD) +1.6 (LL)	
Load Cases		
Sustained load ⁽¹⁾	1.0 (OW+SD) +0.3 (LL)	
Long-term deflection case ⁽²⁾ (Start at the end of the above case)	0.7 LL	
Slab dimensions	5.0 m (length), 5.0 m width, and 150 mm depth	
Mesh size	0.25 m	
NOTES:		
(1) This case considers 30% of the live load contributes to the sustained load. This proportion of the live load is assumed to remain applicable over sufficient period (say two years or more).		
(2) The remaining proportion of the live load should start from the end of the sustained load case.		

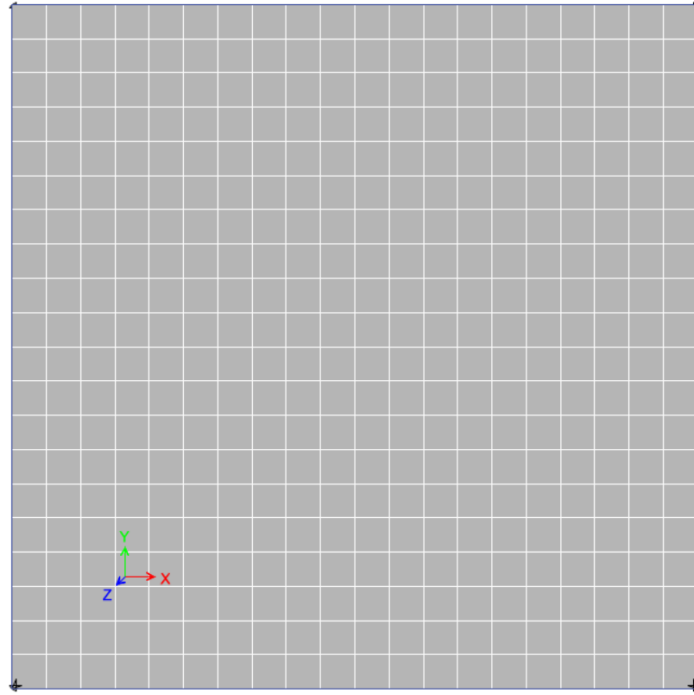


Figure 66: The meshed slab model

The analysis results of the long-term deflection considering the two mixes (M-Ref. and M-PW) are shown in Figure 67. As expected, the low elastic modulus of M-PW makes the slab to deflect more. The percentage increase in deflection for case M-PW is 24.6%. In this specific case, both slabs are passing the condition of the long-term deflection ($L/240$) defined in the ACI 318-19 code [166]. However, the key point that can be inferred from this comparison is that floor slabs made of PWC experience higher deflection values. This becomes critical when the deflection is in the borders of passing the acceptable limits such as flat slab cases where the deflection is usually high.

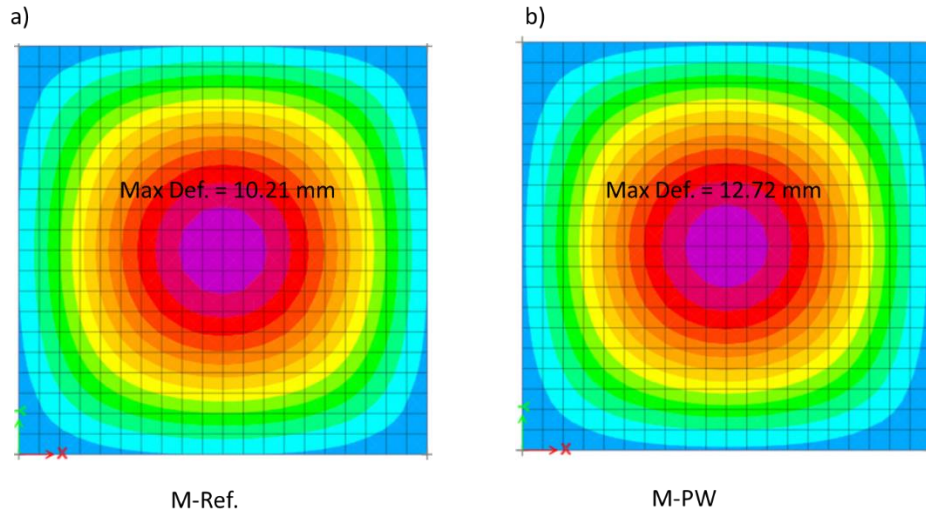


Figure 67: The long-term deflection of the two slabs

To further enhance the comparison, let us find the required M-PW slab thickness such that its long-term deflection will be similar to M-Ref slab of 150 mm. After few attempts in the SAFE model, the required thickness of M-PW was found to be 170 mm. This increase in slab thickness will be reflected in larger concrete volume and, consequently, all the concrete constituents will be higher as shown in Table 16. The table indicates that at a given long-term deflection value, the use of PW in M-PW added additional materials in all the concrete constituents in addition to the PW. It can therefore be concluded that it is not feasible to adopt PWC in floor slabs. Similar conclusion can be drawn if the comparison is made for cases of reinforced concrete beams. Therefore, the use of PW in floor slabs and beams is not recommended.

Table 16: The required constituents to produce two slabs of similar long-term deflection

	M-Ref slab	M-PW slab
Required thickness (mm)	150	170
Concrete volume (m³)	3.75	4.25
Cement (kg)	1389.38	1574.63
SF (kg)	73.13	82.88
SP (kg)	5.85	6.63
NFA	3305.25	2622.25
(kg)		
PW (kg)	0	405.88
NCA (kg)	1323	1499.40
RCA (kg)	1869	2118.20
Water (Ltrs)	832.88	939.68

6.3.3 Degradation at High Temperature Environment

The melting temperature of plastic is about 150° C [167]. This makes it logical that PWC cannot withstand high temperature. For example, a mix containing 30% plastic (similar to M-PW) in addition to the control mix (M-Ref) were gradually heated in a furnace capable of reaching a maximum temperature of 3000°C. The heating process took 2 hours to reach the target temperature (250°C), which was then maintained for an additional 2 hours. Afterward, the furnace was switched off, and the specimens were allowed to cool slowly over a period of 24 hours. Once cooled and becomes at room temperature, the samples were evaluated for cracks and appearance. The samples after heated in furnace as shown in Figure 68 , It can clearly be seen that the mix containing 30% plastic content considerably influenced by temperature relative to the reference mix. It is therefore not recommended to use concrete incorporating PW s in high temperature environments.

a)



M-Ref

b)



M-PW

Figure 68: Heated samples

6.4 Possible Solutions and Recommendations

The possible solutions and recommendations to the identified challenges are given in Table 17. More attention is given to the low elastic modulus as compared to the reference concrete. This suggests the use of PWC in structural members where the long-term deflection is not a major concern. For example, it can be used in deep foundations, where the deflection occurs at the upper portion of the pile as shown in Figure 69, which is the elastic deflection [168] generated for case of 15 m long pile subjected to 500 kN lateral load, and considering M-Ref and M-PW mixes. Another example of applications of PWC is in (CFSTs), where the confinement by the tube enhances the mechanical properties of the concrete. Figure 70 presents the results of CFSTs made of the two concrete mixes (M-Ref and M-PW), and compared with the empty steel tubes. The comparison demonstrated an improvement in the load-bearing capacity of the M-PW CFST relative to the empty tube.

Table 17: Suggested solutions and recommendations to the identified challenges

Identified challenge	Possible solutions or recommendations
The consistency of the fresh PWC	- Controlling the slump value to the lowest acceptable limit. - Development of self-compacted PWC to eliminate the needed vibration. - Investigate possible treatment of the PW to increase the specific gravity to be more than that of water (> 1.0). - Employing mineral additives like SF, combined with PW, can significantly improve the consistency of the mix.
The impact of low elastic modulus	- Avoid using PWC in floor slabs and beams and deflection-sensitive members. - It can be used in foundations, especially deep piles where the majority of the pile is not affected by the lateral deflection (<i>Fig. 7</i>). - It can also be used in concrete-filled steel tubes as the confinement enhances the mechanical properties of the concrete (<i>Fig. 8</i>).
Degradation at high temperature environment	- Avoid using PWC in roofing systems. - It can be used in foundations and slabs-on-grade.

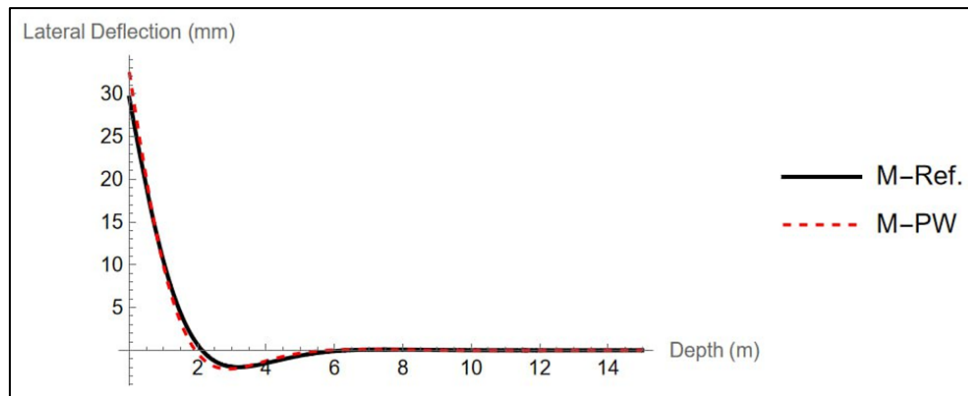


Figure 69: Variation of deflection along depth of the pile

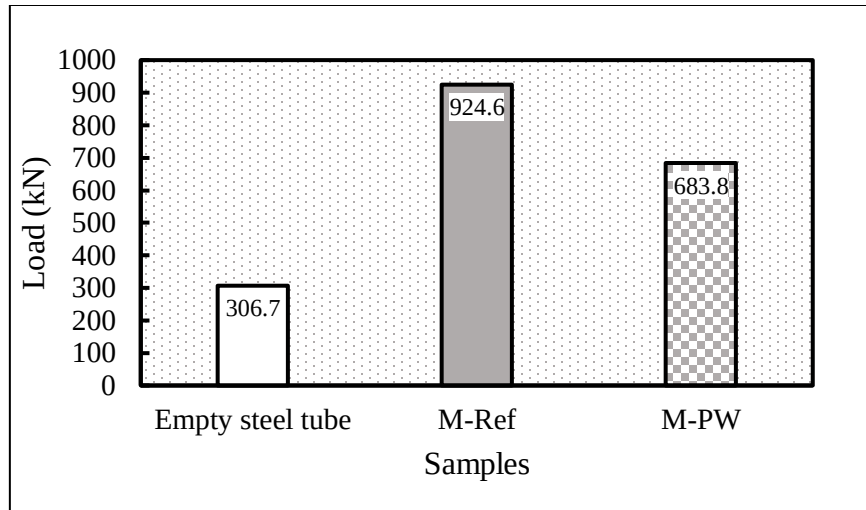


Figure 70: Behavior of concrete filled steel tubes made of M-Ref and M-PW

6.5 Summary

This chapter identified three challenges associated with application of PWC. The identified challenges are: (a) the consistency of the fresh PWC, (b) the impact of low elastic modulus on the application of PWC in flooring system and (c) the degradation at high temperature environments. Suggestions for applications of the PWC in construction industry are provided which mainly focused in its utilization in foundation systems and CFSTs. Future research recommendations are also provided to investigate the possibility of increasing the density of PW before utilization in concrete and development of self-compacted PWC to eliminate the need for vibration.

CHAPTER 7

Summary, Conclusions and Recommendations

7.1 Summary and Conclusions

A comprehensive study was conducted to investigate the behavior of HRAC concrete incorporating RCA and PW. The strength characteristics of this material were first presented to serve as a basis for comparison and discussion of the durability and structural behaviors. Eight concrete mixes were prepared with the natural CAs being replaced by 60% RCA (by volume) in all the mixes. The FAs are replaced by PW in the form of (HDPE) granules at volumetric replacement ratios of 0%, 15%, 30%, and 45%. In addition, SF is used to replace the cement weight at replacement ratios of 0% and 5 % (with and without SF). Performance in terms of mechanical strength and durability were investigated. In addition, the application of the developed HRAC in CFSTs was also investigated. Out of the conducted investigations in this study, the following can be concluded:

- The use of RCA, PW and SF in concrete offer a sustainable solution for incorporating waste materials, enhancing environmental sustainability.
- The mechanical characteristics of concrete, such as its compressive, tensile, and bending strengths (Modulus of rupture), generally decline alongside higher PW content due to insufficient cohesion at the interface of plastic and the cementitious matrix and the weak modulus of elasticity. However, incorporating SF improves strength properties up to a certain level of PW replacement.

- The results of the durability investigation indicate that using PW granules as FAs is not feasible at 30% replacement or higher due to melting at 250°C. Higher HDPE content decreases density while increasing water absorption, porosity, drying shrinkage, and electrical resistivity, and reduces compressive strength, electrical conductivity, and thermal conductivity. SF improves the microstructure and reduces water absorption, porosity, and chloride penetration, and enhances sound absorption up to 30% HDPE.
- CFSTs containing RCA and PW show a decrease in load capacity with increasing plastic content. Most importantly, existing design codes accurately predict load capacity for 0% PW but overestimate it in the presence of PW content, suggesting a need for adjustments. A correction factor was proposed for more accurate design code estimates for cases of concrete incorporating PW.
- Challenges exist when using PW in concrete were identified and discussed. These include the consistency of the fresh concrete, the actual impact of reduced mechanical performance, and melting at higher temperatures. Solutions and possible applications were proposed to promote the adoption of the developed concrete in construction sector.

7.2 Future research and recommendation

- **Explore optimal PW percentage level of replacement:** Investigate the use of intermediate percentages of HDPE PW (between 15% and 30%) to identify the most effective levels that balance workability, mechanical strength, and durability without compromising the concrete's performance or sustainability.

- **Chemical treatment studies:** There is a need to investigate the chemical treatments of PW that could enhance the bonding between HDPE and the cement matrix, potentially improving the concrete's structural properties (mechanical) and resistance. (durability)
- **Life cycle assessment (LCA):** Perform comprehensive LCAs to evaluate the environmental impact, including carbon footprint, of producing and using concrete with HDPE and SF, factoring in resource extraction, production processes, and disposal.
- **Regulatory framework development:** Supporter for the development of updated design codes and standards to better accommodate concrete mixes containing recycled materials, including PW, and establish guidelines for their safe and effective use in construction.
- **Innovative applications in structural members:** Continue to investigate applications for RA and PW concrete, especially in non-load-bearing or low-load structural components, foundation systems, and CFSTs, where their unique properties can be advantageous.
- **Development of self-compacting concrete:** Investigate the feasibility of developing self-compacting concrete incorporating PW to address challenges with consistency and segregation, eliminating the need for vibration during placement.
- recommended to conduct a long-term microstructure analysis to examine the interaction between cement and plastic in the concrete mixture. This will provide valuable insights into the bonding and durability of the composite material.

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